Proceedings of the International Conference on

# Sustainable Built Environments

12th, 13th & 14th December, 2010
Earl's Regency Hotel, Kandy,
Sri Lanka.

**Edited by**

***Prof. M T R Jayasinghe,***
*Professor of Civil Engineering, Department of Civil Engineering, University of Moratuwa, Sri Lanka.*

***Prof. P A Mendis,***
*Professor of Civil Engineering, University of Melbourne, Australia*

***Prof. Ranjith Dissanayake,***
*Professor of Civil Engineering, Department of Civil Engineering, University of Peradeniya, Peradeniya, 20400, Sri Lanka.*

| | | | |
|---|---|---|---|
| University of Moratuwa<br>Katubedda, Moratuwa,<br>Sri Lanka. | University of Peradeniya<br>Peradeniya 20400,<br>Sri Lanka. | University of Melbourne<br>Victoria,<br>Australia | Institution of Engineers<br>Colombo 07,<br>Sri Lanka. |

# Editor's Preface

It is with great pleasure that we announce the organizing of International Conference on Sustainable Built Environments (ICSBE 2010) scheduled for 13th and 14th of December, 2010. This Volume 2 of proceedings of ICSBE 2010 contains the research papers that are presented on 14th of December 2010 along with the extended abstracts of keynote speeches. All the research papers of this Volume 2 have been peer-reviewed. The editors are very much grateful to the authors for contributing research papers high quality. We also acknowledge the financial sponsorship provided by many organizations that has been extremely helpful for organizing a successful international conference.

We are pleased to acknowledge the advice and assistance provided by the members of International Advisory Committee along with many others who volunteered to assist to make this very significant event a success. It is the earnest wish of the editors that this volume of proceedings would serve a very useful service with the research community directly or indirectly involved in studies related to sustainable built environments.

***Prof. M T R Jayasinghe,***
*Professor of Civil Engineering, Department of Civil Engineering, University of Moratuwa, Sri Lanka.*

***Prof. P A Mendis,***
*Professor of Civil Engineering, University of Melbourne, Australia*

***Prof. Ranjith Dissanayake,***
*Professor of Civil Engineering, Department of Civil Engineering, University of Peradeniya, Peradeniya, 20400, Sri Lanka.*

# Table of Contents

## KEYNOTE SPEECHES



## RESEARCH PAPERS PRESENTED ON 14TH OF DECEMBER, 2010

**EXPOSURE RISK ASSESSMENT OF POLYCYCLIC AROMATIC HYDROCARBONS ASSOCIATED WITH THE INDOOR AIR PARTICULATE MATTERS IN SELECTED INDOOR ENVIRONMENTS IN KANDY, SRI LANKA**
D G G P Karunaratne, S.B.A.M.B.Y. Wijayarathna

**CAPACITY BUILDING FOR SUSTAINABLE ENGINEERING PRACTICE: A CIVIL ENGINEERING EDUCATION PERSPECTIVE**
S. Senadheera

**INTEGRATED LAND USE AND MULTIPLE WATER SUPPLY -DEMAND MODELLING FRAMEWORK: A PERI-URBAN CASE STUDY**
B. Nawarathna, H. Malano, B. Davidson, B. Maheshwari

# THE ROLE OF STRUCTURAL FABRICS IN A SUSTAINABLE CONCRETE INFRASTRUCTURE

Ibell T.J.[a], Darby A.P.[b], Orr J.J.[b] and Evernden M.[b]

[a]Corresponding author, BRE Centre for Innovative Construction Materials, University of Bath, Bath, BA2 7AY, United Kingdom, t.j.ibell@bath.ac.uk, Tel +44 1225 386365, Fax +44 1225 386691
[b]BRE Centre for Innovative Construction Materials, University of Bath, Bath, BA2 7AY, United Kingdom, a.p.darby@bath.ac.uk, j.j.orr@bath.ac.uk, m.evernden@bath.ac.uk

**Abstract**
Concrete is the second-most used substance on Earth after water, and the production of cement accounts for at least 5% of the planet's carbon emissions. Concrete has all sorts of excellent properties, which should not be overlooked, but it seems clear that we should be exploiting these fine properties against a backdrop of needing to look carefully at how we manage our concrete infrastructure sustainably. We need to use realistic approaches to understand structural integrity of our existing concrete infrastructure if we are not to needlessly condemn adequate structures. We need to understand how to prolong the life of existing concrete structures in a robust, proven and cost-effective manner, again so that additional carbon- and energy-related costs associated with rebuild are avoided. And we need to design our future concrete structures with the most important property of concrete at the forefront, namely its moudability, something which is seldom exploited, such that efficient use of concrete is ensured. This paper outlines research conducted in the BRE Centre for Innovative Construction Materials at the University of Bath in these areas of structural strengthening and future innovative design of concrete structures. The paper focuses on the role that structural fabric can play in contributing to these aims.

***Keywords:*** *Concrete, fibre-reinforced polymer (FRP), fabric, formwork, strengthening*

## 1. Introduction

The use of concrete creates a profound carbon footprint across the planet. What this ultimately means is that if we already have concrete structures in place, we should do our utmost to ensure that we prolong their lives such that we do not need to rebuild such structures. This is a key requirement. If we then wish to add to our existing building stock using concrete, we should use concrete appropriately and in an efficient manner. This paper addresses both these strands of life extension and innovative concrete structures through presenting research which the authors have conducted recently using fabric to strengthen and form concrete structures.

## 2. Strengthening concrete structures

### *2.1 The use of fibre-reinforced polymer (FRP) materials*

The technique of strengthening existing concrete structures using fibre-reinforced polymer (FRP) materials is mainstream and well documented. Various design guides around the world exist to advice on such strengthening, including the UK's TR55 [1], which was lead-authored by the University of Bath. The basic idea is that one uses FRP material (usually carbon) in the form of fabric, bar, plate or strip to increase the flexural, shear or axial capacity of concrete structures by adhering the FRP to the surface of the concrete, thereby significantly increasing effective reinforcement, either directly in tension or indirectly in the form of confinement. The FRP is extraordinarily strong, stiff, durable and easy to apply. It is expensive, but where time of retrofit is paramount to project costs, the use of FRP is prolific across the world to extend the lifetime of our concrete infrastructure.

But this material does not come without drawbacks. FRP is prone to debonding from the surface of concrete before it reaches its full rupture capacity. It is also very brittle, such that ensuring that the structure still behaves in a ductile manner after strengthening is a crucial design skill, and an area which the University of Bath has researched significantly over many years. There are still question marks over fire, although the concept of fire engineering has largely solved this issue.

But the largest present drawback concerning FRP for strengthening applications is its short history. Thus, we are not yet sure about various important design considerations, because the oldest strengthening projects do not date back further than 30 years, and most are significantly more recent than that. Most strengthening schemes in buildings require that the columns are upgraded for higher axial capacity (and to be more ductile) and that the beams are upgraded for higher flexural capacity. As the flexural capacity of such beams increases, however, so too does the demand on shear capacity. Enhancing shear capacity using fabric is altogether a more difficult thing to do because of brittleness and construction issues. At Bath, we have attempted to focus our efforts on consideration of axial-strength enhancement when strengthening rectangular columns and on the problems associated with size effect when strengthening large structures in shear. Findings from these areas of research aimed at using fabric to strengthen concrete structures are discussed below.

### *2.2 Rectangular wrapped columns*

When circular concrete columns are wrapped using FRP material, there are many potential beneficial effects on strengthening. Flexural tensile strength, shear resistance and axial capacity may all be increased. Further and probably most significant of all, the strain capacity of the concrete is increased due to excellent confining behaviour, leading to improved ductility and increased flexural strength. While square and rectangular columns also benefit from FRP wrapping, the confinement effect is less pronounced than in the circular case, due to the lack of convex curvature along the straight edges of the section. This means that the corners of the section are well confined, but not the entire cross section [2].

Recent tests have shown that the degree of confinement which rectangular columns experience when wrapped is indeed well defined in the new TR55 document, which itself is based on the model suggested by Teng *et al.* [3]. This model suggests the use of a cruciform zone of confinement, although the precise details of this have been modified for the TR55 document.

A major funded research project has just been completed at Bath. Wrapped rectangular columns of realistic size (up to 750mm in side dimension) were tested under various axial and flexural loadings in order to verify the authors' proposed design guidelines for such wrapping in TR55. Figure 1 shows the sorts of specimen which were tested using the BRE facilities. In a nutshell, this research has confirmed that the relatively high level of confinement achievable in wrapped rectangular columns under axial loading only should not be assumed for equivalent columns when also being bent. Importantly, it also appears from the results that we can indeed extrapolate smaller-scale test data for such wrapped columns. This is potentially extremely important, given the vast expense of testing at full scale.

Figure 1: *Large-scale FRP-wrapped column testing*

### 2.3 Size effect on shear strengthening

The use of FRP sheet fabric to strengthen concrete beams, columns and walls in shear is a well-documented technique, and the effectiveness of this system has been tested on many laboratory specimens over several years. However, up until recently, there had been no test conducted anywhere in the world on specimens greater than 600mm deep [4].

This is a major issue. If the FRP sheet is fully wrapped (typical for a column) or U-wrapped and mechanically anchored (just below the flange of a T-beam, for instance), then any shear cracking will lead to local debonding of the FRP. Such debonding will extend the full depth of the FRP, and it will strain between anchored locations as the shear discontinuity widens. If the beam is shallow, as in most laboratory tests, this strain will be substantial for a modest width of shear crack. However, if the beam is deeper (as is usually the case in reality), then the strain in the FRP will be low for a modest crack width. As the crack width increases, so the concrete contribution will reduce due to a reduction in aggregate interlock, so that the shear resistance will drop, even though the strain in the FRP is rising. The authors knew that this issue had to be serious, and went ahead with large-scale testing. Figure 2 shows one such specimen prior to being loaded in shear. It was 750mm deep. A significant size effect was indeed picked up in the testing [5].

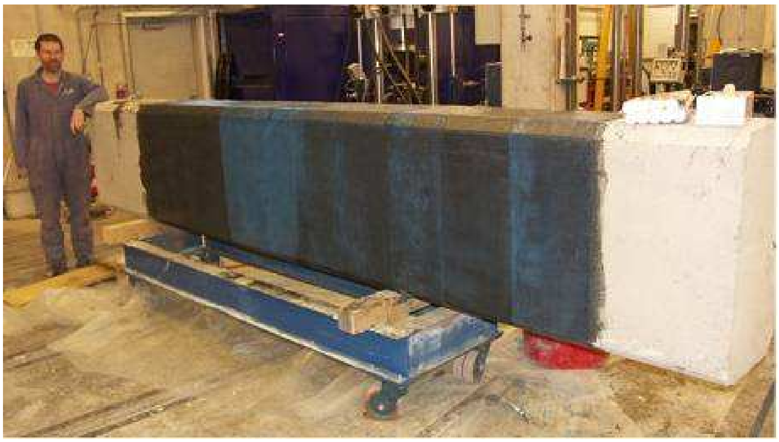


Figure 2: *Fabric-wrapped shear specimen of realistic size*

## 3. Fabric-formed concrete structures

### 3.1 Introduction

The use of fabric formwork for concrete structures can be traced back to the early 1900s, and the methods involved mainly stem from work in offshore and geotechnical engineering. In 1922 it was proposed to use concrete filled fabric bags in the construction of underwater concrete structures but it was not until the late 1960s that any real headway was made in this field, precipitated by the new availability of low cost, high strength, durable synthetic fibres that allowed the forming of complex shapes [6]. Initial interest in the architectural possibilities of fabric formwork can be attributed to the Spanish architect Miguel Fisac, who in 1969 completed the Centro de Rehabilitación para la Mutualidad del Papel (MUPAG) in Madrid. It was here that the first patented method for pre-fabricated fabric formed wall panels was developed. Subsequent developments have occurred simultaneously, yet independently, of each other. Whilst both Kenzo Unno and Rick Fearn have developed successful systems and techniques for fabric formed structures, the most influential work has come from Mark West, founder of the Centre for Architectural Structures and Technology (CAST) at the University of Manitoba in Canada, which is the first research centre dedicated to the development and promulgation of fabric formwork for concrete structures. It is this architecturally-led work that has formed the basis for previous research at the University of Bath. This section of the paper begins by considering the principles behind fabric formwork, before focusing on the current state of the art in design, optimisation and construction of fabric formed beams.

### 3.2 Traditional practice

Concrete has been primarily cast in orthogonal timber or steel moulds since the mid-1800s, resulting in the well established formwork practices that exist today. Rigid formwork systems tend to be simple to construct, but consume more material than an equivalent variable section member, increasing both cost and structural dead weight. Variable section members can feasibly be produced on an industrial scale, but their geometry remains governed by primarily prismatic forms.

### 3.3 Fabric formwork

Forming concrete in a flexible membrane (typically a high strength polyester fabric) provides a simple method for the construction of efficient, optimised and aesthetically pleasing concrete structures that offer several advantages for engineers. The material required for a fabric formed structure is lightweight, cheap and ubiquitous – 12m span beams have been formed using under 10kg of fabric [7]. Pouring concrete into a permeable fabric results in a filtering effect in which air and water are allowed to bleed from the structure, the effect of which is twofold. First, small increases in concrete compressive strength occur that are attributable to the reduction in water:cement ratio. More significant are the increases in surface density that occur as a result of there being very few air bubbles trapped between the formwork and concrete. Increases in surface density prevent in-service moisture and air ingress into the section, thereby slowing corrosion processes and potentially allowing for a reduction in cover to steel in fabric formed structures when compared to their conventional counterparts. The distance to which this 'case hardening' effect extends into the member is unclear at present.

Fabric formwork can be stitched into almost any configuration and the boundary conditions, including support locations and degree of pretensioning, can be altered to achieve the desired form. The construction of façade panels, columns, trusses, shells and beams has already been achieved, as illustrated in Figure 3.

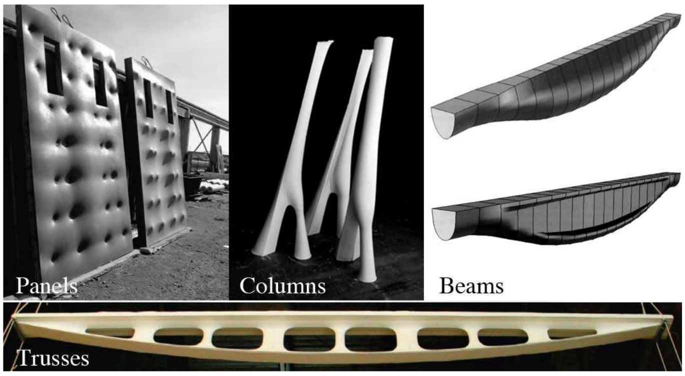



Figure 3: *Fabric formed structures (after West [7}; Garbett [8])*

### 3.4 Design

Design methods for fabric-formed beams are currently in a state of flux. Work at CAST has previously taken an empirical approach, and many beams were not reinforced or tested structurally. The final shape of such a beam is determined by the material properties of the fabric and boundary conditions imposed during construction.

The hydrostatic shape obtained from a given set of these boundary conditions can be accurately predicted using elastic theory, although dynamic relaxation has also been used to model the interaction between fabric and concrete [9]. In addition, Bailiss [10] and Garbett [8] used an empirical method to determine the area, perimeter and shape of fabric structures that was moderately successful. The design of fabric structures has, up to now, been approached primarily from an architectural perspective. Structural verification is now required, and this has been the focus of work at the University of Bath. Garbett implemented a sectional analysis method, as outlined in Figure 4, to design singly reinforced beams that were unreinforced in shear.

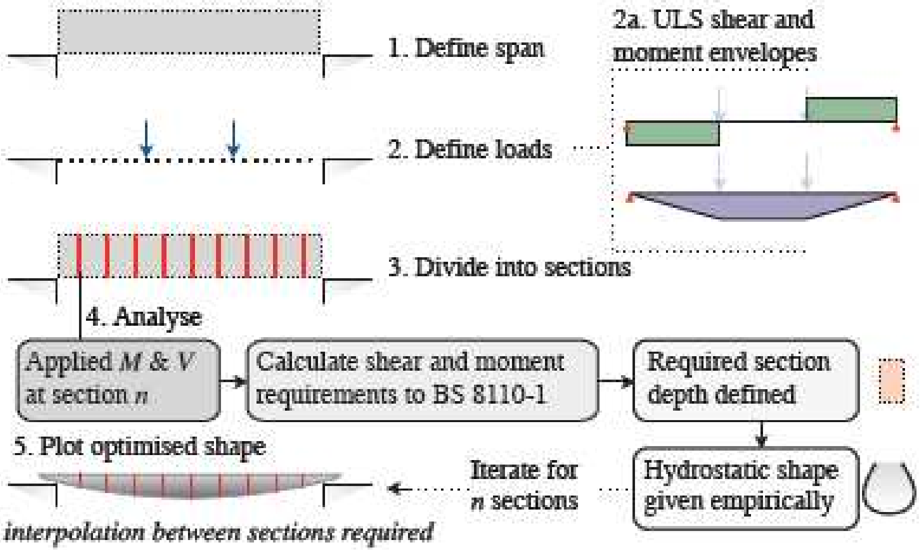



Figure 4: *Sectional design method*

Whilst the flexural strength of a reinforced concrete member can accurately be predicted using the plane section hypothesis, which forms the basis of many concrete design codes, shear design is widely considered to be an unresolved area of concrete technology. The complex cross sections of fabric formed beams make conventional shear links difficult to detail, yet omitting shear reinforcement relies on an accurate prediction of the unreinforced section's shear capacity. In many design codes around the world, the unreinforced capacity is predicted using empirical relationships based on many hundreds of beam tests, none of which were carried out on variable sections, thereby making the accuracy of these codes questionable. However, the use of the modified compression field theory in the Canadian CAN/CSA S6 [11] make this code better suited to the design of variable section members.

Longitudinal reinforcement in fabric formed beams has previously been limited to single or bundled bars, pre-bent to the desired profile. The accurate placement, and provision, of anchorage to such bars has proven to be difficult [8]. The welded end plate (Figure 5) is the most common anchorage connection, yet this leaves longitudinal steel susceptible to corrosion. The use of advanced composite reinforcement and post-tensioning offers a potential solution to this, as discussed later.

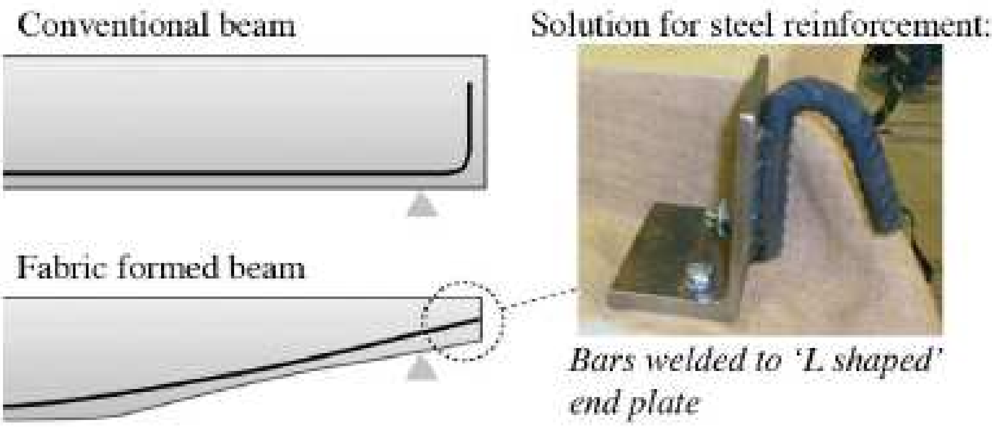



Figure 5: *Present unsatisfactory anchorage connection*

### *3.5 Optimisation*

Optimisation can be considered as the process by which variables are used to determine the best option for a given set of parameters. Physical modelling techniques used in the past have now been all but replaced with numerical simulation methods such as evolutionary structural optimisation, solid isotropic material penalisation and sensitivity analysis that are described in detail elsewhere [12]. Structural and material optimisation is a key component of fabric formwork. Up to now, bending moment-shaped beams (Figure 3) have been optimised using the previously described sectional analysis method and material reductions of up to 50% have been obtained when compared to an equivalent rectangular beam.

This is remarkable given the simplicity of both the design and construction of these beams and offers a real opportunity for material use reductions in entire building systems. More complex approaches utilising evolutionary optimisation are currently being considered that offer two opportunities to further reduce material usage. First, through more accurate modelling of the hydrostatic shape and concrete-fabric interaction during pouring and second through improved analysis of the reinforced concrete section under loading to ensure that material is provided only where it is required. However, computational methods must always consider construction processes to ensure the optimised beam design can feasibly be built using fabric formwork.

### *3.6 Construction*

Construction methods for fabric formed beams are continually improving. This section details four methods for the construction of variable section beams, three of which were developed by researchers at CAST [9]. The spline method (Figure 6) uses a metal bar to vertically pretension a single rectangular sheet of fabric held on a timber forming table. Pretensioning the fabric reduces the volume of concrete in the tension zone, thus providing an optimised design. Beams constructed using this method have previously had a parabolic elevation, although the final layout is determined by varying the locations and magnitude of the applied pretension. The keel mould (Figure 7) uses two sheets of fabric, held vertically and secured between sheets of plywood (the keel) that are cut to the desired beam elevation. The fabric is then draped over a forming table and pretensioned to both obtain the desired shape and to prevent wrinkling during construction.

The pinch mould (Figure 8) is used to create beams and trusses by sandwiching two sheets of fabric between a rigid timber mould. At designated locations protrusions from the timber mould ‘pinch’ the fabric to create openings in the web of the beam. The method allows the rapid construction of optimised beam elements, but constructing the formwork is more labour and material intensive than other methods. In addition, the structural behaviour of these beams is governed by Vierendeel action, which somewhat complicates their analysis. The fourth method, developed by Bailiss and Garbett utilises solely the wet concrete weight to form the beam. By predicting the shape of the fluid filled fabric, fixing points along its perimeter are determined. The fabric is then hung between two supports before being filled with concrete to obtain the desired forms, some of which are illustrated in Figure 3.

### *3.7 Structural tests*

A total of six singly reinforced beams, designed as described in Section 3.4, have been tested in five points bending at the University of Bath [13]. Of these, five were found to fail in shear close to the supports, although two tests failed to reach their design load due to incorrect positioning of the longitudinal reinforcement. Fabrication of the beams was generally successful, and empirical methods were employed to predict the hydrostatic section shape. In general, elastic and plastic methods for the prediction of failure loads were accurate as was the prediction of load-deflection responses by double integration of curvatures along the length of each beam.

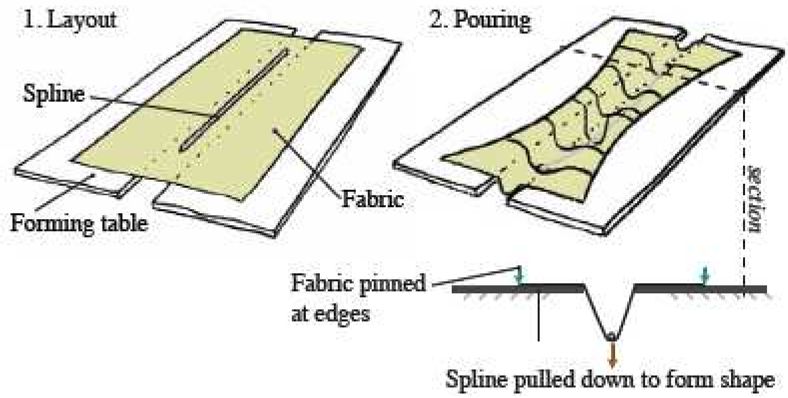



**Figure 6:** *Spline method*

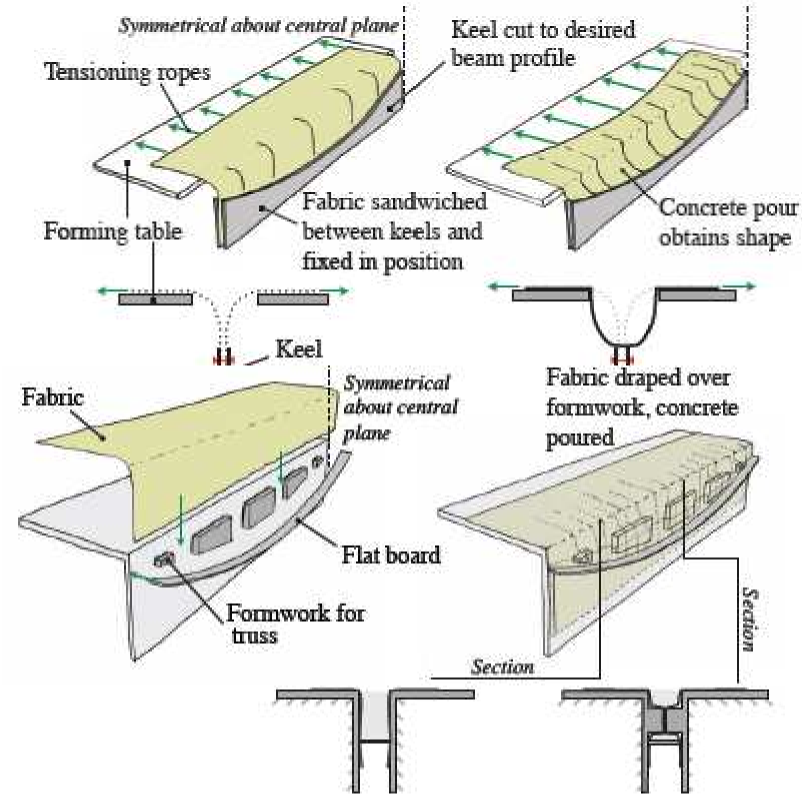



**Figure 8:** *Pinch mould*

The testing highlighted two areas that require further work. The predominance of shear failures in bending moment shaped beams was unexpected and highlights deficiencies in the current design procedure. In addition, anchorage methods for longitudinal reinforcement (Figure 5) leave steel bars exposed to corrosion and cannot be used if advanced composite reinforcement is to be used, an area of future development, discussed below.

### *3.8 The future design of fabric-formed concrete structures*

The field of fabric formwork is by no means limited to beams. Shells, working efficiently in membrane action, offer great advantages, but their design and construction are more complex than bending elements and they are rarely used in commercial building systems. However, the construction of shell structures using fabric formwork is well established at CAST, and work is being undertaken by the authors to investigate the potential for large scale uses of fabric formed shell structures that will bring further material and cost savings to concrete construction. Polymer (CFRP) sheet, acting as reinforcement and permanent formwork, is currently being investigated at the University of Bath. The high tensile strength and durability of advanced composites also makes them a logical alternative to steel as longitudinal reinforcement, but introduces new design problems. As welded end plates cannot be used for the anchorage of longitudinal FRP reinforcement, current research is investigating the use of an innovative wedging anchorage method for FRP bars. See Figure 9. The concept has been proven in cube pull out tests [14] where order of magnitude increases in load and displacement capacity were seen, and verification by beam tests is now required. However, advanced composites have high working strains and are therefore inefficient when used in passively reinforced structures. Advanced composites are most effectively used in prestressed structures, where greater moment capacity can be obtained and the full tensile strength of the tendon utilised. Post tensioned fabric formed beams are an exciting prospect, and offer potential improvements in moment and shear capacity. By sewing ducts into the formwork, tendon positioning within the section can be ensured, and the use of unbonded advanced composite rope bypasses potential corrosion concerns.

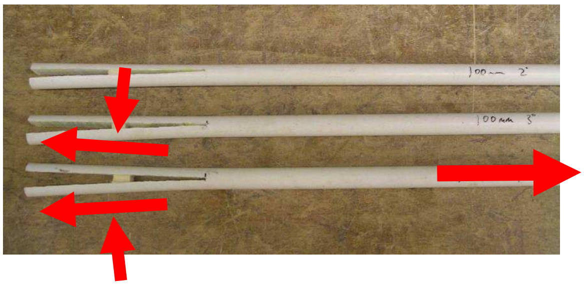


Figure 9: *Wedge anchorage system for FRP bars*

Figure 10: *Possible future fabric-formed structures*

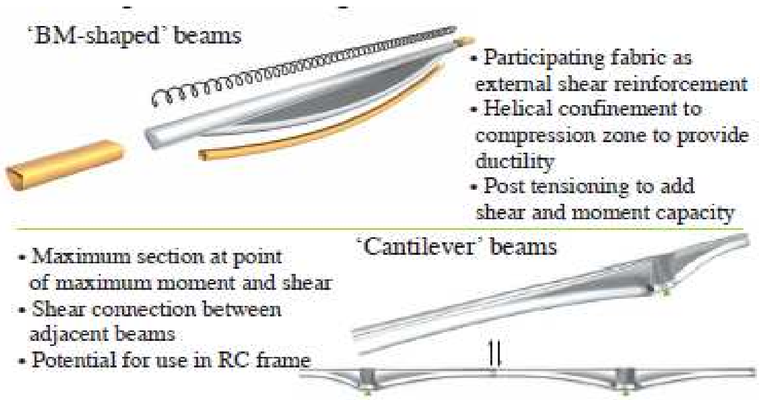



Figure 11: *Future innovations for fabric-formed structures*

## 4 Conclusions

Fabric formed concrete beams offer significant advantages for designers, including material reductions, ease of construction and aesthetic appeal. The forms are predictable and the development of robust methods for their design and optimisation is well underway. New materials, including advanced composites, prestressed reinforcement and fibre reinforced concrete offer additional advantages for fabric formed beams that will be investigated in future work.

## 5 Acknowledgements

The authors wish to gratefully acknowledge the financial contribution to this research which has been received from The Engineering and Physical Sciences Research Council of the UK, from Atkins Global and from the University of Bath. Grateful thanks are also due to all the technical staff in the Department of Architecture and Civil Engineering for their help in experimentation.

### References

[1] Concrete Society 2004. Design guidance for strengthening concrete structures using fibre composite materials. Technical Report 55, Crowthorne. 102pp.

[2] Cluett, J. 2005. The strengthening of rectangular columns using FRP wrap. MEng dissertation, University of Bath.

[3] Teng, J.G., Chen, J.F., Smith, S.T. and Lam, L. 2002. FRP strengthened concrete structures, Chichester, Wiley.

[4] Denton, S.R., Shave, J.D. and Porter, A.D. 2004. Shear strengthening of reinforced concrete structures using FRP composite. *Advanced polymer composites for structural applications in construction*. Woodhead, pp134-143.

[5] Wen, X. 2006. Size effect in shear behaviour of FRP-strengthened concrete beams. MPhil dissertation, University of Bath.

[6] Lamberton, B.A., 1989. Fabric forms for concrete. *Concrete International.* 11(12): 58-67.

[7] West, M., 2007. Construction. Winnipeg: Centre for Architectural Structures and Technology, University of Manitoba.

[8] Garbett, J., 2008. Bone growth analogy for optimising flexibly formed concrete beams. Thesis (MEng). University of Bath: Bath.

[9] Veenendaal, D., 2008. Evolutionary Optimization of Fabric Formed Structural Elements. Thesis (MS). Delft University of Technology: Delft.

[10] Bailiss, J., 2006. Fabric-formed concrete beams: Design and analysis. Thesis (MEng). University of Bath: Bath

[11] CAN/CSA S6:2006. Canadian Highway Bridge Code. CSA.

[12] Rozvany, G.I.N., 2008. A critical review of established methods of structural topology optimization. Struct Multidisc Optim. 37: 217-237

[13] Ibell, T.J., Darby, A.P., Bailiss, J.A., 2007. Fabric formed concrete beams. In: Darby, A.P., ed. ACIC 2007, April 2-4 2007, Bath: University of Bath.

[14] Darby, A.D., Ibell, T.J., Tallis, S., & Winkle, C., 2007. End Anchorage for Internal FRP reinforcement. In: Triantafillou, T.C., ed. FRPRCS-8, July 16-18, University of Patras

# DISCUSSION PAPER: DEVELOPING A RESILIENT BUILT ENVIRONMENT: POST-DISASTER RECONSTRUCTION AS A WINDOW OF OPPORTUNITY

Dr. Richard Haigh

## 1. Introduction

With growing population and infrastructures, the world's exposure to hazards – of both natural and man-made origin – is predictably increasing. This unfortunate reality will inevitably require frequent reconstruction of communities, both physically and socially. At the same time, it will be vital that any attempt to reconstruct after a disaster actively considers how to protect people and their environment to ensure those communities are less vulnerable in the future. In summary, it requires reconstruction of a more resilient built environment. This discussion paper considers what is meant by a resilient built environment, why it is needed, why post-disaster reconstruction presents a window of opportunity, and how reconstruction of the built environment can contribute to broader societal resilience.

For the remainder of this discussion paper and in common with The Centre for Research on the Epidemiology of Disasters (CRED), which maintains the International Disasters Database (EM-DAT), a disaster is a "situation or event, which overwhelms local capacity, necessitating a request to national or international level for external assistance; an unforeseen and often sudden event that causes great damage, destruction and human suffering". For a disaster to be entered into the database at least one of the following criteria must be fulfilled: 10 or more people reported killed; 100 people reported affected; there is declaration of a state of emergency; or, a call for international assistance.

## 2. A global challenge

There are wide-ranging origins and causes to the many disasters that have affected communities across the world with ever greater frequency. The term disaster is frequently associated with geo- and hydro-meteorological hazards, such as hurricanes, earthquakes and flooding. Three main categories of natural disasters account for 90% of the world's direct losses: floods, earthquakes, and tropical cyclones (Munich Re, 2010).

The degree to which such disasters can be considered 'natural' has long been challenged. In their seminal paper entitled "Taking the 'naturalness' out of natural disasters", O'Keefe et al. (1976) identified the cause of the observed increase in disasters as, "the growing vulnerability of the population to extreme physical events", not as changes in nature. However, as Kelman (2009) observes, even as early as 1756, Rousseau, in a letter to Voltaire about the earthquake and tsunami that hit Portugal a year earlier, commented that, nature did not build the houses which collapsed, and suggested that Lisbon's high population density contributed to the toll.

More recently, the links between disasters and climate change have increasingly been recognised. There are growing concerns over the threats posed by climatological hazards such as extreme temperatures, drought and wild fires, and the multi-faceted threats associated with sea level change. The scale of human contribution to climate change may still be open to debate, but there is widespread, although many would argue, insufficient concern from politicians, commentators, researchers and the public alike, over its ability to increase the number and scale of hazards, and the potential for resultant impact on communities world-wide. The World Meteorological Organisation (WMO) figures showed that 2008 was the 10th warmest year since reliable records began, meaning that the 10 warmest years on record all occurred in the past 12 years.

Alongside disasters of so called 'natural origin', many other disasters to affect populations in recent times are unquestionably of human origin. Conflict sometimes results in wars and terrorist acts that match or exceed the losses from any 'natural' disaster. Other types of disaster, often referred to as 'technical', result from equipment malfunction or human error. Although less frequent they still have the potential to cause widespread damage to people and property.

Regardless of the origins and causes, as previously noted by the authors (Haigh and Amaratunga, 2010), the consequences to human society are frequently similar: extensive loss of life, particularly among vulnerable members of a community; economic losses, hindering development goals; destruction of the built and natural environment, further increasing vulnerability; and, widespread disruption to local institutions and livelihoods, disempowering the local community.

In 2008, more than 220,000 people died in events like cyclones, earthquakes and flooding, the most since 2004, the year of the Asian tsunami (Munich Re 2010). Meanwhile, overall global losses totalled about 200 billion USD, with uninsured losses totalling 45 billion USD, about 50% more than in 2007. This makes 2008 the third most expensive year on record, after 1995 when the Kobe earthquake struck Japan, and 2005, the year of Hurricane Katrina in the US. The frequency, scale and distribution of disasters in recent years is further evidence, if any is needed, that hazards – of both natural and man-made origins – are a global problem, threatening to disrupt communities in developed, newly industrialised and developing countries. The developed world cannot afford to be complacent.

But recent disasters also highlight that developing and newly industrialised countries are most at risk: the losses to life and the economy – as a percentage of GDP – are far greater. During the last decade of the 20$^{th}$ Century, direct losses from natural disasters in the developing world averaged 35 billion USD annually (Munich Re 2000). Although a disturbingly high figure by itself, perhaps more worryingly, these losses are more than eight times greater than the losses suffered over the decade of the 1960's.

In part, this high risk felt by developing and newly industrialised countries can be attributed to hazard frequency, severity and exposure. The three main categories of natural disasters that account for the greatest direct losses – as identified earlier, these are floods, earthquakes, and tropical cyclones – periodically revisit the same geographic zones. Earthquake risk lies along well-defined seismic zones that incorporate a large number of developing countries. High risk areas include the West Coast of North, Central and South America, Turkey, Pakistan, Afghanistan, India, China, and Indonesia. Similarly, the pattern of hurricanes in the Caribbean and typhoons in South Asia, Southeast Asia, and the South Pacific is well established. These typically affect Algeria, Egypt, Mozambique, China, India, Bangladesh, Taiwan, Indonesia, Philippines, Korea, Afghanistan, Armenia, Georgia, Iran, Mongolia, Thailand, Argentina, Brazil, Chile, Colombia, Cuba, Ecuador, El Salvador, Guatemala, Honduras, Mexico, Nicaragua, and Venezuela. These examples illustrate that to a significant degree, developing countries are unfortunate in being located in regions that are particularly prone to natural hazards. Of course, this correlation is not entirely accidental. The large number of disasters resulting from this high level of exposure has seriously hindered the ability of these countries to emerge from poverty.

Aside from hazard frequency, severity and exposure, the other contributory factor to disaster risk is capacity. Unsurprisingly, newly industrialised and developing countries both tend to lack the capacity to deal with the threats posed by hazards. This capacity needs to be deployed before the hazard visits a community in the form of pre-disaster planning. Effective mitigation and preparedness can greatly reduce the threat posed by hazards of all types. Likewise, capacity can also be deployed following a major disruptive event. The post-disaster response can impact the loss of life, while timely reconstruction can minimise the broader economic and social damage that may otherwise result.

Although frequently represented as discrete stages, there is also recognition that the same are inter-connected, overlapping and multidimensional (see for example McEntire et al, 2002). In particular: the level and quality of pre-disaster planning will largely determine – positively or negatively – the post-disaster response; and, the effectiveness of post-disaster reconstruction will determine to what extent the community remains vulnerable to the threats posed by hazards in the future. This link between sustainable development and mitigation has been referred to by Mileti (1999) as 'sustainable hazard mitigation.'

With this in mind, although this discussion paper is focused on post-disaster reconstruction, much of what is discussed is intent on ensuring that communities are less vulnerable in the future. The emphasis on reconstruction also recognises that, unfortunately, many communities are left in a perpetual cycle of disasters, as failures in reconstruction efforts prevent them from addressing underlying risk factors.

## 3. Why focus upon the built environment?

As noted in the paper's title, the emphasis of this is discussion is on reconstruction of the 'built environment', but this in no way suggests that reconstruction of the built – or physical – environment should be carried out in a vacuum. Instead, as will be highlighted later, it is vitally important to link the physical requirements with broader social, natural, institutional and economic needs. However, this emphasis does recognise the growing recognition that the construction industry and built environment professions have a significant role to play in contributing to a society's improved resilience to disasters (Haigh et al, 2006; Lloyd Jones, 2006). In order to understand this role, it is necessary to understand what constitutes the 'built environment' and the nature of the stakeholders involved in its creation and maintenance.

The environments with which people interact most directly are often products of human initiated processes. In the 1980s the term built environment emerged as a way of collectively describing these products and processes of human creation. The built environment is traditionally associated with the fields of architecture, building science and building engineering, construction, landscape, surveying, urbanism. In Higher Education, Griffiths (2003) describes, 'a range of practice-oriented subjects concerned with the design, development and management of buildings, spaces and places'.

The importance of the built environment to the society it serves is best demonstrated by its characteristics, of which Bartuska (2007) identifies four that are inter-related. First, it is extensive and provides the context for all human endeavours. More specifically, it is everything humanly created, modified, or constructed, humanly made, arranged, or maintained. Second, it is the creation of human minds and the result of human purposes; it is intended to serve human needs, wants, and values. Third, much of it is created to help us deal with, and to protect us from, the overall environment, to mediate or change this environment for our comfort and well-being. Last, is that every component of the built environment is defined and shaped by context; each and all of the individual elements contribute either positively or negatively to the overall quality of environments.

As previously noted by the Editors (Haigh and Amaratunga, 2010), several important consequences for disaster risk can be identified from these characteristics. The vital role of the built environment in serving human endeavours means that when elements of it are damaged or destroyed, the ability of society to function – economically and socially – is severely disrupted. Disasters have the ability to severely interrupt economic growth and hinder a person's ability to emerge from poverty. The protective characteristics of the built environment offer an important means by which humanity can reduce the risk posed by hazards, thereby preventing a disaster. Conversely, post-disaster, the loss of critical buildings and infrastructure can greatly increase a community's vulnerability to hazards in the future. Finally, the individual and local nature of the built environment, shaped by context, restricts our ability to apply generic solutions.

## 4. Resilience in the built environment

The consequences outlined above serve to underline and support the growing recognition that those responsible for the built environment have a vital role to play in developing societal resilience to disasters. The notion of resilience is becoming a core concept in the social and physical sciences, and also in matters of public policy. But, what does resilience mean? What are the attributes of resilience? What is needed to create a disaster resilient built environment?

The term resilience was introduced into the English language in the early 17th Century from the Latin verb resilire, meaning to rebound or recoil. However, there is little evidence of its use until Thomas Tredgold introduced the term in the early $18^{th}$ Century to describe a property of timber, and to explain why some types of wood were able to accommodate sudden and severe loads without breaking. In 1973, Holling presented the word resilience into the ecological literature as a way of helping to understand the non-linear dynamics observed in ecosystems. Ecological resilience was defined as the amount of disturbance that an ecosystem could withstand without changing self-organised processes and structures.

In subsequent decades, the term resilience has evolved from the disciplines of materials science, the ecology and environmental studies to become a concept used by policy makers, practitioners and academics. During this period, there have been a range of interpretations as to its meaning.

For some, resilience refers to a return to a stable state following a perturbation. This view advocates a single stable state of constancy, efficiency, and predictability, or, as the ability to absorb strain or change with a minimum of disruption (Horne and Orr, 1998; Sutcliffe and Vogus, 2003). For others, resilience recognises the presence of multiple stable states, and hence resilience is the property that mediates transition among these states. This requires very different attributes, as for example advocated by Douglas and Wildavsky (1982), who define resilience from the perspective of risk as, "the capacity to use change to better cope with the unknown: it is learning to bounce back" and emphasise that, "resilience stresses variability". More recently but in a similar vein, Dynes (2003) associates resilience with a sense of emergent behaviour that is improvised and adaptive, while Kendra and Wachtendorf (2003) argue that creativity is vital.

Further discrepancy can be found in the degree to which resilience should be defined in merely passive terms. Douglas and Wildavsky (1982) focus on the ability to simply 'bounce back' from a 'distinctive, discontinuous event that creates vulnerability and requires an unusual response'. Wildavsky (1988) further characterises resilience as the, 'capacity to cope with unanticipated dangers after they have become manifest' and notes that resilience is usually demonstrated after an event or crisis has occurred. Lettieri et al (2009) suggest a 'contraposition' in the literature between two concepts: resilience and resistance. Resilience they argue focuses on after-crisis activities, while resistance focuses on before-crisis activities. These all suggest a reactive approach whereby resilience is considered a 'pattern rather than a prescribed series of steps or activities' (Lengnick-Hall & Beck, 2003). Others stress a positive approach that suggests resilience is more than mere survival; it involves identifying potential risks and taking proactive steps (Longstaff, 2005). The objective is to build resilience by maximising the capacity to adapt to complex situations (Lengnick-Hall & Beck, 2005). Similarly, Paton et al (2001) write of a paradigm shift that accommodates the analysis and facilitation of growth, whereby resilience, 'describes an active process of self-righting, learned resourcefulness and growth'.

Resilience is evidently complex and open to a variety of interpretations but how can it be applied to the built environment? The relationship between disaster risk, resilience and the built environment suggests that a resilient built environment will occur when we *design, develop and manage context sensitive buildings, spaces and places that have the capacity to resist or change in order to reduce hazard vulnerability, and enable society to continue functioning, economically and socially, when subjected to a hazard event*. It is possible to elaborate on this definition by exploring specific characteristics of resilience and how they may be present in the built environment.

Firstly, resilience is seen as the ability to accommodate abnormal or periodic threats and disruptive events, be they terrorist actions, the results of climatic change, earthquakes and floods, or an industrial accident. Identifying, assessing and communicating the risk from such threats and events are therefore vital components. Individuals, communities, organisations and, indeed, nations which are prepared and ready for an abnormal event, tend to be more resilient. Consequently, those responsible for the planning, design and management of the built environment need to understand the diverse hazard threats to buildings, spaces and places and the performance of the same if a disruptive event materialises.

The next characteristic is the ability to absorb or withstand the disturbance while still retaining essentially the same function. This may mean returning to the state or condition that existed before the disturbance occurred, or returning to an improved state or condition. This absorption might be realised through the specification and use of hazard resistant methods, materials and technologies. It might also result from the construction of protective infrastructure, or the protection of critical infrastructure. Such measures may resist the threat, or at least reduce the losses experienced.

As outlined in the opening of this discussion, we live in a world which is constantly evolving, in some cases through natural processes and in other cases through the intervention of mankind. There is common agreement in the literature that systems, organisations and people who are able and willing to adapt tend to be more resilient. Creative solutions, the ability to improvise and the capacity to adapt will be essential in order to address the challenges posed by what is often seen as an unbounded threat.

The ability and willingness to learn is often linked to adaptability and being prepared. The learning may come from studying the lessons of others in a formal manner: by gathering and evaluating data, by conducting research in an objective, independent and balanced manner, and by communicating the findings, conclusions and recommendations.

The ability to absorb or withstand also requires economic and human capacity. A resilient built environment will need to be supported by a strong domestic industry and appropriately skilled professions and trades. A well-developed construction sector and supply chain, which largely comprise of micro, small and medium sized enterprises, provide a strong means to counter the economic shocks that frequently accompany other disasters, while also offering an economic stimulus and livelihood opportunity in the recovery period.

As society becomes more complex, resilient communities tend to be those which are well coordinated and share common values and beliefs. This sense of interconnectedness can be undermined by self-interest and personal gain, resulting in vulnerable societies which are less able and willing to plan for, and react to, disruptive events. Understanding the link between the physical and social environment will be vital in developing connectedness. Culturally sensitive, sustainable and socially responsible planning, design and management of the built environment, have the potential to help develop community cohesion and thus contribute to wider societal resilience.

From this discussion of its characteristics, it is evident that the concept of resilience provides a useful framework of analysis and understanding on how we can plan, design and maintain a built environment that copes in a changing world, facing many uncertainties and challenges. Sometimes change is gradual and things move forward in continuous and predictable ways; but sometimes change is sudden, disorganising and turbulent. Resilience provides better understanding on how society should respond to disruptive events and accommodate change.

## 5. Disasters as a window of opportunity

If this idea of a resilient built environment is appealing, how can it be achieved? A further reason for this discussion’s emphasis on reconstruction is that the post-disaster period provides a window of opportunity to address many of the vulnerabilities usually encountered in a community’s built environment. There are several features of this post-disaster period that can be capitalised upon. Firstly, the disaster has destroyed much of the built environment that was improperly designed and vulnerable, creating a fresh start from which to address disaster risk. Furthermore, the experience gained during the disaster typically generates new knowledge, which brings various stakeholders together around a shared awareness of the nature of risk. The mistakes of previous development policies and strategies are exposed and can be addressed. Next and perhaps even more significantly, the political will and desire to act is almost certainly stronger than usual. Any interest in disaster risk reduction that had been forgotten or side-lined before the disaster, will suddenly gain renewed prominence in the recovery period. In a similar vein, the lack of resourcing for risk reduction, any presence of corruption and otherwise weak institutional structures that allowed a vulnerable built environment to be constructed will have been highlighted. Finally, but perhaps most importantly, the post-disaster period often provides a level of resourcing, including considerable external funding, that would be otherwise unattainable. If properly utilised – something that is by no means certain – this additional resource does afford a major opportunity to reduce vulnerability.

The fact that this window of opportunity exists does not mean that the various actors involved in reconstruction will take advantage of it. Although many, if not all, of these features are usually present following a major disaster, even a cursory glance at the countless studies and evaluations of programming after disasters, provides evidence that it is frequently a missed opportunity.

There are a myriad of reasons as to why these failures occur. Humanitarian principles are primarily concerned with addressing acute human suffering. By necessity, a timely response is essential.

Anything that slows this response is likely to be a problem. Unfortunately, the well-planned reconstruction of a more resilient built environment will take time. Likewise, humanitarian principles also tend to dictate maintaining independence, neutrality and impartiality. This can dissuade actors from highlighting previous failings, which would otherwise create the necessary political will for change.

Effective reconstruction of the built environment is also competing with many other priorities. Poverty alleviation, improved health, and good governance are a few of the many goals usually mainstreamed in the post-disaster recovery period. A more resilient built environment can certainly contribute to these goals, but there will inevitably be a time-lag; other recovery programmes can sometimes appear more appealing due to their ability to deliver short term results. If the window of opportunity is to be taken advantage of, then advocates of a more resilient built environment will need to demonstrate the vital role it plays in helping society achieve much broader development goals.

A further complication is the natural tension between the need for timely reconstruction and a desire to utilise and where necessary develop local capacity. Institutions and local enterprise to plan and construct the built environment may matter, but they are often simply not there. Government, both national and local, is usually called upon to make critical long term planning decisions, and to develop and enforce appropriate building regulations. This expectation is made of institutions that have usually failed to achieve this in far less challenging periods of their electorate's history. The reality is that large scale reconstruction may have to be undertaken during a period soon after a major part of the civil service has perished, or at least been severely disrupted. At a time when even greater demands are being made of the civil service, its employees are sometimes being laid off, with the damage to the local tax base reducing available funding. At the same time, the local construction industry is suddenly called upon to increase its output to meet the needs of an unprecedented programme of reconstruction activity, while simultaneously familiarising itself with less vulnerable methods and materials. Building human resources and local capacity to address these shortfalls and support reconstruction, may take years.

The alternative, to make use of international agencies and private enterprises, understandably raises other concerns. International actors are often accused of poaching the most talented local civil servants and encroaching on a country's independence, while the private sector is accused of disaster profiteering and leaves local industry unable to 'benefit' from the economic opportunities afforded by the disaster.

In summary, there is a window of opportunity, but it is beset with challenges. A pragmatic approach to the development of a resilient built environment needs to include an understanding of these difficulties and their implications for what can actually be done, at least in the short term. While the humanitarian efforts are frequently a rushed process, effective rebuilding for resilience will require reflection, discussion and consensus building. This should not undermine the importance of starting this process early in the recovery phase; indeed, a failure to consider long term reconstruction goals early in the recovery can lead to wasted or misguided effort, as well as undermine efforts for future resilience. Instead, it recognises the importance of a judicious approach that addresses the complexity of creating resilience.

## 6. Asset-based reconstruction

The consequences outlined here serve to underline and support the growing recognition that those responsible for the built environment have a vital role to play in effective disaster planning. It would also appear to be highly desirable for the built environment discipline to be able to contribute to increased resilience through a strategy that is inter-disciplinary in nature. Thus far, the emphasis of this discussion has been on reconstruction of the 'built environment', but as asserted earlier, reconstruction to address a community's physical requirements must be done so in a manner that considers broader social, natural, institutional and economic needs.

The built environment industries are usually associated with a range of critical activities in post-disaster recovery, including temporary shelter and housing after the disaster, and the restoration of critical infrastructure such as hospitals, schools, water supply, power, and communications. However, in order to achieve the challenge laid out earlier – to create a more resilient built environment that can contribute to broader societal resilience – the impact of reconstruction, positively or negatively, needs to be evaluated far more carefully. Disaster planners have begun to realise the link between disaster and development – a large and well-established field relating to physical, social, natural and economic aspects of society. Although reconstruction of the built environment by itself will not eliminate the broad ranging consequences of disasters, there is increasing recognition that the reconstruction process can contribute to the development of communities beyond merely the building of their physical environment.

This potential contribution of the reconstruction process to the broader goal of a more resilient society can be viewed with the aid of the Asset-Based Community Development (ABCD) approach, developed by Kretzmann and McKnight (1993) as a methodology that seeks to uncover and highlight the strengths within communities as a means for sustainable development. The basic tenet is that a capacities-focused approach is more likely to empower a community and therefore mobilise citizens to create positive and meaningful change from within. Instead of focusing on a community's needs, deficiencies and problems, the ABCD approach helps them become stronger and more self-reliant by discovering, mapping and mobilising all their local assets. Few people realise how many assets any community has. The reconstruction process has the potential to utilise and impact, positively or negatively, a community’s assets.

The construction and maintenance of a community’s infrastructure and buildings, or physical assets, are the first obvious impact. These physical assets address material needs (infrastructure, water, housing, waste, energy, transport, work), social and educational needs (schools, play areas, meeting places), and, spiritual or cultural needs (places of worship). Reconstruction of the physical environment is vital to secure sustainable economic development. Further, by incorporating appropriate mitigation measures, effective reconstruction of the physical environment is also an opportunity to reduce the community’s vulnerability to hazards in the future.

Of vital importance will be to secure sufficient capacity or resources to deliver all this reconstruction activity. The challenge posed by the scale of reconstruction can also be viewed as an opportunity: to develop livelihood competencies or human assets in construction related trades. This human asset development is not just required at the trade level; project management and professional skills are also vital. This opportunity can help address a problem frequently encountered following a disaster, particularly in conflict affected environment: how to develop the skills of displaced peoples and ex-combatants who, for a variety of reasons, are unable to return to their original livelihoods?

Reconstruction also enables the development of economic assets within the community through opportunities to initiate market linkages in the construction supply chain. Excessive reliance on external private enterprises can be counterproductive and hinder local economic development. Local intermediary and long term income generation opportunities provided by reconstruction activity may lay an important platform for economic development in the region. Micro, small and medium enterprises are a vital component of any economy and the construction sector is largely comprised of micro and small enterprises. Reconstruction is thus an opportunity to provide market access for these local enterprises.

A community’s natural assets are frequently impacted by reconstruction. On the positive side, locally sourced and contextually appropriate materials can provide an important contribution to reconstruction while also offering market opportunities for local businesses. It is however vital to consider the community’s long term sustainability and thereby ensure that its natural assets are not exploited to the detriment of the community.

Fundamental to the recovery of any disaster affected community is the idea of connectedness. There is growing evidence that collective reconstruction contributes to social cohesion and builds social assets. Reconstruction is an opportunity for cooperation and working across diverse groups, particularly useful in conflict affected environments. Engaging the community in reconstruction has the added benefit of moving them away from being passive recipients of aid, which can increase the sense of helplessness.

Finally, reconstruction can impact a community’s institutional assets. The organisation and coordination of recovery is usually complex because a wide range of activities occur simultaneously with an equally wide range of needs that have to be met, including those of the most vulnerable members of the community. Reconstruction can provide members of a community with an opportunity to influence policies, decisions and interventions that affect them, including assessment, planning, construction and monitoring. Further, the community can develop links to important stakeholders.

## 7. Conclusion

In summary, the reconstruction process will have a far greater impact on the affected community than the physical buildings and infrastructure. An asset-based approach does not remove the need for outside resources. However it will make their use more effective. It will also go a long way to creating the type of resilient society that was put forward earlier as a goal to aspire to. Indeed many of the characteristics of resilience can be developed through an assets based approach. Asset based community development starts with what is present in the community. It concentrates on the agenda-building and problem-solving capacity of the residents and stresses local determination, investment, creativity, and control. Weak communities are places that fail to mobilise the skills, capacities and talents of their residents or members. Ignoring a community's assets during reconstruction may inadvertently leave communities more dependent and ultimately less resilient to the threat posed by future hazards. In contrast, post-disaster reconstruction programmes where the capacities of the community are identified, valued and used, will lay the platform for a more resilient society.

### References

Bartuska, T. (2007) The Built Environment: Definition and Scope, in Bartuska T., and Young, G. (Eds) The Built Environment: A Creative Inquiry into Design and Planning, Crisp Publications.

Douglas, M. and Wildavsky, A. B. (1982) Risk and Culture: An essay on the selection of technical and environmental dangers, Berkeley: University of California Press.

Dynes, R. (2003) Finding order in disorder: continuities in the 9-11 response, International Journal of Mass Emergencies and Disasters, 21(30), 9-23, Research Committee on Disasters, International Sociological Association.

Griffiths, R. (2004) Knowledge production and the research-teaching nexus: the case of the built environment disciplines. Studies in Higher Education, 29(6), 709-726.

Haigh, R., Amaratunga, D., Keraminiyage, K. (2006) An Exploration of the Construction Industry's Role in Disaster Preparedness, Response and Recovery, Proceedings of the Annual International Research Conference of the Royal Institution of Chartered Surveyors (RICS COBRA 2006), The RICS and The Bartlett School, University College London.

Haigh, R. and Amaratunga, D. (2010) An integrative review of the built environment discipline's role in the development of society's resilience to disasters. International Journal of Disaster Resilience in the Built Environment, 1(1), 11-24.

Holling, C. (1973) Resilience and stability of ecological systems. Annual Review Of Ecology And Systematics, 4:1-23.

Horne III, J. and Orr, J. (1998) Assessing Behaviors that Create Resilient Organizations', Employment Relations Today, Volume 24, Number 4, pp. 29–39.

Kendra, J. and Wachtendorf, T. (2003) Creativity in Emergency response to the World Trade Center Disaster in Beyond September 11th: An Account of Post-Disaster Research.

Kelman, I. (2009) Natural Disasters Do Not Exist (Natural Hazards Do Not Exist Either) Version 2, 10 September 2009 (Version 1 was 26 July 2007). Downloaded from http://www.ilankelman.org/miscellany/NaturalDisasters.rtf

Kretzman, J., and McKnight, J. L. (1993). Building Communities from the Inside Out: Centre for Urban Affairs and Policy Research. Chicago: ACTA Publications.

Lengnick-Hall, C. and Beck, T. (2003) Beyond Bouncing Back: The Concept of Organizational Resilience. Paper presented at the Academy of Management, Seattle, WA, 1–6 August.

Lengnick-Hall, C. and Beck, T. (2005) Adaptive Fit Versus Robust Transformation: How Organizations Respond to Environmental Change, Journal of Management, Volume 31, Number 5, pp. 738–757.

Lettieri, E., Masella, C. and Radaelli, G. (2009) Disaster management: findings from a systematic review, Disaster Prevention and Management, Vol. 18 No. 2, pp. 117-136.

Lloyd-Jones, T. (2006) Mind the Gap! Post-disaster Reconstruction and the Transition from Humanitarian Relief, RICS, London.

Longstaff, P. (2005) Security, Resilience, and Communication in Unpredictable Environments Such as Terrorism, Natural Disasters and Complex Technology, Harvard University, Cambridge, MA.

McEntire, D., Fuller, C., Johnston, C. and Weber, R. (2002) A comparison of disaster paradigms: the search for a holistic policy guide, Public Administration Review, Vol. 62 No. 3, pp. 267-81.

Mileti, D. (1999) Disasters by Design: A Reassessment of Natural Hazards in the United States. Washington DC: Joseph Henry Press.

O'Keefe, P., Westgate, K., Wisner, B. (1976) Taking the naturalness out of natural disasters. Nature, vol. 260, pp. 566-567.

Paton, D., Johnston, D., Smith, L., Millar, M. (2001), "Community response to hazard effects: promoting resilience and adjustment adoption", Australian Journal of Emergency Management, Vol. 16 pp.47-52.

Sutcliffe, K. and Vogus, T. (2003) Organizing for Resilience, in: Cameron, K. (ed.), Positive Organizational Scholarship, Berrett-Koehler Publishers Inc., San Francisco, CA, pp. 94–110.

Swiss Reinsurance Company (2000) Natural catastrophes and man-made disasters in 1999: Storms and earthquakes lead to the second–highest losses in insurance history, Sigma, Zurich.

Swiss Reinsurance Company (2010) Natural and man-made catastrophes in 2009, Sigma, Zurich.

Wildavsky, A. (1988) Searching for Safety, New Brunswick, CT: Transaction Books.

# STABILISED SOILS FOR STRUCTURAL APPLICATIONS AND SOME ISSUES ON SUSTAINABLE CONSTRUCTIONS

B. V. Venkatarama Reddy[1]


## ABSTRACT

Construction industry is the largest producer of materials when compared with any other industrial sector. Currently the annual production of cement is about 3 billion tonnes and burnt clay bricks are about 3.5 billion tonnes. Consumption of every tonne of cement requires 5 – 6 tonnes of aggregates and therefore 15 – 20 billion tonnes of aggregates are consumed annually. Therefore, Manufacturing and use of construction materials necessitates consumption of raw material resources and energy. Raw materials are mined from the earth and energy is expended to convert these raw materials into construction products. The consequences are depletion of the raw material resources due to mining. Any activity related with mining is unsustainable. Expenditure of energy causes green house gas (GHG) emissions. Thus construction sector has two problems to address: (1) unsustainable mining of limited raw material resources and (2) GHG emissions. It has been assessed that the built environment alone consumes 30% of raw materials extracted and 40% energy resources. The built environment is responsible for 40% of GHG emissions and 30% of solid waste generation. Majority of the arguments proposed regarding sustainability of construction sector in general and the built environment in particular address the issue of pollution and GHG emission reductions. Without addressing the issue of depleting material resources due to mining there is no meaning in talking about sustainability of construction sector. This presentation will discuss some real issues of sustainability with reference to construction sector and the technology of stabilised soil products (low energy and sustainable products) for structural applications like walls and other building components.

Concepts of sustainability of construction sector particularly with reference to the mining of material resources and energy represent the main focus of the presentation. Energy, emissions and life cycle of some conventional materials are discussed. Case studies of zero carbon foot print vernacular structures and the problems associated with rating systems are illustrated. Broad guidelines on achieving sustainability construction sector are proposed.

Potential of earth based low embodied carbon building products for structural applications in buildings has been illustrated with some examples. Loss of strength on saturation and rain erosion are the two major disadvantages of pure soil based constructions. Hence, there is a need for stabilised soil products for structural components of buildings. Examples of centuries old earthen structures especially multi-storey residential structures are pictorially illustrated. Surge of recent interests in reviving earthen architecture for dwellings with case studies are shown.

Principles soil stabilisation as adopted for the production of stabilised soil blocks and stabilised rammed earth elements would be dealt in greater detail. Density, strength and moisture relationships and their importance in devising good quality stabilised soil blocks and stabilised rammed earth walls forms the main theme of discussions on stabilised soil products for structural applications. Discussions on embodied carbon in stabilised soil products, retrieving clay minerals from such products, recycling and end of life considerations of such products forms the main scientific analysis. The presentation leads to the emergence of some useful guidelines on stabilised earth construction as applicable to sustainable constructions.


[1] Professor, Department of Civil Engineering, Indian Institute of Science, Bangalore 560012, INDIA
E-mail: venkat@civil.iisc.ernet.in

# NANOBEAM MECHANICS: THEORY AND EXPERIMENTAL COMPARISON

N. Rajapakse
Faculty of Applied Sciences, Simon Fraser University, Burnaby, Canada, V5A 1S6
rajapakse@sfu.ca

## ABSTRACT

Various experimental techniques have been used to determine the mechanical properties at the nanoscale, namely, bending tests, nanoindentation tests, resonant excitation tests, etc. [1-3]. Bending of nanowires using an atomic force microscope (AFM) is one of the most popular testing techniques for nanomaterial characterization. Wong et al. [1] performed AFM bending tests to directly measure the force-displacement relation and determined the mechanical properties of cantilever SiC beams by using conventional beam theory. Jing et al. [3] determined the elastic modulus of silver nanowires with diameters ranging from 20 to 140 nm by performing three-point bending tests on suspended nanowires. They found that the apparent Young's modulus of silver nanowires increased significantly with decreasing wire diameter.

Nanobeams are also key components of nanomechanical and nanoelectromechanical systems (NEMS) which are essentially sensors, actuators, machines and electronics at the nanoscale [4]. These devices can be used to measure extremely small displacements and forces that can lead to novel applications in engineering, advanced materials, medicine, computers, communications, etc. Current material processing technology allows for fabrication of NEMS of a few nanometers. The static and dynamic behavior of nanobeams is the principal feature that is exploited in the functional design of NEMS. Beams encountered in microelectromechanical systems (MEMS) are normally analyzed using the classical beam theory as classical theories are applicable at the microscale. However, it has been reported that at the nanoscale the response of beams is size-dependent and the conventional beam theory need to be modified [1-3].

To understand and predict the behavior of nanoscale structures, various modeling approaches have been proposed including atomistic simulation methods. The reason for the size-dependent behaviour at nanoscale is that the fraction of energy stored in surfaces becomes comparable with that in bulk due to the relatively high ratio of surface area to volume of nanoscale structures. Excess energy associated with surface/interface atoms is called surface/interfacial free energy. The ratio of surface free energy $\gamma(J/m^2)$ and Young's modulus $E$ $(J/m^3)$, $\gamma/E$, is dimensional $(m)$ and points to some other inherent parameter of a material [5]. This intrinsic length scale is usually small, in the nanometer range or even smaller. When a material element has one characteristic length comparable to the intrinsic scale, the surface/interface free energy can play an important role in its properties and behaviour. A direct method for analysis of nanoscale structures is to apply atomistic simulations but prohibitive computing cost makes it impractical.

This study is motivated by the need to develop a suitable mathematical model to understand the complex size-dependent behavior of nanobeams observed in experiments [1-3, 6, 7] and the need for a simple simulation tool to analyze beams in NEMS and other nanoscale devices. The classical beam theory widely used to analyze nanobeams does not account for important effects at the nanoscale such as surface energy. Gurtin and Murdoch [8, 9] presented a mathematical model that incorporates the effects of surface and interfacial energy into continuum mechanics.

A mechanistic model based on the Gurtin-Murdoch theory is first presented to analyze thin and thick nanoscale beams with an arbitrary cross-section. The main contribution of the first part of this study are a set of analytical solutions for static response of thin and thick beams under different loading (point and uniformly distributed loading) and boundary conditions (simply-supported, cantilevered and both ends fixed), and the solution of free vibration characteristics of such beams. Complete details of the analytical solution including the formulation of a new beam theory are given elsewhere [10] and its finite element formulation can be found in Ref. [11]. In the second part of this

study, a model for large deflections of thin beams is developed based on the Gurtin-Murdoch continuum theory and applied to examine the experimental results of Nilsson et al [7] and the classical large deflection beam model used by Søndergaard et al. [12]. The formulation of non-linear beam theory and solution algorithm is discussed. It is shown for the first time that good agreement with experiments can be obtained by using size-independent mechanical properties such as the bulk elastic modulus and surface residual stress. The model is then applied to show the influence of end boundary conditions and surface residual stress and resulting softening/stiffening effects. The present study shows that solutions obtained from the classical beam theory require careful interpretation when applied to nanobeams and generalization of nanomaterial behavior on the basis of classical beam models could lead to questionable conclusions.

**Acknowledgement:** The work presented in this paper was supported by a grant from the Natural Sciences and Engineering Research Council of Canada.

## References

[1] Wong E W, Sheehan P E and Lieber C 1997 Science **277** 1971–5.

[2] Cuenot S, et al. 2004 Phys. Rev. B. **69** 165410.

[3] Jing G Y, et al. 2006 Phys. Rev. B. **73** 235409.

[4] Craighead H G 2000 Science **290** 1532.

[5] Yakobson B I 2003 Handbook of Nanoscience, Engineering and Technology CRC Press.

[6] Nilsson S G, Sarwe E L and Montelius L 2003 Appl. Phys. Lett. **83** 990.

[7] Nilsson S G, Borrise X and Montelius L 2004 Appl. Phys. Lett. **85** 3555.

[8] Gurtin M E and Murdoch A I 1975 Arch. Rat. Mech. Anal. **57** 291.

[9] Gurtin M E and Murdoch A I 1978 Int. J. Solids Struct. **14** 431.

[10] Liu C and Rajapakse R K N D 2010 Trans. IEEE Nanotechnology **9** 422.

[11] Liu C, Rajapakse, R K N D and Phani, S 2010 J. Applied Mechanics ASME (in press).

[12] Søndergaard N, Nilsson S G, Guhr T and Montelius L 2007 Nanotechnology **118** 255502.

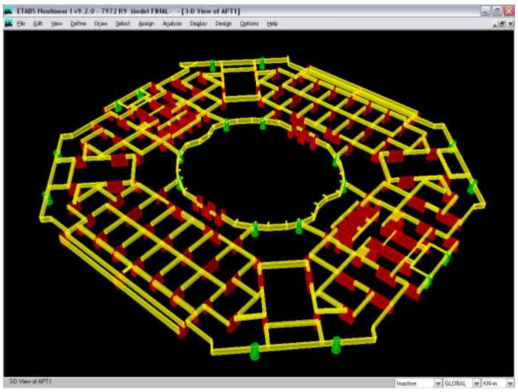
# PALAIS ROYALE, A TREND SETTER

Mr. GIRISH DRAVID
B.Tech.(Civil) 1981, M.Tech.(Structures) 1983, IIT Bombay
Director : Sterling Engineering Consultancy Services Pvt. Ltd., Mumbai, India.

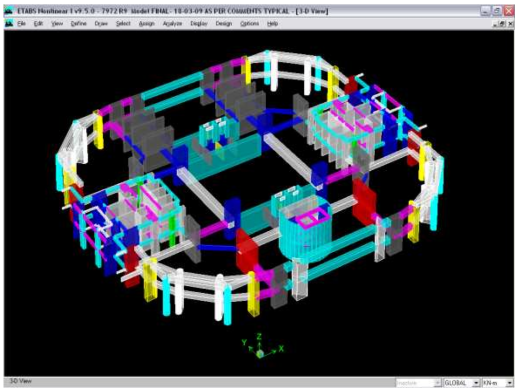
## Introduction:
Palais Royale (pronounced as pa-lai ro-yaal) is a French phrase which means a Royal Palace.

## Location:
Situated in the heart of south Mumbai, Worli Naka, the building with a height of 295 m above the ground level has a total structural height of 325 m from the bottom of the foundation to the top of the elevation cap. The base dimensions of the octagonal prismatic building are 84m x 86 m. The construction area of the building is over three million sq.ft. with 88 slabs.

## Structural Systems:
The residential levels have been provided with a conventional column / beam and solid slab configuration. Presence of an atrium in the centre as a Brahmasthan requirement has provided the structural advantage of the supports being on the periphery.

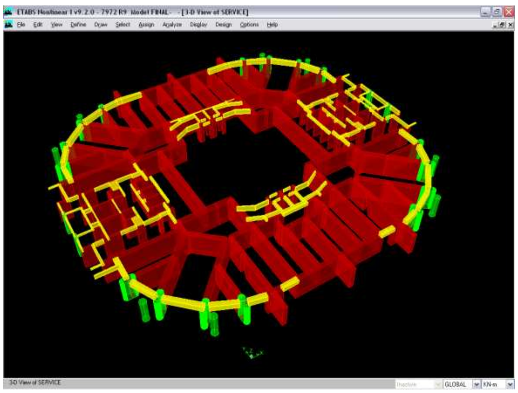


APARTMENT LEVEL FRAMING

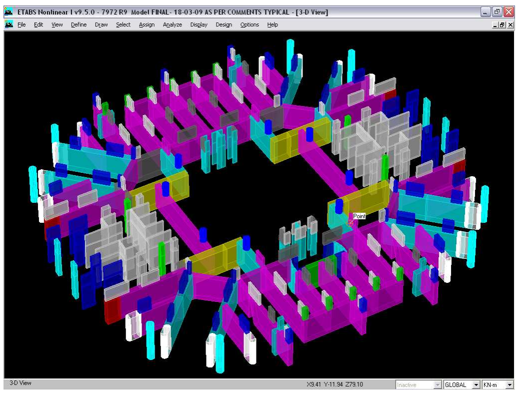


AMENITY LEVEL FRAMING

## Transfer Level:
In order to transfer the loads of the 244 residential columns to the foundation through the 88 lower columns, transfer girders are provided at +76 m level. The depth of these RCC girders is 9 m and the widths are varying from 1200 mm to 1500 mm as per the design and bearing requirements.



TRANSFER GIRDER FRAMING



TRANSFER LEVEL

## The Parking and Amenity Levels:
Structural system below transfer girder level comprises predominantly of Post Tensioned Flat Slab except in the Brahmasthan where the 25 m x 22 m rectangular area is framed by strong posttensioned beams. The amenity areas carry huge loads, the average intensity of the superimposed loads (SDL +

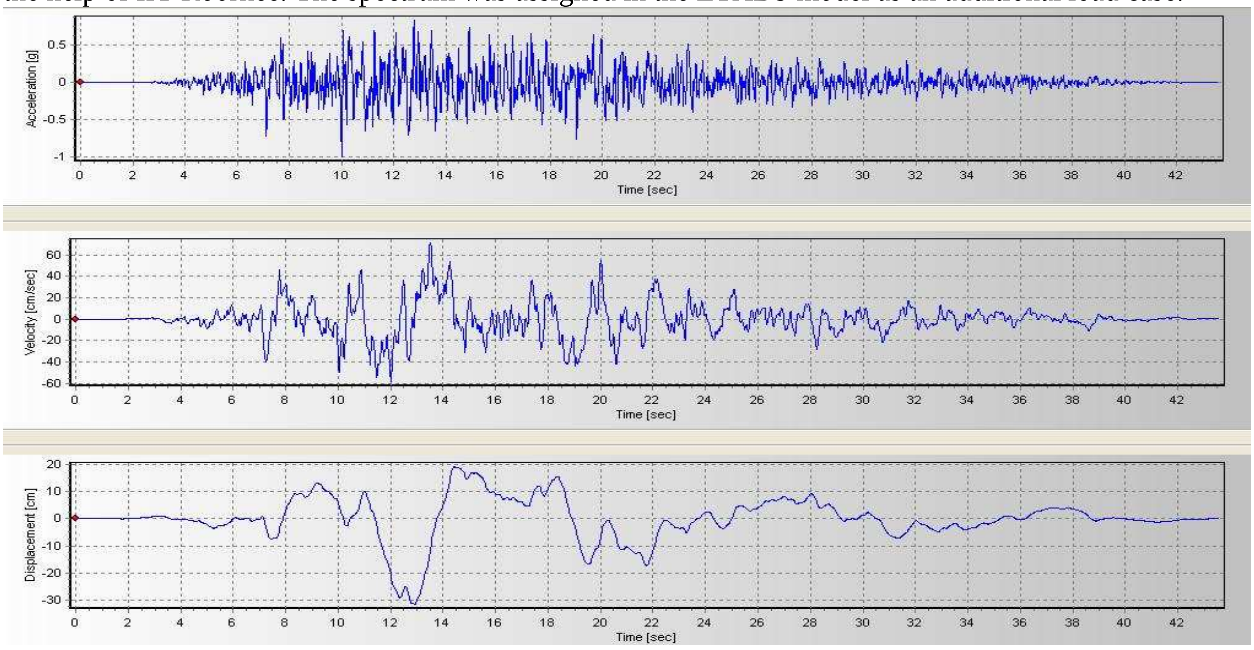
LL) being as high as 35 kN/sq.m at the swimming pool level. The parking levels are designed for the possibility of double stacked parking. Seismic design is corresponding to Earthquake Zone III aimed at operational level performance – maximum allowed drift ratio being 1000 and acceleration in wind limited to 5 milli-g.

## Wind tunnel tests:

Wind tunnel tests by RWDI showed that for a 10 year return period the Total Peak Acceleration in simulated wind conditions was 7.2 milli-g against the criteria of ISO 10137:2007 of 14 milli-g (extrapolated from the criteria for 1 to 5 year return period to 10 year return period).

## Site Specific Seismic Studies :

For the first time in India, site specific seismic studies were conducted for a residential building, with the help of IIT Roorkee. The spectrum was assigned in the ETABS model as an additional load case.



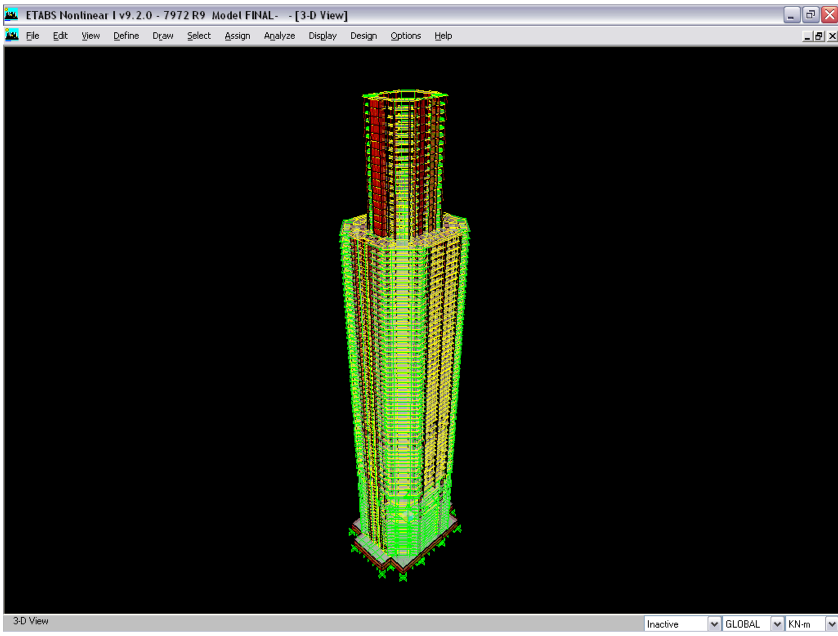

**SITE SPECIFIC TIME HISTORY SCALED TO UNITY**

## Soil Profile:

Soil consultants estimated a safe bearing pressure of 150 T/sq.m with settlement less than 25 mm. Modulus of subgrade reaction of 6500 T/m3 was recommended for the design of raft foundations. Cross-hole velocity test yielded average values of Poisson's Ratio, Young's Modulus and Shear Modulus as 0.32, 5200MPa and 200 MPa respectively showing excellent characteristics of the rock profile. Due to presence of weak soil for the upper 8 to 9 m, soil retention system was erected in the form of contiguous concrete infilled tubular steel piles, held to the bedrock with inclined pre-stressed rock anchors.

## Computer Model:

A combination of shell diaphragm and membrane diaphragm was chosen to simulate framing conditions of the structure, in order to optimize the run time and the computer memory. The flat slabs at the parking and amenity levels have been treated as shell elements contributing to the lateral stiffness.




**ETABS MODEL** **PERSPECTIVE OF THE BUILDING**

At the residential levels, the lateral resistance is derived from the beam/column frame action. Hence, the diaphragm is modeled as a membrane. Cracked section properties were assigned in accordance with the code recommendations.

The foundation raft was analyzed using SAFE, using the reactions obtained from the Etabs Analysis.

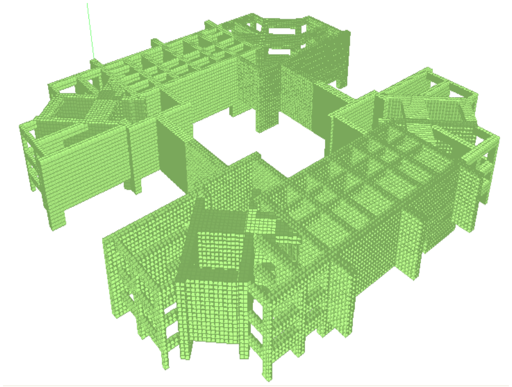
## Analysis Results:

Considering the overall maximum lateral deflection of the building being only 300 mm occurring at the top level, the general performance of the building is well controlled. The massive proportions and the enormous stiffness of the building are evident from the modal frequencies found to be 0.1206 Hz for Mode 1 in Y direction, 0.1349 Hz for Mode 2 primarily in X direction and Mode 3 showing 0.1515 Hz primarily in Z direction as torsion. Fortunately, significant differential elastic shortening of columns and shear walls due to vertical loads was not observed.

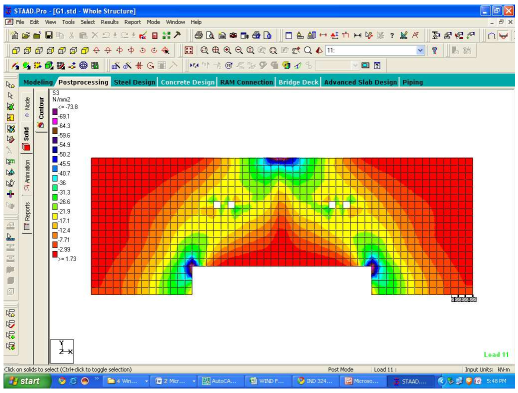
**Salient Aspects of Seismic and Wind Design**

**1. Extensive reference to international guidelines:**
- **(a) CTBUH guidelines for seismic design of tall buildings (2008)**
- **(b) Los Angeles Tall Buildings Structural Design Council guidelines for tall buildings (2008)**
- **(c) Pacific Earthquake Engineering Research Centre – seismic performance objectives for tall buildings (2008)**

**2. Generation of site specific response spectra and time-histories (undertaken for the first time for a civil application in India).**
**3. Palais Royale being treated as a Special Structure as defined by IS-1893 (2002).**
**4. Minimum design base shear scaled to 1 % of the seismic weight.**
**5. Intrinsic damping for seismic & wind design = 1%**
**6. Structural elements modeled using cracked section properties.**
**7. Importance factor of 1.5 used.**
**8. Seismic deflections under DBE controlled to H/750.**
**9. Wind accelerations under 10 year return period wind pegged at 10 milli-g**

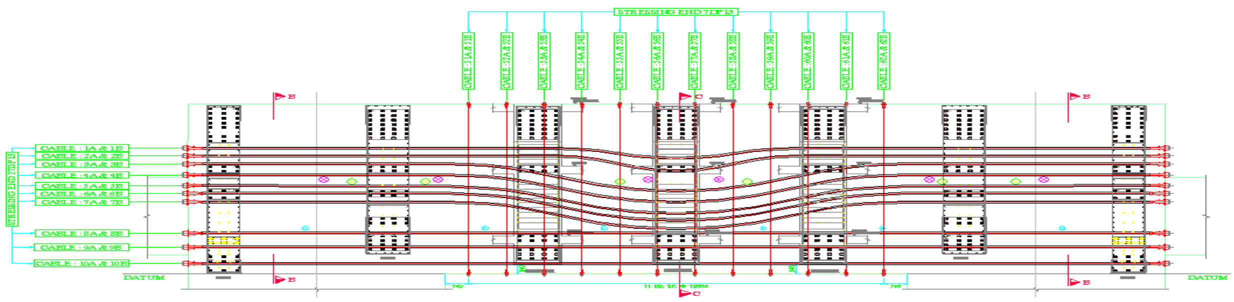


SOLID FINITE ELEMENT MODEL IN STAAD



STRUT-TIE ACTION : STESS PATTERN

The transfer girders were analyzed by Solid Finite Element Method. Both individual girder models and integrated layout model involving all girders and the three floor levels within the girder depth were assembled in STAAD-Pro. The stress patterns clearly indicated that the girders acted in conformity with strut-tie model corresponding to deep beam action. In-depth research was carried out to design the girders, which are probably the largest transfer girders being constructed in the world. The transfer girders have been provided with horizontal and vertical post tensioning to achieve monolithic behavior and deflection control. The post-tensioning is carried out stagewise to avoid excessive upward deflections in early stages of construction.



POST TENSIONING OF TRANSFER GIRDERS

## Concrete Information:

M:80 concrete for columns and shear walls and M:60 concrete have been used for slabs and beams. With the help of an elite team of concrete experts, concrete manufacturers, admixture vendors, contractor's engineers and batching plant operators, innumerable trial mixes were tested for various performance criteria. Eventually, M:80 SCC was finalized with free water cement ratio of 0.225 and free water binder ratio of 0.23. With 450 kg cement content and 168 kg/cu.m fly ash, the target strength was 90 N/sq.mm. Micro silica content was tried starting from 0% and was varied up to 10% to examine the performance. The design was finalized with 5% i.e. 23 kg/cu.m micro-silica content. Minor adjustments are carried out for aggregate quality variation and moisture content on a routine basis.

## Construction Methodology:

M:80 concrete, use of self compacting concrete, using surface retarders, introduction of retarded concrete to avoid cold joints, column cages, compulsory use of couplers for rebar splicing, Automatic Climbing System for Walls and cores etc. are some of the salient aspects of the construction method suggested by Sterling.

## Mock ups:

A practice of setting up true scale mock-ups to study the veracity of the systems was adopted on this project. For example, two mock foundation blocks were cast with 3.5 m depth with reinforcing bars as per the actual design requirement, and were cast, cured and monitored for formwork system, feasibility of using SCC, temperature variation, thermocouple working, segregation characteristics, characteristic strength curve and Evalue. These mock-ups were tested two months in advance before commencing the actual foundation concreting. Similarly, bottom chord of 2 m depth of the overall 9 m deep transfer girder with all the rebars and other embedments was cast, which proved to be enormously helpful in understanding the complexities of rebar placement and in adopting suitable methodology prior to actual work at the 76 m level.

### Present Status:

The construction has now reached 125 m, six typical apartment floors above the transfer girder level.

### List of consultants

| | |
|---|---|
| Architects | Talati and Panthaky Associates, INDIA |
| Structural Engineers | Sterling Engineering Consultancy Services Pvt.Ltd., INDIA |
| Structural Peer Reviewers | CBM Engineers, USA |
| Damper Consultants | Taylor Devices India, USA |
| Geotechnical Consultant | Mr. Jaydeep Wagh |
| MEP Services Consultants | Pankaj Dharkar Associates, INDIA |
| MEP Peer Reviewers | LEHR Consultants International, USA |
| Concrete Technologists | Dr. S.A.Reddi, S.G.Bapat |
| Project Management | Dongre Associates |
| Wind Tunnel Testing | RWDI, Canada |
| Post Tensioning agency | BBR India, Bangalore, India |
| Design Management | Anand Palaye Architects Colasce |
| Acoustics | Smita Chogle |
| Landscaping | Kishore Pradhan |
| Firefighting | SSN Corporation |
| Solar Energy | Entegra Infrastructure Ltd. |
| Lift Consultants | LBA Consultants Pvt.Ltd. |
| Façade Consultants | Fusion Cladding |

### Executing agencies

| | |
|---|---|
| Principal Contractors | Raghuveer Infrastructure |
| Concrete Production | Constromat Consultancy Services |
| Micro silica | Elkem India |
| Reinforcement Fabrication | Ready Made Steel |
| Formwork | MEVA, Germany / Pranav Constructions Limited, India |
| Couplers | Dextra India |
| Reinforcement | Tata, SAIL |
| Concrete Embedments | Halfen-Deha |
| Waterproofing | Nina Concrete Systems Pvt. Ltd. |
| Cladding Material | Du-Pont |

PALAIS ROYALE

# A RELIABILITY BASED APPROACH FOR SUSTAINABLE MANAGEMENT OF PUBLIC BUILDINGS

S.Setunge[1] and K.Zhang[2]

[1]Associate Professor Sujeeva Setunge, Discipline Head, Environmental Engineering,
RMIT University, Melbourne, Australia
[1]E-mail: sujeeva.setunge@rmit.edu.au
[1]Telephone: +61 3 9925 2182 ; Fax: + 61 3 9639 0138

[2]Senior Lecturer, Civil Engineering, RMIT University, Melbourne, Australia
[2]E-mail: Kevin.zhang@rmit.edu.au
[2]Telephone: +61 3 9925 3824: Fax: + 61 3 9639 0138

**Abstract:** Management of aging community buildings is a major cost to many local government organizations in Australia. One of the major challenges is integrating physical or engineering condition ratings with the sustainability issues and community service driven parameters. A new research project continuing at RMIT University in Melbourne is exploring an innovative reliability based approach for deterioration prediction and decision making for sustainable management of community buildings.

The paper will present the practices adopted in management of community buildings by six local government agencies in Australia, identifies the community needs and gaps in knowledge. A new integrated methodology for management of community buildings is presented as well as the development of a software tool for implementation of the methodology in local government. The tool covers a building hierarchy, condition monitoring method, deterioration prediction and a decision making process.

***Keywords:*** *infrastructure management, sustainable buildings, Markov process*

## 1. Introduction

Infrastructure and public assets belonging to Australian community represent a major investment built up over many generations, and are valued at approximately 60 billion dollars. Out of these, community buildings are the second largest class of assets. A major issue currently faced by local government agencies is their inability to predict maintenance and replacement expenditure with a reasonable accuracy within reliable confidence limits, which creates situations where emergency repairs could use the funds kept for routine maintenance creating a vicious cycle of deterioration. Even when the cost is estimated with a reasonable accuracy, funding available is always inadequate to cover the total cost of keeping buildings at an optimum condition. Furthermore, without an overall understanding of the performance of building assets, there is very little opportunity at this stage to link previous performance data to design of new and sustainable buildings.

A research project funded by Australian research Council and six local councils in Australia is aimed at developing a methodology and a software tool for sustainable management of community buildings. In developing the methodology, the current practices of six partner councils were reviewed in detail, gaps identified and a basic methodology has been developed based on a review of major literature. The developed practice has been presented to the partners and comments have been obtained through three workshops of partner organizations. The work presented here covers the research methodology adopted, innovative approaches being developed and the management framework developed.

## 2. Proposed research methodology

Following steps were adopted in developing the model for sustainable management of community buildings;

- Review of literature
- Analysis of current practices of partner organizations
- Collection of existing condition data
- Developing an integrated framework for the management model
- Developing methods for quantifying each of the stages of the management model.
- Validation of the model

## 3. Review of the previous work on building deterioration prediction and condition rating

Prediction of building deterioration requires integrating the behaviour of a number of discrete elements with a vast range of significant influencing factors. As observed by Frangopol et al (2004) life cycle prediction of infrastructure systems under no maintenance or under various maintenance scenarios is quite complex. Current body of knowledge can be summarised as:

1. Approximate methods where conditions of different elements were rated at levels A, B, C and D or 1, 2, 3, 4 through condition inspections. Deterministic life cycle analysis is conducted assuming the time period of progression of deterioration to be fixed in one state.
2. Deterministic methods with modifications for exposure conditions and usage through fixed factors calibrated with data -ISO factorial approach. (Bamforth, 2004, ISO, 2000 and 2001).
3. Semi-empirical methods where probability of elements being in a given condition at a given age is calculated from condition data- MEDIC method (Flourentzou, 2000)
4. Reliability index methods using the mean value first-order second moment (MVFOSM) or the first-order reliability method (FORM), failure rate or time dependent reliability index.
5. Stochastic process models such as Markov decision process and renewal models (Frangopol et al, 2004).
6. Predicting life cycle of assets considering an integration of three drivers such as Market forces, physical deterioration and functional obsolescence – mainly a deterministic approach.

Of the aforementioned modeling methods, no single approach has yet proven to be generally applicable and each method has its advantages and disadvantages (Frangopol et al., 2004). Most powerful methods are reliability based models which are calibrated using data from non destructive testing, which gives continuous physical deterioration curves (Maheswaran et al, 2005, Frangopol et al, 2004). However, these are only available for a few known deterioration mechanisms such as corrosion of reinforcing steel in concrete, sulphate exposure, carbonation etc. On the other extreme, deterministic models which are based on discrete condition rating methods are widely available. A building is a complex system with a large number of discrete elements and often the condition rating is done using a visual inspection which categorises the condition into a discrete rating (ISO, 2000 and 2001). A method which offers a reliable prediction requires a large collection of validation data in order to cover different scenarios, categories of buildings, exposure conditions and element types. Since non destructive testing of all building elements to rate the condition of a building is almost impossible, a method which provides the best accuracy in deterioration prediction and is combined with a decision model based on condition data collected using the traditional discrete approach is the most practical.

## 4. Current practices of partner organisations

There are three distinct practices adopted by the six partner organizations as summarized below:

### *4.1 Reactive maintenance, building valuation and a spreadsheet used for forecasting*

The first approach observed is a maintenance strategy with a reactive decision making model. Building valuation data is used for the decision making. A list of 24 elements is used with a 1 to 5 scale condition rating in valuation. A spreadsheet based model is used in cost forecasting. Decision making process takes stakeholder needs in to account even though politically influenced decision making is also possible.

### *4.2 Physical condition assessment and spreadsheet based forecasting, data used for maintenance*

This method adopts a building maintenance strategy which is at a higher level than the basic reactive strategy. Councils collect the building condition data annually through a contracted inspector. The data collection method used is visual inspection. The data is used in maintenance decision making.

Cost forecast and decision making are based on a spreadsheet tool that considers stakeholder, scoping and design, permits, cost estimation, timeline, community, strategy, commitment, economic, environmental, and social aspects in capital budgeting decisions.

### *4.3 Physical condition data used to derive a deterioration curve*

This method is similar to method 2, however, instead of using condition data just to plan maintenance activities, data is used to generate deterioration curves as well. Curves are generated for five different categories of building components.

### *4.4 Combined assessment of physical condition and valuation*

This method uses a combined approach comprising of valuations and physical condition rating. Physical condition rating of the building is integrated with other influencing factors such as environmental, amenity – equity, service, children's services, grounds & gardens, sewer storm water, housekeeping and safety. Hence, the model finally uses an integrated condition rating for buildings. Maintenance plan and decision making is done based on building categorization, building priority and building weights (e.g. buildings of state significance, regional significance, municipal significance, neighbourhood significance and minor associated). Further, it is interesting to see that the budget allocation process uses involvements of appropriate committees and their consultation.

## 5. Existing condition data

Table 1 shows a sample set of condition data collected form one partner organization. These were collected by a council adopting the methods described in 4.2 above.

Table 1: Sample of condition data collected by a local council

| Building ID | Room Number | Room Function | Component | Ht. | Perimeter | Qty | Unit | Condition Rating |
|---|---|---|---|---|---|---|---|---|
| B001 | B001_000 | External | External Walls | 3.5 | 86.82 | 303.89 | m2 | 2.5 |
| B001 | B001_001 | Change Room | Ceiling | | | 42.45 | m2 | 1 |
| B001 | B001_001 | Change Room | Flooring | | | 42.45 | m2 | 3 |
| B001 | B001_001 | Change Room | Internal Walls | 3 | 28.21 | 84.63 | m2 | 2 |
| B001 | B001_002 | Shower Room | Ceiling | | | 24.83 | m2 | 1 |
| B001 | B001_002 | Shower Room | Flooring | | | 24.83 | m2 | 2.5 |
| B001 | B001_002 | Shower Room | Internal Walls | 3 | 23.5 | 70.5 | m2 | 2 |
| B001 | B001_003 | Kitchen | Cabinetry, Benches & Shelves | | | 3 | Item | 2 |
| B001 | B001_003 | Kitchen | Ceiling | | | 13.65 | m2 | 1.5 |

The data given in table 1 have been collected using the criteria shown in table 2. The criteria shown are quite generic and don't differentiate between critical and non-critical elements. Some of the other councils use a scale between 0 and 10.

Table 2: The condition rating criteria adopted for the data given in table 1

| | | |
|---|---|---|
| 1 | Excellent | The element is as new and can be expected to perform adequately to its full normal life |
| 2 | Satisfactory | The element is sound, operationally safe, and exhibits only minor deterioration |
| 3 | Unsatisfactory | The element is operational but major repair or replacement will be needed soon, that is within one to three years |
| 4 | Failing | The element runs a serious risk of imminent breakdown |

## 6. Integrated framework for building management

After analysis of the current practices, condition rating methods and a comprehensive literature review, the integrated framework shown in Figure 1 has been developed by the research team as a generic flow chart for management of infrastructure. The model is divided into six stages described below.

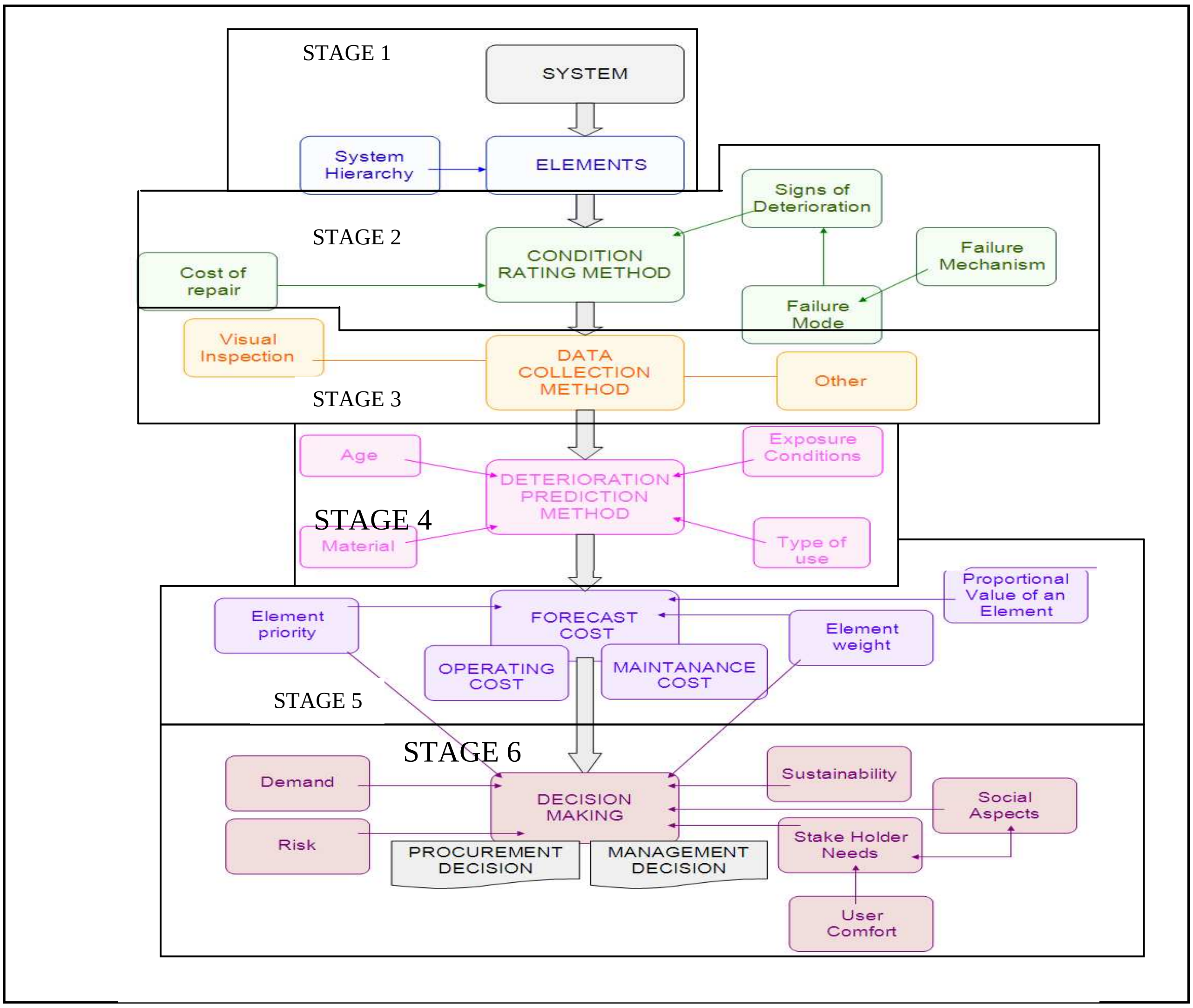



Figure 1. Infrastructure Management Process.

***6.1 Stage 1***

Stage 1 of the process covers the definition of the system and the division of the system into elements which can be inspected and recorded. A typical building system definition using a hierarchical framework is shown in figure 2. IPWEA (2009) guidelines have been used to develop a comprehensive building hierarchy for the project.

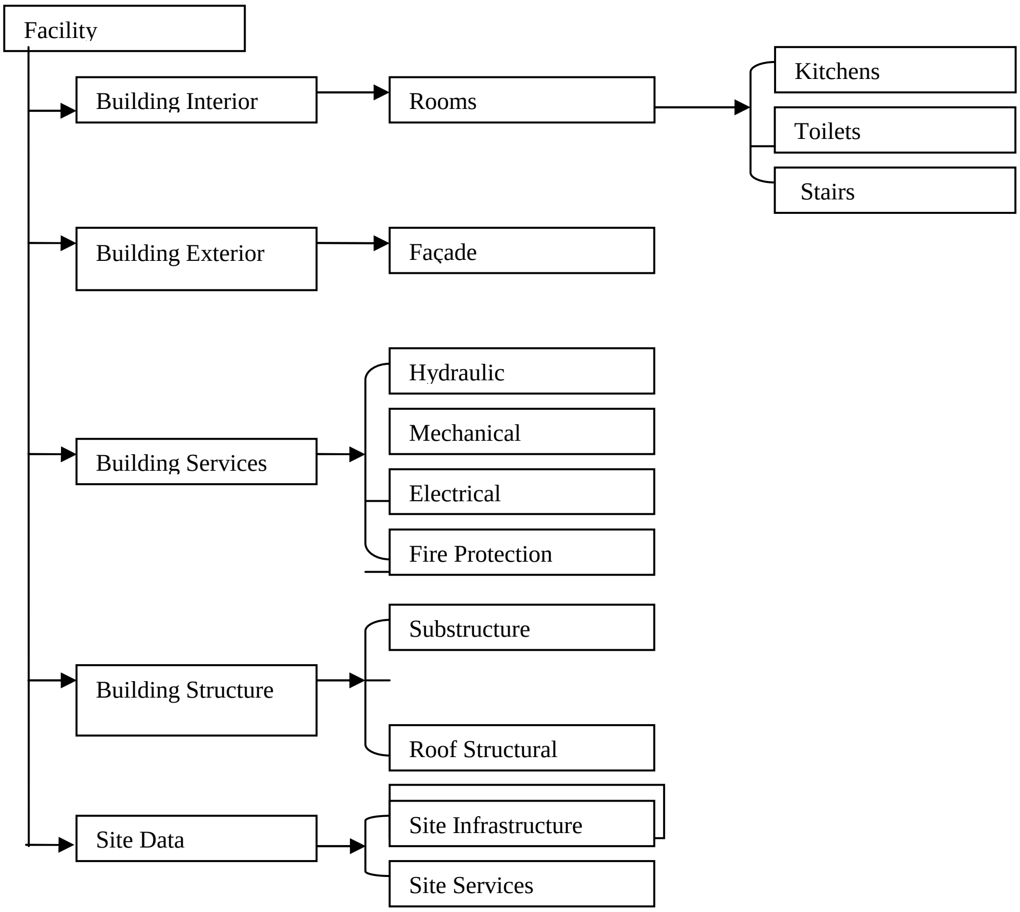



Figure 2: Building Hierarchy

***6.2 Stage 2***

Stage 2 covers the condition monitoring method which requires identification of the condition level, linking the cost of refurbishment/ maintenance to the given rating. After a review of number of approaches adopted and detailed consultation of stakeholders, a five scale condition rating scheme is proposed for the model. The proposed method is easy to implement due to a direct link to cost of work required to change the given condition to condition 1.

Table 3: The proposed condition rating scheme

| | |
|---|---|
| 1 | The element is as new and no work required |
| 2 | The element is sound, operationally safe, and exhibits only minor deterioration With minor repair, it can be changed to condition 1 |
| 3 | The element is operational but a medium scale refurbishment is needed |
| 4 | The element requires major refurbishment to change to condition 1 |
| 5 | The element requires to be replaced |

***6.3 Stage 3***

Stage 3 requires development of a data collection method, which links the specified condition rating and the building deterioration. In the initial stage of the project, the visual inspection methods adopted by the partners will be implemented. More sophisticated methods such as non-destructive testing is also explored.

### *6.4 Stage 4*

The stage 4 of the model requires rigorous analysis to generate the deterioration curves from condition data. A stochastic model based on the Markov process has been proposed for this stage (Sharabah et al, 2008). Discrete Time Markov Chain is a finite-state stochastic process in which the defining random variables are observed at discrete points in time. This chain satisfies Markov property, which means that given that the present state is known, the future probabilistic behavior of the process depends only on the present state regardless of the past. If an element is in state "i", there is a fixed probability, Pij of it going into state j after the next time step. Pij is called a "transition probability". The matrix P whose ijth entry is Pij is called the transition matrix. Transition matrix consist of a set of finite set of state S (1,1,3….n ) and a propriety pi j to pass from state i to state j in one time step t. In Markov chain pi j should satisfy two following conditions

$$p_{ij} \geq 0,$$

$$\sum_j p_{ij} \leq 1$$

This mean if an element is in state i, there is a (Pii) probability that this element will stay in state i, and (1- Pii ) will move to the next state j.

Present state at time t is i: $X_t = i$

Next state at time t + 1 is j: $X_{t+1} = j$

Conditional Probability Statement of Markovian Property:

$$\Pr\{X_{t+1} = j \mid X_0 = k_0, X_1 = k_1, \ldots, X_t = i\} = \Pr\{X_{t+1} = j \mid X_t = i\}$$

Discrete time means $t \in T = \{0, 1, 2, \ldots\}$

| | State1 | State2 | State3 | State4 | State5 |
|---|---|---|---|---|---|
| **State1** | 0.476 | 0.523 | 0.000 | 0.000 | 0.000 |
| **State2** | 0.000 | 0.336 | 0.665 | 0.000 | 0.000 |
| **State3** | 0.000 | 0.000 | 0.290 | 0.710 | 0.000 |
| **State4** | 0.000 | 0.000 | 0.000 | 0.004 | 0.996 |
| **State5** | 0.000 | 0.000 | 0.000 | 0.000 | 1.000 |

Figure 3: Transition Matrix derived for condition of walls

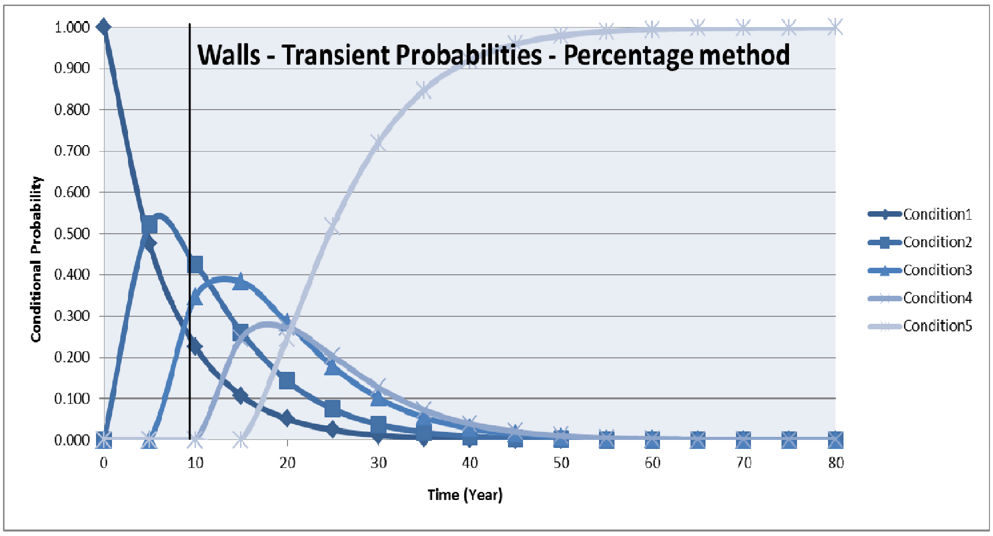



Figure 4: Transient probabilities derived from condition data

Figure 3 shows a typical transition matrix. An initial distribution 'v' is a single row matrix representing the number of elements in each state. In Markov chain after one time step the new distribution will be the result of multiplying initial distribution v by the transition matrix P

Distribution After 1 Step: vP
The distribution one step later, obtained by again multiplying by P, is given by (vP)P = vP2.`
Therefore distribution After 2 Steps = vP2
Similarly, the distribution after n steps can be obtained by vPn
Figure 4 shows the transient probabilities derived from the transition matrix shown in figure 3.

***6.5 Stage 5***

Economic modeling of the refurbishment cost and the maintenance cost are important elements of the model. Since the condition ratings are directly linked to the cost, using figure 4, the total cost at a given age can be determined for the walls analysed here. For example, from figure 4, at an age of 10, 20% of wall elements are expected to be in condition 1, 45% will be in condition 2 and 35% will be in condition 3. The total cost of renewal to condition 1 would be:
0.45 x cost of changing condition from 2 to 1 + 0.35 x cost to changing condition from 3 to 1

***6.5 Stage 6***

Integrated method for decision making forms the final stage of the proposed framework. A fuzzy logic based approach is currently being explored for integration of the economic and financial parameters.

## 7. Validation of the model

Validation of the model can be quite complex especially the decision making components. The fuzzy logic approach proposed will be used with a second set of data sourced from the Municipal association of Victoria to compare the model outcomes with the actual financials of one council over a given year. It is believed that with the availability of more data, as partners implement the methodology, the model can be further refined.

## 8. Conclusions

The paper presented the development of an integrated management model for council buildings in Australia. The method integrates a reliability based deterioration prediction model with a decision making model to provide local government agencies with a working model which can be used for managing community buildings.

### References


1 Bamforth, P. and Alisa, M. (2004) Probabilistic approach for predicting life cycle performance of buildings and civil infrastructure, *Proceedings of the international conference Innovative Materials and Technologies for Construction and Restoration*, Lecce, Italy, June 2-9, pp 553-558.
2 IPWEA (2006) International Infrastructure Management Manual.
3 IPWEA-NAMS.AU (2009) Building Condition and Performance Assessment Guidelines - Practice Note 3- Buildings.
4 Flourentzou F., Brandt E. and Wetzel C. (2000) MEDIC - a method for predicting residual service life and refurbishment investment budgets: *Journal of Energy and Buildings*, 31, 167-170.
5 ISO 15686 (2000) Buildings and constructed assets - Service life planning - Part 1: General Principles, Geneva: International Organisation for Standardisation.
6 Setunge S. and Kumar A. (2009) Estimating residual service life of commercial buildings. P. Newton, K. Hampson and R. Drogemuller (Eds) *In Technology, Design and Process Innovation in the Built Environment*, Abingdon, Oxfordshire, UK, 439-454, Taylor and Francis.
7 Sharabah, A. and Setunge, S. (2008) Effective condition monitoring and assessment for more sophisticated asset management systems. *Proceedings of the Clients driving construction innovation - benefiting from innovation.* Icon.net Pty Ltd, Brisbane, Australia, pp. 174 – 181.

# DEVELOPING A SIMPLE LIFE CYCLE ASSESSMENT (LCA) TOOL TO ASSESS CLIMATE ADAPTIVE BUILDINGS

Fernando K C A

Department of Chemical & Process Engineering, University of Moratuwa, Katubedda, Sri Lanka
chalaka@postgrad.unu.edu, TP : +94 77 354 1047

**Abstract:**
Climate change adaptations already identified as a key priority globally. Sustainable building practices like climate adaptive buildings and green buildings are immerged more prominently supporting climate adaptation strategies. A quantitative assessment through scientifically accepted LCA method supports to justify the environmental investments in these new constructions models. Introducing a simple LCA tool assessing the global warming potential calculating through greenhouse house gas emission per selected functional unit supports to find the environmental savings or degradations. Scope can be select to cover both constructing & use phases. LCA outputs will further reinforce the sustainable building initiatives both ecologically and economically.

***Keywords:*** *life cycle assessment, climate adaptive buildings, green buildings, climate change adaptation, climate change mitigation*

## 1. Introduction

Frequent natural disasters and changes in the environment indicate the climate change is not a myth. Most of the Asian nations are in the mostly climate vulnerable region. Climate change adaptation and mitigation practices already considered as a timely priority in the national strategies by majority of the nations. Countries are also in the process of either developing climate resilient national policies or reviewing the existing ones in order to improve more. These cover mitigation practices more lean towards developed - high greenhouse gas (GHG) emitting nations and adaptation actions for the climate vulnerable countries. Mostly the mitigation practices are implemented jointly through industrial and private partnerships while adaptations commonly lead by government intervention. Adaptive capacity varies between countries depending on social structure, culture, economic capacity, geography and level of environmental degradation. (UNFCCC, 2007)

## 2. Background Problem

Building climate resilience into new property will avoid unnecessary climate-related damages and costs, as the impacts of climate change begin to be felt more intensely. (Torbay Council, 2010) One of the major impacts of climate change - sea level rise is likely to have adverse impacts on: buildings and tourism. (UNFCCC, 2007). More focus to constructions were given under adaptation strategies; including buildings, dams and other flood management initiatives, since their higher vulnerability in disaster conditions. Millions of people could become homeless in the Asia-Pacific region by 2070 due to rising sea levels, with Bangladesh, India, Vietnam, China and Pacific islands most at risk. (Perry, 2006) New building construction requires considering wetter winters, drier summers, extreme rainfall events, rising sea levels, intensification of the urban heat island effect and higher wind speeds depend on the location. These impacts are changed not only the building architecture also the material consumption. Green building is a solution to mitigate challenges cause by climate change. (United Nations Economic Commission for Europe, 2009) Similarly green buildings are supported climate adaptations, consideration of structure to be adaptive to anticipated climate change. Some buildings are already being developed so that they will be able to resilient to future climate change extreme events. The European Commission has also identified the future market opportunity for climate adaptive buildings (Dalton & John, 2008). Analysis of environmental benefits and losses of climate adaptive building efforts will help to find the macro picture rather than stick to the green building or climate adaptive building concepts alone. Also it will justify more on investing on climate adaptive buildings which is still in the starting level of developing nations.

## 3. Research objectives and Scope

The objective of the research is developing a simple LCA tool as a strong justification method for decision makers and related stakeholders on rationalizing the newly constructing buildings in climate adaptive or green building architectural concepts considering the GHG aspect which directly supports the climate change impacts. Research outcomes could make more transparent evaluation of buildings and harmonize the climate adaptive buildings and green buildings a way forward GHG reducing and sustainable consumption and material focused constructional methods. Through this research spatial planners and building architects will be mostly benefited in order to make designs environmentally biased and reducing impacts which simultaneously reduces the cost (both capital and recurrent). Local government bodies will also interested to quantitatively justify the environmental positive (towards carbon neutral and beyond) benefits of these buildings in the scope of climate change mitigation and adaptation. The final data will be more important for the environmental economists, in order to economically justifying the environmental impacts which the most is commonly considered as burdens

## 4. Research methodology

Throughout the study ISO 14040:2006 standard – 'Environmental management -- Life cycle assessment -- Principles and framework' was selected as the LCA standard which also the research methodology. United Nations defined "Life Cycle Assessment (LCA) is an analytical tool for the systematic evaluation of the environmental aspects of a product or service system through all stages of its life cycle. LCA provides an adequate instrument for environmental decision support. A reliable LCA performance is crucial for a life cycle economy. The International Organization for Standardization (ISO) completed a whole series of Life Cycle Assessment standards in 2002, the 14040 series." (ATIS Exploratory Group on Green, 2010) This ISO 14040:2006 standard which is the most commonly accepted and practiced internationally, describes the principles and framework for life cycle assessment (LCA) which includes: defining of the goal and scope of the LCA, the life cycle inventory analysis (LCI) phase, the life cycle impact assessment (LCIA) phase, the life cycle interpretation phase, reporting and critical review of the LCA, limitations of the LCA, the relationship between the LCA phases, and conditions for use of value choices and optional elements (ISO, 2006). Standard was initially introduced in 1997 and reviewed 1998, 2000.and finally by 2006. However defining of a specific 'scope' is critical in these studies, due to variable of building parameters are mostly depended on the geographical location, purpose and based on socio cultural influences.

## 5. Research framework

The research framework is split in to three steps, in order to study more deeply and align with the selected methodology ISO 14040: 2006. LCA objectives, goal and the scope are covered in the step one while life cycle inventory and analysis (LCI) covered under second step. The third step is the life cycle impact assessment (LCIA) while the lifecycle interpretation pillar is embedded to each of these steps in order to elaborative delivered.

### *5.1.1 Step 01*

LCA objective is also similarly defined aligning to the research objective which is 'developing a simple LCA tool as a supportive method for decision makers and related stakeholders on justifying the newly constructing buildings in climate adaptive or green building architectural concepts considering GHG emissions as the impact category'. The goal of the LCA is, construct and propose a simple LCA tool for the above purpose. The LCA tool will be introduced to measure the environmental impact of climate adaptive or green buildings for considering two scenarios; selecting traditional buildings as the baseline study. This tool will help to compare two scenarios; baselines study and improved climate adaptive or green building. From a single building to a housing scheme can be selected as the study area for the LCA study.

LCA scope was defined as 'material to use' type which life cycle focused on both constructional and used phase impacts. Environmental impacts of material transportation from the generated point to building are also within the LCA scope. 'Use phase' environmental impacts will be more focused due to access to actual data collection possibilities and also varied due to climate change impacts (in long run). In this research, the environmental impacts due to disposal or recycling of the building after the lifetime will not be considered, mainly due to it is still not in the national implementation agenda of developing (more climate vulnerable) countries. Hence this will be a typical LCA framework to 'cradle to use' in general LCA for products. Selection of the specific scope 'from material to use' phase will support to assess both environmental impacts on adaptation phase and mitigate actions.

Life cycle impacts can be categorized under few LCA functionalities as United States Environmental Protection Authority (USEPA) reported in their Tool for the Reduction and Assessment of Chemical and Other Environmental Impacts (TRACI, 2007 version). They are: primary energy, acidification potential, eutrophication potential, global warming potential, human health respiratory effects potential, ozone depletion potential, weighted raw resource use and photochemical smog potential. In order to align with the goal and objective of this LCA study 'global warming potential' was selected which is indicated as anthropogenic GHG emission per unit time and unit area as the functional unit. All the computations, comparisons and interpretations is based on this functional unit. This indicator is introduced as 'BuiLCA-CC' in order to represent buildings – LCA – climate change and defined as below equation (1). Unit of this indicator is derived as '$kgCO_2e/m^2$'. The system boundary of the product life cycle shall exclude the GHG emissions associated with: human energy inputs to processes and/or preprocessing (BSI, 2009) however an uncertainty may influence the final figures which require certain statistical control during primary data acquiring (LCI) and calculation and interpretation (LCIA) phase. Once calculated BuiLCA-CC for a building, it will also support the climate change evaluation schemes for buildings.

BuiLCA-CC = (Construct phase GHG + Use phase GHG) / area of the building (1)

Use phase GHG will only consider for one year which the backward latest from the calculating time.

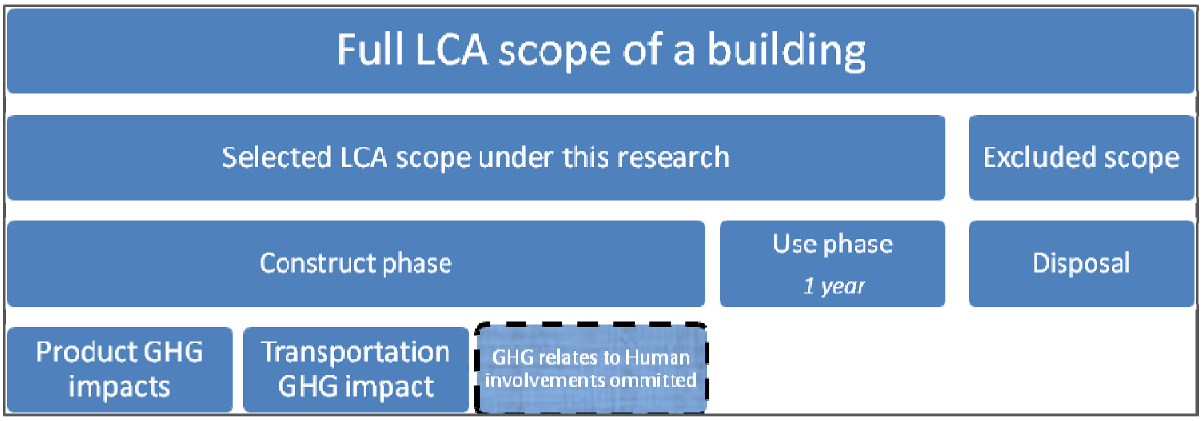



***Figure 1: Scope of the LCA tool***

Climate change vulnerability extremely differs with geography, climate patterns and the social status of a location. For an effective LCA comparison, considering all above facts are critical. Especially for the 'use phase' an emphasis should be given to select the similar socio-economical status. The LCA tool (initially) will not have the facility to compare geographic, socio, economic and other variations. For the baseline study a housing complex or a representative house can be selected from the location where focused climate adaptive or green building complex is located (which is the best) or from a similar geographical region. The tool will be selected under 'single family residential category' to be more focused on the integrated community based adaptation impacts. The building life expectancy also required to similar for a LCA comparison in order to make a one to one comparison.

***5.1.2 Step 02***

The life cycle inventory and analysis process is covered under this step. The life cycle inventory preparation process was again split in to two pillars in order to minimize the complexity of the study. The first part of the LCI is fully focused on the construct phase which consist of GHG emission from materials and material transporting phase and the second part is to focus on 'use phase' impacts. The inventory preparation and calculations will be done accordingly to the following tables. LCI preparation is recommended annual basis in order to have more rationalized values. The data acquisition is highly important in the LCI phase which is always necessary to use actual data other than an extreme difficulty to find. Following two tables Table 1 to Table 4 are shown the proposed data sources. For the construct phase GHG calculations ‘bill of material record’ is a critical data source. Generally this set of data is only available for buildings constructed by professional contractors.

**Table 1: LCI – Construct phase - Product related GHG calculation**

| Parameter | Material | Specific product GHG emission | Used quantity | Product phase GHG for material |
|---|---|---|---|---|
| | i | $a_i$ | $q_i$ | $P_i$ |
| Unit | NA | $kgCO_2e$ /ton $product_i$ | ton | $kgCO_2e$ |
| Information source | Bill of material record | LCA or PCF (product carbon foot printing) data bases | Bill of material record | Calculation $P_i = a_i \times q_i$ |

Initially the cement, steel, timber, sand, tile, paint and metal were selected for the LCA tool, keeping the provision to expand once researching in more complex cases. In some cases instead of cement, sand and metal, ‘supplied concrete’ can be directly used upon the application on site. The below table elaborate the calculation of GHG in material transportation phase. This covers the transportation related GHG from the supplier to building location.

**Table 2: LCI – Construct phase – Material transport related GHG calculation**

| Parameter | Material | Used quantity | Average loaded weight per trip | Round the trip distance | Specific GHG emission for the used vehicle | Transport phase GHG emission |
|---|---|---|---|---|---|---|
| | i | $q_i$ | $w_i$ | $d_i$ | $v_i$ | $T_i$ |
| Units | NA | ton | ton/trip | km/trip | $kgCO_2e/km$ | $kgCO_2e$ |
| Information source | Bill of material record | Bill of material record | Actual or estimated | Actual or estimated | LCA or Carbon foot printing data bases | Calculation $T_i = (q_i / w_i) \times d_i \times v_i$ |

Calculation of construction phase GHG is equal to summation of Product phase GHG ($\sum P_i$) for material and Transport phase ($\sum T_i$) GHG emission with unit $kgCO_2e$.

$$\text{Construct phase GHG} = \sum (P_i + T_i) \qquad (2)$$

LCI calculating GHG foot printing for ‘use phase’ was also focused under Step 02. Main GHG source of the ‘use phase’ is from energy consumption. The raw (primary) data required are electrical energy consumption (in kWh), use natural gas (in $m^3$), liquid petrol gas (LPG) (in liter) and other sources. The thermal energy which consider is for building heating purpose only and not used for cooking since the thermal requirement for cooking significantly depend on societal structure and economical background. Following two tables describe the use phase energy related GHG LCI – data requirements and calculations separately.

**Table 3: LCI – Use phase – Thermal energy related GHG calculation**

| Parameter | Energy source | Used quantity | Lower calorific value | Fuel specific carbon factors | GHG emission for thermal fuel |
|---|---|---|---|---|---|
| | i | $u_i$ | $c_i$ | $f_i$ | $TE_i$ |
| Unit | NA | ton | GJ/ton | $kgCO_2e$ /GJ | $kgCO_2e$ |
| Information source | Primary data | Primary data | Intergovernmental Panel on Climate Change (IPCC) ‘lower calorific values | IPCC default ‘fuel specific carbon factors’ | Calculation $TE_i = u_i \times c_i \times f_i$ |

Electrical energy usage related GHG emission is calculated as per the below table. However availability of accurate and updated secondary data is important in this calculation process.

**Table 4: LCI – Use phase – Electrical energy related GHG calculation**

| Parameter | Utilized electrical energy | Grid carbon factor | GHG emission for thermal fuel |
|---|---|---|---|
| | $e_i$ | G | $EE_i$ |
| Unit | kWh | $kgCO_2e$ /MWh | $kgCO_2e$ |
| Information source | Primary data | Whichever the latest from the local electrical supplying body, national reports or latest United Nations Framework for Climate Change Convention (UNFCCC – CDM) reports. In cases none of the above sources are available IPCC (2007) | Calculation $EE_i = e_i \times g$ |

Calculation of use phase GHG is equal to summation of GHG emission from thermal energy ($\sum TE_i$) and from electrical energy utilization ($\sum EE_i$) with unit $kgCO_2e$.

$$\text{Use phase GHG} = \sum (TE_i) + \sum (EE_i) \quad (3)$$

In order to calculate the functional unit measuring the area of the selected house or building is critical. Building area is measured in SI unit $m^2$. Calculation of the area the building required a similar approach to both scenarios in order to compare in one to one basis. For this parameter measuring the area of the living space of the house (including bed rooms, visitors' area, wash rooms and kitchen) is required. It is also possible to extend to other parts of the house depend on the architect design however it is further require to be a similar approach in both scenarios. The final data for both scenarios will be presented in the unit's $kgCO_2e/m^2$ per year as the functional unit of the study as per the equation (1).

#### *5.1.3 Step 03*

This step covers the LCIA phase. The final two sets of data will be available in 'one to one' comparison mode in order to assess scenario wise emission both in absolute and specific GHG emission. These two figures are used to assess the environmental soundness of both scenarios. Absolute emission figures can also used to compare the GHG emission reductions or increasing during annually. The climate adaptive constructional investments and the return can also be assessed in both building scenarios using the BuiLCA-CC indicator. Since the indicator is based on two main GHG streams: constructional and use phases, it provides the facility of sector level analysis.

## 6. Verification of Results

Used equations during the LCI process are only derived using fundamental theories. Since validating of such equations might not require necessarily. The data which use to feed to this model require certain validation steps. Especially the functional unit of this study is entirely depended on the constructional, transportation and use phase inputs which require a statistical data validation before input to LCI. Also before using of 'use phase' annual figures it require to introduce a control limit with a co-efficient of variation interval to remove outliers to justify the effort of calculating such parameters. During LCI phase the use of secondary data is also high in this study, mainly due to find the constructional phase GHG emissions. Accessing latest and reliable data sources and selecting the exact product related emission is also important in this exercise which enhances the data validation process.

## 7. Conclusion and proposed future work

Introducing and developing a LCA tool to assess the climate adaptive buildings is definitely supporting to justify the investments in quantified environmental factors. These findings will diminish the resilient to invest on climate adaptation strategies in building sector which is an important requirement currently. BuiLCA-CC can also use as an indicator for environmental competitions as an unbiased estimator to assess the environmental friendliness with respect to 'global warming impact'. The model will be useful for the green building architects to improve their future models more ‘carbon friendly’ similarly to justify if more capital expenses require on green building constructions, through recurrent carbon (ie environmental) savings derived through global carbon costing models.

The research can be further expanded in two different axes. While increasing the number of functional units and integrated eco indicator for green building can be introduced. This will support on applying certification schemes like LEEDS (Leadership in energy and environmental design). The other aspect is to link with an environmental cost benefit analysis model and justifying the adaptation cost by showing the GHG saving during impact stages and disaster reduction. GHG emission can be quantified in economical terms using developed models like "the model of the Eco-costs / Value Ratio" which is identified as a future addition to this tool.

For developing countries the findings will be further important on justifying the green building costs with respect to environmental savings benefited from mitigation aspects by using latest climate change models and selected building adaptation strategies.

## Reference

ATIS Exploratory Group on Green. (2010). *ATIS Report Reviewing ICT Life Cycle.* Washington DC: Alliance for Telecommunications Industry Solutions.

BSI. (2009). *Specification for the assessment of the life cycle greenhouse gas emissions of goods and services.* London: British Standards Institution.

Dalton, K., & John, R. (2008). Towards More Sustainable Tall Buildings. *Council on Tall Buildings and Urban Habitat (CTBUH) 8th World Congress 2008* (p. 10). Dubai: CTBUH.

ISO. (2006). *ISO 14040:2006 Environmental management - Life cycle assessment - Principles and framework.* Geneva: International Organization for Standardization.

Perry, M. (2006, October 09). *http://www.commondreams.org/headlines06.* Retrieved 09 22, 2010, from www.commondreams.org: http://www.commondreams.org/headlines06/1009-06.htm

Torbay Council. (2010, September 14). *Climate Change Adaptations*. Retrieved September 20, 2010, from http://www.torbay.gov.uk: http://www.torbay.gov.uk/es/index/business/planning/planningservice/sustainableconstruction/scadaptation.htm

UNFCCC. (2007). *Climate Change : Impacts, Vulnerabilities and Adaptation of Developing Countries.* Bonn: UNFCCC.

United Nations Economic Commission for Europe. (2009, October 21). *Information*. Retrieved September 20, 2010, from www.unece.org: http://www.unece.org/press/pr2009/09tim_p08e.htm

## About the Authors

**Fernando K C A,** B.Sc. (Hons.) Peradeniya, is currently reading for his MSc at the Department of Chemical & Process Engineering, University of Moratuwa. Author currently working as the Head of Consulting & Operations – Carbon Consulting Company, Colombo, Sri Lanka

# EARNED VALUE MANAGEMENT SYSTEM AS A PROJECT MANAGEMENT TOOL FOR MAJOR MULTI-DISCIPLINARY PROJECTS

Dr. P.B.G. Dissanayake[1]

[1] Department of Civil Engineering, Faculty of Engineering, University of Peradeniya
E-mail: pujitha@civil.pdn.ac.lk

**Abstract:** The Earned Value Management System (EVMS) is a useful management tool available for project managers to monitor and control major multi-disciplinary projects. EVMS measures project performance by comparing the amount of work planned against the amount of work actually carried out and the actual costs incurred. EVMS combines the work scope, schedule and the cost elements of a project and facilitates the integrated reporting of a project's progress and the cost status. The earned value concept was first introduced by industrial engineers working in American factories over a century ago and this concept was used to manage the production cost of commercial industrial products. The basic concepts of earned value were adopted by the United States Air force in the early 1960s and later endorsed by the United States Department of Defence in all major system acquisitions. Earned Value Management is a technique that can be applied to the management of all infrastructure projects, in any industry, while employing any contracting approach.

This paper summarizes the basic concepts and the theory of EVMS, briefly explains how EVMS can be implemented for a multi-disciplinary project, the challenges encountered during implementation and its benefits to a project as a project management tool.

## 1 Introduction

The management of multi-disciplinary infrastructure projects requires monitoring and control tools for effective project control. The Earned Value Management System (EVMS) is one of the tools for measuring project performance by comparing the amount of work planned against the amount of work actually done and the actual cost incurred. EVMS integrates the scope, cost (or resource) and schedule to help the project team assess project performance (PMBOK Guide, 2002). The concept of earned value management system was first introduced by industrial engineers working in American factories over a century ago (Fleming & Koppelman, 1999). This concept was used to manage the production cost of commercial industrial products. The basic concepts of earned value were originally adopted by the United States Air force in the early 1960s and by 1967, the United States Department of Defence formally endorsed the use of earned value management in all major system acquisitions (Fleming & Koppelman, 1999). The basic concepts of earned value management have not changed since 1967 (Christensen, 1999). Earned Value Management is a technique that can be applied to the management of all capital projects, in any industry, while employing any contracting approach (Fleming & Koppelman, 2002). The Earned Value Management System has been successfully implemented for the project management and control of a major mult-disciplinary infrastructure development project in Hong Kong (Dissanayake, 2007).

To understand the concept of EVMS, it is important to compare EVMS with the traditional method of project cost management. In the traditional approach there are two data sources, the budget (planned) expenditures and the actual expenditures. The comparison between the budget versus the actual only indicates what was planned to be spent and the actual amount spent at a given time. But this does not give any idea about how much work has been done. Therefore, in the traditional approach there is no measure of the physical amount of work performed. It does not indicate anything about what has been achieved for the money spent. In EVMS, unlike in the traditional approach, there are three data sources, the budget (planned), actual expenditure and the "earned value" which is the physical work done at a given time. Therefore, in EVMS the planned value of the work could be compared with the earned value and actual cost. The three basic definitions used in EVMS are as follows:

Budgeted Cost of Work Scheduled (BCWS) = Planned Value
Budgeted Cost of Work Performed (BCWP) = Earned Value
Actual Cost of Work Performed (ACWP) = Actual Cost

During the planning stage of a project, a time phased budget is developed based on the estimated cost of various elements of the project and the works programme. This time phased budget serves as a performance measurement baseline against which the project progress is monitored. At a given point in time on the date of analysis, the value of the performance measurement baseline becomes the BCWS. The BCWS is compared with the BCWP and ACWP, all expressed in terms of monetary values. The following variances are used to measure project performance.

Cost Variance (CV) = BCWP – ACWP

CV is an indicator of expenditure measured against the completion of the corresponding work scopes. If CV = 0, the performance is on target, if CV > 1.0, it indicates a favourable performance and if CV < 1.0, it indicates a cost overrun.

Schedule Variance (SV) = BCWP – BCWS

SV is an indicator of the schedule status as compared to the plan in terms of a monetary value. If SV = 0, then the project is progressing as planned, if SV > 0, it indicates the project is ahead of schedule and if SV < 0, it indicates the project is behind schedule.

Fig. 1 illustrates the graphical representation of the above earned value parameters.

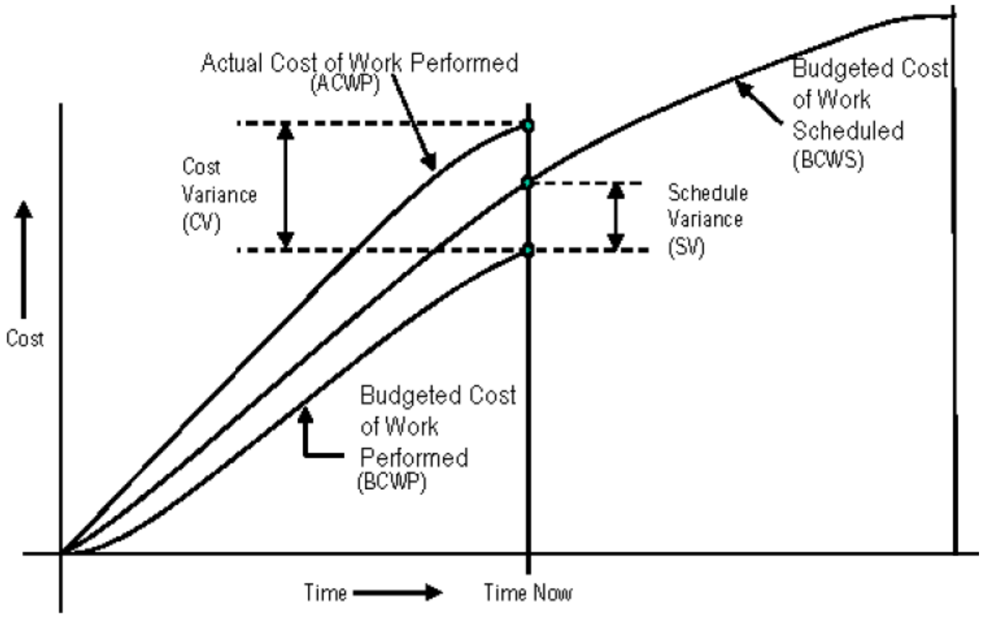



Fig. 1 – *Graphical Representation of Earned Value Parameters*

The following indices are used to measure project performance.

Cost Performance Index (CPI) = $\dfrac{\text{BCWP}}{\text{ACWP}}$

If CPI = 1.0, then the performance is on target, CPI > 1.0, then the performance is exceptional and if CPI<1.0, then the performance is substandard.

Schedule Performance Index (SPI) = $\dfrac{\text{BCWP}}{\text{BCWS}}$

If SPI = 1.0, then the performance is on target, SPI>1.0, then the performance is exceptional and if SPI<1.0, then the performance is substandard. The two indices CPI and SPI may give results that may seem contradictory if CPI >1 and SPI < 1. In this situation, the project is within budget which is good but it is behind schedule. This indicates that money was saved because not enough work was done. Therefore, the Cost-Schedule Index (CSI) is introduced (Barr, 1996).

Cost Schedule Index (CSI) = CPI x SPI = $\dfrac{\text{BCWP}^2}{\text{ACWP x BCWS}}$

It is said that if the CSI value in general is greater or around 1.0, a project is not having any serious problems (Barr, 1996).

## 2 Implementation of EVMS

In order to implement EVMS for an infrastructure project, it is important that the following steps are followed.

### *2.1 Detailed Work Breakdown Structure (WBS)*

The development of the WBS is considered as the cornerstone of effective project planning, execution, controlling and reporting (US DOE, 2003). Therefore, it is important that a detailed WBS is developed for the whole project that will include all the major elements and their sub-elements with further detailed breakdown of the sub-elements as required. Any major infrastructure project would essentially be a multi-disciplinary project and hence a large number of consultants and contractors would be involved in the project at various stages. Therefore, it is important that all these parties who would be participating in the project be included in the WBS. The WBS should include all work that is carried out under the project that will incur a cost to the project. Therefore, all consultants and contactors having separate contractual arrangements with the client need to be included in the WBS.

### *2.2 Integrated Programme (Schedule)*

Developing an Integrated Programme (IP) is an essential step in the implementation of EVMS. It should include all the elements, sub-elements etc. included in the WBS. A comprehensive IP should be developed by the project manager. It is also possible to get the help of the individual consultants and contractors to prepare the programmes for their own portion of works (project programmes) that will form the IP.

### *2.3 Formal guideline for the preparation of Project Programmes (PP)*

Any major multi-disciplinary infrastructure project will include a large number of consultancies and works contracts and the development of IP will be a collective exercise. It is therefore essential that a formal guideline is prepared for the development of PP (that will form the IP). This will ensure that once PPs are prepared, they could be integrated to form the IP. The guideline should cover the activity identification numbering scheme, activity coding structure (in-line with WBS), activity identification codes, calendar codes and resource identifications. This formal guideline will form the backbone of the EVMS and hence should be prepared with great care. The formal guideline should include a copy of the WBS, guidelines for the preparation of individual PPs, the incorporation of project cost information, the procedure for the integration of PPs and the updating of project progress and cost data.

### *2.4 Activities for Earned Value (EV) Measurement*

It is important to note that it is not practical to use all activities in the IP to monitor EV at the end of each reporting period. Therefore, the project manager needs to select a reasonable number of activities that will be assigned with cost data and will be used to measure the EV parameters. The Earned Value Elements are to be selected in such a way that when the progress and cost status are reported against these Earned Value Elements, it should reflect the overall progress and cost status of each contract package.

### *2.5 EV Measuring Techniques*

It is important that EV measuring techniques are established for each type of activity before carrying out project monitoring work. The determination of earned value depends on the type of effort, whether it is discrete, apportioned or on the level of effort.

#### *2.5.1 Discrete effort*

There are three basic earned value methodologies applicable to the discrete effort. They are based on valued milestones, standard hours and management assessment. A typical example would be a contract with a milestone payment arrangement where a pre-determined monitory value will be earned once a particular milestone is achieved.

#### *2.5.2 Apportioned effort*

Apportioned effort is work for which planning and progress is tied to other efforts. The budget for apportioned account will be time-phased in relation with the resource plans for the base account. A typical example would be in a software design project where the task manager would apportion computer costs to the design and coding effort.

#### *2.5.3 Level of Effort*

Level of effort is work scope of a general or supportive nature for which performance cannot be measured or impracticable to measure. Resource requirements are represented by a time-phased budget schedule in accordance with the time the support will likely be needed. A typical example would be the head office staff work-hours spent on administration work for a particular project.

## 3 Application of EVMS

EVMS was used as a management tool for the project monitoring and control of a major multi-disciplinary infrastructure development project in Hong, the Hong Kong Science Park Phase 2 Development. The author, a former employee of Maunsell AECOM (a Project Manager of Hong Kong Science Park Phase 2 Development) was responsible for the implementation of the EVMS for the above project. Hong Kong Science Park Phase 2 was the second phase of a 3 phased development project funded by the government of Hong Kong at a total cost of more than US$ 1.5 billion. Upon completion of the Science Park, a total Gross Floor Area of 330,000 m2 will be provided for office and laboratory facilities for applied research and development of the four strategic industries, namely Information Technology and Telecommunications, Electronics, Precision Engineering and Biotechnology.

A formal guideline, the Protocol for Integration of Programmes, was developed to include the detailed WBS covering all consultancies and works contracts, the essential steps in the preparation of PPs, the setting-up of the IP and the subsequent updating the IP with progress and cost information. A formal mechanism was set-up and implemented in the preparation of initial PPs and the subsequent updating of progress and cost data at the end of each month by the respective consultant / contractor under the close supervision of the Project Manager and the Resident Site Staff.

Primavera Project Planner (P3) was used in the preparation of all PPs, as it had the capability of integrating PPs to form an IP and also its ability to handle time and cost data. The necessary clauses were written into the respective agreements of consultancies and works contracts to comply with the Protocol for Integration of Programmes in preparing their programmes.

The WBS developed for Hong Kong Science Park Phase 2 Development included 3 no. consultancy agreements, 2 no. foundation works contracts, 4 no. main works contacts, 6 no. specialist works contracts and 5 no. nominated supply contracts.

Since EVMS was used to report the progress and cost status of the project to the client (Hong Kong Science and Technology Parks Corporation), it was important that ACWP should reflect the cost to the client rather than the cost incurred by the consultants or the contractors. Therefore, ACWP was defined as the total cost of payments that have been certified at any given time. In the case of the works contracts, it is the total amount of work that the Quantity Surveyor has certified at any given time. In the case of the consultancies, it was the total amount certified by the Project Manager. For other payments that were directly handled by the client, the ACWP was defined as the total amount issued for payment by the client.

To facilitate the measurement of BCWP, it was necessary to develop a systematic method of breaking down the total cost and to assign the cost to the specific activities within each PP. In the case of consultancies, the schedule of fees was the basis for distributing the consultant’s fees among the activities. In the case of works contracts, the contractors were required to propose the breakdown of the cost of works based on the bill of quantities and to assign cost figures to a reasonable number of activities within each PP. The requirement was to have a manageable number of activities to measure and report the earned value at the end of each month.

## 4 Results

Fig. 2 shows earned value data (BCWS, BCWP and ACWP) from the commencement of the project (June 2001) up to June 2007 when most of the site work were substantially completed. As illustrated in Fig. 2, the original baseline for the BCWS was used from the commencement of the project up to April 2003 which included the BCWS of the Client and the Project Manager. In May 2003, the original baseline was revised (1st revision) to take account of the award of the consultancy agreements which changed the baseline of the BCWS.

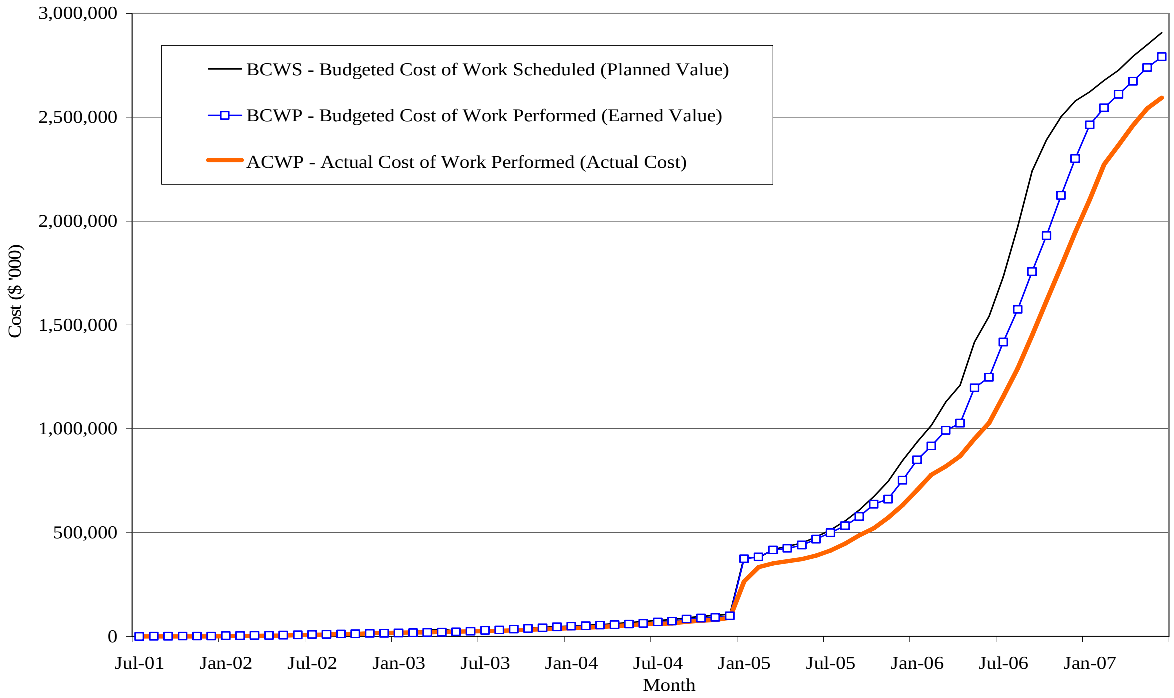



Fig. 2 – *Earned Value Graph (Up to June 2007)*

Fig. 3 shows the Cost Variance (CV) and the Schedule Variance (SV) from the commencement of the project up to June 2007. The value of CV is positive and it indicates that less money has been spent. This is mainly due to the payment arrangement where completed work is considered paid only after the Quantity Surveyor has certified the work and hence there is a lag of one month. The SV is negative and it indicates that the works are behind schedule. This is mainly due to the slow progress of works under several main works contracts and specialist contracts. Since September 2006 onwards, the negative SV has sharply reduced with the improved progress status of the works contracts.

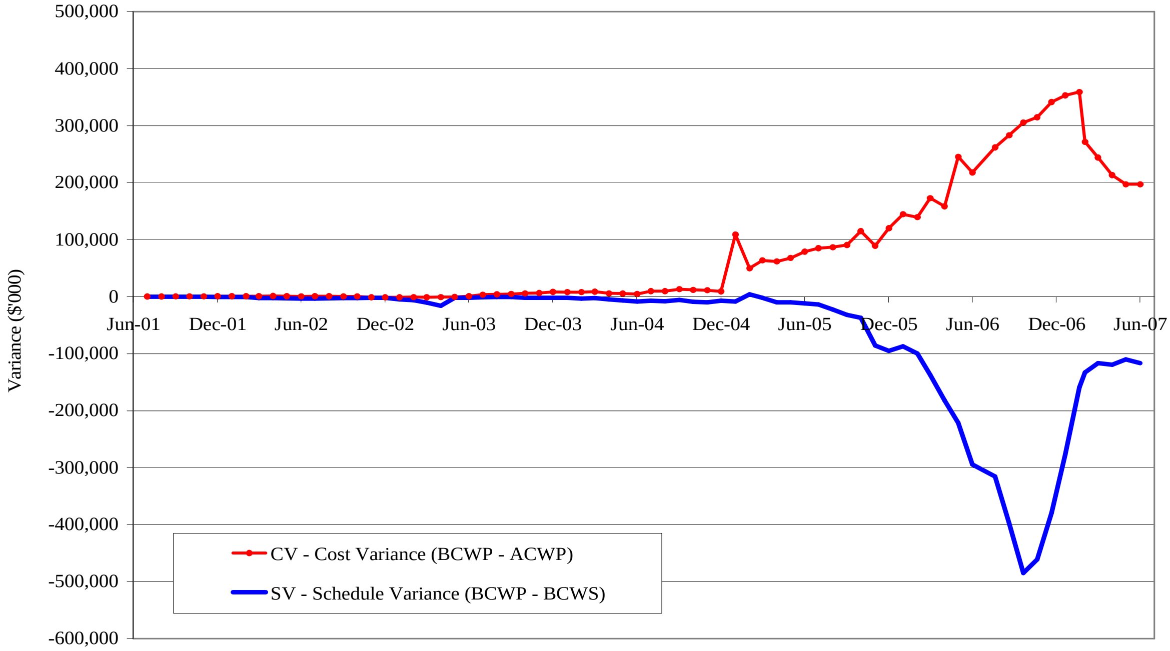



Fig. 3 – *Cost Variance (CV) and Schedule Variance (SV)*

Fig. 4 shows the Cost Performance Index (CPI) and the Schedule Performance Index (SPI) from the commencement of the project up to June 2007. It is noted that the CPI has stabilized around 1.2. This is largely due to the works already carried but yet to be certified. A value of CPI > 1 is considered as a favourable situation. The SPI is around 0.96 in June 2007 and had been in the range between 1.0 and 0.8 during the preceding four years. A value of SPI < 1 indicates that the project is behind schedule. The SPI has been gradually slipping since June 2005 following the commencement of the major main works contracts. The SPI has been increasing since September 2006 with the improvement in the progress of works contracts.

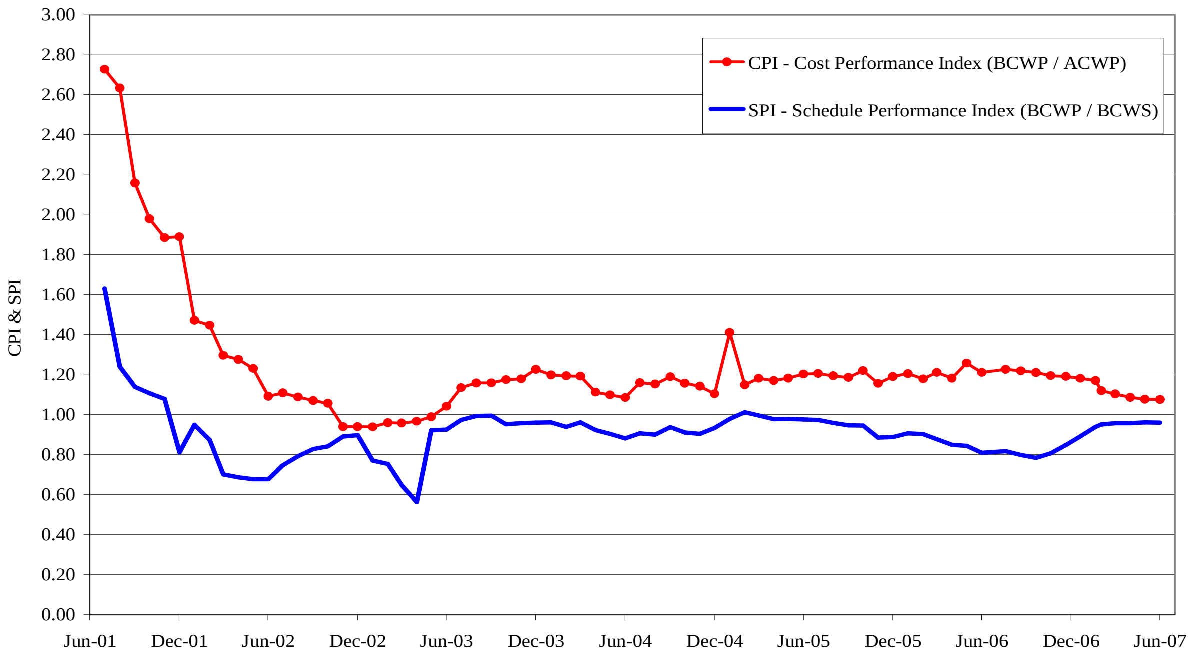



Fig. 4 – *Cost Performance Index (CPI) and Schedule Performance Index (SPI)*

Fig. 5 shows the Cost-Schedule Index (CSI) from the commencement of the project up to June 2007. The value of CSI has been above or around 1.0 with the exception from March 2002 to June 2003. During this period the approval of the Master Layout Plan for the Science Park was deferred by the government of Hong Kong and hence the postponement of the award of the design consultancies. If the CSI value is greater or around 1.0, it indicates that the project is not having any serious problems (Barr, 1996).

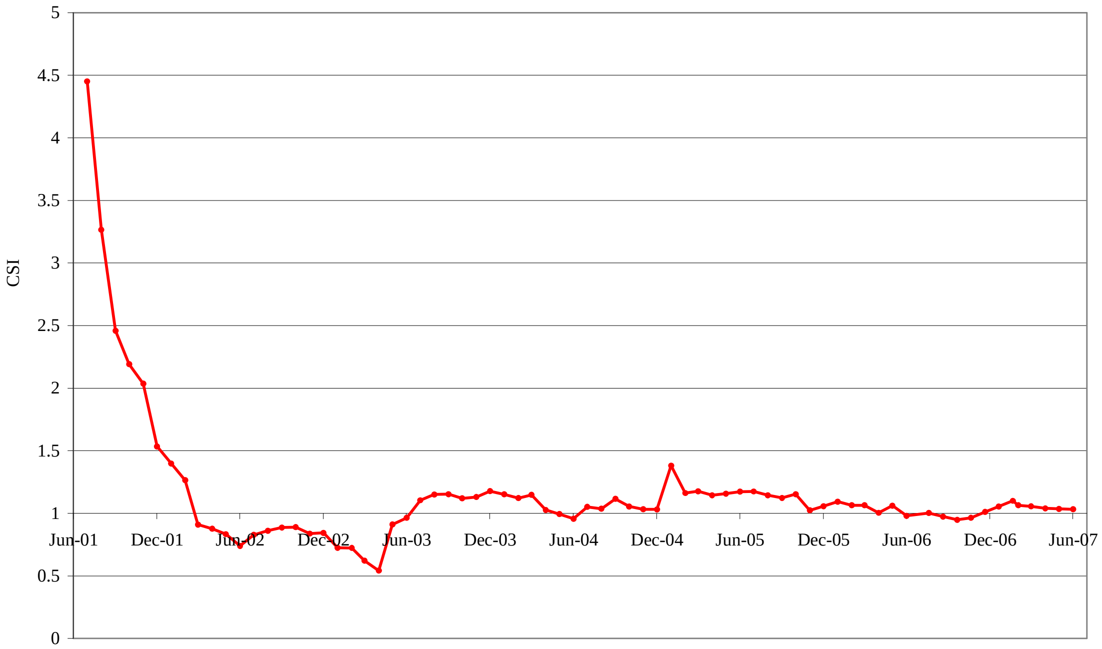



Fig. 5 – *Cost Schedule Index (CSI)*

Fig. 6 shows a plot of the Cost Performance Index (CPI) versus the Schedule Performance Index (SPI). Most of the points are located in the quadrant where the project is behind schedule and under spent. However, a few points are within the least desirable quadrant where the project is behind schedule and overspent. This situation prevailed prior to the award of the design consultancies in the first half of 2003.

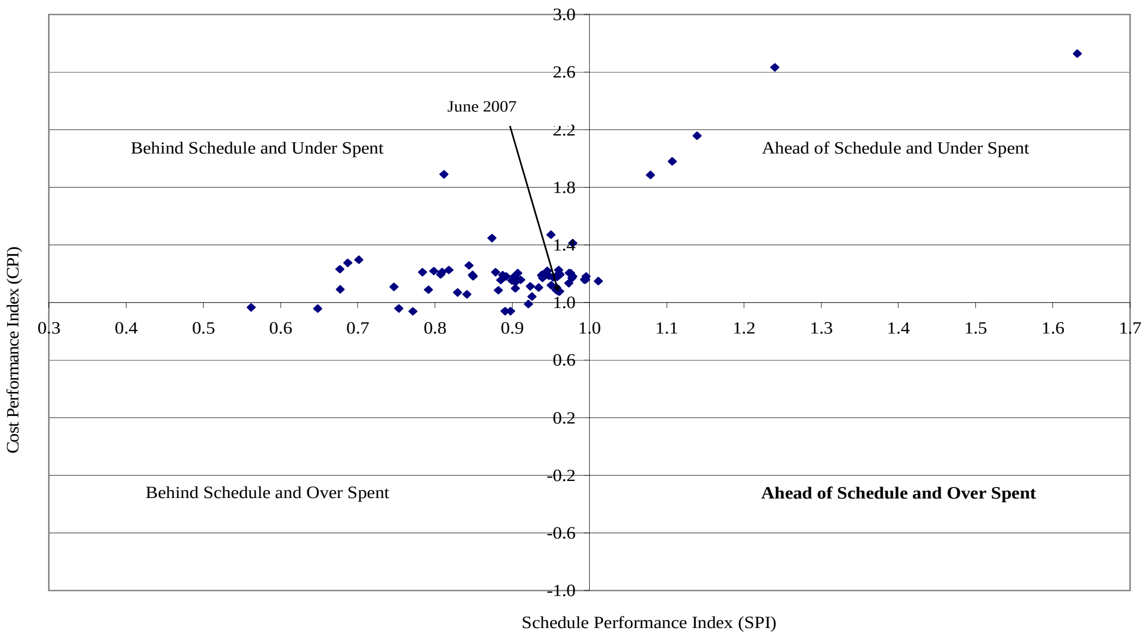



Fig. 6 – *Cost Performance Index (CPI) Vs. Schedule Performance Index (SPI)*

## 5 Conclusion

SV and the SPI shown in Fig. 3 and Fig. 4 indicate that the overall project progress was slightly behind schedule. But based on the value of CSI shown in Fig. 5, it could be concluded that the project was not having any serious problems with regard to project progress and cost status as the value of CSI is above or around 1.0. The earned value results point out that the management should focus mainly on the SV and the SPI.

Since EVMS was relatively new to Hong Kong, considerable effort by the Project Manager was required to educate the consultants, the resident site staff and the contractors on the application of EVMS and the requirements outlined in the Protocol for Integration of Programmes in preparing the PPs. The main difficulty encountered during the implementation of EVMS was the slow progress achieved by the contractors in getting their PPs approved early on to ensure a smooth and timely integration into the IP. Until such time, the consultant's construction programme was used to generate the earned value parameters for these works contracts. It was noted that this arrangement contributed some degree of error to the earned value parameters at the early stage of the project. It was also observed that some activities in the works programmes especially within the specialist contracts had large floats where the early finish and late finish dates were far apart. This too contributed to the negative SV as the earned value data have been generated based on the early finish dates rather than the late finish dates. Therefore, it is important to ensure that contractor's works programmes are not overly optimistic. However, it was possible to overcome the above challenges and successfully implement the EVMS as a project management tool for Hong Kong Science Park Phase 2 Development. It can be concluded that EVMS could be used as a project management tool for any multi-disciplinary infrastructure project as an efficient management tool for project monitoring and control.

### References


Barr, Z. (1996), "Earned Value Analysis: A Case Study", *PM Network*, December 1996, 31-37.

Christensen, D. (1999, Summer), "Using the Earned Value Cost Management Report to Evaluate the Contractor's Estimate at Completion", *Acquisition Review Quarterly*, 19, 283-296.

Dissanayake, P.B.G. (2007), "Application of Earned Value Management System for Hong Kong Science Park Phase 2 Development", *Proceedings of the PMI Global Congress Asia Pacific 2007, Hong Kong, 29-31 January 2007*.

Fleming, Q.W., Koppelman J.M. (2002), "Earned Value Management – Mitigating the Risks Associated with Construction Projects", *Program Manager, March – April 2002*, 90-95.

Fleming, Q.W. and Koppelman J.M. (1999, October), "The Earned Value Body of Knowledge", *Proceedings of the 30th Annual Project Management Institute 1999 Seminars and Symposium Philadelphia, Pensylvania, USA retrieved on 25 August 2006*.

Project Management Institute (2000), "A Guide to the Project Management Body of Knowledge (PMBOK®) (2000 Edition)", *Newton Square, PA, Project Management Institute*.

U.S. Department of Energy, Office of Management, Budget and Evaluation (2003, June), "Work Breakdown Structure", *Project Management Practices (Rev.E, June 2003)*.

# REFORMING HEI FOR THROUGH-LIFE SUSTAINABILITY OF CONSTRUCTION PROFESSIONALS

Thayaparan[1], M., Siriwardena[2],M.L, Kulatunga[3],U., Amaratunga[4],R.D.G. & Malalgoda[5],C.I.

[1]Researcher, School of the Built Environment, Maxwell Building, The University of Salford, The Crescent, Salford, Greater Manchester, M5 4WT, UK.
[1]E-mail: m.thayaparan@salford.ac.uk (*corresponding author*)
[1]Telephone: +44-161-2956939; Fax: + 44-161-2955011

[2]Lecturer, School of the Built Environment, The University of Salford, UK.
[2]E-mail: m.l.siriwardena@salford.ac.uk
[2]Telephone: +44-161-2957052; Fax: + 44-161-2955011

[3]Lecturer, School of the Built Environment, The University of Salford, UK.
[3]E-mail: u.kulatunga@salford.ac.uk
[3]Telephone: +44-161-2956943; Fax: + 44-161-2955011

[4]Professor, Associate Head of International Development, Head of Centre for Disaster Resilience School of the Built Environment, The University of Salford, UK.
[4]E-mail: r.d.g.amaratunga@salford.ac.uk
[4]Telephone: +44-161-2954471; Fax: + 44-161-2955011

[5]Postgrdaute researcher, School of the Built Environment, The University of Salford, UK.
[5]E-mail: c.i.malalgoda@edu.salford.ac.uk
[5]Telephone: +44-161-2956941; Fax: + 44-161-2955011

**Abstract**: BELLCURVE research project aims to promote the concept of 'lifelong university' in modernising Higher Education Institutes to be more responsive to labour market skills needs by continuously improving the skills and knowledge of the construction professionals. This paper briefly explains improving such responsiveness of HEIs through governance reform. Initial conceptual framework and the research methodology are illustrated. In responding to labour market skills requirements, the need for sector and context specific skills and knowledge to the construction professionals is emphasised. Lifelong learning on Disaster Management and Quantity Surveying sectors are considered as proposed case study areas.

**Keywords:** Disaster Management, Quantity Surveying, Lifelong Learning, Higher Education, Governance Reform

## 1. Introduction

The mismatch between graduate skills and labour market requirements has been identified as one of the main factors behind graduate unemployment and employer dissatisfaction, particularly in the Built Environment (BE) sector [1]. Some advances have been made in recent years to incorporate the roles of construction professionals into topics such as climate change and sustainability.

This paper is based on the EU funded project titled Built Environment Lifelong Learning Challenging University Responses to Vocational Education (BELLCURVE). BELLCURVE aims to modernise the Higher Education Institutions (HEIs) in order for them to be more responsive to the labour market skills needs. In doing so, the focus of this paper is on the role HEIs play in continuous improvement of the skills and knowledge on disaster management and quantity surveying among the construction professionals.

Section 2 provides an introduction to the BELLCURVE project and the need to reform governance followed by the initial conceptual framework and research methodology. Section 3 explains the case studies on lifelong learning in the built environment, with particular focus on disaster management and quantity surveying sectors. Finally the conclusion and the way forward of the project are provided.

## 2. Built Environment Lifelong Learning Challenging University Responses to Vocational Education (BELLCUREVE)

### *2.1 Introduction of BELLCURVE*

BELLCURVE (Built Environment Lifelong Learning Challenging University Responses to Vocational Education) is an EC (European Commission) funded research project currently being conducted at the Centre for Disaster Resilience, School of the Built Environment, University of Salford, UK, in collaboration with Department of Construction Economics and Property Management, Vilnius Gediminas Technical University, Lithuania and Department of Building Production, Tallinn University of Technology, Estonia.

BELLCURVE aims to promote the concept of 'lifelong university' in modernising Higher Education Institutes (HEI) to be more responsive to labour market skills needs. 'Lifelong university' encourages graduates who are either employed or unemployed to inform their university on labour market skill requirements. This will provide the opportunity for HEIs to be appropriately responsive to provide the required mix of skills for the labour market through training / retraining programmes. The rationale behind the existence of this project is mainly the issues associated with the mismatch between graduate skills and labour market requirements and its impact on graduate unemployment and employer dissatisfaction in the Built Environment sector. The universities are expected to offer innovative curricula, teaching methods and training/retraining programmes which include broader employment-related skills along with the more discipline specific skills. This requires a much clearer commitment by universities to lifelong learning opportunities. Lifelong learning presents a challenge, in that it will require universities to be more open in providing courses for students at later stages in the life cycle.

In addressing this, BELLCURVE considers 'student engagement' as a continuous through-life process rather than a temporary traditional engagement limited by the course duration. This through-life studentship defines the essence of the new innovative "Lifelong University" concept, whereby providing an opportunity for learners to acquire and develop skills and knowledge enabling responds to changing construction labour market needs on a continuous basis. Universities will not become innovative and responsive to change unless they are given real autonomy and accountability. This demands a reform in governance systems based on strategic priorities to respond labour market needs effectively while promoting lifelong learning agenda.

In this context, the project focuses on governance reforms in HEIs delivering Built Environment programmes across the EU, emphasising the ERASMUS programme's objective "to contribute to the development of quality lifelong learning and to promote high performance, innovation and a European dimension in systems and practices in the field". To achieve this objective, the existing interactions between the HEIs and the labour market are to be investigated and any improvements that could possibly be imposed on the nature of such interactions needs to be analysed. This demands the concept of lifelong university to be structured into a framework, identifying the possible components which will either directly or indirectly have an impact on the way the lifelong university has to function. The objectives of this project are therefore formulated as, to develop a framework for HEI's to promote the concept of lifelong university in capturing and responding to labour market skill needs in the Built Environment; to refine, test and validate the developed framework through existing HEI Built Environment programmes/ sectors; to provide recommendations on governance reform for HEIs to become 'continuing education centres' for graduates while responding to labour market skill needs.

### *2.2 The Need to Reform Governance*

The objective of this project is directly linked various strategies such as Lisbon strategy, EU 2020, Education and Training 2010, and Modernisation agenda for universities. Europe faces major structural challenges such as globalisation, climate change and an ageing population. The economic downturn has made these issues even more pressing. In order to address these challenges, Lisbon Strategy was set out, based on a consensus among Member States, to make Europe more dynamic and competitive, in a sustainable way and while enhancing social inclusion. The Lisbon strategy thus aims

to stimulate growth and create more and better jobs, while making the economy greener and more innovative [2]. The ’EU2020’ Strategy, the successor to the Lisbon Strategy, highlights education as a key policy area where collaboration between the EU and Member States can deliver positive results for jobs and growth. This strategy shows how the EU can come out stronger from the crisis and how it can be turned into a smart, sustainable and inclusive economy delivering high levels of employment, productivity and social cohesion [3]. If Europe is not to lose out to global competition in the education, research and innovation fields, this crucial sector of the economy and of society needs in-depth restructuring and modernisation. In this framework, higher education has an important role to play. Governments and higher education institutions are looking for ways to creating better conditions for universities. At the same time, the strategic framework for European cooperation in education and training (‘ET 2020’), adopted by the Council in May 2009, underlines the need to promote the modernisation agenda for higher education to improve the quality and efficiency of education and training (Council of the European Union).
The main areas for reform identified in the agenda are [3]:

- Curricular reform: The three cycle system (bachelor-master-doctorate), competence based learning, flexible learning paths, recognition, mobility.
- Governance reform: University autonomy, strategic partnerships, including with enterprises, quality assurance.
- Funding reform: Diversified sources of university income better linked to performance, promoting equity, access and efficiency, including the possible role of tuition fees, grants and loans.

Since BELLCURVE focuses on integrating the construction labour market skill needs to the modernisation agenda of the HEIs in the Europe, the vision to increase corporation between the higher education and the enterprises is the core of this project. Challenges faced by construction enterprises are fed to the European higher education agenda through the lifelong learning feedback loop, thereby ensuring the subject content of the European HEIs is dynamic, and of high quality, to address the market needs [4].One of the main areas of reform as identified in the modernisation of agenda is governance reform which is where the focus of the BELLCURVE lies. Governance of higher education has both direct and indirect links with the curriculum and funding systems. The reform in governance might therefore have an impact on the way a curriculum is developed and delivered and on the system of funding, and vice versa. In terms of response to the changing labour market requirements, the governance reform proposed through this project ensures that the HEIs will be more agile and dynamic in providing the appropriate mix of skills and knowledge, to the target audience at the appropriate time.

### *2.3 Conceptual Framework and Research Methodology*

*“A conceptual framework explains, either graphically or in narrative form, the main things to be studied – the key factors, constructs or variables – and the presumed relationships among them”* ([5];p18). Accordingly, this project has developed an initial conceptual framework and this will be continuously improved as the project progresses. The Figure 1 illustrates the initial conceptual framework.

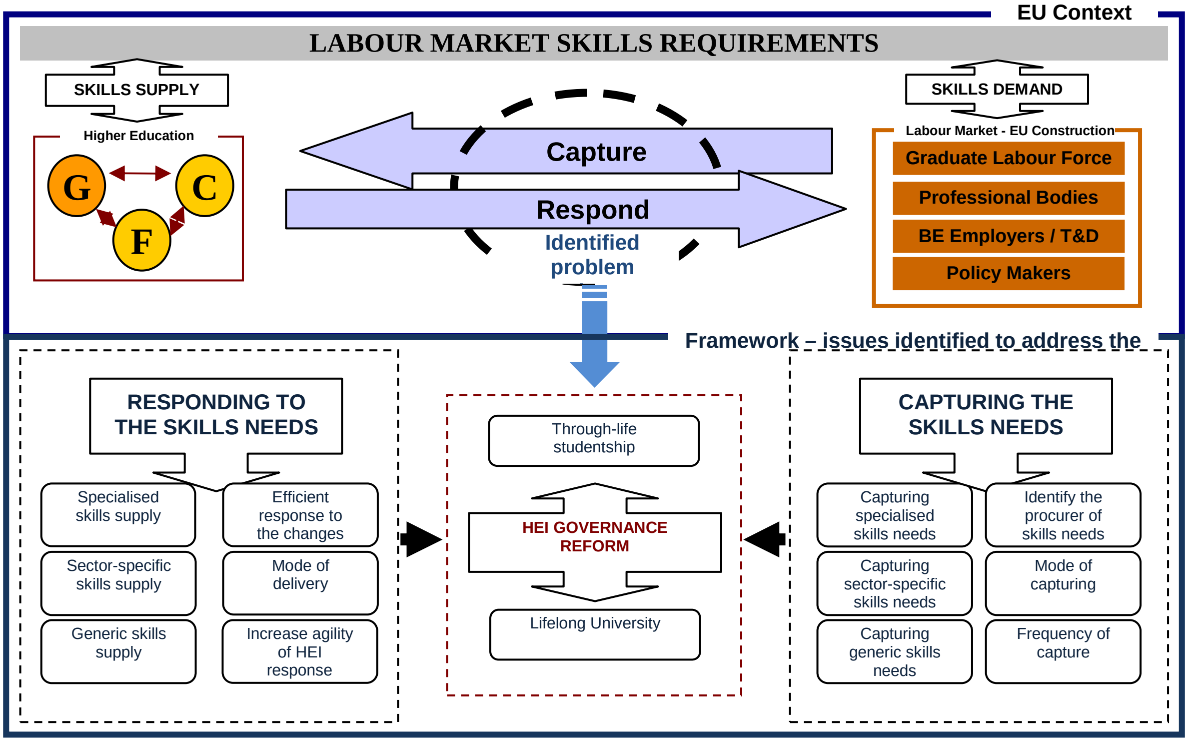



**Figure 1: Conceptual framework**

The labour market skills requirements for built environment professionals are perceived with the demand and supply side issues. The HEIs, being a body for knowledge creation and sharing, are expected to fulfil the labour market requirements. However, the problem was spotted within the process of capturing the skills requirements of the EU construction labour market and the process of appropriately responding to such requirements by HEIs, despite that HEIs are one of the major suppliers of skills and knowledge. BELLCURVE will address this problem by developing a framework to capture and respond to the skills requirement, giving particular attention to governance reform.

As shown in Figure 1, all three areas of reform that are Governance (G), Funding (F), and Curriculum (C) are identified as the major components to deal with within the higher education system. Nevertheless, the major focus of the research will be on governance reform where it aims to minimise the mismatch identified between the skills demand and the skills supply. In this regard, three major elements such as capturing skills need (Demand), Responding to the skills needs (Supply) and HEI Governance reform have been identified within the initial framework as shown in Figure 1. Key issues associated with these 3 elements will be analysed in order to address or minimise or resolve the identified problem. This will be done through 4 phases such as framework development, framework refinement, framework validation and research conclusion. Since this involves a process of framework development, a design science approach ([6],[7],[8]) is used as the most appropriate overall research methodology for this project.

Figure 2 below indicates how the 3 key stages of the BELLCURVE framework development process (i.e. framework development, framework refinement, framework validation) synchronise with the main phases of design science research methodology.

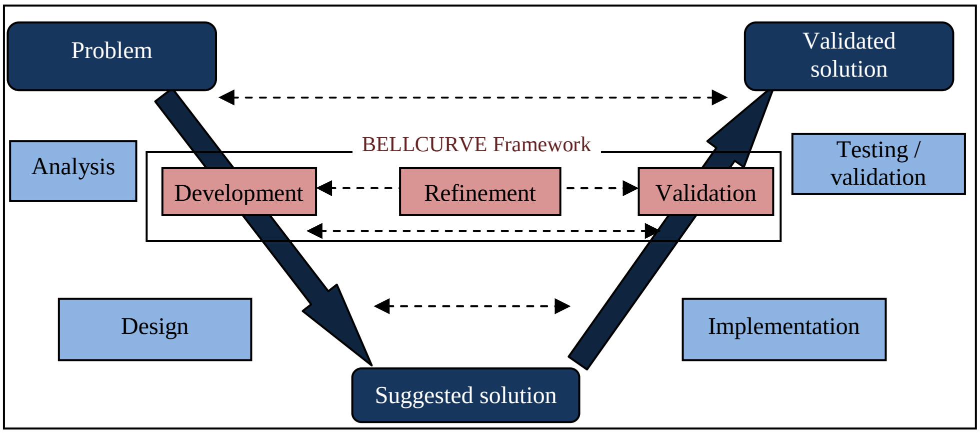



**Figure 2: BELLCURVE framework development thorough a design science approach**

In order to produce the initial input for the framework, a thorough literature analysis was conducted. This helped to identify the issues associated with the framework development. In addition, questionnaire surveys with graduates in the EU construction and interviews with Higher education lead officers, professional bodies, construction employers will further confirm the identified issues with the framework development. The developed framework will then be refined based on expert interviews and focus group. The purpose of this phase is to ensure that the developed framework captures all the important components associated with the identified research problem. Once the framework is developed and refined, then it needs to be validated for its practicality. A case study strategy has been chosen to achieve this purpose. Case studies will be conducted on built environment sectors such as quantity surveying, disaster management, civil engineering, and construction management in order to validate the framework. This paper introduces the case studies on disaster management and quantity surveying sectors which will be conducted by the University of Salford.

As a contribution of the research carried out in all 3 phases, recommendations will be provided on governance reform for HEIs to become continuing education centres for graduates while responding to labour market skills needs. These will be in the form of best practice guidelines and policy documents which will finally be disseminated to the stakeholders of the EU HEIs and construction labour market. This will ultimately lead the HEIs to provide lifelong learning to the graduates and in turn to become lifelong universities.

The next section explains the case studies selected by the University of Salford to validate the BELLCURVE framework.

## 3. Sustaining Built Environment Education: Case Studies on Lifelong Learning on Disaster Management and Quantity Surveying Sectors

Construction labour market, due to its labour-intensive, multi-disciplinary and highly fragmented nature, relies highly on the skills and competencies of its workforce. As it involves workers with various disciplinary backgrounds, the industry uses a wide range of technical and managerial skills. The labour market requirements of the construction industry are of dynamic nature, changing from time to time, due to various factors. In addition, the recent developments in the economic recession have made a reduction in the labour demand and vacancy levels for construction workers. The employers are thus trying to achieve the maximum utilisation with the minimum numbers of workers [4]. This has resulted in the existing construction workers to concentrate more on acquiring or developing new skills in order to retain in the industry and to meet various skills demand. Hence, possessing up-to-date skills and competencies has become a vital role in the construction sector.

### *3.1 Case Study Area 1: Disaster Management*

Disasters cause a considerable amount of damage around the world every year [9]. There has been an increase in the number of natural disasters over the past few years, and the impact in terms of human, structural and economic losses has increased considerably. Disasters create significant challenges to the EU which includes the loss of lives and hindering the social economic capacity of the member countries and also of the union as a whole. According to CRED & UNISDR [10] in the past 20 years, 953 disasters killed nearly 88,671 people in Europe, affected more than 29 million others and caused a total of US$ 269 billion economic losses. Compared to the rest of the world, economic loss per capita is high in Europe mainly because it is very densely populated. Disaster scholars who have investigated the relationship between development and vulnerabilities have identified that the impact of disasters are likely to increase in the future [11].

Considering the impact of disasters towards the built environment, it is evident that most of the material damages of disasters have been on engineering related facilities of the built environment such as buildings, roads, bridges, water supply plants, communication and power services, harbours, etc. and therefore clearing, salvaging, rehabilitation and reconstruction work fully or partly require serious effort of the construction sector. On the other hand, prevalence of disasters is related to how the built environment is planned, designed, built, maintained and operated [12]. Therefore the severity of the impact due to disasters can be linked to unplanned development of built environment. For example, though the Chile earthquake in February 2010 was far stronger than the one that struck Haiti in January 2010, the damage was much more contained comparatively [13].

In accordance with what has been discussed, the need to respond, recover, rebuild or reinstate the built environment affected by disaster can be identified as a major challenge for the countries affected by disasters. Construction industry and built environment disciplines have a major responsibility in responding to the above context. Apart from the physical construction process the knowledge and the experience of the construction professionals are essential in the disaster mitigation process [12].

One of the main reasons to focus on disaster management sector in the context of lifelong learning is due to the widespread agreement in the literature that disaster management is a continuous process and has no specific end point. According to Haigh and Amaratunga [14] the built environment discipline at each stage of disaster management process has invaluable expertise and key role to play in the development of society's resilience to disasters. Further, the construction professionals are generally expected to possess specific knowledge and expertise to act effectively in a disaster situation. The main reason is the peculiar nature of disaster reconstruction. Some factors that explains the unique nature of disaster reconstruction are short time for rebuilding; low cost; use of local resources; well developed communication links and relationships including trust and respect between parties [15]. In addition, disaster reconstruction differs from normal construction based on funding arrangements, project planning and monitoring, stakeholder involvement, and adaptation of disaster risk reduction strategies ([4], [12], [16]).

In this context, educating the construction professional to make them act efficiently and effectively in a disaster situation is vital. HEIs delivering Built Environment programmes have a major responsibility to provide specific skills and knowledge that are necessary to be acquired and apply in a disaster situation by the construction professionals. The lifelong learning opportunities further enhance this provision as it will facilitate the HEIs to act as a continuing education centres providing skills and knowledge in a dynamic environment.

### *3.2 Case Study Area 2: Quantity Surveying*

Quantity surveying skills sector has undergone significant changes over the past decade. Although, it was initially considered as the main profession for quantifying construction works in projects, quantity surveyors today undertake a spectrum of work ranging from providing investment appraisals to construction project management. Trends in building economics that have occurred during the latter part of the twentieth century made an impact on the changing roles of quantity surveying profession. This can be seen clearly in Table 1.

Table 1: *Chronology of developments in building economics (Ashworth [17]; p29)*

| Date | Building Economics | Other Developments | Practice |
|---|---|---|---|
| Pre - 1960s | Building Bulletin: Cost study (1957)<br>Building price books<br>RICS Cost Research Panel | Post-war building boom | Approximate estimating<br>Bills of quantities<br>Final accounts |
| 1960s | Const Studies of elements<br>Cost limits and allowances<br>Value for money in building<br>Building Cost Information Service<br>The Wilderness Group | Cost-benefit analysis | Elemental bills<br>Operational bills<br>Cut and shuffle<br>Cost planning<br>Standard phraseology |
| 1970s | Cost-in-use<br>Cost modeling<br>Contractor's estimating<br>Cost control | Measurement conventions<br>Data coordination<br>Building maintenance information<br>Buildability<br>Value-added tax/taxation<br>Bidding strategies<br>Computer applications<br>Undergraduate surveying degrees | Computer bills<br>Formula methods of price adjustment<br>Cash flow forecasting<br>Engineering and construction |
| 1980s | Life-cycle costing<br>Cost data explosion<br>Cost engineering techniques<br>Accuracy in forecasting<br>Value engineering | Coordinated project information<br>Procurement systems<br>European comparisons<br>Construction industry analysis<br>Postgraduate education<br>Single-point responsibility | Project management<br>Post-contract cost control<br>Contractual procedures<br>Contractual claims<br>Design and build |
| 1990s | Value management<br>Risk analysis<br>Quality systems<br>Expert systems | Facilities management<br>Commercial revolution<br>Single European market<br>Building sustainability<br>Information technology | Fee competition<br>Diversification<br>Blurring of professional boundaries<br>Development appraisal |
| 2000s | Benchmarking<br>Added value in building and design<br>Whole-life costing | IT in construction<br>Knowledge management | Rethinking construction<br>Lean construction<br>Facilities management |

In addition, changes in market, construction industry, client needs and profession posed threats and opportunities to the profession. A report titled 'The challenge of change' produced by the former Quantity Surveyor's Division of the RICS, provided warning to the profession that if the profession did not adapt to change then it would not exist in the future [18]. The quantity surveyors have subsequently begun to explore new potential roles. The traditional role of quantity surveyors, which is still practised by some and especially on small and medium sized projects, can be briefly described as a measure and value system [19]. Apart from the traditional roles, there were other evolving roles in the profession with increased importance and emphasis on meeting clients' needs. This involves quantity surveyors to work on procurement, design cost planning, whole life costing, value management, and risk analysis and management. Since the buildings have become more engineering services oriented, emphasis was placed on measurement, cost and value of such services. Other evolved roles have also included project and construction management, facilities management, contractual disputes and litigation [19]. The role of quantity surveyors are expected to develop in future due to the factors such as client focus, development and application of information and communication technologies, research and its dissemination, graduate capability and practice size. With particular focus on graduate capability Ashworth and Hogg ([19]; p13) say "*the number of graduates in quantity surveying is unlikely to change significantly in the short term from the reduced numbers experienced in the late 1990s. The relative shortage in supply has already had the effect of increasing salaries. Those graduates who have a good technical understanding, a broader use of* *business skills and a commitment towards lifelong learning are likely to be in high demand. For other graduates they will need to make themselves either more valuable to practices and contractors or less expensive"*. In this context, considering quantity surveying as a possible case study area for lifelong learning is dully justifiable, due to the changing and increasing skill requirements of the profession.

### *3.3 Supply side issues*

Built environment requires a diverse range of professionals teaming up to deliver the products and services. Therefore, education and training of such professionals is a major aim of most built environment (BE) educational programmes in HEIs, which has resulted in competency based education being a major influencing factor for the design and conduct of such programmes. There have been various efforts to promote integration between built environment disciplines including that of Latham [20] and Egan [21] reports. Three key barriers to interdisciplinary studies, namely faculty structures, staff relationships, resource pressures and the influence of external accreditation bodies have been reported [22]. Engagement with the industry is seen as a requisite to rapidly changing industry requirements ([23],[24],[25]). Construction Knowledge exchange initiative centred on Continuing Professional Development (CPD) and action learning are used in the industry [23]. Further, the lifelong learning is an emerging concept of acquiring new skills throughout the life of an employee. The CITB Construction Skills [26] has identified that more employers are supporting the lifelong learning and have begun to use associated products and toolkits. Little has been realised by the HEIs to adopt lifelong learning within their education system, despite the fact that lifelong learning is a core concept in modern education. In this context, it is vital to explore the role of HEIs in the lifelong learning and how could they continuously support the construction workers, throughout their life time, through training and re-training programmes. This research will also help HEIs to increase the duration of their student-engagement, which is presently limited to the course duration.

## 4. Conclusions and way forward

BELLCURVE research project aimed at modernising the HEIs thorough governance reforms in order for them to be more responsive to the labour market skills needs, considers skills requirements in the fields of disaster management and quantity surveying sectors. The peculiarities of disaster contexts require specific skills. The evolving and developing roles of quantity surveying profession also demand specialised skills. The HEIs are partially responsible in meeting these skills requirements for which lifelong learning has been proposed as an appropriate approach. The conceptual framework of BELLCURVE recognises the interwoven nature of governance, curriculum and funding, and therefore will inform and guide the future stages of the research accordingly.

## 5. Acknowledgments and notes

BELLCURVE research project has been funded with support from the European Commission. This publication reflects the views only of the authors, and the Commission cannot be held responsible for any use which may be made of the information contained therein.

Prof. Dilanthi Amaratunga is the principal investigator of this project. Dr. Chaminda Pathirage, Dr. Kaushal Keraminiyage, Dr. Udayangani Kulatunga & Mohan Siriwardena of the School of Built Environment, University of Salford, United Kingdom; Prof. Arturas Kaklauskas of Department of Construction Economics and Property Management, Vilnius Gediminas Technical University, Lithuania; and Prof. Irene Lill of Department of Building Production, Tallinn University of Technology, Estonia are co-investigators of this project. More information on BELLCURVE can be obtained from the project website http://www.disaster-resilience.salford.ac.uk/bellcurve.

### References

1. OECD - Organisation for Economic Co-operation and Development, 2008, OECD Employment Outlook, OECD.
2. Europa Press room (2010), available at http://europa.eu/press_room/press_packs/lisbon_strategy/index_en.htm [Accessed August 2010]
3. European Commission (EC) (2010), Available at http://ec.europa.eu/education/higher-education/doc1320_en.htm [Accessed August 2010]

4. Thayaparan, M., Siriwardena, M., Malalgoda, M., Amaratunga, D., Kaklauskas, A., and Lill, I. (2010), Reforming HEI to improve skills and knowledge on disaster resilience among construction professionals, The proceedings of the Construction, Building and Real Estate Research Conference of the Royal Institution of Chartered Surveyors (COBRA), Dauphine Université, Paris, 2-3 September 2010.
5. Miles, M.B. and Hurberman, A.M. (1994), Qualitative Data Analysis, 2nd ed., Sage Publications, London.
6. Van Aken, J. E. (2004). 'Management Research Based on the Paradigm of the Design Sciences: the Quest for Tested and Grounded Technological Rules', Journal of Management Studies, 41(2), pp. 219–246.
7. Van Aken, J.E., (2005), Management Research as a Design Science: Articulating the Research Products of Mode 2 Knowledge Production in Management, British Journal of Management,16, pp 19–36
8. Van Aken, J.E., Berends, H. and Van der Bij, H. (2007), Problem Solving in Organisation: A methodological Handbook for Business Students, Cambridge University Press, Cambridge.
9. Ofori, G. (2001), Construction in Disaster Management [online]. National University of Singapore.
10. CRED and ISDR, (2009), Floods, droughts and storms: a major threat for European countries [online]. Centre for research on the epidemiology of disasters and International Strategy for Disaster Reduction – ISDR. Available from: http://www.preventionweb.net/english/professional/news/v.php?id=8229 [Accessed 13 May 2010].
11. Aini, M.S. and Fakhrul-Razi, A. (2010 in press), Development of socio-technical disaster model, Safety Science, DOI.
12. Bosher, L., Dainty, A., Carrillo, P. and Glass, J. (2007), Built-in resilience to disasters: a pre-emptive approach, Engineering, Construction and Architectural Management, 14(5), Pp 434-446.
13. Witte, B. and Llana, S.M. (2010), Chile earthquake much stronger than Haiti's but far less damage. Why? [online]. Available from: http://www.csmonitor.com/ World/Americas/2010/0227/Chile-earthquake-much-stronger-than-Haiti-s-but-far-less-damage.-Why [Accessed 15 February 2010].
14. Haigh, R. and Amaratunga, D. (2010), An integrative review of the built environment discipline's role in the development of society's resilience to disasters, International Journal of Disaster Resilience in the Built Environment, 1(1), Pp. 11-24.
15. Masurier, J.L., Wilkinson, S., and Shestakova, Y. (2006), An analysis of the alliancing procurement method for reconstruction following an earthquake, Proceedings of the 8th US national conference on earthquake engineering, 18-22 April, California, USA.
16. Freeman, P.K. (2004), Allocation of post disaster reconstruction financing to housing, Building research and information, 32(5), Pp. 427-437.
17. Ashworth, A. (2004), Cost Studies of Buildings, 4th ed., Pearson Education Limited, England.
18. Powell, C. (1998), The Challenge of Change, Royal Institutions of Chartered Surveyors, UK.
19. Ashworth, A. and Hogg, K., (2007), Willis's Practice and Procedure for the Quantity Surveyor, 12th ed., Blackwell Publishing, UK.
20. Latham, M. (1994), Constructing the Team. London: The stationary office.
21. Egan, J. (1998), Rethinking Construction, [Online], Available: http://www.mosaicprojects.com.au/PDF/rethinking_construction.pdf, [Accessed March 2010]
22. Wood G. (1999), Interdisciplinary working in the built environment education, Education + Training, 41(8), Pp 373-380
23. Heesom, D., Olomolaiya,P., Felton,A., Franklin, R., and Oraifige, A. (2008), Fostering Deeper Engagement between Industry and Higher Education: Towards a Construction Knowledge Exchange Approach, Journal for Education in the Built Environment, 3(2), Pp 33-45
24. Lambert, R. (2003), Lambert review of business-university collaboration, London: HM Treasury.
25. Leitch,S. (2006), Prosperity for all in the global economy – world class skills, London: HM Treasury
26. CITB - Construction Skills (2009), CITB - Construction Skills: Annual Report and Accounts 2008 [Online], Construction Industry Training Board, Available from http://www.official-documents.gov.uk/document/hc0809/hc03/0326/0326.pdf. [Accessed May 2009].

# EMERGING RESEARCH ON WOMEN'S EMPOWERMENT IN POST DISASTER RECONSTRUCTION

Nirooja Thurairajah[1]
[1] PhD researcher, School of the Built Environment, University of Salford, M5 4WT,UK
n.thurairajah2@salford.ac.uk

Dilanthi Amaratunga[2]
[2] Professor, School of the Built Environment, University of Salford, M5 4WT, UK
r.d.g.amaratunga@salford.ac.uk

**Abstract**
The seeming randomness of the occurrence of disaster, its impact and uniqueness of events demand dynamic, real-time, effective and efficient solutions from the field of disaster management and thus making this topic necessary. Although disaster management efforts are designed to benefit both men and women, in real practice a larger share of benefits and resources goes to men and women continue to remain marginalised. The lack of involvement of both men and women in disaster management has exposed them to more potential dangers. Recent studies have reflected the need for gender consideration in disaster management and emphasised its importance in building disaster resilient communities. Inclusion of women's contribution to the post disaster reconstruction is a major concern for policy makers and researchers in building disaster resilient communities.

A mid-term review of the International Decade for Natural Disaster Reduction in the Yokohama world conference on natural disaster reduction highlighted the need for community involvement and empowerment of women at all stages of disaster management programmes as an integral part of reducing community vulnerability to natural disasters. Women are less likely to migrate to different places during post disaster phase due to their domestic responsibilities and other cultural systems which lead them to more vulnerable positions. This indicates the need to include women's needs and contribution in post disaster reconstruction. The concept of empowerment can be illustrated as a social process in addressing the above since it occurs in relation to others and as an outcome it can be enhanced and evaluated against expected accomplishments. In this context, this study seeks to identify and investigate the emerging research need in the knowledge on women's empowerment in the post disaster reconstruction in Sri Lanka. The study has been based on a theoretical as well as practical ideas obtained through a comprehensive literature review. It is expected that the study will serve as a platform for researchers who are interested in building disaster resilient communities in Sri Lanka.

*Keywords: Disaster, Empowerment, Reconstruction, Research, Sri Lanka, Women*

## 1. Introduction

In less than half a decade, the world has witnessed numerous catastrophes which took away many hundred thousands of lives and caused huge damages to the economy with unimaginable human sufferings. In addition to man made disasters, experts predict that the rising global temperature from climate change is yet to cause severe natural disasters around the world. One of the unprecedented disasters that occurred within the last decade was the 2004 Indian Ocean Tsunami. When a disaster strikes, the impact of the disaster depends mainly on the socio economic conditions of the particular community in addition to the scale of the cause itself. The extent of impact of the 2004 Tsunami on communities was immense mainly due to the hazards and vulnerabilities that existed in those affected areas. Hence, there is a need to include local needs and local knowledge on existing hazards and vulnerabilities in order to enhance the resilience of communities against future disasters. Although many governments take preventive actions to face disasters, the lack of incorporation of communities into these developments lead to further vulnerabilities within those areas.

Sri Lanka has faced many natural disasters such as floods, cyclones/wind storm, drought, land slide, epidemic, etc. During the nineteenth century, Sri Lanka has faced more number of flooding than other disasters. However, apart from Tsunami, more number of people were killed from windstorm during early decades. The Tsunami that occurred on the Boxing day in 2004 killed 35,322 people and displaced about a million people in 13 districts of the country (Government of Sri Lanka and Development partners, 2005). Besides the human loss, Tsunami caused extensive loss to dwellings and infrastructure, and interrupted livelihood activities and assets that were used for business purposes. According to a joint report of Government of Sri Lanka and Development partners (2005), the socio economic impact was the greater consequence of Tsunami as it compounded with the previously existing vulnerabilities.

The lack of involvement of both men and women in disaster management has exposed them to more potential dangers (Childs, 2006). A core and often neglected aspect of the post disaster reconstruction phase has been the lack of inclusion of women and other vulnerable groups into rebuilding and community development. In addition, studies have reflected the need for gender consideration in disaster management and emphasised its importance in building disaster resilient communities (Ariyabandu and Wickramasinghe, 2003; Delaney and Shrader, 2000). Ariyabandu and Wickramasinghe (2003) state that disasters affect women and men differently due to different roles and responsibilities undertaken by them, and the differences in their capacities, needs and vulnerabilities. In most of the instances, although disaster management efforts are designed to benefit both men and women, in practice a larger share of benefits and resources goes to men while women continue to remain marginalised.

Although, the official statistics on the social conditions of women in Sri Lanka indicate the possibility of utilising their capacity during the post disaster reconstruction women’s participation in this phase is often a neglected element especially at managerial level. In this context, the study based on which this paper is written, seeks to identify and investigate the emerging research need in the knowledge on women's empowerment in the post disaster reconstruction in Sri Lanka. This study has been based on a theoretical as well as practical ideas obtained through comprehensive literature review. In achieving the aim, the study initially describes the concepts involved in disaster management and later identifies the context of post Tsunami reconstruction and women’s status within post Tsunami settings in Sri Lanka. Further, the study elaborates the concept of empowerment and its role in enhancing women’s position. Finally, the study examines the promising research need in women’s empowerment during the post disaster reconstruction in Sri Lanka.

## 2. Post disaster reconstruction in Disasters management

The report by United Nations Environment Program (cited Eilperin, 2009) insists the need to protect the world from increasing global temperature which is currently projected to rise up to 6.3 degrees Fahrenheit by the end of the century. The report further states that even if the countries accomplish their most ambitious climate pledge they will not be able to reduce the temperature rise. This will increase the occurrence of tropical cyclones and heavy rainfall, and it is predicted that the sea level may rise by up to nearly a metre. This indicates the need to build disaster resilient communities in order to face the future. The current number of occurrences of natural disasters and the scale of their damage are drawing the attention of various sessions and meetings around the world. Within disaster management settings, the need for building disaster resilient communities has been increasingly highlighted since vulnerabilities and needs of communities can only be identified through a process of direct consultation and dialogue with the communities concerned as those communities can understand local realities and contexts better than outsiders (Haghebaert, 2007).

Although disasters and devastations are not new to Sri Lanka, the extent of devastation to properties and lives in one single event took the breath of many on the 26$^{th}$ of December 2004, not only within the country but also around the world. Even though Sri Lanka had the experience of dealing with natural disasters such as floods, landslides and occasional cyclones, Tsunami took many by surprise by its scale of devastation. In the process of building disaster resilience, the built environment plays a major role during the post disaster reconstruction. The post disaster reconstructioncan provide many opportunities in rebuilding social and economic status of the community (Thurairajah *et al.*, 2008). Hence, it is important to understand the post disaster reconstruction within disaster management and to examine the contexts of post disaster reconstruction in order to carry out effective developments for disaster resilience.

## 3. Disaster management

United Nations (2003) describes a disaster as a severe disruption of the functioning of a community or a society causing extensive human, material, economic or environmental losses which goes beyond the ability of the affected community or society to cope using its own resources. Delaney and Shrader (2000) have acknowledged that disaster management is a circular model in which disasters and development are intertwined. Although in disasters it is difficult to differentiate between different stages, policy makers and researchers have identified a disaster cycle for management purposes. Ariyabandu and Wickramasinghe (2003) state that a disaster management cycle includes the shaping of public policies and plans that either modify the causes of disasters or mitigate their effects on people, property and infrastructure. According to Ariyabandu and Wickramasinghe (2003), disaster management is a collective term encompassing all aspects of planning for and responding to disasters which includes both pre and post disaster activities. Although different scholars use various ways of naming the phases, generally the disaster cycle includes disaster mitigation and prevention, emergency, rehabilitation and reconstruction. Each phase in the cycle requires particular types of interventions and programming.

During disaster prevention, the activities that are related to elimination or reduction of the probability of the occurrence or reduction of the effects from unavoidable disasters are carried out (Delaney and Shrader, 2000). The mitigation process includes building codes, vulnerability analysis, zoning and land use management, building safety codes, preventive health care and public education. During the emergency phase, relief agencies focus on preventing additional loss of life through actions such as search and rescue. The rehabilitation phase that follows is characterised by medium term interventions such as construction of temporary shelters and provision of basic foods to the affected population. On the other hand, the reconstruction period includes the long-term, and often substantial, investments in rebuilding the physical and social infrastructure of affected regions (Delaney and Shrader, 2000).

In the recent past the number of disaster occurrences has increased (Altay and Green, 2006). This urges the policy makers and researchers to focus on enhancement of society's capacity to withstand disasters in order to reduce damage to both human and material resources. Within the disaster management settings, post disaster recovery and reconstruction can provide windows of opportunity for physical, social, political and environmental development not only to reconstruct the impacted areas, but also to improve the socio-economic and physical conditions of the impacted population in the long term (International Labour Organisation, 2003). However, in practice, too often disaster responses have not contributed to long-term development but they actually subvert or undermine it (Bradshaw, 2001; Anderson and Woodrow, 1998). This, results in lengthy post disaster reconstruction activities and the development opportunities are lost. Therefore, there is a need for built environment to adapt strategies to increase effectiveness and efficiency in post disaster reconstruction. Previous research found that despite the improvements in the emergency response to natural disasters, permanent reconstruction is often inefficiently managed, uncoordinated and slow to get off the ground (Jones, 2006). Further, in some occasions the constructed buildings or infrastructure do not provide the required level of service to the community or act as additional hazards for future disasters. This bespeaks the need to focus on the post disaster reconstruction and the involvement of affected communities in improving disaster resilience for future needs.

## 4. Post disaster reconstruction in Sri Lanka

The tidal waves that were created by a series of earthquakes that occurred in the sea near Sumatra, Indonesia on the 26$^{th}$ of December 2004 haunted many people. These tidal waves struck the Eastern, Southern and Northern coasts of Sri Lanka and also parts of Western coasts sweeping people away, causing flooding and destruction of infrastructure. The government of Sri Lanka has recognised the importance of having proper measures for rebuilding permanent infrastructure in order to carry out effective Tsunami recovery (Government of Sri Lanka and development partners, 2005).

The Tsunami damaged or destroyed more than 100,000 houses, which amounted to 13% of the total housing stock in coastal administrative divisions. Sri Lanka faces many challenges during its reconstruction phase. Although guaranteed external assistance seemed to be more than adequate for reconstruction costs, the subsequent problems with regard to relief payments, providing credit facilities, distribution of funds, coordination of reconstruction activities, and mismanagement of funds hindered the reconstruction progress (Jayasuriya *et al*., 2005). Further, poor coordination among domestic and external agencies has created serious problems in providing humanitarian assistance to people and in balancing sensitive issues in political arena. The study on post Tsunami recovery process in Sri Lanka (Ratnasooriya *et al.*, 2007) highlighted that housing reconstruction efforts have not succeeded in achieving the targets due to lack of consultation among all stakeholders, unawareness of those affected of their entitlements, confusion caused by the revision of the buffer zone and the resulting additional demand for housing, escalation of the cost of building materials, limitation on the capacity of the local construction industry, and the lack of sustained commitment of some of the donor agencies.

There was an extensive damage to the national roads by the Tsunami. Tsunami damaged a total length of nearly 700 km, representing nearly 5% of the total national road network since many national roads were located near to coastline. Even prior to Tsunami, an estimate of nearly 60% (Jayasuriya *et al*., 2005) of the entire road network was in a deteriorated condition due to lack of maintenance and damage and neglect during the 20 years of civil war, particularly in the north and east. The total damage of Tsunami to the road sector was estimated to be approximately US$50 million (Jayasuriya et al, 2005). It was found that the donors were quick in committing funds for road rehabilitation.

Another important infrastructure facility that was severely damaged was railway infrastructure. Tsunami caused disruption to rail services in the north eastern, eastern and southern corridors. However, damages to north eastern and eastern corridors were not severe compared to the southern corridor. The total damage to rail track, railway infrastructure and rolling stock was estimated to be US$ 26 million (Jayasuriya *et al*., 2005). During post Tsunami reconstruction process, the restoration of rail services in the southern corridor was considered to be a speedy achievement. In addition to the prevailing water shortage in certain areas, Tsunami affected the water supply and sanitation systems of those areas. Although many efforts had been taken to restore this service and provide additional supply, it was observed that due to the relocation of communities some existing networks need to be expanded and parts of them have become redundant (Ratnasooriya *et al.*, 2007). Under the water sector, the government highlighted the challenges with regard to sustainable maintenance of water/gully bowers and packaged water treatment plants, securing local counterparts funding, commencement of sanitation studies and development of sewerage for new settlements, further improvement of hygiene practice and strengthening significantly the sanitation sector (Government of Sri Lanka and development partners, 2005).

The report by Government of Sri Lanka and development partners (2005) states that the national construction industry does not have required number of contractors, equipment, skilled workforce, modern management practice and access to necessary finance in order to maintain the required speed of the entire Tsunami reconstruction work. Hence, the government proposed to engage and team up with international contractors and to provide training to local contractors in order to solve the above problems and to develop the affected community. Further, under the cash for work schemes by Non-Governmental Organisations (NGO), the community based organisations and small contractors were encouraged to get trained in labour based contracts to reduce the pressure on main contractors and to improve the quality of infrastructure.

Reconstruction process can play a major part in not only developing the affected area but also for future occurrence of disasters. The poor level of existing social and physical infrastructure facilities can turn hazards into disasters or the inappropriate development can itself be the cause of disasters. Hidellage (2008) emphasised that although many houses and infrastructure facilities were constructed, the effectiveness of their use do not provide adequate return to them (Asia Pacific Forum on Women Law and Development, 2006). This indicates the importance of considering the needs of local communities and including their local knowledge into reconstruction.

The social condition of Sri Lanka is much better than other South Asian neighbouring countries (Department of Statistics and Census, 2005) in the accomplishment of human development goals. Life expectancy in Sri Lanka is 72 years (Department of Statistics and Census, 2005). Further, the granting of free education facilities to the entire population has made a rapid upliftment in literacy levels, and given an opportunity for both the rich and the poor alike to pursue higher education. This makes the literacy rate in Sri Lanka to about 91.5%. According to Department of Census and Statistics (2009) the male literacy rate is 92.8% and female literacy rate is 90.3%. Hence, adequate measures should be taken to utilise the current social conditions in order to deliver a better environment for the community especially issuing their own resources. The inclusion of women can provide opportunities to develop required skills and income earning opportunities for their enhancement. The following section looks into the challenges that women face during post disaster reconstruction.

## 5. Women's challenges in post disaster reconstruction

Though, decisions regarding resource allocation, enforcement of land and building regulations, and investment on economic and social development are made with an intention to satisfy both genders many studies have highlighted the existing inequality in distribution chains and implementation phase. Most importantly, women's contributions to post disaster resilience have long been under estimated. Further, similar to using a generic term 'he' especially in written documents, linguistically females are subsumed under male. Pyles (2009) recognised that a core and often neglected element of disaster recovery has been the rebuilding and community development phase. Morrow and Peacock (1997) recognised that low income and marginalised communities are likely to suffer from downward spiral of deterioration after a disaster. Further, Sundet and Mermelstein (1996) found that high poverty rates in communities were associated with failures to survive. This can be seen in many occasions within the research on disaster. Therefore, in order to enhance women's position and to improve the post disaster reconstruction this section of the paper identifies and examines the problems that women face in post Tsunami reconstruction.

It has been widely stated that women have been mostly affected by the Tsunami and in many occasions they have been referred as vulnerable groups (Ariyabandu and Wickramasinghe, 2003; APWLD, 2006; Women's coalition for disaster management, 2005). While investigating the challenges that women face during post disaster it can be seen that, under different phases of disaster management cycle women's needs and challenges are different. However, while some needs may not continue to next phase the other continue to remain till a solution is given. Further, these challenges are interconnected. This section of the study mainly looks into the challenges that are directly related to disaster reconstruction apart from other challenges that are not directly related to disaster reconstruction such as poor access to health and other services, violence against women, other human rights issues, etc. Although the challenges under second category do not directly fall under reconstruction activities the link and interconnections between benefits and activities link them together.

During planning and designing of shelters, women find that poor procedures in capturing women's demands and their ways of living lead to construction of inappropriate houses (Women's coalition for disaster management, 2005). Further, guidelines used by the agencies/institutions were not clear about the definitions and about the people to whom that the support can be provided. For example, a government initiated agency which worked on disaster reconstruction claimed that it will encourage 'household-driven housing reconstruction' while it does not clearly define the word household, especially where extended families live in the same house (Women's coalition for disaster management, 2005). Women's coalition for disaster management highlighted the importance of providing compulsory criteria for including women in decision making bodies in order to avoid dismal representation. Since certain organisations such as the Village Rehabilitation Committees and Divisional and District Grievance Committees play a very important role in the reconstruction such as, taking responsibility for making the beneficiary lists, administration and disbursal of grants, and resolution of disputes, it is important to maintain representation from all in order to avoid any discrimination. Further, Women's coalition for disaster management (2005) emphasised that tsunami recovery, rehabilitation and construction process has to be based on the promotion and protection of rights rather than on a 'victim focus' which is limited on a welfare and dependency approach.

Further, time constraints in utilising the loans given for reconstruction process added additional burden to people who are in affected families. The eligibility for special loans was based on the capacity to pay back the loan rather than on the vulnerability of people whose accommodation has been destroyed by Tsunami. The increase consumption of alcohol by men lead to misuse of funds allocated for reconstruction purposes (Women's coalition for disaster management, 2005). This shows the need to consider the equal distribution of funds to both men and women and to maintain a monitoring mission in order to provide effective distribution of funds for the purpose.

According to the study by National committee on women (2006), it was found that female headed households face discrimination in terms of their civil status, family and community support, property ownership, and access to resources. Patriarchal systems that exist within the community suppress women's legal rights such as property rights and land titles. Since land titles are allocated to the head of household who is generally registered as being male made concerns over the entitlements of women within the reconstruction phase. Although the Sri Lankan law does not state that male is the head of household, the patriarchal systems tend to locate women in secondary position within the family based household (National committee on women, 2006). However, government payments and interventions in the post Tsunami context target the family based household as the unit that receives payments. Further, the head of household is eligible to receive these benefits. This leads to women in a more marginalised position. A woman is usually recognised as a head of household especially in the Tsunami affected families only when her spouse departed or who is unable to provide support to the family (National committee on women, 2006).

Women's participation in reconstruction of dwellings is not always anticipated. Many women from certain parts of the affected communities mainly carry out their income earning activities in their houses. Their lack of alternative housing, and also with other cultural factors forced them to live in marginalised positions. Lack of experience/knowledge on construction of houses and their dependency on others to complete the project led them to more vulnerable positions. Further, misuse of the constructed houses for women's by others made them more vulnerable. In addition, it was found that lack of knowledge on the usage of new technology within their houses did not offer any benefits to them (Hidellage, 2008).

Above problems that women face in post disaster reconstruction bespeak the need to address women's position within post disaster reconstruction. Bearing in mind the social conditions of women and the opportunities that the post disaster reconstruction can offer (Thurairajah *et al*., 2008) to the community, the concept of empowerment can be applied in order to enhance women's position. The following section describes the construct of women's empowerment prior to the discussion on its application in the post disaster reconstruction.

## 6. The construct of women's empowerment

The origin of empowerment as a form of theory was traced back to the Brazilian humanitarian and educator, Paulo Freire (1973 cited Hur, 2006) when he proposed a plan for liberating the oppressed people through education. Although, Paulo did not use the term empowerment, his emphasis on education as a means of inspiring individual and group challenges to social inequality provided an important background for social activists who were concerned about empowering marginalised people (Parpart, et al. 2003). The concept is conceived as the idea of power since it is closely related to changing power by gaining, expending, diminishing, and losing (Page and Czuba, 1999).

## 7. Meanings of empowerment

Empowerment has been defined in several ways by many authors for different contexts. Even though the meaning of the terms delegation and empowerment may look similar they are different to each other. While describing empowerment, Nesan and Holt (1999) state that, empowerment is more a philosophy than a set of tools or management principles to be readily applied to business organisations. Though the term empowerment has been used frequently in management literature, it is been defined in several ways by organisations and scholars. Accordingly, empowerment is a diverse concept which is open to a number of different interpretations. During the last decade the term has become a widely used word in the social sciences across many disciplines such as community psychology, management, political theory, social work, education, women studies, and sociology (Lincoln et al. 2002).

Handy (1993) explains empowerment as encouraging people to make decisions and initiate actions with less control and direction from their manager. Avrick and colleagues (1992) state empowerment as giving authority commensurate with their responsibilities to initiate positive change in their organisation. This demands total commitment, involvement, support and trust from management. While explaining about empowerment, Rubinstein (1993) states that every individual is responsible for acceptance or rejection of the quality of prior work; self inspection and control of current work; and acceptance or rejection of finished work. In the above studies the authors have explained the term from a similar perspective within the management of organisations.

In a study within the construction industry, Nesan and Holt (1999) collectively define empowerment as the process of giving employees the authority to take decisions, relating to their work processes and functions, and within the limits provided by management, but requiring them to assume full responsibility and risk for their actions. Further they state that, empowerment is not an act or incident that can visibly or physically happen, but it is employees' perception or realisation that they believe in, and control what happens to their work processes; and that they are capable of controlling those processes efficiently. Even though Eylon and Bamberger (2000) view empowerment from two different perspectives: a cognition (psychological approach) or social act (sociological approach), in their gender related study, they accept that empowerment cannot be neatly conceptualised as either a cognition or social act.

## 8. Women's empowerment

The concept of women's empowerment begins from the understanding that women's empowerment is about the process by which those who have been denied the ability to make strategic life choices can acquire such an ability (Kabeer, 1999). According to Magar (2003) women's empowerment is an outcome of a process whereby individual attitudes and capabilities, combined with collaborative actions, and reciprocally influenced by resources results in a transformation to the desired achievements. Kabeer (1999) describes women's empowerment as a process by which women acquire the ability to make strategic life choices in terms of three interrelated dimensions that include resources (preconditions), agency (process) and achievements (outcomes).

Magar, in her study on empowerment approaches to gender based violence, constructed a framework using the findings from earlier studies (Kabeer, 1999; Stein, 1997). In her framework, she highlights individuals' attitudes and capabilities, which allow participation in various types of collaborative behaviour which leads to empowerment. Under this framework the empowerment process comprises of two levels: the level of individual capacities observed in individual attitudes and capabilities and the level of group capacities (Magar, 2003). Individual attitudes (self-esteem and self-efficacy) along with specific types of skills, knowledge, and political awareness, are key ingredients to achieving empowerment at these two levels. Self-efficacy or agency is defined as the experience of oneself as a cause agent, not in terms of skills but rather in terms of one's judgment of what one can do with whatever skills one has (Bandura, 1995).

In a study within rural India Roy and Tisdell (2002) refers women's empowerment as a process by which women can gain power to diminish the forces of institutional deterrents considerably to their development. Further, they state that the right to land is an important factor for women's empowerment as it is a more permanent source of income and it indicates that the person has a long-term interest in preserving the fertility of the land and therefore will be interested in investing in land.

Furthermore, when income is higher this will increase the person’s capacity to spend on consumption of food, housing, education, health and other necessities which will uplift the living conditions.

In Sri Lanka, the concept of empowerment is becoming an important concept to address women’s difficulties especially to those who are in more vulnerable state. The introduction of Millennium Development Goals made a tremendous impact on the way the construct of women’s empowerment has been viewed at political level to enhance women’s representation and participation in productive activities. Currently, non-governmental organisations are working on women’s development in order to address women’s safety and health related issues and, few of them are involved in enhancing their status with regard to their incoming earning activities particularly within the small cottage industries. Women’s representation in Sri Lankan Parliament stands at 4.05% and in Local Government 1.9 %. The percentage of women in Municipal Councils and the Urban Councils is 3.0% and 3.4% respectively (Women's International League for Peace and Freedom, 2008). In Sri Lanka, Ministry of Child development and women’s empowerment is the governmental ministry which is responsible for women’s development. Under this ministry, National Committee on Womendrafted a women’s rights bill in 2003 covering a wide spectrum of activities for district representation and powers to resolve women’s issues. However, the bill was subjected to many debates and ultimately went out of the scene with the dissolution of Parliament in 2004.

In a study on the difficulties of women who work in factories, the concept of empowerment was considered while exploring the violence against women free trade zone. Factory women who participated in this research (Hancock, 2006) rated violence against women as a key way to measure women's empowerment. This indicates seriousness of the problem itself at the societal level and provides a firm need for women's empowerment. Although, government’s one of the ministry has interest on women’s empowerment they do not have any legislations specifically on women’s empowerment. However, compared to earlier days enhancement of women’s position within the society has been recognised and interest to take further actions have been considered to lessen the gender based violence which act as a major barrier to women’s empowerment.

According to a study on rural women in Bangladesh, Parveen and Leonhäuser (2004) describe empowerment as an essential precondition for the elimination of poverty and upholding of human rights, in particular at the individual level it helps building a base for social change. A study on empowering women through community development approach views empowerment as a multidimensional and interlinked process of change in power relations to expand individual choices and capacities for self-reliance (Mayoux, 2003 cited Acharya et al., 2005). In order to address females submission, silence, sacrifice, inferiority and obedience, problems in female illiteracy, lesser mobility of women on employment in Nepal, the project considers the concept of empowerment of women through facilitating self-help group activities which are truly self-reliant, literacy programmes, group savings and credit programmes. Although many organisations work on women's empowerment, the application of this concept in developing countries has its interconnected key issues, such as the role of culture, tradition, education, religion and economics.

## 9. Discussion: Women’s empowerment in post disaster reconstruction

The unprecedented Tsunami affected many buildings and infrastructure such as houses, roads, bridges, railway tracks, fishing ports, landing centres, small scale industrial units, hotels located near to seaside, irrigation systems, etc. In addition to the physical damages, the increase in gender violence and gender insensitive procedures during the post disaster reconstruction led women to a more vulnerable state. Further, women’s dependency on other sections of the community for support in reconstruction, and management of their finance, the patriarchal systems that exist within the society also led them into a more marginalised position in the post disaster reconstruction. In certain cases, for example, women’s needs and demands weren’t included in the planning of houses. The existing customs and cultural systems with regard to legal rights of land and properties together with administrative process made women into economically disadvantaged positions. In addition to above, there was a need to provide sustainable income generation for living for women and their dependents.

Acar and Ege (2001) found that during post disaster phase, there is a ‘double suffering’ on women, created by natural as well as social, economic and cultural forces which shape the way they experience natural disasters. Further, they found that women in patriarchal societies, developing economies and traditional cultural contexts are precisely in this position. Acar and Ege (2001) recognised that gender-based prejudices, patriarchal values and behaviour patterns are likely to take new vigour and scope during post disaster context when the conditions of mass anxiety, helplessness and insecurity felt in the face of life threatening disasters. In addition, they found that this reinforces the communities to follow the same old familiar patterns of behaviour and tends to reject the differences.

Enarson and Fordham (2001) state that exclusion of women’s full participation in forming disaster will-resilient communities will hardly lead to its success. Further, they emphasised that the reconstruction of safer communities cannot be done with elites or technical specialists, but through regular consultation with women across deep divides of class and culture and of women and men working together toward a common future. Exclusion of women will create ‘gender-blind’ post disaster reconstruction. This will simply build women’s subordination which will leave the communities even more vulnerable to future disasters.

Due to lack of experience and exposure in handling the post disaster reconstruction process women tend to fall into more susceptible positions. However, identification of their need and their capacity to contribute to the reconstruction process can uplift their current economic and social position within the society. During reconstruction phase, expertise from those who are knowledgeable about women’s human rights in the areas of resource use, work and employment, immigration, housing, health and reproduction are required in order to build a more resilient community (Enarson and Fordham, 2001). Since the Institute for Construction Training and Development of Sri Lanka provide educational courses and training to communities to develop their construction related skills, an initiative focusing on the process of bridging the available support and women’s interest and knowledge can develop women’s empowerment using the reconstruction phase as a platform.

Bearing in mind the literacy rate of women and other social conditions of women in Sri Lanka, and the introduction of policies and laws such as Sri Lanka Disaster Management Act, No 13 of 2005, Tsunami Housing Policy and Tsunami (Special Provisions) Act; and amendments and reinforcement of laws in the registration of death and declarations against gender based violence could create a better environment for women’s empowerment. Further, the operation of Ministry of Child development and women’s empowerment, Women’s bureau and National committee on Women can contribute to for its development. In addition to above, many non-governmental organisations are emphasising the gender consideration in reconstruction process. However, it is rarely evident that any initiatives have been undertaken to empower women within reconstruction from built environment’s perspective. Since post disaster recovery and reconstruction can provide windows of opportunity for physical, social, political and environmental development not only to reconstruct the impacted areas, but also to improve the socio-economic and physical conditions of the impacted population in the long term (International Labour Organisation, 2003) efforts should be taken to bridge the gap existing between women’s empowerment and community resilience through this phase. Within this gap further studies can be done on how women’s contributions can be incorporated into the post disaster reconstruction that can enhance the disaster resilience while improving women’s position within the community. While carrying out this research a detailed study needs to be done on the promoters and hindrancesof women’s empowerment that exists within the post disaster reconstruction in addition to the facilitators. The existing systems can play a major role in developing the application of the concept. Since the application can not be done on a similar way through out the country a framework can be developed on how the concept can be adopted into the national and local levelespeciallyby considering various factors that lead the difference.

The marginalised positions of women bespeak the strong need to uplift them socially and economically, and since the post disaster reconstruction can offer opportunities to reconstruct a better environment, women’s representation in reconstruction process need to be strongly supported. In this context, the application of the concept of empowerment can provide an environment to women to acquire the ability to make strategic life choices in terms of their own life which will uplift their status and reduce their vulnerabilities. This will not only uplift the conditions of women but also the community around them and improve the post disaster reconstruction. Although, currently many projects and studies have been carried out on women's empowerment in livelihood sector, the utilisation of reconstruction process for women's empowerment is hardly evident within the literature available in this area apart from its recognition for further development. Therefore, on one hand studies need to focus on linking the opportunities that the post disaster reconstruction can offer and the capacities of women that can be utilised for the above. This can lead to satisfying the needs of the construction sector and the needs of women from the affected community not only for the reconstruction phase but also for their living. Moreover, further studies can be carried out on women's capacity development within the built environment.

## 10. Conclusion

For the first time in history, Sri Lanka experienced the devastating effects of a Tsunami which was caused by giant waves created from massive earthquakes in Sumatra. In addition to huge physical damages it caused severe human sufferings. However, the post disaster reconstruction offers opportunities to the communities to enhance their positions. This study recognises that women's empowerment is one of the key factors in the reconstruction process towards sustainable ways of living and that can address vulnerabilities, promote justice and reduce other risks. Although this study specifically focused on women, the study does support that both women and men must be empowered as disaster decision makers at all levels. However, careful considerations should be made in cases where men's and women's interests are different and women's political voice is still too rarely heard. This can not only develop women but also others who are dependant on them.

In the process of empowerment, women need to be facilitated for self reliance. Since the post disaster reconstruction can offer opportunities to build disaster resilient communities it is wise to search for hidden resilience displayed by communities affected by disasters through themselves. Therefore, further studies should be undertaken without any delay to bridge the gap existing between women's empowerment and community resilience by researching on how women's contributions can be incorporated into the post disaster reconstruction which can enhance disaster resilience while improving women's position in the community. This will require conscious strengthening of local knowledge and wisdom while finding solutions to problems. This process can develop economic possibilities while supporting political, social and economic empowerment of women without much of dependency on externally parties in the long run. This can lead government to take initiatives to develop women and reduce the vulnerabilities.

In this context, while considering the current conditions and support that exists within Sri Lanka, there is a strong need to research into the ways in which women can be empowered. Research into women's empowerment in the post disaster reconstruction can not only contribute to the knowledge but also add tremendous value to increase women's self reliance which can decrease their dependency that in many occasions leads them to vulnerable occasions. Similar to the creation of the Ministry of National Disaster Management and Human Rights to cope with future disasters the Tsunami can act as a catalyst in developing women's empowerment during the post disaster reconstruction in Sri Lanka.

## References

Acar, F. and Ege, G. (2001) *Women's Human Rights in Disaster Contexts: How can CEDAW help?*. United Nations. Available from: http://www.un.org/womenwatch/daw/csw/env_manage/documents/EP4-2001Oct25.pdf [Accessed 30th September 2009]

Acharya, S., Yoshino, E., Jimba, M. and Wakai, S. (2005) Empowering rural women through a community development approach in Nepal. *Oxford University Press and Community Development Journal*. 42(1). Pp. 34-46.

Altay, N. and Green, W.G. (2006) OR/MS research in disaster operations management. *European Journal of Operational Research*. 175. Pp. 475 – 493.

Anderson, M. and Woodrow, P. (1998) *Rising from the Ashes: Development Strategies in Times of Disaster*. 2nd ed. London: IT Press.

Ariyabandu, M.M. and Wickramasinghe, M. (2003) *Gender dimensions in disaster management*. Colombo: ITDG South Asia Publication.

Asia Pacific Forum on Women Law and Development (2006) *Tsunami aftermath- Violations of women's human right Sri Lanka.* Colombo.

Avrick, A.J., Manchanda, Y.P. and Winter, E.F. (1992) Employee empowerment: world class test team. *Proceedings of American production and Inventory Control society*. USA.

Bandura, A. (1995) *Self-efficacy in changing societies*. New York: Cambridge University Press.

Bradshaw, S. (2001) Reconstructing Roles and Relations: Women's participation in reconstruction in post-Mitch Nicaragua. *Gender and Development*. 9(3). Pp. 79 – 87.

Childs, M. (2006) Not through women's eyes: photo-essays and the construction of a gendered tsunami disaster. *Disaster Prevention and Management*. 15(1). Pp. 202-212.

Delaney, P. L. and Shrader, E. (2000) *Gender and Post-Disaster Reconstruction: The case of Hurricane Mitch in Honduras and Nicaragua*. Report prepared for the World Bank. Available from: www.anglia.ac.uk/geography/gdn [accessed on 25/10/2007].

Department of Census and Statistics. (2009) *Bulletin ofLabour Force Statisticsof Sri Lanka*. Available from: http://www.statistics.gov.lk/samplesurvey/Bullet2009q2.pdf [Accessed 25th September 2009]

Department of Census and Statistics. (2005) *Social Conditions of Sri Lanka*. Available from: http://www.statistics.gov.lk/social/social%20conditions.pdf [Accessed 20th September 2009]

Eilperin, J. (2009) New Analysis Brings Dire Forecast Of 6.3-Degree Temperature Increase. *The Washington Post.* Available from: http://www.washingtonpost.com/wp-dyn/content/article/2009/09/24/AR2009092402602.html [Accessed 10th October 2009].

Enarson, E. and Fordham, M. (2001) From women's needs to women's rights in disasters. *Environmental Hazards.* 3. Pp. 133 – 136.

Eylon, D. and Bamberger, P. (2000) Empowerment cognitions and Empowerment acts. *Group and Organisation Management*. 25(4). Pp. 354-372.

Government of Sri Lanka and Development partners, (2005) *Post Tsunami Recovery and Reconstruction*. Available from: http://siteresources.worldbank.org/INTTSUNAMI/Resources/srilankareport-dec05.pdf [Accessed 20th September 2009].

Hancock, P. (2006) Violence, Women, Work and Empowerment. *Gender, Technology and Development*. 10(2). Pp. 211-228.

Handy, M. (1993) Freeing the vacuums. *Total Quality Management*. June 1993. 11.

Haghebaert, B. (2007) Working with vulnerable communities to assess and reduce disaster risk. *Humanitarian Exchange.* 38. Pp. 15-16.

Hidellage, V. (2008) Women's state within post Tsunami reconstruction. [Interview] (Personal communication, October 2008)

Hur, M.H. (2006) Empowerment in terms of theoretical perspectives: Exploring a typology of the process and components across disciplines. *Journal of Community Psychology*. 34 (5). Pp 523–540.

International Labour Organisation. (2003) Gender in Crisis Response. Report prepared for ILO InFocus Programme on Crisis Response and Reconstruction. (Available online: http://www.ilo.org/public/english/employment/crisis/download/factsheet9.pdf [Accessed on 23/05/2008].

Jayasuriya, S. et al. (2006) *Post-Tsunami Recovery: Issues and Challenges in Sri Lanka*. ADB Researcher paper series. No 71. Available from: http://www.adbi.org/files/2005.10.dp39.tsunami.recovery.srilanka.pdf [Accessed 20th September 2009].

Jones, T. L. (2006) Mind the Gap! Post-disaster reconstruction and the transition from humanitarian relief. London: RICS.

Kabeer, N. (1999) Resources, agency, achievements: Reflections on the measurement of women's empowerment. *Development and Change*. 30. Pp 435– 464.

Lincoln, N.D., Travers, C., Ackers, P. and Wilkinson, A. (2002) The meaning of empowerment: The interdisciplinary etymology of a new management concept. *International Journal of Management Reviews*. 4(3). Pp 271–290.

Magar, V. (2003) Empowerment approaches to gender-based violence: women's courts in Delhi slums. *Women's Studies International Forum*. 26 (6). Pp 509 – 523.

Morrow, B.H. and Peacock, W.G. (1997) Disasters and Social Change: Hurricane Andrew and the Reshaping of Miami?'. In: Peacock, W.G., Morrow, B.H. and Gladwin, H. (eds) *Hurricane Andrew: Ethnicity, Gender and the Sociology of Disasters.* Pp. 226–42. New York: Routledge.

National committee on women. (2006) *Gender and the Tsunami*. Colombo: United Nations Development Fund for women.

Nesan, L.J. and Holt, G.D. (1999) *Empowerment in Construction*. Hertfordshire: Research Studies press limited.

Page, N. and Czuba, C.E. (1999) Empowerment: What is it?. *Journal of Extension*. 37(5). Pp 24–32.

Parpart, J.L., Rai, S.M. and Staudt, K. (2003) Rethinking empowerment: Gender and development in a global/local world. New York: Routledge.

Parveen, S. and Leonh¨auser, I. (2004) Empowerment of Rural Women in Bangladesh - A Household Level Analysis. Conference on *Rural Poverty Reduction through Research for Development and Transformation* 5th - 7th October 2004, Berlin: Deutscher Tropentag.

Pyles, L. (2009) Community organising for post disaster social development: Locating social work. *International social work*. 50(3). Pp. 321-333.

Ratnasooriya, H.A.R., Samarawickrama, S.P and Imamura, F. (2007) Post Tsunami Recovery Process in Sri Lanka. *Journal of Natural* Disaster Science. 29(1). Pp 21-28.

Roy, K.C. and Tisdell, C.A. (2002) Property rights in women's empowerment in rural India: a review. *International journal of social economic.* 29 (4). Pp. 315-334.

Rubinstein, S.P. (1993) Democracy and quality as an integrated system. *Quality Progress*. September edition. USA. Pp. 51-55.

Stein, J. (1997) *Empowerment and women's health: Theory, methods and practice*. London: Zed Books.

Sundet, P. and Mermelstein, J. (1999) Predictions of rural community survival after a natural disaster: Implications for social work practice. *Research in social work and disasters.* Pp. 57-70.

Thurairajah, N., Amaratunga, D.G. and Haigh, R., (2008) Post disaster reconstruction as an opportunity for development: women's perspective, CIB W89: International Conference in *Building Education and Research,* 10th - 15th February 2008, Kandalama.

United Nations International Strategy for Disaster Reduction (2003) *Turning the tide on disasters towards sustainable development.* 2003 World Disaster Reduction Campaign. Geneva.

Women's coalition for Disaster management – Batticaloa (2005) Concerns of displaced women's welfare and rights. *Options*. 1. 25th of September. Pp. 38-41.

Women's International League for Peace and Freedom (2008) Sri Lanka: Women, Political Empowerment and Concerns. *Daily Mirror*. 8th of September. Available from: http://www.dailymirror.lk/DM_BLOG/Sections/frmNewsDetailView.aspx?ARTID=25230 [Accessed 10th October 2009].

# GLOBAL CHANGE AND ITS EFFECT ON COASTAL WATER RESOURCES – CASES OF THE ASIA PACIFIC REGION

A/Prof. Gregory Shahane De Costa
Open Polytechnic of New Zealand
Gregory.decosta@openpolytechnic.ac.nz

Prof. Bithin Datta
Department of Civil Engineering, I.I.T. Kanpur, India

A/ Prof Mark Porter
University of Southern Queensland, Australia

Dr. Hamaguchi
DPRI, Kyoto University, Japan

**Abstract**

While global change is directly affecting water resources, it is also well know that anthropogenic impacts on hydro-geological systems can result in long term harm and the degradation of the resource if they are not adequately managed. While this witnessed around the world, management options to prevent increasing damage to the surrounding environment are being developed on an individual site basis. Salinity intrusion with the eventual degradation of both land and water quality is one of the most common examples of this type of problem. Due to increase in demand to fresh water resources, ground water is being exploited, and coastal ground water systems hydraulically connected to the ocean necessarily has to cope with salinity intrusion. This paper presents our observations and analyses of salinity intrusion at selected areas in the Asia Pacific region, namely in New Zealand, Australia, Japan and India. It discusses the characteristics of each site, simulates some sites using state of the art soft ware, and analyzes the impacts on the environment. It also presents the management practices used to mitigate the resulting damage on the environment at each site.

*Keywords – Salinity intrusion*

## 1. INTRODUCTION

The most significant impact of global change is its effect on water resources. A United Nations report has warned that, coastal degradation will put at risk ecosystems which support over half the world's economies, unless coastal management strategies are implemented. It goes on to say that "terminal" disaster looms in many coastal areas unless "unless we introduce much more effective management immediately." “Coastal marine ecosystems have declined progressively in recent times due to the increase in human populations and their accompanying development of coastal regions. This, accompanied by increasing climate change, is putting enormous pressure on the coastal ecosystems” say the authors of the report. "By 2050, 91% of the world’s coastlines will have been impacted by development," It says that, "We believe that use of scientific and traditional knowledge, together with better understanding of the economic value of healthy coastal ecosystems, can help change the political discourse that eventually determines societal pressures. Although the situation is dire, there is reason for hope. Our understanding of the ecological functioning of the coastal ocean is quite good, and we have a basic kit of useful management tools at our disposal."

Coastal aquifers are important sources of water in coastal regions. As population density in many coastal areas increased, need for fresh water also increased. Along with the population, industrial and agricultural growths in these areas accelerate the exploitation of groundwater. Over exploitation of groundwater from coastal aquifers may result in intrusion of saltwater in the aquifer. This is mainly due to excess withdrawal of groundwater compared to the recharge rate, and unplanned pumping locations and pumping patterns. Saltwater intrusion often results in loss of fresh potable water, loss of water for irrigation, increase in soil salinity etc. This results in even possible relocation of habitants from villages due to non-availability of productive soils and drinking water effectively changing the catchments characteristics including its socio economic characteristics.

On the whole, contamination of coastal aquifers may lead to serious consequences on environment, ecology and economy of that region. This research endeavours to investigate changes in coastal zones caused by salinity intrusion, and first, would assess and predict the long term salinity intrusion situation, and then, simulate aquifer variations, and thereafter develop a model that would facilitate the policy makers to take optimal decisions with multiple objectives to manage the changes occurring in coastal zones.

## 2. CASE STUDIES

### *2.1 Whaiwetu aquifer, Wellington New Zealand*

There are three principal groundwater areas in the Wellington region: Lower Hutt Valley, Kapiti Coast and the Wairarapa Valley. Secondary groundwater areas include: Upper Hutt, Mangaroa Valley, Wainuiomata Valley and Sections of the Eastern Wairarapa Coastline. Aquifers in all of these areas are found in unconsolidated alluvial, aeolian, and beach sediments of varying grain size. Minor aquifers are also found in limestone and fractured greywacke in some areas of the region.

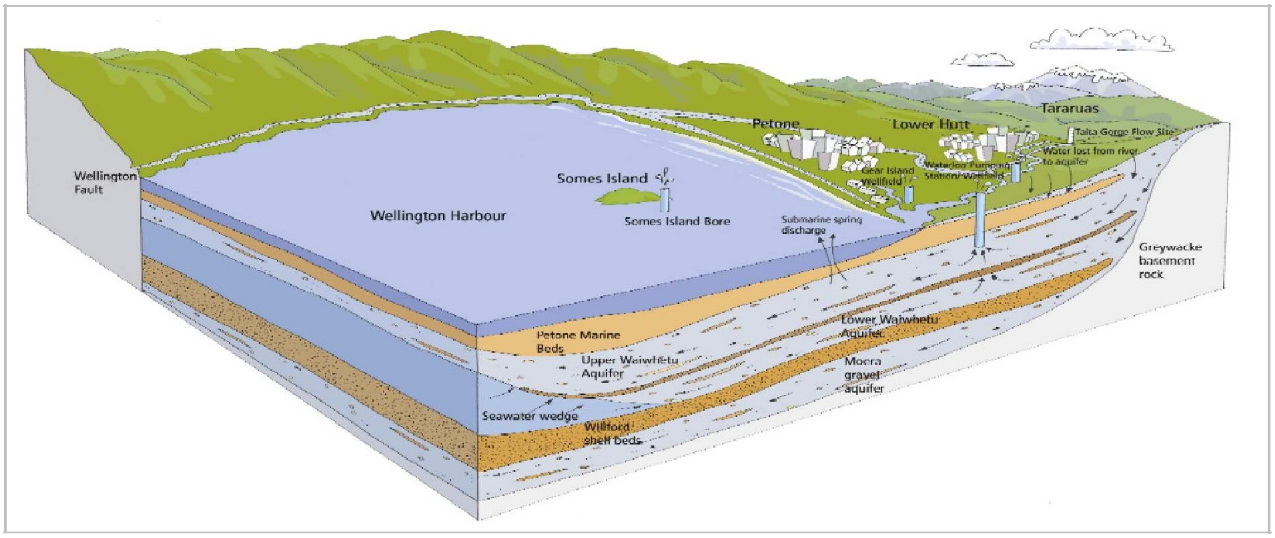



**Figure 1. Lower Hutt Groundwater System**, Source: Wellington Regional Council Report.

#### *2.1.1 Long term variations*

The Waiwhetu aquifer is located beneath the Hutt Valley and extends well into the Wellington harbour. Greater Wellington extracts 40% of it's water requirements from ground water. The risk of salinity intrusion is emphasized by the level of abstraction from the Waiwhetu aquifer, estimated to be as 80 to 90% of the total through flow. The variation of Conductivity and Total Dissolved Solids for the points Somes Island, Petone and Seaview are shown in Figures 1 and 2.

*Figure 2 – Variation of Conductivity (Jun 93–Sep01)*

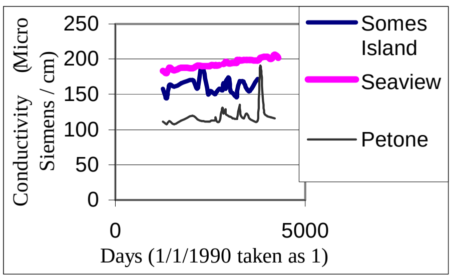



*Figure 3 -Variation of Total Dissolved solids (Jun 93–Sep 01)*

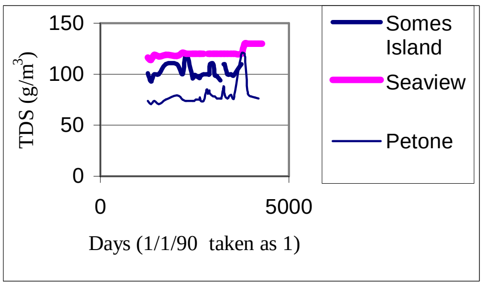


*Table – 1 Variation and Forecast at Seaview*

| | Present Values | Average Values | Slope% (Gradient) | Coefficient of Correlation | Forecast 1/1/2010 | 1/1/2025 |
|---|---|---|---|---|---|---|
| Conductivity | 202 | 195.61 | 0.75 | 0.94 | 229.6 | 270.6 |
| Total Dissolved Solids | 130 | 122.73 | 0.54 | 0.83 | 147.3 | 176.9 |
| Cl | 19 | 17.42 | 0.09 | 0.85 | 21.6 | 26.8 |
| Na | 18 | 17.62 | 0.05 | 0.75 | 20.0 | 22.8 |
| Hardness | 55 | 49.81 | 0.34 | 0.77 | 65.4 | 84.3 |

It can be seen that there is a very high correlation for all these factors with time. In the case of conductivity it is as high as 0.94, giving an $r^2$ (square of the Pearson product moment correlation coefficient) value of 0.88. The gradient of, the variation of these factors and particularly conductivity, being positive and high indicates that there is a continuous rise in these factors. The regression results can be extrapolated to indicate that, the Conductivity of the water at Seaview will cross the threshold from fresh into the medium saline category by the year 2025. Analysis of Somes Island and Petone reveal that there is no significant correlation between either conductivity or TDS and time at Somes Island. The aquifer cap in and around Seaview could be thinner, or fractured to allow saline water intrusion to take place. As it was clear that the aquifer is susceptible to salinity intrusion detail investigation was performed and presented as follows. Here, PMWIN has been adopted to develop a three dimensional model in the study area. The model is used to understand the ground water movement and the risk of saltwater intrusion. Based on resulted conditions, the paper highlights some of the possible approaches to use for enhancing the sustainability of ground-water resources.

#### *2.1.2 Simulating Monitoring and Management of Seawater Intrusion*

PMWIN model package is used to test different assumptions on how the system may develop in the future. Since the future is uncertain, some assumptions about the evolution of the main source/sink terms need to be made resulting in different future scenarios. The complete set of scenarios provides a wide insight into the long term sustainability of existing pumping rates under different conditions. Furthermore, they provide information about where and which additional corrective measures are needed. Many corrective measures can be considered. Basically, they can be grouped into: reduction of groundwater pumping; increase of recharge; relocation of pumping wells; and in the case of coastal aquifers, additional engineering solutions to restore groundwater quality (e.g. hydraulic barriers).

#### *2.1.3 Control of the water abstraction*

Control of abstraction to manage the levels recommended by different studies is the traditional method to control seawater intrusion in Waiwhetu Aquifer. The new model has a more detailed layer structure and re-designed boundaries. The information and analysis on the aquifer system has been used to review the critical level for hydraulic heads on the aquifer. Donaldson, I.G. and Campbell, Cussins and Phreatos studying the aquifer's flow under different stresses calculated a critical level for hydraulic heads on the aquifer. Based on the results, control of abstraction to manage the levels was recommended.

The model simulations showed a noticeable effect of seawater intrusion for the dry period.
Assuming that the recharge is zero (aquifer recharge only from river bed seepage, no recharge from raining or other sources), after five years period with constant abstractions of 15000, 25000 and 50000 mc/day for each of the wells, the results show a considerable advance of the seawater intrusion.

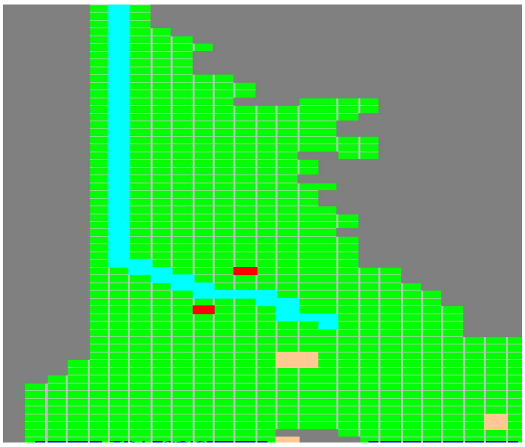


*Figure 4. Seawater intrusion.*
No abstraction,no recharge.

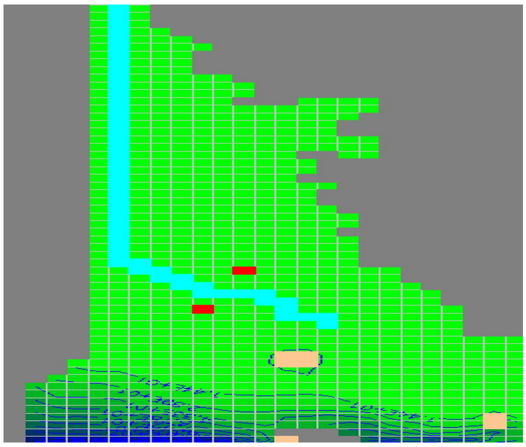


*Figure 5. Seawater Intrusion.*
15000mc/day each well. No recharge.

Blue line is sea water location, green zone is fresh water zone

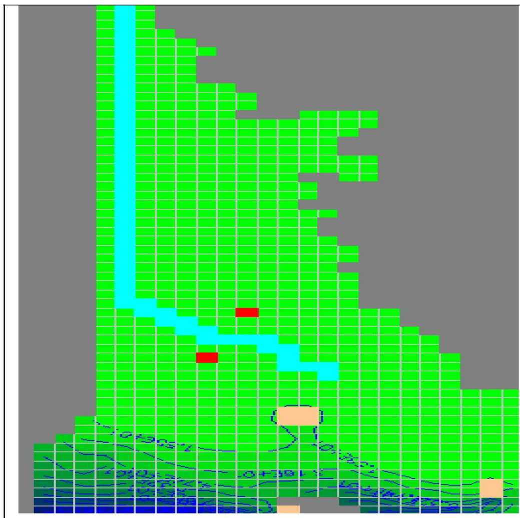


*Figure 6. Seawater intrusion.*

25000mc/day each well. No recharge.
Blue line is the seawater location.
green zone is freshwater zone

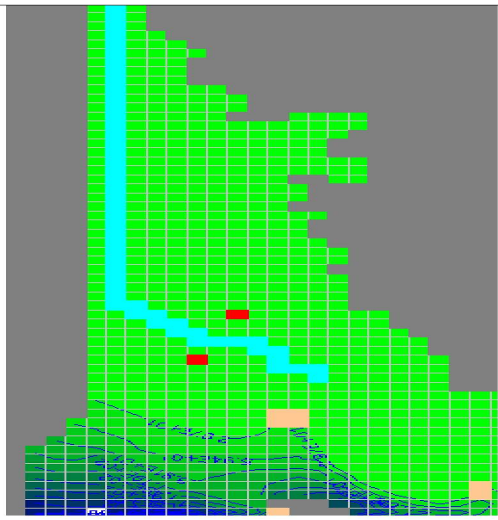


*Figure 7. Seawater intrusion.*

50000mc/day each well. No recharge
Blue line is the seawater location.
green zone is freshwater zone

***2.1.4 Recharging the Aquifers***

This investigation deals with artificial recharging of the unconfined part of the aquifer. Natural replenishment of aquifers occurs very slowly. Therefore, withdrawal of groundwater at a rate greater than the natural replenishment rate causes declining of groundwater level, which may lead to decreased water supply, contamination of fresh water by intrusion of pollutant water from nearby sources, seawater intrusion into the aquifer of coastal areas, etc. Artificial recharge may be defined as an augmentation of surface water into aquifers by some artificially planned operation. Possible adverse effects of the excess recharging may lead to the growth of water table near the ground surface and causes several types of environmental problems, such as water logging, soil salinity, and may affect natural aquifer storage and recovery systems. If the recharge is increased to 400mm/years, the seawater intrusion is less significant for the abstraction of 15000, 25000mc/day but considerable for 50000 mc/day.

### *2.2 The Bunderberg aquifer Australia.*

Bundaberg is a regional centre on the coast of Queensland, 360 km north of the state capital, Brisbane. The Bundaberg aquifer provides a major water supply for domestic consumption and irrigation. The aquifer is located under the Burnett and Elliott river systems and is hydraulically connected to the ocean. It is known to suffer some intrusion from the sea. The State Government has constructed an extensive surface irrigation scheme to reduce the pressure on the aquifer, and pumping of groundwater is controlled by license. The variation in conductivity between 1/1/90 and 7/8/2002 at four wells in the Elliott head area of the Bundaberg ground water system is presented.

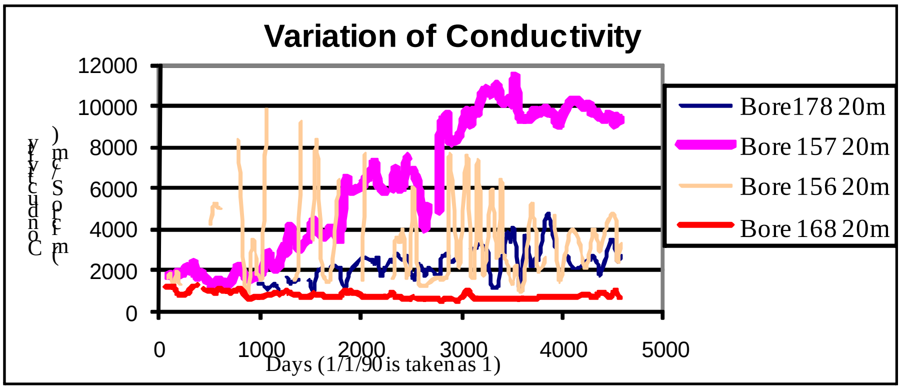



*Figure 8 - Conductivity at Elliot Heads*

*Table – 2 Variation of Conductivity at Elliott Heads.*

| | Present Values | Average Values | Slope% (Gradient) | Coefficient of Correlation | Forecast 1/1/2010 | Forecast 1/1/2025 |
|---|---|---|---|---|---|---|
| Well 156 | 3302 | 3447 | 0.22 | -0.13 | 2395(n/a) | 1187(n/a) |
| Well 178 | 2708 | 2375 | 0.41 | 0.55 | 4241 | 6511 |
| Well 157 | 9308 | 6130 | 2.35 | 0.94 | 17,953 | 30862 |
| Well 168 | 734 | 843 | 0.086 | -0.64 | 387 | 0 |

The coefficient of correlation for all the wells, except number 156 is reasonably high. Well 157 has the highest correlation value of 0.94, giving an $r^2$ value of 0.88. Well 156 is closest to the ocean and is very susceptible to the external variation of tides, while 157 is inland by about 1.3km. Well 168 is further inland by 1.1-1.2km; its results are influenced by freshwater flow from inland to the ocean, which explains the decreasing conductivity. It is thought that the rising conductivity in 157 and 178 could be due to two factors: (a) intrusion from the Elliott River (there is a tidal flat nearby); or (b) the aquifer in the vicinity of 157 is more clayey and thus responds more slowly to changes than a highly permeable aquifer, say at 156. Well 168 furthest from the coast is associated with an 18 m thick aquifer. Its conductivity values are decreasing and should reach the fresh potable water range in the near future. Its decreasing salinity may be caused due to inland flow, the greater proximity to the coast and the effect of stringent water management practices employed in this region. The analysis of the Bundaberg aquifer suggests that it is possible to manage saline water intrusion. If the aquifer is managed well, not only will the degree of intrusion be reduced but also the recovery of the ground water quality is possible.

### *2.3 Andra Pradesh India*

In the state of Andhra Pradesh, saltwater intrusion is widespread in the Delta regions of the eastern coast. Cities/towns affected by deteriorating groundwater quality due to increase in salinity are Vijayawada, Guntur, Tenali, and south regions of the Krishna River. This study area is known for large amount of groundwater withdrawals for agriculture and aquaculture. The increasing salinity in the explored groundwater aquifers is a matter of concern. A 3-D, transient, density dependent, finite element based flow and transport simulation model is implemented for the selected area in Nellore District, in Andhra Pradesh, India. This area is extensively utilizing pumped water from the underlying aquifers for agricultural, domestic and aqua cultural uses. The simulation model is calibrated using observed head and concentration data. The calibrated model is then utilized for evaluating the impact of adapting few pumping strategies for controlling the saltwater intrusion process.

The geographical location of study area is show below. The study area falls under alluvium soil type. These soils comprises of admixtures of sand, silt and clay in various proportions. The quartz pebbles are invariably encountered at different depths in almost all places in alluvial areas. It is generally light brown to pale gray and sandy in nature. The thickness of coastal alluvium is very large as evident from the exploratory wells drilled by Central Ground Water Board (CGWB) in this area. Bedrock was not encountered even at drilling depths ranging from 250 to 500 m.


*Fig – 9 Location of the aquifer*

Saltwater intrusion is already occurring in this area. This is mainly due to excess withdrawal of groundwater for domestic, agriculture and aquaculture uses. For the past 5 years the growth of aquaculture industries is very high, which requires huge amount of water. The only usable water source in this area is groundwater. The observation data by State Groundwater Department, Nellore suggest that water the table is going down every year. In addition to high pumping, there is no good amount of rains for the past 4 years (2001-2004). This further accelerates the groundwater deterioration. The only source of groundwater recharge is through rainfall. The data collected for this study area are briefly described below.

A 3D, transient, density dependent, finite element based flow and transport simulation model, FEMWATER is implemented for simulating the coupled flow and transport processes of saltwater intrusion in a coastal aquifer in Nellore district of Andhra Pradesh, India Available data for a selected study area of around 355 km$^2$ was collected from different agencies to be used as input data for implementing the numerical simulation model for the study area. Due to the scanty nature of available data and questionable reliability of all available data the best but subjective judgment was used in selecting the data for implementing the model.

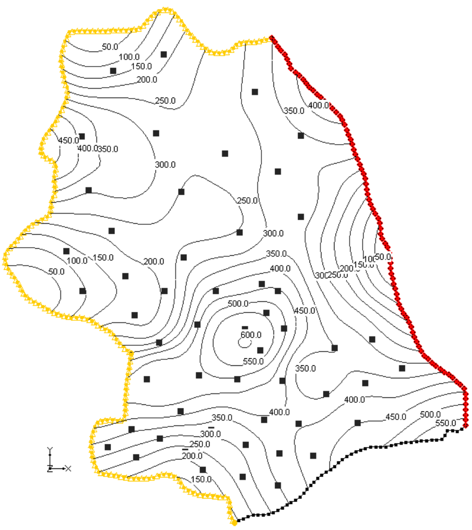


*Fig. 10. Observed salt concentration contours July 2001*

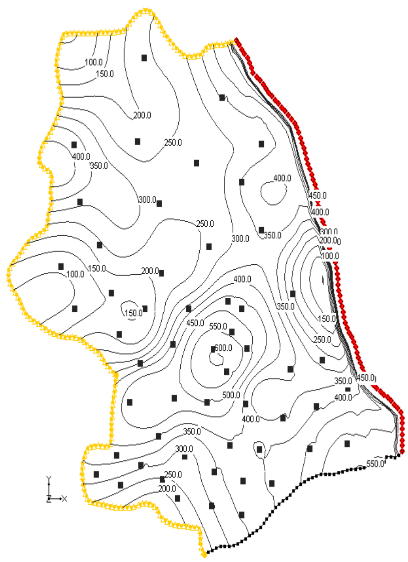


*Fig 11. Simulated salt concentration contours July 2001*

The numerical model was calibrated for two years time period, between July 2000 and July 2002, both in terms of hydraulic heads and salt concentration. The aquifer was considered heterogeneous in terms of vertical stratification. Both flow and transport are considered transient. Withdrawal from aquifer is estimated based on available data, and assuming an increasing trend over the period of calibration and validation. The calibrated simulation model was used to predict the saltwater transport scenario in the study area at future time periods. This predicted head and concentration values show the future saltwater intrusion patterns if the present trend of pumping continues. These results also show if the withdrawal rate continues to increase over time it may have detrimental effect on the salt concentration in the study area.

### *2.4 Salinity intrusion in Japan*

The Ogawara lake, located in the north end of Japans main island Honshu, indicates salinity intrusion during particular periods, Ishikawa et. al., (2001). The lake is hydraulically connected to the sea by means of the Takase river which is 6 km long. During the summer months due to tidal effects sea water intrudes the lake and thereafter, the upper layer of low salinity flows towards the lake exit while the bottom layer of high salinity flows in the opposite direction, due to the balance between effective gravity towards the centre of the lake as well as the pressure gradient towards the exit.

Clams, which breed in this lake and used for a Japanese soup (Miso) require a certain degree of salinity for breeding. A dynamic balance which develops at certain locations facilitates sea water intrusion and higher salinity in shallower areas resulting in favourable conditions for breeding of clams. This is a positive implication of salinity intrusion caused by a natural phenomenon.

A study done by Tokuoka., et. al. (2000), on the Gono river in the Shimane prefecture indicates that the fresh - saline water interface of adjacent subsurface aquifer systems demonstrate good positive correlation to the movement of the fresh-saline water interface of the surface river system. This implies that utmost care must be taken in any intervention of the hydro geological system, as any intervention in either would impact on both the water systems.

A study done by Tsumi et. al., (2001), in the western part of Fukouka city, demonstrates that the degree of sensitivity of salinity intrusion in coastal aquifers to direct recharge of rain water and irrigation water, is high in lowlands but low in high lands. Surface development work such as construction on low lands would result in reduced seepage and thereby impact on salinity intrusion in coastal aquifers. In addition, systems used to irrigate and drain water in low lands express varying sensitivities to salinity intrusion. The impact of such activity on high lands is much lesser. Therefore, not only surface development work, inclusive of irrigation and drainage systems, but also, their location within the surface system, plays a major role in salinity intrusion.

The aquifer system in the Izena Island in northern Okinawa prefecture is the main source of water for domestic and agricultural consumption. It has to be exploited to meet the needs of the island. As it is imperative that this aquifer continuously yield potable water, exploitation of this source has to take place in a sustainable manner. A study done by Ru et. al., (2001) indicates that, even though large percentage of fresh water flow through the system in to the ocean, the formation of large cones of depression due to continuous abstraction causes the upward journey of the saline - fresh water interface and the gradual contamination of the aquifer. This could be mitigated by construction of impervious subsurface dams at appropriate locations, which assists not only in the increase in speed of recharge of the aquifer by reducing the speed of outward flow, but also impedes the upward movement of the saline water fresh water interface.

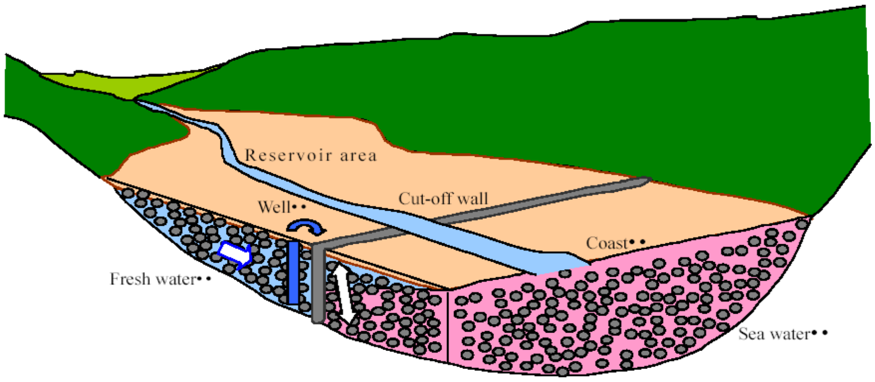



*Fig – 12 Schematic diagram of underground dammed aquifer*

## 3. SUMMARY AND CONCLUSION

The analysis of the Waiwhetu aquifer suggest that there is considerable dependency as well as stress on the aquifer and it has the potential to degenerate and cross the thresh hold category limit of saline water unless properly managed. The Bundaberg analysis indicates that implementation of aquifer management practices can prevent the degeneration of the aquifer over a long period of time. It is not only possible to prevent degradation, but also to facilitate the recovery of water quality even to the point of eventually yielding potable water. The Ogawara river and the breeding of clams in high salinity areas reveals a positive impact of salinity intrusion caused by a natural phenomenon. The Ogawara case also indicates that numerous complexities exist in salinity intrusion and the difficulty in analyzing the impact of intervention. The Gonokawa case reveals that surface and aquifer water systems are correlated and utmost care must be taken in intervention of the hydro geological system, as any intervention in either the surface or the aquifer water systems would impact on both the systems. From the Kitakyushu case it could be said that, not only surface development work inclusive of irrigation and drainage system, but also their location within the surface system, play a major role in salinity intrusion of coastal aquifers.
From the case of the North Okinawa Island aquifer system it could be concluded that, the important criteria in a hydro geological sense is not the degree of exploitation of aquifers, but the equilibrium of the aquifers and this could be achieved not only by naturally maintaining hydraulic gradients but also even by means of intervention such as creation of impervious subsurface dams.

This paper presents modelling of the Waiwhetu aquifer, and expresses the stresses under which the aquifer is subject to. PMWIN model has been used to simulate the aquifer, and some innovative scenarios have been investigated to identify possible solutions to reduce the risk of seawater intrusion.

The result of the simulations shows that the risk of sea water intrusion can not be reduced only by controlling the level of abstraction particularly if demand continues to increase and recharge decrease, but augmentation of recharge is a superior and viable alternative which facilitates abstraction as well. River banks, infiltration basins and injection wells have been simulated and the result show that the using a combination of different techniques abstraction could be maximized. The simulation studies show that abstraction can be double the actual current abstraction with no risk of seawater intrusion. Model Simulations indicate that implementation of aquifer management `practices and varying methods to augment the recharge are alternatives that could be considered for the Waiwhetu aquifer if it is subject to higher abstraction levels, lower recharge and salinity intrusion.

### *Acknowledgements*

The authors most sincerely thank Mr. Peter Butcher, Head of Engineering & Construction, Open Polytechnic of New Zealand for the assistance given in conducting this research. The authors also thank the DPRI, Kyoto University and The Open Polytechnic of New Zealand for providing the necessary facilities to do the research. The authors thank Mr. David Dempster of the Queensland Department of Natural Resources for his assistance in providing data.

## REFERENCES.

[1] De Costa. S., Porter. M., and Jones. A., (2003) : Comparative analysis of salinity intrusion-Case of the Waiwhetu aquifer New Zealand and the Bundaberg aquifer Australia, Proceedings of the 30th Congress of the IAHR, Theme B, pp.565-572.

[2] De Alwis. K., and Panabokke. C., (1972) : Hand book of the soils of Sri Lanka

*[3] Ishikawa. T., Tsuruta. Y., Nishida, S., and Narita, M. (2001) : Salt water behaviour near the exit of lake Ogawara, in relation to the breeding of Corbicula Japonica, Proceedings of the 29th IAHR Congress, Vol. 2.*

[4] Kuruppuarachchi. D., (1995) : Impact of irrigated agriculture on ground water resources of Sri Lanka, Proceedings of the Sri Lanka Association of the Advancement of Science, pp.49-66.

[5] Ru. Y., Jinno. K., Hosokawa. T., and Nakagawa. K., (2001) : Study on effect of subsurface dam in coastal sea water intrusion, First International Conference on salt water intrusion and coastal aquifers, Paper No. 92.

[6] Tokuoka., T., Sampei, Y., and Ueno. H.,(2000) : Saline water intrusion at estuary river and its relation to the under ground water, Case study at Gonokawa, Shimane Prefecture, http://www.survas.mdx.ac.uk/pdfs/3tokuoka.pdf

*[7] Tsumi. A., Jinno. K., and Oheda. Y., (2001) : Changes in fresh and saltwater movement in a coastal aquifer by land surface alteration, First international conference on saltwater intrusion and coastal aquifers, Paper No. 20.*

[8] Tsuruta. Y., Ishikawa. T., and Hayasaka. H.,(2000): Stochastic analysis of seawater intrusion into lake Ogawara, http://kfki.baw.de/conferences/ICHE/2000-Seoul/pdf/51/PAP_077.PDF

# LAKE GREGORY, ALIEN FLORA AND URBAN AQUA-ENVIRONMENTS IN A MISTY CITY OF SRI LANKA

Shirantha, R. R. A. R[1]., Amarathunga, A. A. D[2]. and Pushpa Kumara, N. W. J[3]

[1]Research Officer, Inland Aquatic Resources and Aquaculture Division, National Aquatic Resources Research and Development Agency (NARA), Crow Island, Mattakkuliya, Colombo 15, Sri Lanka
[1]E-mail: r_shirantha@hotmail.com
[1]Telephone: +94-11-2521005; Fax: + 94-11-2521005

[2]Research Officer, Environmental Studies Division, National Aquatic Resources Research and Development Agency (NARA), Crow Island, Mattakkuliya, Colombo 15, Sri Lanka
[2]E-mail: Deeptha@nara.ac.lk
[2]Telephone: +94-11-2521009; Fax: + 94-11-2521009

[3]Research Assistant, Inland Aquatic Resources and Aquaculture Division, National Aquatic Resources Research and Development Agency (NARA), Crow Island, Mattakkuliya, Colombo 15, Sri Lanka
[3]E-mail: JPkumaraik@yahoo.com
[3]Telephone: +94-11-2521005; Fax: + 94-11-2521005

**Abstract:** Lake Gregory in the uppermost hill capital of Sri Lanka is a man made water-body built to enhance the site esthetic beauty. Now it has to act as a waste sinker, having lost its ability to supply bathing/ recreational/agricultural water. A series of short studies on its floral composition and water quality parameters were done from 2007 to 2009 through direct visual observations and the APHA standard methods to quantify its current ecological conditions and future sustainability. Floral composition of the lake was found to dominate by Alien Plant Species (APS) demonstrating rapid spatial changes from time to time. *Myriophyllum* sp., *Polygonium* sp, *Salvinia molesta*, *Eichhornia crassipes*, *Ceratophyllum demersum* and *Pistia stratiotes* were the dominant APS of which *E. crassipes* was found to occupy nearly 1/3 of the lake. Others were in localized patches. There was a reduction of native water Lilly plant cover and amplification of extensive mat forming of APS *Polygonium* sp., *Ludwigia* sp. and *Altermanthera philoxeroides* in water edge areas. *Wedelia trilobota, Ulex europaeus* and *Colocasia esculenta* were found to form thickets in peripheral areas displacing the grass land gradually while modifying the structure and function of the lake. APS in water edge areas was found to facilitate sediment accumulation in greater amounts while converting lake into a semi-terrestrial area and choking in flow channels. Now its open water area estimated was <0.5 $Km^2$. Lake water quality was found to deplete rapidly, where most parameters exceeded permitted levels i. e. pH 7.8, EC 103.5 μS/cm, ammonia 0.38 mg/l, nitrate 3.7 mg/l and phosphate 1.04 mg/l. The condition was attributed to dense APS patches. It is clear now that the lake is persisting in a meso to eutrophic condition. Comparably less similar conditions were recorded in down stream at Meepilinama (*Nanu Oya;* BOD 8.14 mg/l, pH 7.10, DO 7.00 mg/l, EC 99.72 μS/cm, ammonia 0.14 mg/l, nitrate 2.34 mg/l and phosphate 1.54mg/l) in spite of receiving high quality water from a natural forest stretch. It appears that water quality depletion is so severe at the lake that it enables its down stream to self purification at this extension. Heavy use of agrochemicals and fertilizers in riparian areas and high silation were found as main pollution causes. The additional contribution of APS colonies blocking the air-water interface resulted low oxygen levels (DO <5.2 mg/l). Dumping huge quantities of waste as well as outflows from neighboring hotels, houses, etc. were other known threats which contributed to make the lake sometimes unpleasant. Discarding unwanted vegetable parts into the lake areas was another big problem as it adds on to the organic pollution. Altered floral composition in Lake Gregory is mainly due to heavy loading of nutrients and sedimentation from crop/tea farming and ecotourism based activities. Therefore, control/mitigation of such activities and removal of APS from time to time are highly recommended in addition to dragging as the lake is now rapidly loosing its ecological sustainability.

**Key words:** Gregory Lake, water pollution, alien plant, ecological risk.

## 1 Introduction

### *1.1 Alien flora in freshwater ecosystems*

Of the world, nearly almost all natural ecosystems are under stress particularly freshwaters may be the most endangered of all as they have lost a greater propotion of their speices and habitats compare to other ecosystems on land or in the oceans (www.wri.org). Industrial discharges as well as agricultural and urban run off are pervasive strees on frehswater ecosystems since they alter the physical (water temperature, water flow and light availability), chemical (nutrients, oxygen consuming materials and toxins) and biological environmets in freshwaters (exotic species) (www.towards-sustainability.co.uk/issues/built). The biological alternation in freshwaters are largely supported by alternations of their physical and chemical factors mainly due to siltation and organic pollution which facilitates some alien species to colonize and pose significant threats causing an ultimate effect on their sustainability (www.iucn.org).

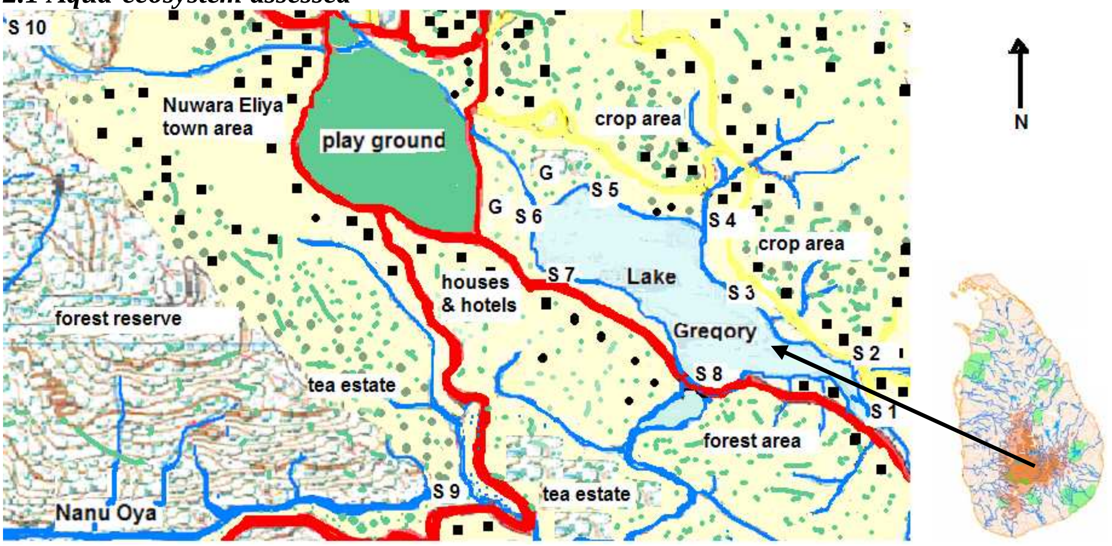
Alien species *i.e. non- native/non-indigenous/foreign/exotic* can be a species, sub species or lower taxon occurring out side of its natural range. Alien Plant Species (APS) in freshwaters are concern; there are several instances that they have invaded and possess threats to functions and structures of natural, semi natural and irrigational ecosystems of the world causing terrific damage to some of them (www.iucn.org). However, APS are important as indicators in disturbed waters since most of them are tend to colonize certain places where elevated levels of nutrients, heavy metals and etc. are encountered due to obvious point causes of pollution (Bambaradeniya et al. 2002). Hence APS colonization depends on site quality and detection of their colonization in a freshwater body is very significant to get an exact idea on existing quality depletion or pollution trends.

***1.2 Rational for the study***

Sri Lanka harbours a thousands of man made water bodies of which the Lake Gregory in the uppermost hill capital Nuwara Eliya is a major tourist destination in South East Asia and it is a monument of the British rule in Sri Lanka (*www.jetwinghotels.com).* The potato cultivation introduce in the early 1970s and vegetable farming caused extensive environmental damage to the Nuwara Eliya urban area as well as to the lake Gregory causing massive erosion of surface soil and up loading of nutrient rich run off into the lake (www.encyclopedea.com). The alienation of surrounding lands of the lake went by political favour regardless of the environment and the natural beauty of the Nuwara Eliya town. As a result the lake Gregory became a waste sinker as well as a sufferer due to stress condition built by agricultural base activities as well as due to rapid urbanization. Now it is likely to deviate from all levels of standards for its sustainability proving ideal place to colonize for some destructive alien aquatic plants. In recent past several projects have been implemented to maintain Lake Gregory but no avail (www.jetwinghotels.com*)*. The reason may be due to lack of thorough understanding on causes and effect on its eco-quality depletion. Since it is reasonable to carry out an eco-monitoring assessment in the lake to identify causes and effects on eco-quality depletion, the present study was carried out with an objective of recommending suitable management strategies to bring the lake into a manageable level.

## 2. Material and methods

***2.1 Aqua-ecosystem assessed***




Scale 1: 50 000

Figure 2.1 Map of Sri Lanka showing the location of the Nuwara Eliya and water sources of the Gregory Lake (■ houses & hotels, G – grass land and **S** sampling point).

Lake Gregory is a man made water-body in a most superficial sight in 06°53’.816N and 080°52’.536E in the uppermost hill capital *Nuwara Eliya* (1200 m Mean Sea Level) in Sri Lanka (Figure 2.1). It was built in 1874 by then British Governor by crossing the Thalagala Oya which is one of the uppermost streams of the Nanu Oya that empties into the Kotmale Oya; a tributary of the River Mahaweli of Sri Lanka (*www.encyclopedia.freedictionary.com).* In the past the lake was used to feed a mini-hydropower plant at Black Pool area. Since the hydro-power plant was closed later, enhancing the sight esthetic beauty and use as a sport fishing ground become main target functions of the lake. The Gregory lake is of an area of 30 hectares and its catchment is of several natural streams that originate from the Piduruthalagala mountain range in North-western, Kandapola area in North-eastern and Magatota area in South-eastern. In addition to these the lake gets water from several inflow culverts from Nuwara Eliya municipal area. From South-western side the lake connects to the Nanu Oya which encounters several other perennial streams flow through natural forest stretches as well as from plantation areas (figure 2.1).

***2.2 Study sites selected***

A total of eight sampling points covering al most all significant inflow channels into the lake *i. e. including natural streams and culverts that are of obvious point source pollution* were selected for the study (figure 2.1). The site 1 is a 3$^{rd}$ order perennial upstream originates in Magastota area where intensive vegetable farms are found. The site 2 is a 2$^{nd}$ order perennial stream flows through Magastota tea estate as well as vegetable crop areas. The site 3 is a small inflow channel. The site 4 is 3$^{rd}$ order stream flows through Hawa Eliya crop area. The site 5 is a stream at the army camp site. The site 6 is a 3$^{rd}$ order perennial relatively large stream (Thalagala Oya) originates in the Piduruthalagala mountain peak. The site 7 is a culvert from Nuwara Eliya town side. These sites (6 and 7) are encountering obvious point sources of pollution due to agricultural and urban run off. The site 8 is a reservoir site close to the main road. The site 9 is a riverine site at the Nanu Oya at Black Pool junction. At this point the Nanu Oya receives water from neighbouring annual crop lands and tea plantation areas as well as from a natural forest stretch at Kelegama.

***2.3 Monitoring of Alien Plant Species***

The study sites were visited at least in three monthly intervals from April 2007 to March 2009 and the floristic composition at the sampling points selected was studied through *in situ* direct observation. Their spatial distributions were stretched out on a schematic diagram of the lake. Later temporal variation in occupancy area of APS during the entire study period was roughly assessed.

***2.4 Monitoring of some physio-chemical parameters***

At each study occasion some water quality measurements; dissolved Oxygen (*Orion 830A* DO meter), pH (*Orion 260A* pH meter), temperature (thermometer incorporated in DO meter), electrical conductivity (*HANNA HI 8733t* conductivity meter) and alkalinity (tritration method) were made *in situ.* In the laboratory nitrate, phosphorus and ammonia level were analyzed following the standards methods given in APHA (APHA, 1998). Bio-Chemical Oxygen Demand (BOD) was measured using Aqua Lytic BOD sensors.

***2.5 Assessment of site condition***

Finally assessment of trophic status of the reservoir sites was evaluated referring to the eutrophication survey guideline values given for water quality parameters for lakes and reservoirs (table 2.1).

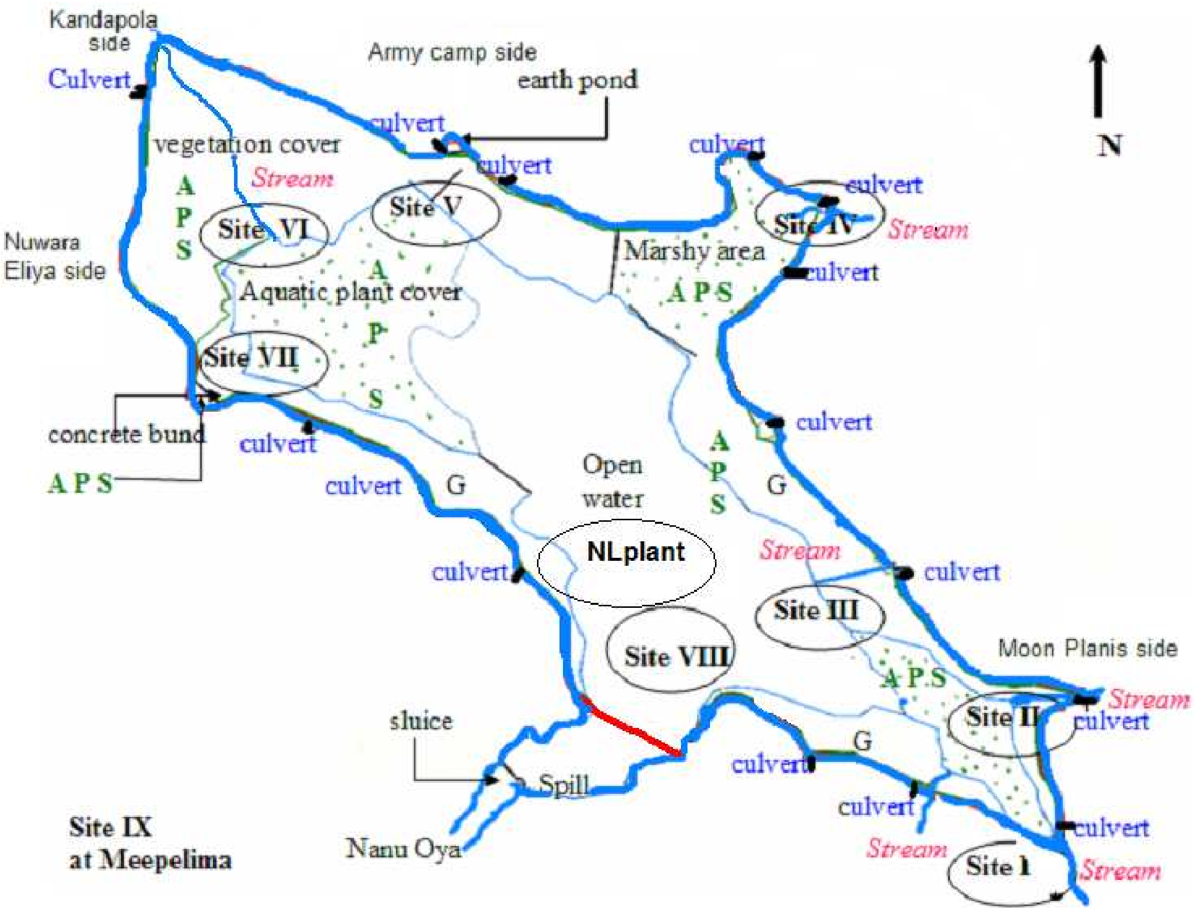
*Table 2.1 Eutrophication survey guidelines for lakes and reservoirs (source: Mason 1996)*

| | **Oligotrophic** | **Mesotrophic** | **Eutrophic** |
|---|---|---|---|
| Total phosphorous (mg/l) | < 0.01 | 0.01- 0.02 | > 0.20 |
| Total nitrogen (mg/l) | < 0.20 | 0.20 - 0. 50 | > 0.50 |
| Secchi depth (m) | > 3.7 | 3.7 – 2.0 | < 2.0 |
| Hypolimnetic dissolved oxygen (% saturation) | > 80 | 10 – 80 | < 10 |
| Chlorophyll-a (mg/l) | < 4 | 4 – 10 | > 10 |
| Phytoplankton production (g $Cm^{-2}$ $d^{-1}$) | 7 -25 | 75 - 250 | 350 - 700 |

## 3. Results

### *3.1 Floristic composition of the lake*

During the entire study period a total of 14 APS were recorded from the Lake and their site vise abundances are given in table 3.1. The potential impacts of those APS are also given in the table 3.1. The variation in spatial distribution of APS patches in the lake at the beginning of the study (in 2007) and at the end of the study (in 2009) is respectively shown in figure 3.1 and 3.2. Majority of APS were noxious aquatic weeds of which *Eichhornia crassipes* and *Altermanthera philoxeroides* were found to colonize in all the study sites. However, they were found to form dense thickets in certain sites i. e. in site 1, 2, 3, 7 and 8 which were found to on a point source of pollution mostly due to nutrient rich run off. *Myriophyllum* sp. was found to colonize largely in the sites 1 and 7 though it was firstly recorded in early 2008.




Scale 1: 2500

Figure 3.1 Map of Gregory lake showing the sampling points and distribution of Alien Plant Species (APS) and Native Lilly plant (NLplant) in 2007 (G – grass land).

*Salvinia molesta* was found in site 5 and site 6 only. However, there was a considerable reduction of native water Lilly plant cover and amplification in APS cover due to extensive mat forming of *Polygonium* sp., *Ludwigia* sp. and *Altermanthera philoxeroides* in water edge areas along the lake. *Ulex europaeus* and *Wedelia trilobota* were found in water edge areas forming dense thickets and found to invade grass land very rapidly in peripheral area of the lake. Relatively small patches of unidentified small weeds were recently (in late 2009) found in site 2 and site 3. Densely grown patches of *Colocasia esculenta*, unidentified shrub and *Cyprus* grass species were found to facilitate sedimentation leading to clog the inflow channels of the lakes. They were found in water edge areas as well as semi aquatic peripheral areas where grazing cattle and horse/pony are often found.

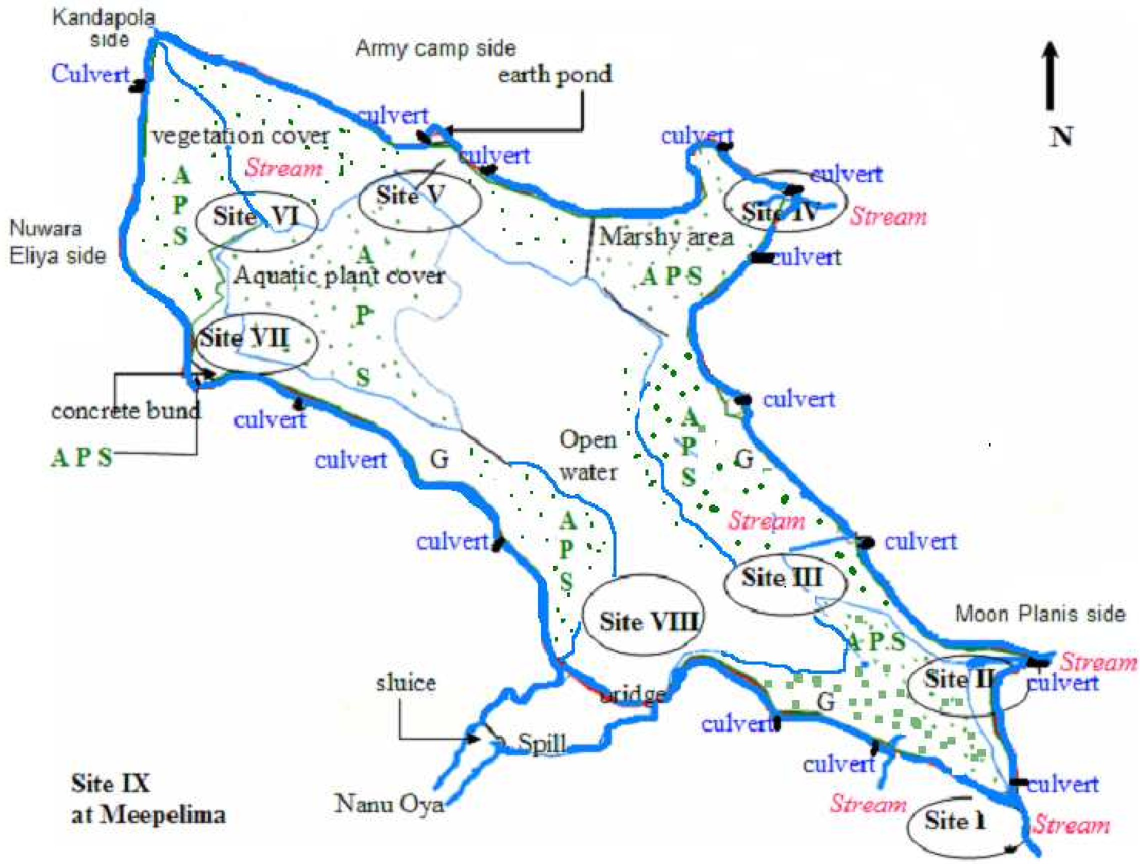



Figure 3.2 Map of Gregory lake showing the sampling points and distribution of Alien Plant Species (APS) in 2009.

*Table 3.1 Floristic composition; Alien Plant Species (APS) and native plant species (NPS) (abundance: + small fragmented patch, ++ large patch, +++ forming thicket/mat, – not recorded) at each study site in the Gregory Lake in Nuwara Eliya during the study period and their potential impacts (FT- form thickets, FM - form mats, IG – invade grass land, FSA -* facilitate *sediment accumulation, CFC - choking in flow channels and CWS - cover water surface and ● NLI listed as national invasive plant (Source: Marambe 2001),*

| Plant species | Site 1 | Site 2 | Site 3 | Site 4 | Site 5 | Site 6 | Site 7 | Site 8 | Possible impacts | NLI |
|---|---|---|---|---|---|---|---|---|---|---|
| *Altermanthera philoxeroides* | +++ | - | + | + | - | ++ | +++ | +++ | FT, IG, FSA, CFC, CWS | ● |
| *Ceratophyllum demersum* | + | - | - | + | - | + | + | + | FT, FSA, CFC | ● |
| *Colocasia esculenta* | +++ | +++ | + | ++ | + | +++ | + | ++ | FT, IG, FSA, CFC | ● |
| *Eichhornia crassipes* | +++ | ++ | +++ | +++ | + | ++ | ++ | +++ | FM, FSA, CFC, CWS | ● |
| *Mayaca* sp. | + | - | - | - | - | + | + | - | FT, CFC | - |
| *Myriophyllum* sp. | ++ | - | + | ++ | - | + | +++ | ++ | FT, FSA, CFC, CWS | - |
| *Ludwigia* sp. | + | + | + | + | + | ++ | +++ | + | FM, CWS | - |
| *Pistia stratiotes* | + | - | - | + | - | + | + | + | FM, FSA, CFC, CWS | ● |

*Table 2.1 cont.*

| Plant species | Site 1 | Site 2 | Site 3 | Site 4 | Site 5 | Site 6 | Site 7 | Site 8 | Possible impacts | NLI |
|---|---|---|---|---|---|---|---|---|---|---|
| *Polygonium* sp, | - | + | ++ | ++ | + | ++ | ++ | ++ | FT, IG, FSA, CFC, CWS | - |
| *Salvinia molesta,* | + | - | - | ++ | - | ++ | + | ++ | FM, FSA, CFC, CWS | ● |
| *Ulex europaeus* | + | - | +++ | ++ | + | + | - | ++ | FT, IG, | ● |
| *Wedelia trilobota* | - | - | + | + | + | ++ | ++ | ++ | FT, IG, CFC FSA, | ● |
| Unidentified sp. 1 | + | - | ++ | + | + | ++ | + | + | FT, IG, FSA, | - |
| Unidentified sp. 2 | + | - | ++ | ++ | - | + | - | - | FT, CFC | - |
| *Cyprus* sp. | - | + | ++ | ++ | ++ | +++ | ++ | ++ | | - |
| *Nymphea* sp. (native Lilly) | - | - | - | - | + | - | ++ | + | | |
| **Total APS** | **11** | **04** | **10** | **13** | **07** | **14** | **12** | **12** | | **08** |

### *3.2 Results of chemical monitoring*

*Table 3.2 some physio-chemical parameters of water at the sampling sites selected average for the entire study period (LA: average for lake sites and site 9; Nanu oya site).*

| | | | | | Lake site | | | | | Stream site |
|---|---|---|---|---|---|---|---|---|---|---|
| **Water quality parameter** | **Site 1** | **Site 2** | **Site 3** | **Site 4** | **Site 5** | **Site 6** | **Site 7** | **Site 8** | **LA** | **Site 9** |
| Water temperature (ºC) | 19.70 | 20.70 | 18.6 | 18.90 | 17.5 | 20.0 | 17.6 | 17.2 | 18.76 | 19.4 |
| pH | 7.85 | 7.57 | 7.8 | 7.91 | 7.98 | 7.55 | 7.82 | 7.53 | 7.75 | 7.10 |
| DO (mg/l) | 3.39 | 4.46 | 6.77 | 5.05 | 4.47 | 4.43 | 6.50 | 6.35 | 5.20 | 7.00 |
| BOD (mg/l) | 9.20 | 10.5 | 7.21 | 12.5 | 12.42 | 10.25 | 8.20 | 7.58 | 9.72 | 8.14 |
| Alkalinity ($CaCO_3$ mg/l) | 32.8 | 31.60 | 36.4 | 14.0 | 18.40 | 92.60 | 19.60 | 21.05 | 33.31 | - |
| Nitrate (mg/l) | 05.3 | 06.90 | 7.20 | 3.2 | 3.0 | 2.02 | 1.4 | 1.03 | 3.7 | 2.34 |
| Phosphate (mg/l) | 1.10 | 1.75 | 0.98 | 2.03 | 2.31 | 3.15 | 0.49 | 4.48 | 1.04 | 1.54 |
| Ammonia (mg/l) | 0.35 | 0.33 | 0.31 | 0.18 | 0.26 | 0.17 | 0.8 | 0.56 | 0.38 | 0.14 |
| EC (μS/cm) | 94.5 | 108.8 | 126.6 | 128.2 | 85.57 | 85.3 | 105 | 95.2 | 103.38 | 99.72 |

The values for some water quality parameters average for the entire study period for the lake sites and for the Nanu Oya site assessed are given in table 3.2. The lowest DO value (3.39 mg/l) was recorded at the lake site 1 whereas the highest DO value (6.77 mg/l) was recorded at the lake site 3. All the lake sites assessed were recorded relatively higher value for BOD <7.0 mg/l indicating an organic pollution condition in the lake. The nutrient concentrations were considerably high in all the lake inflow channels assessed except in site 7 that flows through Nuwara Eliya municipal area. Though there was a significant variation in the water quality parameters in lake sites assessed, the lake was found of poor quality water since most parameters (average for entire lake) exceeded the permitted levels for ambient water quality i. e. pH 7.8, EC 103.5 μS/cm, ammonia 0.38 mg/l, nitrate 3.7 mg/l and phosphate 1.04 mg/l. Less similar results were recorded in the more down stream site assessed (site 9); BOD was 8.14 mg/l, pH was 7.10, DO was 7.00 mg/l, EC was 99.72 μS/cm, ammonia 0.14 mg/l, nitrate 2.34 mg/l and phosphate 1.54 mg/l.

### *3.3 Identified causes for low down of ecological integrity of the sites assessed*

The identified causes for deterioration of water quality of the lake sites assessed as well as in riverine site assessed are given in table 3.3. It is clear from the table 3.3 that reservoir sites are encountering more impacts that lead to rapid water quality depletion due to agricultural and urban run off.

*Table 3.3 identified causes for water quality depletion in the lake site assessed and Nanu Oya site at Meepilimana (possible effects are given parentheses).*

| | **Lake sites** | | | | | | | | **Stream site** |
|---|---|---|---|---|---|---|---|---|---|
| **Identified causes** | **1** | **2** | **3** | **4** | **5** | **6** | **7** | **8** | **9** |
| Colonization of APS (*modifying the structure and function of the lake*) | +++ | ++ | ++ | +++ | + | ++ | ++ | + | - |
| Converting riparian tea/forest area into annual crops (*soil erosion and high siltation*) | + | + | + | + | + | + | + | + | ++ |
| Excessive use of agrochemicals and fertilizer (*loading of organic pollutants, heavy metals and other toxic compound*) | ++ | + | + | + | + | + | + | + | ++ |
| Dumping of unwanted vegetables (*loading of organic substances and harmful microbs/pathogen*) | ++ | ++ | + | + | + | + | + | + | + |
| Dumping waste due to eco-tourism (*loading bio and non bio degradable substances*) | + | + | ++ | + | + | + | ++ | ++ | + |
| Mixing of urban/hotel sewage/drainage channels (*loading of organic/inorganic substances and harmful microbs/pathogen*) | + | + | + | + | ++ | ++ | ++ | ++ | + |
| Graze by cattle/horse (*loading of organic/inorganic substances and harmful microbs/pathogen*) | - | + | ++ | +++ | + | ++ | ++ | - | - |
| Boat riding *(mixing of oil and grease)* | - | - | + | - | - | + | ++ | ++ | - |

(Severity: + low, ++ moderate and +++ high)

## 4. Discussion

Plants are an important part of healthy diverse aquatic ecosystems as they play a major role in maintaining the integrity of lakes, ponds, streams and etc (Dash 2001). However, t*here are several instances in many parts of the world that the natural balance between vegetation and other aquatic organisms is disrupted when exotic plants are introduced and they become noxious weeds particularly in lakes and urban drainage channels* (www.*aquanic.org). S*pread of alien aquatic plants such as *Eichhornia crassipes*, *Salvinia molesta*, *Hydrilla verticillata*, *Najas marina* into many places in Madu Ganga estuary due to accumulation of nutrients through agricultural runoff and discharge of organic wastes into south-western costal area in Sri Lanka was discussed by Bambaradeniya et. al (2002). They have shown that alien plants species sometimes can be invader particularly that form thickets and shade out native vegetation, and thereby displacing the natives gradually. The reason they have given for that kind of invasiveness is a heavy load of excess nutrient run off that comes through nearby cinnamon cultivation and waste run off. More or less severe scenario was observed in the Lake Gregory as it was found as eutrophic (tables 2.1 and 3.2) water body since some of parameters assessed from the lake sites have exceeded their meso trophic levels. Signifying a hypo-eutrophic condition, an extra elevated level of nutrients (nitrate and phosphate) was recorded at some lake sites assessed particularly in sites 1, 2, 4 and 5 which were observed to under dense APS colonies. All these sites are having point sources of organic/chemical pollution due to agricultural based activities. Since the riparian area is under intensive potato, horticulture and vegetable cultivation, farmers apply fertilizer and agrochemicals at least once in two days, hence the excess amount or all if it is raining washed off into the lake subsequently leading to surface water to become a nutrient rich. In addition to this the existing APS colonies probably disturb water circulation process within the lake. Due to relative small area, the lake it-self is having a poor water mixing ability (Silva 1996). The integrated condition is largely favoured by floating root APS forming dense mats and thickets in preferred areas. Due to functional and structural adaptation of these APS leads to gradual convert of water edge area into a land area as well as leads to further depletion in lake eco-quality.

Adding of non-agricultural based organic run off and waste onto the lake water is another critical impact identified. The eco-tourism based activities are among the major sources of them. The neighbouring hotels, guest houses and hose-hold drainage channels were found as contributories in urban run off that bring a considerable amount of waste into the lake daily. Decomposition of them due to microbs activities results significantly elevated level of BOD and reduced level of DO. At the same time it would release high amount of nutrients (Mason 1996) that consequent for dense mat forming of existing APS. This probably be faster leading to drastic fluctuation of dissolved oxygen due to reduction of air water interface which less facilitate to Oxygen to dissolve in water. The large APS covers especially made with well-known troublesome aquatic noxious weeds like *Ludwigia peploides*, *Myriophyllum* and *Eichhornia* (www.aquanic.org/management) may possibly attributed to the comparatively low levels of DO values in sites 1, 2, 4 and 6. During the study period the above perennial herbs were found to grow in moist to wet riparian areas in the lake, spread to form mats on the sediment, or floats ascending in the water edge areas. However, comparatively higher concentration of dissolved Oxygen (6.67 mg/l) was recorded at site 3. The reason may be due to more open water area with small patches APS. For that reason, too it clear that occupancy area of APS and their spread is extremely couple with lake nutrient profile that significant on lake quality.

Certain APS were found to choke waterways totally especially at sites 1, 2, 4 and 6 facilitating sedimentation and clog waste excluding light penetration into the lake. Especially *Ludwigia* predominately colonizing in these sites was found to entrap plastic bottles and other containers due to blanket in water surface, sometimes making the lake-site unpleasant. This has profound effects on communities of native plants and animals in the water. They also interfere with animal access for drinking water, human access for swimming and boating. The condition was critical in site 7 as dense APS patches have already converted the water edge area into a semi terrestrial land where *Cyprus* grass was observed to colonize rapidly. It has already extended onto the native Lilly plants cover. In initial stages of the study it was found in relatively deep areas of the lake very close to the site 7 (figure 3.1 and 3.2) . Due to high siltation, the particular area became a very shallow within a very short period, and rapidly invaded by few APS namely *Eichhornia crassipes, Ludwigia, Cyprus* and *Altermanthera philoxeroides.* Now few of native Lilley plants can be seen among *Cyprus* sp. that confined to a very little area. It is clear that the APS at the extreme levels have entirely modified the structure ad function of the lake ecosystem in many ways. Sometimes APS can produce substances that are toxic (www.nps.gov/plants/alien) i. e. allopathic to others thereby make the soil unsuitable for original native species. So that it might be one of reasons for APS thrives into a native Lilly plant area. However, further studies need to be carried out to come a definite conclusion regards.

The cattle/horse feeding in marshy areas in the lake might facilitate adding of organic and inorganic load through their dung and urine that adds onto the pollution load. The reason for relatively high ammonia level (≤0.3 mg/l) at sites 2 and 3 possibly attributed by cattle forage. Decaying unwanted vegetable stumps which was observed to be intensified in harvesting period, is significant as it adds extra organic pollution load into the lake. The condition was found to severe in site 2 where the elevated level of BOD (10.5 mg/l) was recorded probably due to additional organic inputs brought by vegetable stump releasing an increased amount of nutrients (nitrates 6.9 mg/l and phosphates 1.75 mg/l). This possibly stimulates massive growth of APS in lake peripherals thereby becoming a deal cattle feeding ground.

The water edge area at town side was largely colonized by *Cyprus* sp. and *Eichhornia* that facilitate fast sediment deposit. Not only but some floating species such as *Salvinia*, *Pistia* were found to accumulate greater amount of sediments. Since most of them possess physiological and structural adaptations to grow in organically polluted muddy conditions (www.nps.gov/plants/alien) the area is now having a plentiful cover of APS. More recently *Myriophyllum* sp. thrives in the area. *The condition could be severe in near future as it is among world worst invasive plant species (*www.nps.gov/plants/alien). It *was found to dense in site 1 where Magastota river empties in to the lake with high nutrient load i. e. 5.3 mg/l nitrate, 1.10 mg/l phosphate and 0.35 mg/l ammonia. This condition possibly attributes to rapid colonization of Myriophyllum*.

Present result also showed that deviation in some other water quality parameter such as pH and conductivity from the ambient standards might be directly couple with organic pollution. The heavy metal levels in the lake can possibly be high though it was not assessed during the present study, as the lake is routinely subjected loading of them through agrochemicals as well as combustion of fossil fuel. The eco-quality depletion scenario originated at the Gregory lake site seems to be forwarding to its down stream stretch since more or less comparable water quality depletion trend observed at the Nanu Oya though water gets diluted at this run there is no sufficient enough purification process to bring river water into a good quality as the river encounter addition threats at this stretch. Further more the landscape changes due to encroachment, illegal cultivation of tea and annual crops and coverting of tea plantation into annual crop are attributing to elevated levels BOD was 8.14 mg/l, pH was 7.10, DO was 7.00 mg/l, EC was 99.72 µS/cm, ammonia 0.14 mg/l, nitrate 2.34 mg/l and phosphate 1.54 mg/l. Since this riverine area is very steep having fast flowing waters, APS are unable to colonize very rapidly but river bankers may be affected.

*The APS have already interfered with recreational activities such as fishing, boating and swimming and with enjoyment of the natural beauty of this unique water resource.* According to the www.nps.gov/plants/alien web site most APS are tolerant of many water pollutants thereby they tends to invade disturbed areas where native plants cannot adapt to the alteration. Further APS does not spread rapidly into undisturbed areas where native plants are well established. It is well understood from the present study we have already altered the chemical, physical as well as biological environments in the Gregory Lake and we have created a new and unnatural niches where APS thrives. The ornamental plant industry has been identified as source of APS into the lake. Yet no prompt action has been taken in the near past to mitigate routinely up loadings of pollutants into the lake and we further alter the environments that are ideal for more APS. The activities in related to the growing human population pressure are fundamentally threatening the lake giving considerable stress on its ecology. Once it has being a sustainable built aqua environment though it is now being threatened. It is a big challenge to the city as it has reduced a potential to develop the city as a sustainable built eco-tourism site.

## 5. Conclusions

The colonization of APS in eutrophic Gregory Lake in uppermost reaches of the Mahaweli river is due to agricultural based activities and urban run off. Since further colonization of APS can possibly be a great threat actions should immediately be taken to bring the lake environments into manageable levels as it seems to be losing its sustainability very rapidly.

## 6. Recommendation for future of the lake:

The Gregory Lake was once a famous tourist attraction site in Sri Lanka where visitors truly feel a refreshing sense of peace with additionally magnificent reverse views of the surrounding area (www.jetwinghotels.com). Since the lake has been recognized as a priority wetland of the uppermost hill country of the island several conservation/reclamation management plans being developed during the last decade by the central government and the Nuwara Eliya urban council. Under the circumstances, several projects were implemented to maintain Lake Gregory from 1978 to 2001, but to no avail yet the lake is covered with destructive aquatic plants. Since the Lake Gregory, adorns Nuwara Eliya city, it should be given a facelift to improve tourist potentials. According to the results of present monitoring study there is a severe depletion in sustainability of the lake mainly due to destitute quality in lake water largely attributed by heavy load of nutrient pollutants. The condition attribute to intensified growth in steady populations of APS. According to the cause and effect studies (table 3.3), nearby crop lands and hotels are critical problem to the lake as they bring plenty of nutrient and organic waste through their drainage canal systems. Therefore, it is recommended to initiate comprehensive monitoring study to document the physio-chemical, hydrological and biological status of the lake at least in two month intervals. The local administrators and government conservation agencies should take immediate steps to mitigate harmful practices that degrade the lake. A work should be initiated to control loading of silt into lake. It can be achieved by constructing long term persist effective silt traps (concrete ones) at all in-flows for significant entrapping of sediment. Appropriate control/mitigation actions should be taken on agriculture based activities and to avoid further colonization of new APS. The Nuwara Eliya Municipal Council should initiate awareness programs on proper waste removing and management. In addition to these it is highly recommended to remove APS from time to time for effective removal of excess nutrient through bio-remediation process. Since the dragging of the lake sediment in an appropriate manner could help to remove load of heavy metals and other pollutants too it also highly recommended. The local administrative should regulate the number of boats operation in the lake as it could leads to oil/geese contamination.

**Acknowledgements**

We are very thankful to Mr. Regi W. Fernando of Environment Studies Division of NARA and Mr. A. J. Jayatissa, Miss R. L. Liyanage and Mr. Stanly Fernando of Inland Aquatic Resources and Aquaculture Division of NARA for their assistance given through out this work.

## References

APHA, (1998)."Standards Methods for the Examination of Water and Wastewater", 20th Edition, American Public Health Association, Washington, D.C, USA.

*Bambaradeniya, C. N. B., S. P. Ekanayake, L. D. C. B. Kekulandala, R. H. S. S. Fernando, V. A. P. Samarawickrama and T. G. M. Priyadharshana (2002),* An Assessment of the status of biodiversity in the Maduganga Mangrove Estuary, *Occasional papers of IUCN Sri Lanka, No. 1: 49Pp.*

Dash, M.C. (2001), Fundamentals of Ecology, 2nd Edition, Tata McGraw-Hill Publishing Company Limited, New Delhi, Indiea.

http//www.epa.vic.gov.au/sustainable Built Environment. (accessed on 04 August 2010).

*http://* www.nps.gov/plants/alien/fact/mysp1.htm *(accessed on 21/07/2010)*

http://www.aquanic.org/management%20practices/.../aquaticplantmanagement.htm *(accessed on 21/07/2010)*

http://www.encyclopedea.thefreedictionary.com. (accessed on 05/08/2010).

*http://*www.encyclopedia.com/doc/1P3-1707342891.html (accessed on 21/07/2010*)*

http://www.esc.nsw.gov.au/weeds/Sheets/aquatic/A%20Ludwigia (accessed on 18/08/2010)

http://w*ww.jetwinghotels.com/jetwingstandrews/gregory_lake.htm (accessed on 12/08/2010)*

http://www.towards-sustainability.co.uk/issues/built/> (accessed on 08/08/2010).

http://www/iucn.org/themes/ssc/pubs/policy/invasiveEng.htm (accessed on 12/08/2010)

http:/www.wri.org/trends/freshwat.html (accessed on 12/08/2010)

Marambe, B. (2001), "Alien invasive plants: Present status and Management", Paper presented at the workshop to Increase Awareness among Customs Officers on Monitoring of Biodiversity related matter, 22nd September, Colombo.

Mason, C. F. (1996), "Water Pollution Biology". In: *Pollution Causes, Effects and Control* 3rd Edition (Harrison, R. M. ed.) 66 – 91, The Royal Society of chemistry, University of Birmingham.

Silva, E. I. L. (1996), "Water quality of Sri Lanka, *A review on twelve water bodies*". Institute of Fundamental Studies, Kandy, Sri Lanka, 141

## About the Authors

**R. R. A. R. SHIRANTHA,** B.Sc. (Sp). Kelaniya, M. Phil. Kelaniya is a Research Officer at the Inland Aquatic Resources and Aquaculture Division, NARA, Sri Lanka. Her research interests are in the areas of freshwater conservation biology, biodiversity and use of aquatic macro-invertebrate as bio-indicators.

**A. A. D. AMARATHUNGA,** B.Sc. (Agri.) (Sp). Peradeniya, M. Sc. in Environment & Forestry Peradeniya is a Research Officer at the Environmental Studies Division, NARA, Sri Lanka. His research interests are in the areas of wetland ecology & management, water pollution & bio indicators, biodiversity & water quality in River basins & lagoons and environmental impact assessment.

**N. W. J. Pushpa Kumara** is a Research Assistant at the Inland Aquatic Resources and Aquaculture Division, NARA, Sri Lanka. His research interests are in the areas of natural resource management and Aquaculture.

# NUISANCE ALGAE IN WATER SUPPLY PROJECTS IN SRI LANKA

Senalie Sethunge and Pathmalal M Manage

Department of Zoology,
University of Sri Jayewardenepura, Gangodawila, Nugegoda, Sri Lanka.
E-mail: path2007ma@yahoo.com

**Abstract**
Sri Lanka is a tropical continental island which consists of 103 rivers and about ten thousand man made tanks. There are over 250 water supply systems constructed using these water bodies since later part of the nineteenth century, reservoirs which are used extensively for domestic and agricultural practices. It is reported that most of these water sources are constantly becoming contaminated with different types of algae making the water unsuitable for human consumption. The present study was carried out to identify toxin producing, filter clogging, taste and odor forming nuisance algae and some physico-chemical parameters in some selected water bodies namely Labugama, Kalatuwawa, Parakramasamudra, Kondawatuwana, Mahaweli intake at Neelapola and Kantale. Sampling for algae and physico cemical parameters were carried out for a period of one year from April 2009 to April 2010. The results of the present study showed that all physico-chemical parameters in the water bodies were within the drinking water quality standards recommended for Sri Lanka. However Species diversity and density of phytoplankton was different in the water bodies. In Labugama and Kalatuwawa, total algae population consisted 10% of cyanotoxin producing *Microcystis aeruginosa,* 10%, 20% and 60% of taste and odor forming *Peridinium* sp., *Microcystis* sp. and *Staurastrum* species respectively. In contrast, 89% of cyanotoxin producing cyanobacteria in Kondawatuwana tank, 50% in Parakramasamudra, 40% in Neelapola and 95% in Kantale were detected. Total filter clogging algae in Parakramasamudra, Neelapola, Kondawatuwana and Kantale were 29%, 69%, 72% and 15% respectively while taste and odor forming algae were 22%, 36%, 72% and 11% respectively. Among the water bodies under study Neelapola recorded the highest occurrence (34%) of filter clogging diatom while the highest percentage of cyanotoxin producing *M. aeruginosa* (62%) and *Cylindrospermpsis* sp. (85%) were recorded from Kondawatuwana and Kantale tanks respectively. During the study period, the lowest percentages of toxic producing algae were recorded from Labugama and Kalatuwawa reservoirs which are exclusively used for drinking purposes indicating their suitability as drinking water sources. In contrast, the highest percentages of toxin producing, filter clogging and taste and odor forming algae were recorded in Kondawatuwana drinking water tank. As for the other water bodies studies showed that there is no immediate threat due to algae.

*Key words: cyanotoxin, filter clogging, nuisance, phytoplankton, taste & odor forming,*
Corresponding Author: Path2007ma@yahoo.com

## 1. Introduction

Cyanobacteria are a group of prokaryotes found all over the world (Carmichael, 1994) and grow in light habitats and prefer neutral or alkaline conditions. Algae have an economic importance and considered as major primary producers which contribute globally to soil and water fertility. Use of algae in food production and in solar energy conversion holds promising potential as a solution for energy crisis (Skulberg, 1995). The great increase in population and the rapid development of agriculture and industry have caused a phenomenal increase in the use of water in recent years and have brought about many difficulties in meeting the demand for water. In addition industrial, agricultural and economic development of the world may on the other hand cause nutrient enrichment in water bodies which may enhance the massive production of algal blooms. These algal blooms may have wide range of social, economic and environmental problems such as formation of foul odours (Silvey & Roach, 1975), depletion of oxygen in underline waters and potential harmful effect on human health due to production of algal toxins (Manage et al, 2009; Gorham & Carmichael, 1988; Falconer, 1989, 1996; Song et al., 1998), water chemistry and subsequent fish kills (Bury et al., 1996), accumulation of toxin in aquatic animals (Lykke & Kirsten, 1999 (*Daphnia magna*)), death of animals (Galey et al., 1987 (Cattle); Matsunaga et al., 1999 (bird), 1988; Zohary & Roberts, 1989).

Among the cyanobacteria, filamentous *Anabaena, Oscillatoria, Nostoc, Aphanizomenon, Hapalosiphon, Cylindrospermopsis* and unicellular *Microcystis* sp. are well known all over the world as common bloom-forming cyanobacteria in ponds, rivers, lakes, reservoirs which are used for irrigation and drinking purposes. Also, it has been documented that some species of algae and cyanobacteria can become nuisance species (Carmichael, 1996). Some species of blue green algae and green algae which produce secondary metabolites lead to foul tastes and odors in water and some species of diatoms causes filter clogging problems in water purification systems when they become abundant in the environment (Palmer, 1959). Furthermore, over 100 of cyanobacteria species belonging to 40 genera have been recognized as toxic. Among them *Microcystis*, *Anabaena*, *Lynbya*, *Nostoc*, *Nodularia*, *Cylindrospermopsis*, *Oscillatoria* are some of the most common toxigenic cyanbacteria species (Jayatissa et al, 2006, Pathmalal, 2009).

Sri Lanka, has 103 rivers radiating from hill country and about ten thousand man made tanks and several water supply systems have been constructed since later part of the 19th century.(Ferdinando, 2006). Some drinking water bodies have built by damming across the rivers and some are by diverting the rivers. For example, the Kalatuwawa reservoir which supplies water to Colombo and greater Colombo area, has been built by damming across the Kelani River, while Amban ganga feeds Parakramasamudra which supplies water to Polonnaruwa area through a diversion anicut (Ferdinando, 2006). Most of these water bodies are used both as drinking sources and also for irrigational purposes. However some reservoirs are used exclusively as drinking sources (eg: Labugama and Kalatuwawa).

Recent studies have been revealed that some of the water bodies in Sri Lanka have contaminated by toxigenic cyanobacteria (Jayatissa et al, 2006, Pathmalal, 2009). In such studies, about forty species of cyanobacteria belonging to twenty four genera have been reported from drinking, irrigation and recreational water bodies. Of these *Microcystis aeruginosa* was recorded as the most common nuisance cyanobacteria in most of the water bodies in Sri Lanka (Silva and Wijeratne, 1999; Pathmalal, 2009).

The objective of the present study was to identify the nuisance algae in some selected water bodies which are being used as drinking water sources.

## 2. Materials and Methods

### *2.1 Study area*

The Kalatuwawa, Labugama, which are used for drinking purposes, Parakramasamudra and Kantale reservoirs; Neelapola intake of Mahaweli river and Kondawatuwana tank which are used for drinking and irrigation purposes were selected to collect water and plankton samples for the present study. The water bodies are situated in lowland wet zone i.e Labugama and Kalatuwawa reservoirs and low land dry zone i.e Parakramasamudra and Kantale reservors, Neelapola intake, Kondawatuawna tank of the country (CEA, 2006).

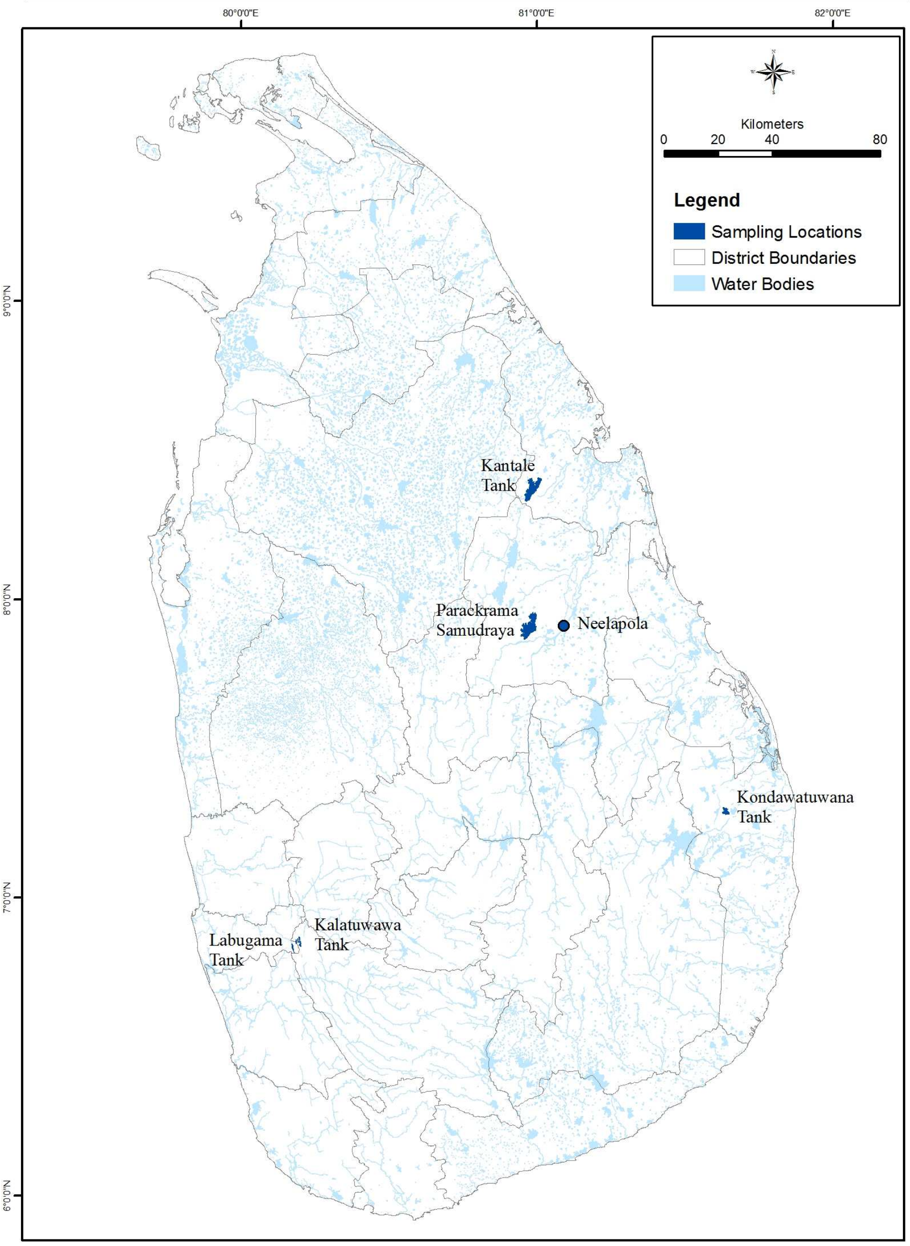
Labugama reservoir is an impoundment of Wak Oya in the Kelani basin and has a catchment area of 2835 acres at an elevation of 360 ft above the sea level (Ferdinando, 2006). Kalatuwawa reservoir was build-up by damming across a sub tributary of Kelani River and its submerged area is 482 acres and the catchment area is 3320 acres (Ferdinando, 2006). Parakramasamudra is fed by perennial water source from Ambanganga through a diversion anicut and the capacity is 1110,000 Acft (Kamaladasa, 2007).

Kondawatuwana tank is a shallow and stagnant water body situated in Ampara district and used for drinking and irrigation purposes. Kantale reservoir receives irrigation water diverted from Mahaweli River and Neelapola intake of Mahaweli River distributes water to eastern part of the country and expects to develop as drinking water source under the secondary town and rural community base water supply and sanitation project of 4th phase of the ADB project.

### *2.2 Sampling*

Sampling was carried out from April 2009 to April 2010. Water samples in triplicate were collected during 9.00 am to 11.00 am at 10 cm below the surface. Water temperature, pH, conductivity and dissolved oxygen were measured at the site using a thermometer, pH meter (330 I/ Set, WTW Co., Weilheim, Germany), a conductivity meter (340A-Set 1. WTWCo., Weilheim, Germany) and an

oxygen meter (Oxi 320/ Set, WTW Co., Weilheim, Germany) respectively. Spectrophotometric analysis of nitrogen in the form of nitrate ($NO_3^{-1}$-N),) and total phosphate ($PO_4^{-3}$-P), were done in the laboratory.
To determine species composition of algae, five surface plankton samples from five sampling points in each water body were collected using 55µm plankton net while moving the boat for a known period of time. Immediately after collection, a 100 ml sample was fixed with acidified Lugol's solution to a final concentration of 1% and kept overnight for natural sedimentation. Enumeration was done using a




Sedgwickrafter counting chamber under a light microscope.

**Fig. 1 sampling locations of the present study**

## 3. Results

pH ranged between 7.3-8.2 both in Labugama and Kalatuwawa reservoirs. pH in Parakramasamudra ranged between 7.8- 8.5 and in Kondawatuwana, Kantale and Mahaweli Neelapola intake the pH values remained with in the range of 7.3 to 7.6. Mean temperature of Kalatuwawa and Labugama reservoirs and Parakramasamudra were 29.0 ±0.02 ºC during the study period. In contrast, Kondawatuwana (30.1 ±0.2 ºC), Neelapola (32.3 ±0.02 ºC) and Kantale (30.0 ±0.04 ºC) reservoirs showed higher mean temperatures compared to the lowland wet zone reservoirs. Mean conductivity of Labugama and Kalatuwawa remained at 16.8±0.02 µS/cm and the mean conductivity values of Parakramasamudra, Kondawatuwana, Neelapola and Kantale reservoirs varied and are shown in Table 1. The concentration of alkalinity, total phosphate and nitrate in each water body was different and given in the table(Table 1). All physico-chemical parameters of the water bodies tested were within the drinking water standard recommended by NWSDB and BOI.

**Table1:** *Mean values of some selected physico-chemical parameters in the reservoirs during the study period.*

| **Water body** | **pH range** | **Temperature ºC** | **Conductivity µS/cm** | **Alkalinity ppm** | **$PO_4^{-3}$ $mgl^{-1}$** | **$NO_3^{-1}$ $mgl^{-1}$** |
|---|---|---|---|---|---|---|
| Labugama | 7.8-8.5 | 29.0±0.02 | 16.8±0.2 | 30±0.6 | 0.06 | <0.1 |
| Kalatuwawa | 8.12-8.32 | 29.0±0.02 | 16.9±0.11 | 42±0.5 | 0.07 | <0.1 |
| Parakramasamudraya | 7.8-8.5 | 29±0.2 | 215±1.5 | 107±0.9 | 0.46 | <0.1 |

| | | | | | | |
|---|---|---|---|---|---|---|
| Kondawatuwana | 7.3-7.6 | 30.1±0.2 | 113±0.9 | 49±0.8 | 1.5 | <0.1 |
| Neelapola | 7.3-7.6 | 32.3±0.06 | 191.1±2.5 | 91±0.6 | 0.44 | <0.1 |
| Kantale | 7.2-7.6 | 30.0±0.04 | 327.3±1.6 | 125±1.1 | 0.60 | <0.1 |
| Recommended BOI Standards (maximum permissible level) | 6.5-9.0 | | 3500 | 600 | 2.0 | 45 |

**Table 2:** *Toxin producing, filter clogging and taste and odor forming algae found in the present study*

| **Reservoir** | **Toxic algae** | **Filter clogging algae** | **Taste & Odor forming algae** |
|---|---|---|---|
| Labugama | **Cyanobacteria**<br>*Microcystis aeruginosa** | **Cyanobacteria**<br>*Microcystis* sp. | **Cyanobacteria**<br>*Microcystis* sp.<br>**Chrysophyta**<br>*Peridinium* sp. |
| Kalatuwawa | **Cyanobacteria**<br>*M.aeruginosa** | **Cyanobacteria**<br>*Microcystis* sp. | **Cyanobacteria**<br>*Microcystis* sp.<br>**Chrysophyta**<br>*Peridinium* sp. |
| Parakrama samudraya | **Cyanobacteria**<br>*M. aeruginosa**<br>*Cylindrospermopsis* sp.*<br>*Anabaena* sp.** | **Bacillariophyta (Diatoms)**<br>*Melosira* sp.<br>*Fragilaria* sp.<br>**Cyanobacteria**<br>*Anabaena* sp.<br>*Microcystis* sp. | **Cyanobacteria**<br>*Microcystis* sp.<br>**Chrysophyta**<br>*Peridinium* sp. |
| Kondawatuwana | **Cyanobacteria**<br>*M. aeruginosa**<br>*Cylindrospermopsis* sp. | **Cyanobacteria**<br>*Microcystis* sp. | **Cyanobacteria**<br>*Microcystis* sp. |
| Neelapola | **Cyanobacteria**<br>*M. aeruginosa**<br>*Cylindrospermopsis* sp.*<br>*Anabaena* sp.** | **Bacillariophyta (Diatoms)**<br>*Melosira* sp.<br>**Cyanobacteria**<br>*Microcystis* sp.<br>*Anabaena* sp. | **Bacillariophyta (Diatoms)**<br>*Melosira* sp.<br>**Cyanobacteria**<br>*Microcystis* sp. |
| Kantale | **Cyanobacteria**<br>*M. aeruginosa**<br>*Cylindrospermopsis* sp.* | **Bacillariophyta (Diatoms)**<br>*Melosira* sp.<br>*Navicula*sp.<br>**Cyanobacteria**<br>*Microcystis* sp. | **Cyanobacteria**<br>*Microcystis* sp. |

- *Hepatotoxic (microcystins/Cylindrospermposins**Nurotoxic (Anatoxin-a/ Anatoxin (s)*

Table 2. illustrates the species composition of toxin producing, filter clogging and odor forming algae detected in the present study. Species composition of algae was different in each water body. All water bodies tested in the present study were contaminated with toxin producing, filter clogging and taste and odor forming algae during the study period (Table 2).

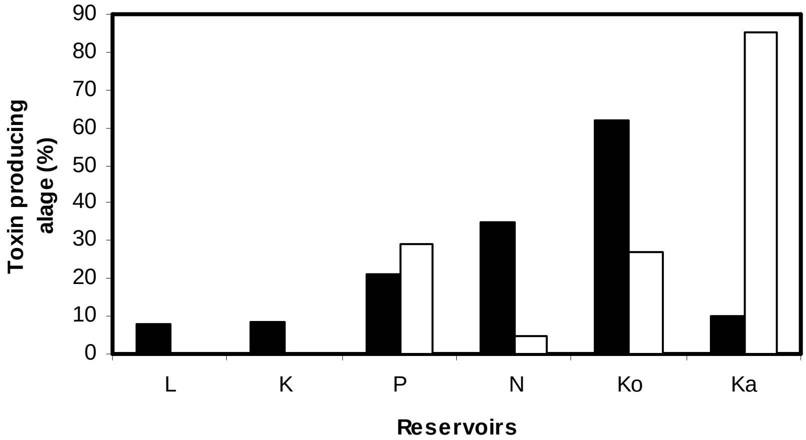



**Fig 2:Percentage of toxin producing algae in Labugama (L), Kalatuwawa (K), Parakramasamudra (P), Neelapola (N), Kondawatuwana (Ko), and Kantale (Ka) reservoirs ( ■*M. aeruginosa,* □*Cylindrospermopsis* sp.)**

The toxin producing algae, *M. aeruginosa*, 8% and 8.5% of the total algae population respectively in the Labugama and Kalatuwawa reservoirs (Fig.2). Comparatively, percentages of *M. aeruginosa* (21%) and filamentous *Cylindrospermopsis* sp. (29%) in the total algae population were high in Parakramasamudra. The highest density of *M. aeruginosa* (62%) was detected in Kondawatuwana tank while 27% of *Cylindrospermopsis* sp. In the Neelapola Mahaweli water intake the two species *M. aeruginosa* (35%) and *Cylindrospermopsis* sp. (5%) were reported and in Kantale reservoir *M. aeruginosa* (10%) was low compared to other low land dryzone reservoirs. *Cylindrospermopsis* sp (85%) was the dominant cyanobactera found in Kantale reservoir.
*Microcystis* sp. which is considered as filter clogging algae recorded in Labugama and Kalatuawawa reservoirs were 9.5% and 10% respectively (Fig.3). In the Parakramasamudra *Microcystis* sp. (21%) *Melosira* sp. (7%), *Aanabaena* sp. (0.5%) and *Fragilaria* sp. (0.1%) were recorded as filter clogging algae and in the Neelapola Mahaweli water intake considerable densities of *Microcystis* sp. (35%) and *Melosira* sp. (34%) were recorded. In Kondawatuwana *Microcystis* sp. (72%) was identified as the dominant species and in Kantale reservoir three filter clogging algae namely *Microcystis* sp. (10%), *Melosira* sp. (5%) and *Navicula* sp. (0.5%) were detected during the study period.

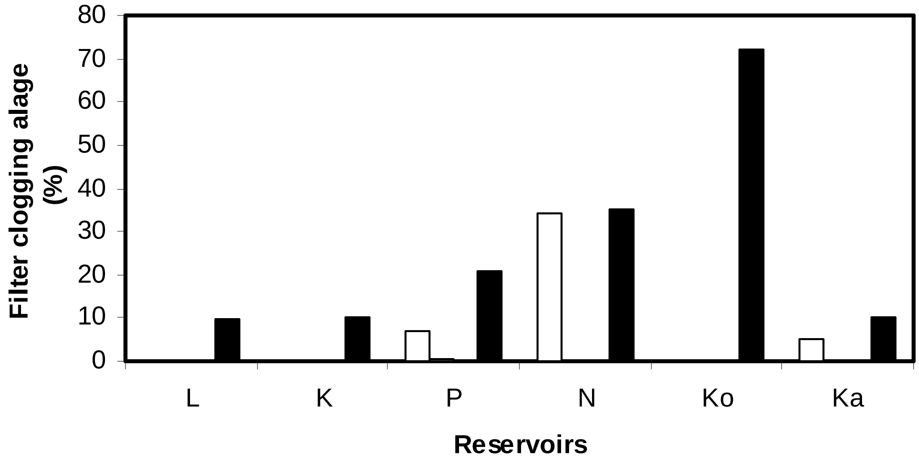



**Fig 3:Percentage of filter clogging algae in Labugama (L), Kalatuwawa (K), Parakramasamudra (P) Neelapola (N), Kondawatuwana (Ko) and Kantale (Ka) reservoirs (** ■ *Microcystis* sp., □ *Melosira* sp.)

In Labugama reservoir *Microcystis* sp. and *Peridinium* sp. were detected as taste and odour forming algae and they constituted 9.5% and [illegible]% of the total algae population respectively (Fig.4). *Peridinium* sp., *Microcystis* sp. and *Staurestrum* sp. were identified as taste and odour forming algae in Kalatuwawa reservoir and they contributed 15%, 10% and 45% of the total algae population respectively. Percentage contribution of taste and odour forming *Microcystis* sp. was detected as 21% and *Peridiniun* sp. and *Scenedesmus* sp. were less than 0.5% in Parakramasamudra where as in Neelapola Mahaweli water intake 35% of *Microcystis* sp. and 0.85% of *Scenedesmus* sp. were recorded. In addition, 1.2% of *Closterium* sp. was also recorded as taste and odour forming algae in Neelapola and 0.2% of *Scenedesmus* sp. (data not shown) and 10% of *Microcystis* sp. were recorded in Kantale reservoir. In Kondawatuwana 72% of *Microcystis* sp. was recorded as taste and odor forming algae.

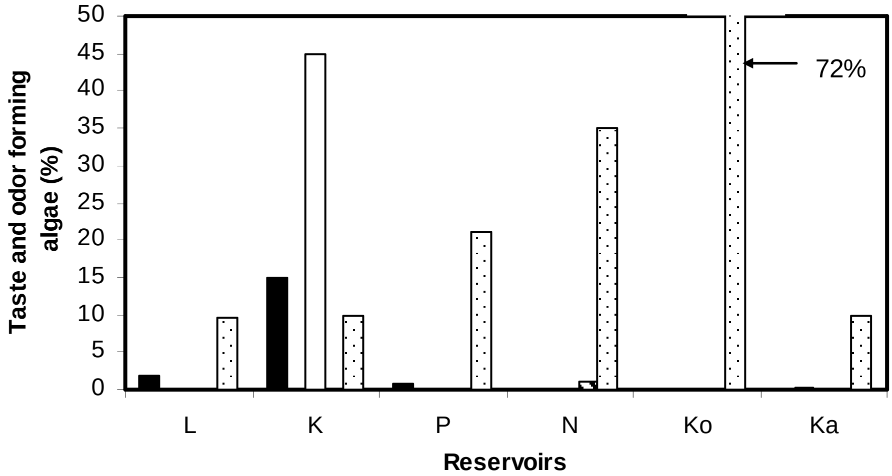



**Fig 4: Percentage of Taste and odor forming algae in Labugama (L), Kalatuwawa (K), Parakramasamudra (P) Neelapola (N), Kondawatuwana (Ko) and Kantale (Ka) reservoirs( ■ P*eridinium* sp., □ *Staurastrum* sp., ⬚ *Microcystis* sp., □ Closterium sp.)**

## 4. Discussion

Filter clogging, toxin producing, taste and odor forming algae were detected in different percentages in different water bodies. Previous studies have shown that the phytoplankton community varied from reservoir to reservoir irrespectively of their geographical locations and hydrological regime (Silva and Wijeratne, 1999). pH, temperature, conductivity and alkalinity values remained within the drinking water quality standards during the study period. These parameters were also favourable for growth of algae. The generally accepted phenomenon is that the occurrence of cyanobacteria is triggered by nutrient enrichment (Hutchinson, 1973).For instance the Kondawatuwana tank had higher concentrations of total phosphate (value the recommended standards) compared to the other water bodies which may have led to have high growth of cyanbacterium *M. aeruignosa* than all other water bodies.

The most common toxin producing algae *M. aeruginosa* in Labugama and Kalatuwawa reservoirs were the lowest compared to the other water bodies which show less or no contamination of cyanotoxins. This is a good indication that the water in these two reservoirs is suitable for drinking and domestic uses. The low levels of *M. aeruginosa* may be due to limiting nutrients such as nitrates and phosphates for the cyanobacteria growth in the catchment of these two reservoirs. It is known that most toxin producing and bloom forming cyanobacteria needs high concentrations of phosphorous for their mass growth and it has been reviewed that in lowland reservoirs in Sri Lanka, phosphorous is known to be a limiting factor (Jayatissa et al, 2006) and both Labugama and Kalatuwawa reservoirs situated in lowland wet zone thus, the limited phosphorous concentrations of the reservoirs may be the reason for the low abundance of bloom forming *Microcystis* species.However the two reservoirs, showed highest percentages of taste and odor forming *Peridinium* sp. *and Staurastrum* sp. *Staurastrum* sp. gives a grassy odor to drinking water while *Peridinium* sp. and *Microcystis* sp. give fishy and septic odors to water when they are abundant (Palmer, 1959). The results of the present

study is indicating that continuous monitoring of algae and water quality of these two reservoirs is needed to evaluate the level of algae density to ascertain the potential impact of odor forming algae.
The results of the present study showed that the highest percentages of toxin producing, filter clogging and taste and odor forming algae such as *M. aeruginosa*, other *Microcystis* sp. and *Cylindrospermopsis* sp. are recorded in Kondawatuwana water tank. This is an indication that the water in this reservoir contaminated, toxin producing, filter clogging and taste and odor forming algae thus needs a proper treatment scheme before using as drinking and domestic water source. The generally accepted phenomena is that nitrogen limited aquatic environments promote the growth of cyanobaceria when phosphorous is not limited (Silva and Samaradivakara, 2005). In Kondawatuwana nitrogen limitation may not be the single factor of having high percentages of cyanobacteria.
Compared with the other water bodies studied, a considerable percentage of nuisance algae were present in Parakramasamudra which is situated in lowland dry zone. This may due to the effect of hydrological flushing and dilution on the limnetic biomass with the water level in Parakramasamudra which governs the high densities of phytoplankton (Silva & Schiemer). Dissolved silica and its relationship to other reactive micro nutrients may be a key factor regulating the growth of centric diatom *Aulacoseira granulate* in the humid tropics (Adeniji, 1977, Silva & Samaradivakara, 2005). In Neelapola water intake the highest percentages of filter clogging algae, *Melosira* sp. was reported and it may be due to the concentration of dissolved silica during the rainy season resulting an increase in the relative proportion to total phosphorous which in turn may have enhanced the growth of *Melosira* sp.
The findings of the present study indicate that favorable physico-chemical factors and nutrients, especially phosphorous which is a limiting factor have favored the growth of algae particularly certain species which have increased to nuisance levels which are capable of producing toxins, clogging filters and forming foul odors and tastes in these water sources under study.
Therefore, the results of the present study emphasises the importance of carrying out further monitoring studies, including catchment and runoff monitoring to identify the potent nutrient input to reservoirs, water quality and reservoir management practices and ascertain potential impact of anthropological and natural activities on water quality which positively affect nuisance algae production. These information can help in introducing control measures to control the nuisance algae in order to supply safe drinking water for future generations.

## Acknowledgements

This study was financially supported by the University of Sri Jayewardenepura, grant in aid for research (ආ.ශි.පු./06/පර්/2009/20) for the university academics. We grateful to the National Water Supply and Drainage Board (NWSDB) for their support during the study period. Also, we thank academic and non academic members of the department of zoology for their support extended to the research work.

## References

1. Adeniji,H.A.(1977), Preliminary study of the silica content and diatom abundance in Kainji Lake, Nigeria. In: Interaction between sediments and freshwater (H.L. Goltermann ed.), p.331-334,Junk & PUDOC, The Hague.
2. Bury, N.P., Eddy, F.B & Codd, G.A. (1996), "Stress responses of brown trout, Salmo turtle to the cyanobacterium, *Microcystis aeruginosa*", *Environmental Toxicology and Water Quality* 11 (3): 187-193.
3. Carmichael, W.W. (1994), "The toxins of cyanobacteria", *Scientific Am Jan*. 270: 78-86.
4. Carmichael, W.W. (1996), Toxic *Microcystis* and the environment. In Toxic *Microcystis*.ed.Watanabe, M, p.1-13. Florida: CRC press.
5. Central Environmental Authority (CEA) (2006), The World conservation Union (IUCN), International Water Management Institute (IWMI), Colombo (2006), p. 114-115, National wet land directory of Sri Lanka.
6. Falconer, I.R. (1989), "Effects of human health of some cyanobacteria (blue-green algae) in reservoirs, lakes and rivers", *Toxicity Assessment* 4: 175-184.
7. Falconer, I.R. (1996), "Potential impact on human health of toxic cyanobacteria", *Phycologia* 35: 6-11.
8. Ferdinando, D.N.J. (2006), Water supply and sewerage development in Sri Lanka. p. 26 -32, Centenary commemoration publications of Irrigation Department.

9. Galy, F.D., Beasley, V.R., Carmichael, W.W., Kleppe, G., Hooser, S.B & Haschek, W.M. (1987), "Blue-green algae (*Microcystis aeruginosa*) heapatotoxicosis in dairy cows", *American Journal of Veterinary research* 48 (9) 1415-1420.
10. Gorham, P.R & Carmichael, W.W. (1988), Hazards of freshwater blue-green algae (cyanobacteria). In: Lembi, C.A & Waland, J.R (Eds) Algae and Human Affairs. Cambridge University press, Cambridge, p403 Granhall U, Berg B (1972), "Antimicrobial effects of *Cellivibrio* on blue-green algae", *Arch Mikrobiol* 84: 234-242.
11. Hutchinson, G.E. (1973), "Eutrophication", *American scientist* 61: 269-278.
12. Jayatissa, L.P., Silva, E. I.L., McElhiney, J & Lawton, L. A. (2006), "Occurrence of toxigenic cyanobacterial blooms in fresh waters of Sri Lanka", *Systematic and Applied Microbiology* 29: 156-164.
13. Kamaladasa, B. (2007), Irrigation development in Sri Lanka, p.14-16 Centenary commemoration publications of irrigation Department.
14. Lykke. T & Kirsten, C. (1999), "Accumulation of Microcystin in *Daphnia magma* feeding on toxic *Microcystis*", *Archiv fuer Hydrobiologia* 145: 447-467.
15. Matsunaga, H., Harada, K.I., Senma, M., Ito, Y., Yasuda, N., Ushida, S & Kimura, Y. (1999), "Possible cause of unnatural mass death of wild birds in a pond in Nishinomiya, Japan: Sudden appearance of toxic cyanobacteria", *Natural Toxins* 7 (2): 81-84.
16. Mervin Palmer, C. (1959), Algae in water supply.p14-16, Bureau of State Services, Division of Water Pollution Control, Robert A. Taft Sanitary Engineering Center. Health Service Publication No. 657.US Government Printing Office, Washington 25,DC
17. Pathmalal M Manage (2008), Rapid detection of Microcystin in the field. Dec. 27-28, International Symposium of Forestry and Environment Sciences, Kalutara, Sri Lanka.
18. Pathmalal M. Manage, Chritine Edwards, Brajesh K. Singh and Linda A. Lawton (2009), "Isolation and identification novel microcystin degrading bacteria", Accepted for Applied Environmental Microbiology AEM. 01928-09.
19. Skulberg, O.M., Codd, G.A & Carmichael, W.W. (1984), "Toxic blue-green algae bloom in Europe", *A growing problem, Ambio* 13: 244-249.
20. Silva, E.I. L & Samaradivakara S.R.M.S. (2005), "Limnology of Kandy Lake before the outbreak of a cyanobacteria bloom in May 1999. 111 Phytoplankton composition and succession", *Sri lanka J.Aquat.sci*.10: 55-71.
21. Silva, E. I. L & Schiemer, F. (2000), "Hydraulic changes of three reservoirs (Minneriya, Udawalawa and Victoria) in Sri Lanka", *Sri lanka* J. *Aquat. Science* 5:75-86.
22. Silva, E.I.L & Wijeratne, M.J.S. (1999), "The Occurrence of Cyanoabcteria in the Reservoirs of Mahaweli basin in Sri Lanka", *Sri lanka J. Aquat*. Science 4:51-60.
23. Silvey, J.K & Roach, A.W. (1975), The taste and odour producing aquatic actinomycetes. CDC Crit. Rev. Environ. Control, 233-273.
24. Song, L., Sano, T., li R., Watanabe, M.M., liu Y& kaya, K. (1998), "*Microcystin* production of *Microcystis viridis* (Cyanobacteria) under different culture conditions", *Jpn J Phycol* 46:19-23.
25. Zohary, T & Roberts, R.D. (1989), "Diurnal mixed layer and the long-term dominance of *Microcystis aeruginosa*", *J Plankton Res* 11: 25-30.

# HEALTH HAZARDOUS: NITRATE-N IN GROUNDWATER AND SOIL IN INTENSIFIED AGRICULTURAL AREAS

T. Jeyaruba[1], M. Thushyanthy[2]

[1]Post graduate student, Post Graduate Institute of Agricultulture, Peradaniya, Sri Lanka.
[1]E-mail: agrosruba2000@yahoo.com
[1]Telephone: +94-776618095

[2]Senior Lecturer, Department of Agric. Engineering, University of Jaffna, Jaffna, Sri Lanka.
[2]E-mail: thushym@jfn.ac.lk
[2]Telephone: +94-21-2222404; Fax: + 94-21-2222973

**Abstract:** The study was focused on estimation of nitrate-N in groundwater and soil in intensive agricultural areas mainly on Valikamam East, Jaffna, Sri Lanka. Groundwater samples from sixty eight wells were collected from the intensive agricultural areas and an analysis was done periodically from July 2007 to February 2008 throughout dry and wet season for nitrate-N concentration. Out of sampled area, some of the areas were selected for soil sampling to see the nitrate level in the soil. Nitrate-N in the groundwater and soil was determined by brucine method. The nitrate- N vary in all the months in sixty eight wells and values were ranged from 0.1mg/l to 17.83 mg/l. Out of sixty eight tested wells, 80% of the wells were not recommended for drinking water in intensified agricultural areas and all the wells were accepted for irrigation requirement. High concentration of nitrate-N was observed till 0- 40 cm of soil profile and the concentration was low below the top layer. There was a good correlation between soil nitrate-N to groundwater nitrate-N.

***Keywords:*** *Soil Nitrate- N, groundwater Nitrate-N, Intensive Agriculture, Limestone*

## 1. Introduction

Groundwater plays major role in fresh water consumption of human in several countries, including Sri Lanka. Groundwater contamination is a major problem where the places mainly depend on groundwater sources for drinking purposes. The contamination occurs in several ways. Among them inorganic nitrate pollution in groundwater is one of the most common pollution scenario. Groundwater is the major natural water resource in the Jaffna Peninsula, Sri Lanka and it is used for domestic, agricultural and industrial purpose. The population of Jaffna Peninsula is entirely dependent on the groundwater resources for all the purposes with seasonal rainfall. The limestone aquifer has several isolated caves and caverns capable of storing groundwater without evaporation losses. The availability of fresh water is limited and the entire groundwater is generated from percolated rainfall and it forms a fresh water lens above the sea water. After the rainfall, 10-15% of rain water runs off and about 40-48% is lost by evaporation, only 30-32% of rainfall is left over for groundwater recharge [1]. Among the available water in the Peninsula, requirement of 80% is being extracted from the limestone aquifer through open shallow dug wells and deep tube wells [2].

Increasing of the population, the demand of water is also relatively increasing and various human activities have been causing several serious problems, such as nitrate pollution, saline intrusion and bacterial multiplication[3]. In 1983, Gunasegaram [2] studied extensively groundwater contamination in the Jaffna Peninsula and found that the nitrate levels exceeded WHO limits, which is due to the mixing up of abundant nitrogenous waste matter and synthetic and animal fertilizers reaching the shallow groundwater table. This was supported by Mageswaran and Mahalingam in 1984 [4] that high nitrate-N content was in the well water and soil. In 1985, Dissanayake and Weerasooriya [5] pointed out in hydro geochemical atlas of Sri Lanka that Jaffna Peninsula has the highest nitrate content among the groundwater of Sri Lanka. Studies conducted in Jaffna by Nagarajah *et al*. in 1988[6] also substantiated the high concentration of nitrate in groundwater.

Excess nitrate in drinking water affects especially infants and older children, pregnant and nursing mothers. An increasing level of nitrate in groundwater induces health related problems. Increased nitrogen in the soil also may cause serious health problem because some plants such as carrots could store this excess nitrate then reduce it partly to nitrite within it self. The nitrite could convert haemoglobin to methaemoglobin or produce nitrosamines and thus the carrots containing excess nitrite is health hazard [4]. Hence the study was focused with the objective of assessment of nitrate-N in groundwater and soil in the intensive agricultural areas of Jaffna Peninsula.

## 2. Materials and Methodology

### *2.1 Selection of the well and collection of water and soil samples*

In the intensive agricultural areas, totally sixty eight wells were selected randomly from different cropping discipline mainly in Valikamam East, Jaffna, Sri Lanka . All the selected wells were used not only for irrigation but also for drinking purpose. Groundwater samples were drawn from 15 cm below the surface area of the wells by water sampler for a period of eight consecutive months beginning from July 2007 to February 2008, at monthly interval. Samples bottles were prepared to collect the water samples to meet prerequisites of chemical analysis.

Six locations were selected to collect soil sample to analyze nitrate-N concentration at different land use. The selected locations were near the selected wells and with in the field. Table 1 shows the locations, depths up to which the soil sample was collected and the land use of the field. The maximum depth was tried up to 150 cm in the soil profile. But the collection of soil samples in the paddy field was not possible beyond 25 cm by auger. At the same time, some of the high land crop fields also failed to collect soil samples up to 150 cm due to the interruption of stones.

*Table 1: Soil sampling locations with land use and depth*

| Place | Crop type | Depths (cm) |
|---|---|---|
| Thirunelvely | High land crops | 25, 50, 75, 100, 125, 150 |
| Thirunelvely | High land crops | 25, 50, 75, 86 |
| Kopay | High land crops | 25, 50, 75, 100, 125, 134 |
| Irrupalai | Paddy field | 25 |
| Neervely | Banana field | 25, 50, 75, 100, 125, 150 |
| Neervely | Perennial crops | 25, 50, 75, 100, 125 |

### *2.2 Chemical analysis of water and soil samples*

Nitrate, ammonium and nitrite were extracted from soil by common reagent 2M KCl [7]. The nitrate-N content of groundwater and soil extract was determined colorimetrically using the Brucine method [8]. Rainfall data was obtained from meteorological department, Jaffna during the study period as secondary data to see the correlation between rainfall and measured nitrate-N in the groundwater. Height of the water surface from the reference point was measured at every sampling time by measuring tape.

### *2.3 Statistical analysis*

All the measured data were statistically analysised by using SAS program version 8.0. The significant different between months and season were observed for nitrate-N.

## 3. Results and discussion

### *3.1 Nitrate-N in groundwater in the intensive agricultural areas*

Of the sixty eight wells measured, results showed that 20 % of well water was with nitrate-N content of less than 8 mg/l and 12 % were within the critical range of 8 mg/l to 10 mg/l and 68 % were with value of above 10mg/l. The nitrate-N was ranging from 0.1 mg/l to 17.83 mg/l. Figure 1 shows the mean nitrate-N concentration with deviation in all selected wells. The highest value of nitrate-N was observed as 17.83 mg/l at Kondavil. Most of the wells were exceeded the WHO standard [9] of drinking water quality. These wells are mainly used for agriculture. But, when the farmer and family members reside within the farms and labourers who works in their farms uses the well water for drinking. The higher deviation of nitrate-N (Figure 1- Well 36) was due to flowing of runoff water into the well since it has not extended wall above the soil surface. Runoff water collects all fertilizers over the land area, which leads to higher variation of nitrate-N in groundwater. If top of a well is not constructed to divert surface water away from a well, nitrate-N can enter the well from above and increase its concentration in the water. In fact, in many of the farm wells sampled, the tops were either not constructed high enough or were badly damaged so that surface water could easily enter the wells during rainy seasons.

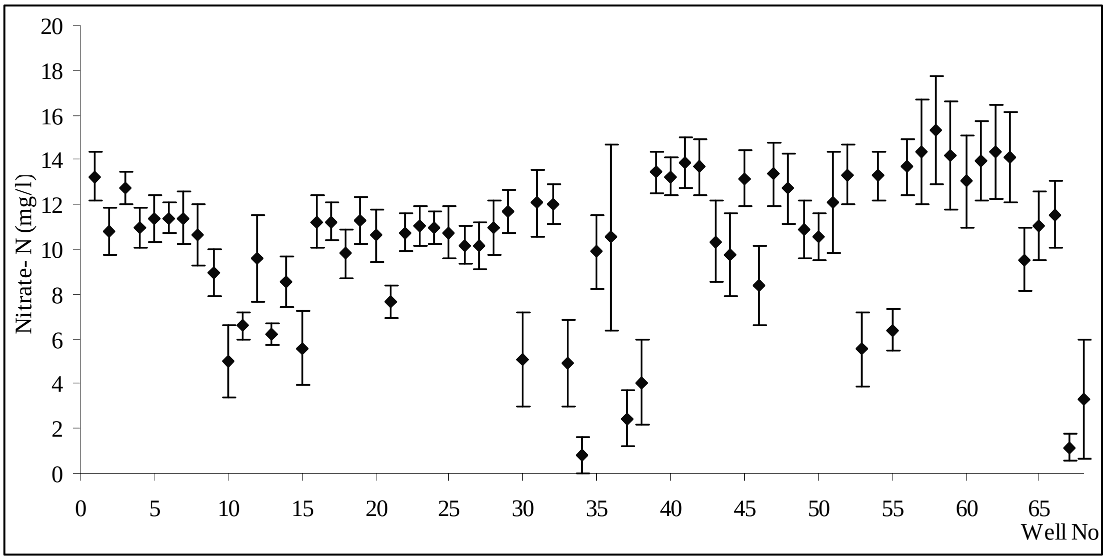



*Figure 1: Mean nitrate-N concentration in groundwater in intensive agricultural areas*

Nitrates are variously associated with diseases like methaemoglobinemia, gastric cancer, thinning of blood vessels, aggressive behavior and hypertension. Sivarajah [10] mentioned that higher incidence of cancer in Jaffna Peninsula due to higher nitrate level in the groundwater. Panabokke [11], in five year study on the geographical pathology of malignant tumour in Sri Lanka, was presented data on investigation of 24,029 biopsy specimens. According to this study, Northern Province showed the highest incidence (184 per 100,000 populations) of malignant tumours in biopsy material among the nine provinces of Sri Lanka.

High nitrate levels recorded in well waters of the Jaffna Peninsula’s agricultural areas was very likely related to the intensive cultivation practiced in that region. It is a well known fact that farmers in this region apply very large amounts of animal wastes, green manures and crop residues in addition to heavy applications of inorganic fertilizers and agrochemicals. Additionally, irrigation from wells is also provided at a higher rate and frequency. Water is applied to the crops (chilli, onion, tobacco, vegetables etc) through flood irrigation. Also, the limestone aquifers are covered by a thin mantle of highly permeable red yellow latosols, rapid movement of any nitrate-N not utilized by crops can reach the aquifers resulting in high nitrate levels.

The recommendation of WHO [9] for nitrate-N for irrigation purposes is in the range of 5 to 30 mg/l. All the wells concentration was less than the recommended level of WHO. The presence of high nitrate in the irrigation water also effects the concentration of nitrate in the vegetable product.

The above mentioned problem occurs not only in Jaffna Peninsula but also some other parts of the Sri Lanka. Vaheesar [12] showed that the highest nitrate content was observed at Mamunai, Batticaloa district as 96.60 $mg/dm^3$ and out of tested thirty three wells, 85% of the wells contained nitrate concentration under the safe level and only 15% of the wells had nitrate content of greater than 45 $mg/dm^3$. Kurupuarachchi and Fernando [13] stated that increase in nitrate concentration is approximately 1 – 2 mg/l per year in Kalpittiya. Finally the study was concluded that 80 % of the well was not recommended for drinking in intensified agricultural areas and all the wells were accepted for irrigation requirement.

***3.2 Temporal variation of nitrate- N with rainfall***
The variation of water level from the soil surface was from 0.41 m to 3.16 m during July to December. Figure 2 shows fluctuation of groundwater table with rainfall in some selected wells. All the wells show the same pattern of fluctuation.

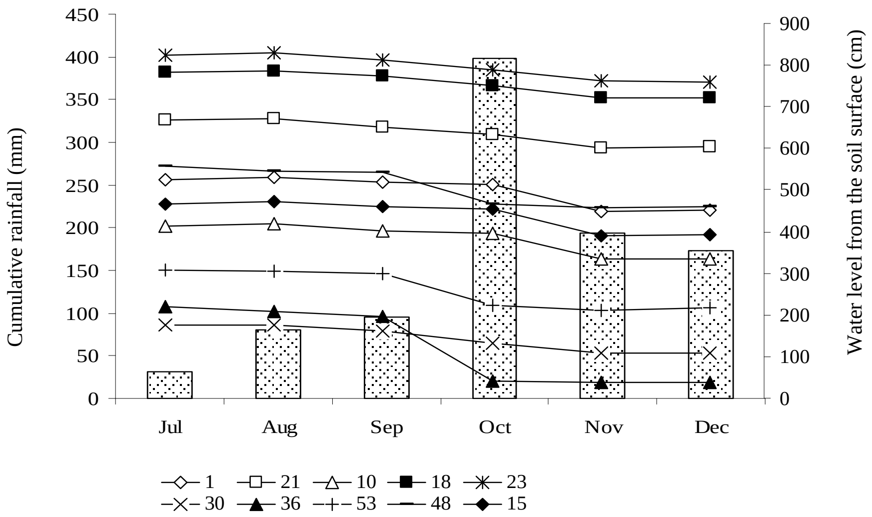



*Figure 2: Fluctuation of groundwater table with rainfall in some well*

Kuruppuarachi *et al.* [14] showed that nitrate concentration of groundwater in dug wells exceeded the limit in agricultural lands at Kalpitiya in regosols. That groundwater nitrate concentration showed a pronounced seasonal variation with peak values as high after the period of *Maha* rains (October to February) and is associated with a general rise in the water table. A smaller increase in nitrate concentration during April – June may occur as a result of excessive irrigation during the *Yala* season with consequent leaching of the nutrients.

The concentration of nitrate increases is lesser amount of situation during dry season, because of gradual slow leaching of fertilizers. After rainfalls starts in October, the concentration of nitrate was increased in the agro well. Increment in agro well is due to leaching from adjoining cultivated lands with high application rate of fertilizers and also due to more agricultural activities during rainy season. During rainy season, the soil will be wet enough up to the water table for nitrate leaching.

Figure 3 shows the fluctuation of nitrate-N concentration in groundwater with rainfall in selected wells. The highest concentration of nitrate nitrogen occurred during the October after that the concentration was reduced during November because of high recharge to the well which dilutes the concentration of nitrate in high land and mixed crop .Again the concentration was increased during December due to the continuous leaching of nitrate –N from the soil. Nandasena *et al.* [15] reported that the rainfall influences the distribution of nitrate-N in the groundwater by raising or lowering of the groundwater table. Rainy season coming just after a well-aerated condition of a soil, easy migration of nitrate-N from topsoil into the relatively shallow water table could occur which results in high concentrations of nitrate-N in groundwater.

Table 2 shows the statistical analysis of variation among monthly data. Significance different between monthly mean nitrate-N value of October significantly differed from July, August, November and December. Monthly mean nitrate values was not significantly differed between September and October while monthly mean nitrate value of the September was not significantly differed from July, August, November and December. Significant effect of October may be due to the effect of the heavy rainfall which influences the recharge. It was supported by high water level in the well (Figure 2). Monthly mean nitrate values of the water samples not significantly differed among the seasons while there was significant interaction found between the season and months.

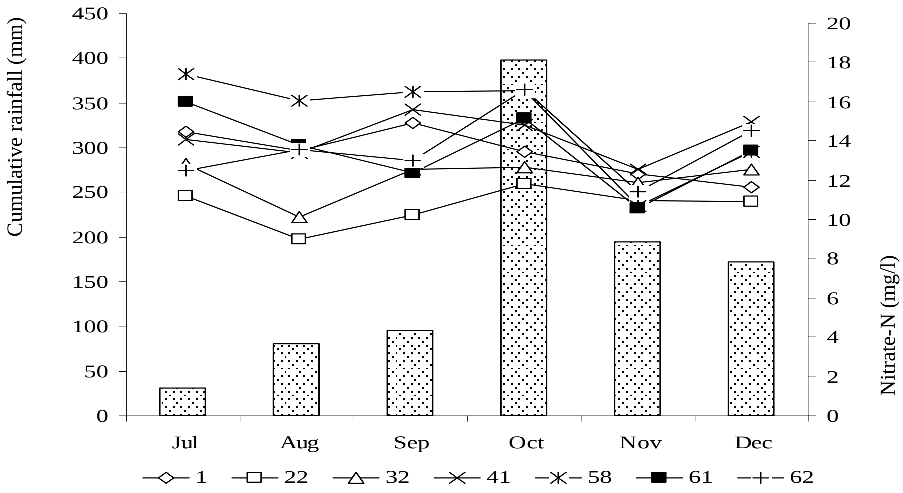



*Figure 3: Fluctuation of nitrate-N concentration in groundwater with rainfall*

*Table 2: Statistical analysis of groundwater nitrate-N among monthly data* ($p < 0.05$)

| Months | Mean nitrate-N |
|---|---|
| July | 9.68 [b] |
| August | 9.65 [b] |
| September | 10.79 [ab] |
| October | 11.37 [a] |
| November | 9.50 [b] |
| December | 10.03 [b] |

Means with same letter aren't significantly different in Duncan's grouping.

### *3.3 Nitrate-N in soil*

Figure 4 shows the presence of nitrate-N in the soil in different cultivated area. High concentration of nitrate nitrogen was observed up to 40 cm of soil profile and the concentration was low below the top layer. Because normally the organic and inorganic fertilizers were incorporated within the top soil surface which results in high concentration in top soil for all type of land use. The concentration was very high within the profile 0 – 25 cm in paddy soil. Premanandarajah *et al.*, [16] reported that the addition of organic manure increases nitrogen retentions capacity and reduces nitrate loss by leaching in sandy soils, therefore crops can efficiently utilize the applied fertilizer and residual N will remain in the soil for next crop.

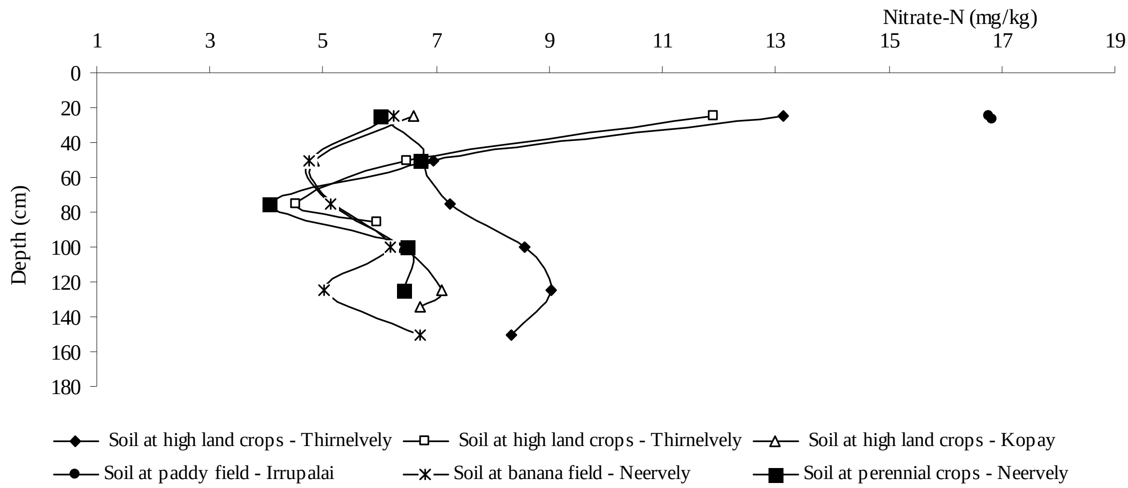



*Figure 4: Nitrate-N in soil in different land use classes*

### *3.4 Nitrate-N in groundwater and soil in different land use*

Nitrate-N in the groundwater and soil in the different land use classes is shown in Figure 5. There was a good correlation between soil nitrate-N to groundwater nitrate-N except paddy land use. Even though the concentration of nitrate-N was high in the paddy land use there was no leaching to the groundwater because leaching was restricted due to the presence of hardpan. De silva and Ayomi [17] reported that low nitrate content despite of intensive vegetable cultivation in Malsiripura in Kurunegala district due to the characteristics of the soil which consists of high clay and less pores which restricts the free leaching of nutrients to the shallow groundwater. Poorly drained soils can reduce the risk of groundwater contamination even in areas with high nitrogen input.

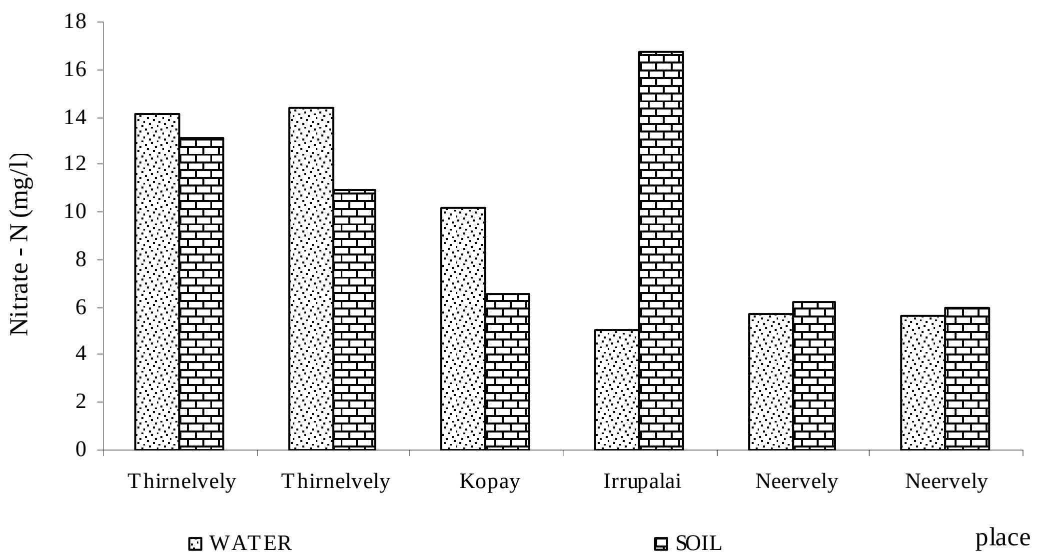



*Figure 5: Nitrate-N in soil and water in different places*

Wijewardena [18] revealed that the low nitrate – N content in the drinking waters of the up country, Sri Lanka where intensive vegetable cultivation is practiced may attribute to heavy textural fraction in ultisol. Loss of nitrate to groundwater is regulated by the amount of infiltration of the soil and water movement in the soil profile. Accordingly, nitrate mobility is high for moderately well drained soil and low for a poorly drained soil. The study area consists of red yellow latosol with the porosity of 46 % and infiltration rate 430 mm/hr [19], which facilitates free leaching of nutrients to the shallow groundwater.

## 4. Conclusion

The nitrate- N varies in all the months in sixty eight wells and values were ranged from 0.1 to 17.83 mg/l. The highest value of nitrate-N was observed as 17.83 mg/l at Kondavil. Out of sixty eight wells, 80% of the well was not recommended for drinking in intensified agricultural areas and all the wells were accepted for irrigation requirement. High concentration of nitrate nitrogen was observed up to 40 cm of soil profile and the concentration was low below the top layer. There was a good correlation between soil nitrate-N to groundwater nitrate-N except paddy land.

### References


1. Navaratnarajah, V., "Water problems in the Jaffna peninsula", 20th WEDC International Conference in Affordable water supply and sanitation, 1994, Colombo, Sri Lanka, pp 191- 193.
2. Gunasekeram, T., "Groundwater contamination and case studies in Jaffna Peninsula, Sri Lanka", 1983, Water Resources Board, Colombo, Sri Lanka.
3. Elankumaran, C., "Groundwater pattern of Jaffna-Valigamum region", University of Jaffna, Sri Lanka, 1994, pp 1- 25.
4. Mageswaran, R., Mahalingam, R., "Nitrate nitrogen content of well water and soil from selected areas in the Jaffna Peninsula", *Journal of Science Council*, Sri Lanka. 1984. Vol: 11, No: 1, 1984, pp 269-275.
5. Dissanayake C. B., Weerasooriya, S. V. R., "Hydro geochemical atlas of Sri Lanka", Natural resources, 1985, The energy and science authority of Sri Lanka, Colombo, pp 17-26.
6. Nagarajah, S., Emerson,B. N., Abeykoon, V., Yogaligam, S., "Water quality of some wells in Jaffna and Kilinochchi with special reference to nitrate pollution", *Tropical Agriculture*,. 1988, Vol: 44, pp 61-73.
7. Keeny, D. R., Nelson, D. W., "Nitrogen- Inorganic forms", In: Page, A. L., *et.al*. (Ed) Method of soil analysis, part 2**.** Second edition. American society of agronomy, 1982, Soil science society of America. Inc Madison, Wisconsin USA. pp 643-679.
8. Taras, M. J., "Nitrogen", In: Bolts, D.F. (Ed) Chemical analysis, Calorimetric determination of non-metals, 1958, New York: Inter Science publishers Inc, Vol: VIII, pp 75-160.
9. World Health Organization, "Guidelines for drinking water quality", Third edition, 2004, Geneva.
10. Sivarjah, N. "Health related problems of water pollution in Jaffna", 2003, International Workshop on Environmental Management in North-East SriLanka, pp 89-94.
11. Panabokke, R. G., "The geographical pathology of malignant tumours in Sri Lanka – a 5- year study", *Ceylon Medical Journal* , 1984, Vol: 29, pp 209-224.
12. Vaheesar, K., "Nitrate and Fluoride content in groundwater in the Batticaloa district", *Journals of Science Eastern University, Sri Lanka*, 2001, Vol: 2, No.1, pp 9-15.
13. Kuruppurachchi, D. S. P., Fernando, W. A. R. N., "Impact of agriculture on groundwater quality: leaching of fertilizers to groundwater in Kalpitiya Peninsula", *Journal of Soil Science, Sri Lanka*, 1999, Vol: 11, pp 9-15.
14. Kuruppuarachchi, D.S.P., Fernando, W.A.R.N., Lawrence, A.R., "Impact of Agriculture on groundwater quality in Kalpitiya Peninsula in North-Western dry zone of Sri Lanka", Proceedings of the Symposium on Irrigation and water resources, 1990, pp 199-213.
15. Nandasena, K.A., Gunawardhana, D.K.S.N. , Kendaragama. K.M.A., "Groundwater quality of three agro-ecological zones of Sri Lanka", *Journal of soil science society of Sri Lanka*, 2005, Vol: 17, pp 20-30.
16. Premanandarajah, P., Nandasena, K. A., Thedchanamoorthy, K., "Effect of organic manure on nitrate pollution of groundwater and soil nitrogen", International Workshop on Environmental Management in North-East SriLanka, 2003, pp 63-64.
17. De Silva, C. S., Ayomi, N., "Impact of intensive vegetable cultivation on agro-well water quality in Malsiripura region of Kurunagela District", 2004, Water professional symposium, Geo-informatics society of Sri Lanka, pp 121-133.
18. Wijewardena, J. D. H., "Impact of intensive vegetable cultivation on drinking water quality in the upcountry region of Sri Lanka' 1998, Proceedings of the national conference held at the BMICH, Colombo, Sri Lanka, pp 177-189.
19. Joshuwa, Y. D., "Physical properties of soil as related to irrigation and water management", In proceedings of FAO Sri Lanka, 1979, National seminar on water management and control at farm level, Colombo, Sri Lanka, pp 18-23.


### About the Authors


**Jeyaruba Thiparanjan,** B.Sc. (Hons.) in Agriculture, University of Jaffna. At present I am doing M.Phill (CB Project) in Integrated Water Resource Management (IWRM) in Post graduate Institute of Agriculture, University of Peradaniya, Sri Lanka.

**M. Thushyanthy,** B.Sc. Hons. Peradaniya., M.Phill. University of Peradaniya, reading for her Ph.D. at the University of Jaffna. Her research interests are soil and water engineering and groundwater management.

# TEMPORAL VARIATION OF NITRATE POLLUTION IN AGRO WELLS IN VAVUNIYA DISTRICT, SRI LANKA WITH SPECIAL REFERENCE TO KANTHAPURAM

P.Loganathan

Head, Department of Biological Science, Faculty of Applied Science,
Vavuniya Campus of the University of Jaffna, Sri Lanka
puvanalogan@gmail.com

**Abstract**

Management of good quality ground water is a prime factor for sustenance of life. Use of nitrate contaminated drinking water is well known risk factor for infant methemoglobinemia and various types of human cancer. There is a possibility of contamination of nitrate nitrogen in agro wells due to intensive use of inorganic fertilizer in permeable soil with shallow ground water. Objective of the study was to measure the nitrate nitrogen in selected agro wells and observe the temporal variation. Twenty wells were selected randomly from the Kanthapuram areas who have been cultivating vegetable crops for long period of time. Monthly water samples were collected from December 2008 to February 2010 to analysis Nitrate Nitrogen. Samples were drawn from the wells approximately at the levels of 30cm depth below the free water surface. Nitrate nitrogen was determined by colorimetric method (Brucine method). Mean Nitrate nitrogen in the study area was 12.2mg/l from December 2008 to June 2009 and 95 % of the wells were above WHO permissible limit of 10mg/l. The depth of the well varied from 6 m to 12 m and around 70% of the farmers apply the inorganic fertilizer above the recommended level. Intensive use of fertilizer and shallow ground water in permissible soil could be the reason for high nitrate nitrogen in agro wells. But mean nitrate nitrogen from August 2009 to February 2010 was 1.2 mg/l. Phyto remediation of nitrate nitrogen in shallow agro wells by available perennial plants around the wells during severe drought period from May 2009 to September 2009 could be the reason for low nitrate nitrogen. Mean nitrate nitrogen from December 2008 to June 2009 was significantly higher than mean nitrate nitrogen from August 2009 to February 2010 after severe drought ($p= 0.000$).

Keywords: Nitrate pollution, agrowells, phyto remediation, intensive use of fertilizer

## 1.0 Introduction

The Vavuniya district is located in the low country dry zone with the mean temperature of 28 ° C and annual rainfall of 1400mm. It is an agricultural area and people use surface and ground water for irrigation purpose. In Kanthapuram area, people have been engaged in cultivation of vegetable crops for more than 30 years using inorganic fertilizer. There is a possibilities of contamination of nitrate nitrogen in agro wells due to intensive use of inorganic fertilizer for vegetable cultivation in permeable soil with shallow ground water. Nitrate contamination of ground water has become a serious problem in northern part of Sri Lanka where intensified agriculture is being practiced (Nagarajah *et al.*, 1988). A study of the incidence of various types of human cancer in relation to nitrate concentrations in Sri Lanka revealed a significant positive correlation for stomach, small intestine, oesophagus and liver cancers (Dissanayake and Weerasooriya 1987). In human body, nitrate is turned into nitrite. Nitrite then reacts with certain substrates such as amines, amides and amino acids to produce nitroso compounds, which have been found in numerous animal studies to carcinogenic ( U. S. Dept of Health and Human services 1998). Preliminary study shows nitrate nitrogen in this area was high than WHO standard of 10mg/l. Therefore objective of the study was to measure the nitrate nitrogen of the agro wells with temporal variation.

## 2.0 Material and Methods

### *2.1 Collection of water samples from agro wells*

Twenty agrowells were selected from Kanthapuram area who has been cultivating agricultural crops for long period and water samples were collected once in two months from December 2008 to February 2010 to analysis Nitrate Nitrogen ($NO_3$- N), Electrical Conductivity (EC), and pH. Samples were drawn from the wells approximately at the levels of 30cm depth below the free water surface and each sample was poured into a bottle after rinsing it twice with the same sample and covered with a lid and transported to the laboratory at the Department of Biological Science of the Vavuniya Campus for chemical analysis. The depth and diameter of wells were measured.

### *2.2 Analysis of Water*

Electrical conductivity and pH were measured by environmental prop and $NO_3$- N was determined by colorimetric method using Brucine method (Taras, 1958). Nitrate-N analysis was done within twenty four hours after collection of sample. Total hardness was determined by titrimeric method using Ethylene Diaminete Tera Acetic Acid (EDTA method). Pair wise t test was performed to observe the significant difference between nitrate nitrogen in two seasons from December 2008 to June 2009 and August 2009 to February 2010.

## 3.0 Results

### *3.1 General characteristics of agro well*

The depth of wells varied from 6m to 12m with the mean value of 8.6m and diameter of wells varied from 3.5m to 7.6m with the mean value of 5.3m. There was a rocky layer in the well No 18 below 6 meter from ground level.

### 3.2 General characteristics of soil

The texture of the soil was sandy loam with 72% sand , 4 % silt and 24 % clay . The bulk density and the particle density of the soil were 1.66 and 2.7 g/cm3 respectively. The soil of the experimental site was classified as Reddish Brown Earth.

### 3.3 pH of agro well water

The pH of the wells varied from 6.7 to 7.9 from with the mean value of 7.1 from December 2008 to February 2010 (Figure 1).

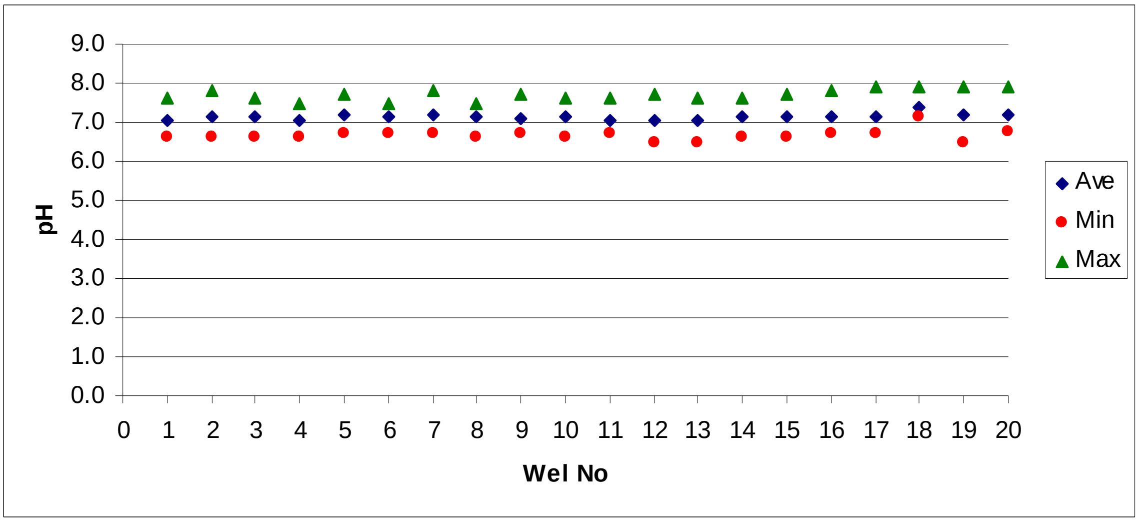



Figure 1 : Temporal variation of pH in agro wells

### *3.4 Electrical Conductivity(EC) of agro well water*

The EC of the wells ranged from 0.65to 1.56 $dsm^{-1}$ with the mean value of 1.00 $dsm^{-1}$ and 60% of the wells' EC were above 1 $dsm^{-1}$ (Figure 2). Low EC was observed in Well No 3, 11 and 17.

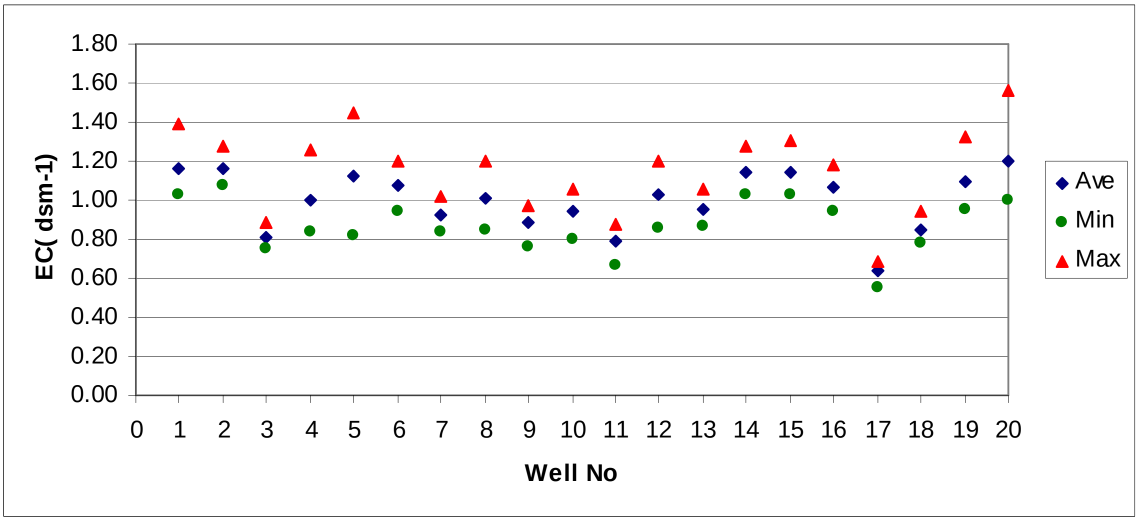



Figure 2 : Temporal variation of EC in agro wells

### *3.5 Nitrate Nitrogen in agro well water*

Mean Nitrate nitrogen in agro wells varied from 4.5 to 15.1 with the mean of 12.2mg/l from December 2008 to June 2009 and 95 % of the wells were above WHO permissible limit of 10mg/l (Figure 3). Nitrate nitrogen in well No 18 varied from 3.5 to 5.9 with the mean of 4.5 mg/l during this period. But it was varied from 0.5 to 2.3 mg/l from August 2009 to February 2010 with the mean of 1.2 mg/l.

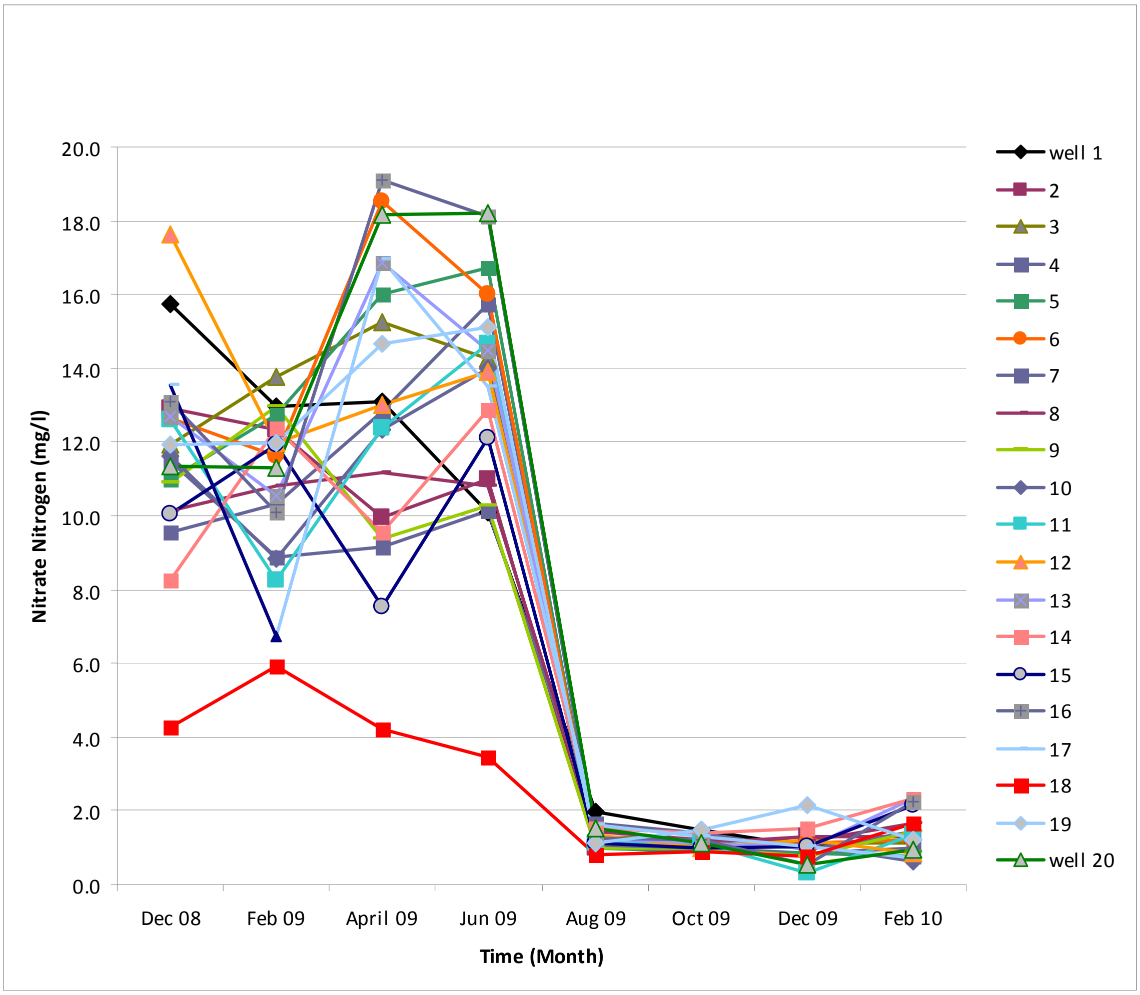



Figure 3 : Temporal variation of $NO_3$- N in agro wells

Mean nitrate nitrogen from December 2008 to June 2009 was significantly higher than mean nitrate nitrogen from August 2009 to February 2010 after severe drought (p= 0.000) (Figure 4).

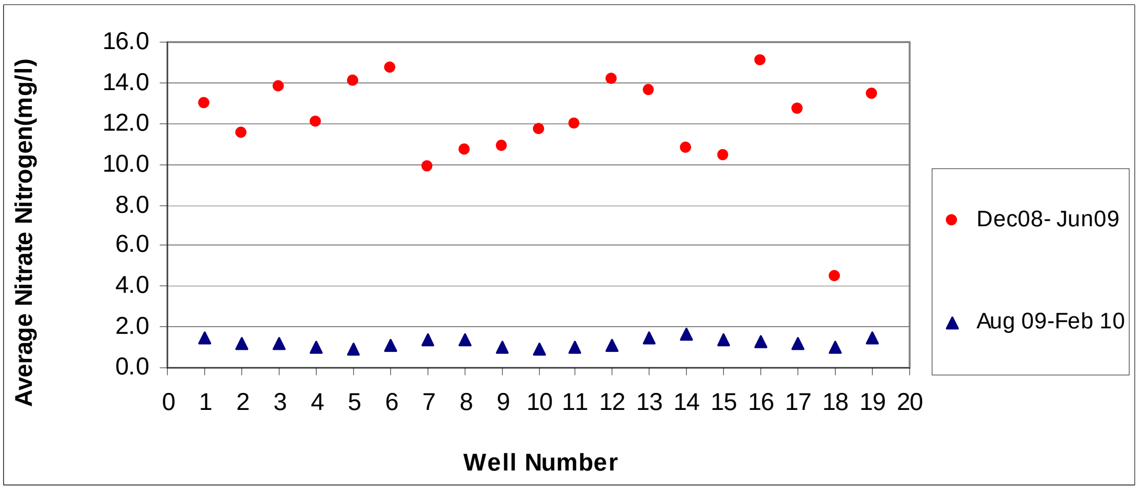



Figure 4 : Average nitrate nitrogen in agro well water from Dec 08 to June 09 and Aug 09 to Feb 2010

## 4.0 Discussion

### *4.1 pH and EC of agro well water*

The neural nature was observed in almost all the well water through out the period and all the wells show pH within the WHO permissible limit of 6.5 – 9.0 irrespective of the months. Therefore water could be used for domestic and agricultural purpose. Electrical Conductivity of water was within the WHO permissible limit of 3.5 $dsm^{-1}$ for drinking water. No significant temporal variation was observed in pH and EC. Therefore water could be used for drinking purpose with out any health hazards in relation to dissolved salts.

### *4.2 Nitrate Nitrogen in agro well water*

The depth of the well varied from 6 m to 12 m and around 70% of the farmers apply the inorganic fertilizer above the recommended level in addition to organic fertilizer. Intensive use of fertilizer and shallow ground water in permissible soil (sandy loam) could be the reason for high nitrate nitrogen in 95% of agro wells. Nagarajah and *et al* (1988) also identified the same problems in agrowells in Jaffna district. Panapokke (2005) identified 6 types of ground water aquifers in Sri Lanka and out of this shallow Regolith aquifer of hard rock region is available in Vavuniya district. Amarasinghe & De Silva (2006) stated shallow wells are very vulnerable to ground water pollution.

The sudden decreased in nitrate nitrogen was observed from August 2009 to February 2010 in all the wells. During these periods there was severe drought and shortage of ground water occurred due to sudden fluctuation of population in Vavuniya district after arrival of internal displaced people. Water level of the wells varied from 1-2 feet in almost all the wells during severe drought period and household members said that once they extracted available water they have wait for next day. There was no water to remove the nitrate ions by leachate from top soil to bottom. Phyto remediation of nitrate nitrogen in shallow agro wells by available deep rooted perennial plants around the wells during severe drought period from May 2009 to September 2009 could be the reason for low nitrate nitrogen. Ground water quality improvement through ecosystem management research by Melvani (2008) in Kalpitiya, Sri Lanka noticed that nitrate nitrogen in water from experimental well decreased from 58.5 mg/l to 12.1 mg/l in four years.

## 5.0 Conclusions

Results of the experiment indicate that there is high nitrate pollution in agro well waters in the study area from December 2008 to June 2009 due to high and continuous chemical and organic fertilizer application, frequent irrigation, soil type (Sandy loam) and shallow ground water. But it was less during severe drought period due to lack of water to remove nitrate ions by leachate in the soil and phytoremediation of nitrate nitrogen in well water by perennial plants. Management of fertilizer and cropping system are prime factor that determine the ground water pollution.

***Acknowledgements***

I wish to thank Vavuniya Campus of the University of Jaffna for the financial assistance through research grant and also express my sincere gratitude to Rector of the Campus and Dean of the Faculty who have given opportunity to carried out the research. Last but not least, I owe my thanks to my colleagues, students and other members who have helped me in various way to complete this research successfully.

### *References*

Amarasinghe.S.R and C.S De Silva., 2006. "Water quality assessment of agro wells in Vavuniya District for the use of Agricultural and Domestic purposes". Water Professional Day symposium. October 2006.

Dissanayake.C.B.1998. "Nitrates in ground water in Sri Lanka"- implications fro community health. Journal of the Geological Society of Sri Lanka, Vol 1 1998 p,80-84.

Melvani.K 2008. Bioremediation ensure clean water. NEO synthesis research centre, Sri Lanka.

Nagarajah, S., N. Emerson, and S.Yogalingam, S. 1988. "Water quality of some wells in Jaffna and Kilinochchi with special reference to nitrate pollution". *Tropical Agriculturist*. 144:15-20

Panabokke, C.R., and A.P.G.R.L.Perera. 2005. "Ground water resources of Sri Lanka". Water Resources Board , Sri Lanka.

Taras, M.J.,1958. "Nitrogen. In: chemical analysis. Vol. VIII. Calorimetric determination of non-metals", Ed. D. F. Bolts, 75-160. New York: Inter Science publishers Inc.

U. S Dept. of Health and human services, Public Health Service. 1998. National Toxicology Program, 8th Report on Carcinogens.

USEPA (US Environmental Protection Agency), 2000. "Drinking water standards and Health advisories", Office of water U S .Environmental Protection Agency Washington, D.C., EPA 822-B-00-001.
114:61-79.

WHO (World Health Organization), 1971. "International Standard for Drinking water". WHO Geneva, Switzerland. 70pp.

# MICRO IRRIGATION TECHNOLOGY: A REMEDY FOR GROUNDWATER MANAGEMENT IN JAFFNA PENINSULA

Christy Nilani, L[1]., Thushyanthy, M[1]. and S.Sivakumar[2]

[1]Department of Agricultural Engineering, Faculty of Agriculture, University of Jaffna
[2]Department of Agriculture Extension office, Thirunelvely, Jaffna Sri Lanka
thushym@jfn.ac.lk

**Abstract**

The study was conducted with the objective of estimating groundwater saving, irrigation intervals and duration for cabbage, a field trial was carried out with sprinkler irrigation to see the performance of yield. The field posses the main soil type of Calcic red yellow latosol and located under $DL_3$ region. The ten treatments were carried out including farmers' practices, morning sprinkler irrigation for 15 and 25 minutes and morning and evening sprinkler irrigation for15 and 25 minutes with two varieties analyzed by two factorial randomized block complete design. Irrigation duration of 15 minutes sprinkler irrigation with Green coronet variety field was record highest yield of 4.53 kg/m$^2$ and 15 minutes sprinkler irrigation with K – Y cross variety field was record lowest yield of 3.94 kg/m$^2$. Morning sprinkler irrigation was statistically not significant different from morning & evening sprinkler irrigation and these two treatments were statistically significant different from ridge and furrow irrigation for two varieties. Finally 15 minutes sprinkler irrigation was selected as best treatment and followed by 25 minutes sprinkler irrigation. Green coronet variety more response to sprinkler irrigation than K – Y cross variety. The depth of water application was higher in ridge and furrow irrigation than sprinkler irrigation. The saving of 69.31% of groundwater was accounted under sprinkler irrigation system with 15 min duration compared to ridge and furrow irrigation method. Adaptation of sprinkler irrigation is more economical and water saving than the ridge and furrow irrigation.

***Key words:*** *Groundwater****,*** *Sprinkler irrigation, yield response, cabbage*

## 1.0 INTRODUCTION

Increasing water scarcity in Sri Lanka, together with evidence of its inefficient use and increasing competitive demand has given momentum to the call to treat water as an economic good. One of the technical mechanisms available to improve the efficiency of irrigation water use is adoption of micro irrigation technologies to reduce losses at distribution and at on-farm water management. It was found that on farm irrigation efficiency was about 90% under properly designed and managed drip irrigation system, 80% for sprinkler irrigation and only about 45% for surface irrigation methods (Sivanappan, 1994). The annual water resource of the island has been estimated as 4.32 million ha m and present withdrawal is about 20 percent mainly for agricultural purposes. However, the increased demand for industrial and domestic water will result in a reduction in water diversions to agriculture.

Water application uniformity is essential for an efficient agriculture especially in regions where water resources are limited and precipitation is not main source to respond water demand. Hassanli *et al*., 2010 indicated that irrigation methods has key role in efficient use of water but still there is limited information on their application on crop performance. One of the best methods to increase the efficiency and the uniformity of irrigation is the use of micro-scale irrigation techniques for irrigating the agricultural lands. In micro-irrigation, water will be supplied on demand to the effective root zone of plants with high efficiency (Sanchez *et al.*, 1994). Micro Irrigation plays an important role in the management of crops to obtain the maximum yield from lesser quantity of water, chemicals and fertilizers compared to other forms of irrigation (Aheeyar *et al.*, 2004). Dharmasena and Karunainathan, 2004 stated that agro-well water is utilized for growing chilli, onion, fruits vegetables by smallholder farmers and the current trend is to cultivate fruits and vegetables by using micro irrigation systems at commercial level.

Groundwater can provide supplementary irrigation in many areas of the dry zone except the Northern district, Jaffna district in which groundwater is the major irrigation source (Srimanne, 1967 and De

Silva, 1996). The average annual rainfall of Jaffna is about 1200 mm. Normally, rainfall period restricted to 3-4 months of this area. Groundwater use has exceeded safe limits in most areas of Jaffna where sustainable irrigation depends on maintaining a delicate balance between recharge and extraction (Rajasooriyar *et al*., 2002). The Jaffna farmers face difficulties in irrigation interval and duration when use micro irrigation (Jayapiratha *et al.*, 2010). They do not have recommended intervals and durations.

### *1.1 Objectives of the study*

Adaptation of micro irrigation is important in Jaffna Peninsula to conserve the quality and quantity of groundwater. Hence the objectives of the study was selected as to determine the irrigation duration and irrigation interval of sprinkler irrigation in cabbage crop and its influence on yield component of two different varieties of cabbage with determination of groundwater saving under sprinkler irrigation in comparison with surface irrigation

## 2.0 MATERIALS AND METHODS

### *2.1 Measurement of system parameters*

Research study was conducted in a field located at District Agricultural Training Center at Thirunelvely in Jaffna district where cabbage are cultivated under sprinkler and surface method irrigated conditions. Randomly selected rotary head type sprinklers were fitted on laterals with equal spacing with riser height of 42 cm in the sprinkler system. The system parameters, such as discharge rate of the nozzle, the average wetted area and the depth of water applications were measured. The commonly used measurement tool to determine the uniformity of sprinkler systems is catch can test (Li *et al*., 2005). Once the data are collected by catch cans, a number of different calculations can be performed. For the measurement of uniformity of water distribution, twenty five catch cans were placed around sprinkler. Sprinklers were allowed to operate for 30 minutes and total collected water in the cups was measured by using a measuring cylinder. For the calculation of uniformity of water distribution from rotating head sprinklers, a formula developed by Christiansen (Sivanappan, 1987) was used.

$$Cu = 100\left(1.0 - \frac{\sum X}{mn}\right)$$

In which,

$C_u$- Co-efficient of uniformity
m - Average value of all observations (average application rate), mm
n - Total number of observation points
X – Numerical deviation of individual observation from the average application rate, mm.

Calculated uniformity coefficient values were plotted and compared with internationally accepted value of 85%.

### *2.2 Collection of weather parameters*

The weather records such as rainfall, relative humidity, wind velocity, sunshine hours and Temperature were collected from Thirunelvely, Meteorological station in Jaffna, for the January, February, March and April 2008 to of study period 2009 to see the suitability of the sprinkler system .

### *2.3 Treatments*

The cabbage was selected and grown as the test crop for this experiment. In this crop, K-Y cross and Green coronet varieties were selected. Two irrigation methods were selected as treatment. The ridge and furrow method was selected as control because most of the farmers planting the cabbage in ridges and furrows irrigation. The research was done with the following ten treatment combinations (Table 1) and three replicates. The experimental data was analyzed statistically following factorial randomized complete block design. Control was designed as every fourth day to Ridge and furrow.

**Table 1:** *Combination of treatment*

| Treatment | Irrigation interval (every day) | Irrigation duration(minutes) | variety | Irrigation type |
|---|---|---|---|---|
| $T_1$ | Morning & evening | 15 | K - Y cross | Sprinkler |
| $T_2$ | Morning & evening | 15 | Green coronet | Sprinkler |
| $T_3$ | Morning & evening | 25 | Green coronet | Sprinkler |
| $T_4$ | Morning & evening | 25 | K - Y cross | Sprinkler |
| $T_5$ | Morning | 15 | K - Y cross | Sprinkler |
| $T_6$ | Morning | 15 | Green coronet | Sprinkler |
| $T_7$ | Morning | 25 | Green coronet | Sprinkler |
| $T_8$ | Morning | 25 | K - Y cross | Sprinkler |
| $T_9$ | - | - | Green coronet | Ridge & furrow |
| $T_{10}$ | - | - | K - Y cross | Ridge & furrow |

Except irrigation duration and irrigation interval all other cultural activities such as nursery management, planting, weed control, fertilizer application and chemical application were maintained the same for all treatment plots. The ridge and furrow was irrigated at three days irrigation intervals to represent farmer's practices. The discharge rate of the pump and duration of irrigation were measured during each furrow irrigation time to get the total depth of water irrigated.

### *3.4 Measurement of yield parameters*

Out of two hundred and forty, forty eight samples of each treatment were selected randomly. The following yield parameters; mean head weight, plant height, head diameter, head height, yield, root length were measured.

### *3.5 Statistical analysis*

The experimental data was analyzed statistically following randomized complete block design and factorial randomized complete block design by the use of SAS computer software package at 5%level.

## 3.0 RESULTS AND DISCUSSION

### *3.1 Measurement of system parameter*

Mean diameter of the wetted area was 6.4 m at 42 cm riser height. The mean discharge rate of sprinkler nozzle was 0.1467 lit/sec and the depth of water applied 33 mm/hour. The christiansen uniformity coefficient value was 92.42% and it was acceptable since the value was greater than the best internationally accepted uniformity coefficient value of greater than 85% (Gupta *et al*., 2001).

### *3.2 Variation of weather parameters*

The study area belongs to dry zone low country ($DL_3$) agro climatic region where the soil is calcic red latosols. The average temperature, rainfall and wind velocity were 27 $^oC$, 356.9 mm and 4.56 km/h respectively. The average maximum temperature was 31.65 $^oC$ with standard deviation of ± 1.65 $^oC$. The highest maximum temperature was 34.7 $^oC$ and lowest maximum temperature was 26.4 $^oC$. This climatic condition is preferable for growth of the cabbage crop. Out of 91 days, in total 356.9 mm rainfall was received with twelve rainy days. The relative humidity was varied from 48% to 98% and average RH was 67.16% with standard deviation of ± 9.32%. The average wind speed was 4.56 km/h and standard deviation ± 2.89 km/h. Most of the days, the speed of the wind was less than10 km/h. Hence there was no any influence of wind speed in uniformity coefficient of sprinkler irrigation.

### *3.3 Response of irrigation treatment on yield performance*

Analysis was done in two ways. First, six treatments were considered as morning sprinkler irrigation, morning & evening sprinkler irrigation and ridge and furrow irrigation for two varieties. Another analysis was done within the sprinkler irrigation treatment. Four treatments were considered (Morning sprinkler irrigation – 15 min & 25 min and morning & evening sprinkler irrigation – 15 min & 25 min). The mean head weights ranged from 4.81 to 8.43 kg/m$^2$. The highest mean head weight at 1.69 kg and 1.66 kg were obtained in morning and morning and evening sprinkler irrigation for Green coronet varieties respectively (Figure 1). Morning sprinkler irrigation was statistically not significant different from morning & evening sprinkler irrigation and these two treatments were statistically significant different from ridge and furrow irrigation for two varieties. The lowest head weights were obtained from ridge and furrow irrigation with K – Y cross variety and Green coronet variety which had 1.13 kg, 0.96 kg respectively. Mean time, the higher yield was received in green cornet variety than K-Y cross verity. According to the other analysis the highest mean head weight of 1.69 kg was obtained in morning 15 minutes sprinkler irrigation. The lowest mean head weight at 1.65 kg was obtained in morning & evening 25 minutes sprinkler irrigation (Figure 2).

Mean head weight and mean plant height were significantly differed in sprinkler irrigation and ridge and furrow irrigation but mean head width, mean head height and mean root length were not significantly different within these treatments for K – Y cross (Table 2). Mean head weight, mean plant height, mean head width, mean head height and mean root length were significantly differed for in sprinkler irrigation and ridge and furrow irrigation for green coronet (Table 3). But all the measured characters were significantly differed between two varieties because of its varietal characters.

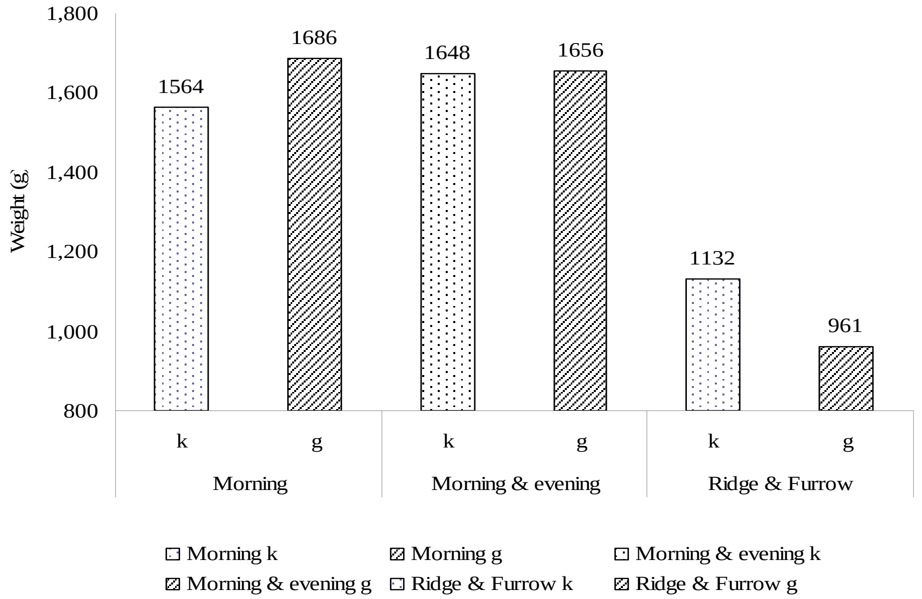



**Figure 1:** *Mean head weight in different irrigation systems*

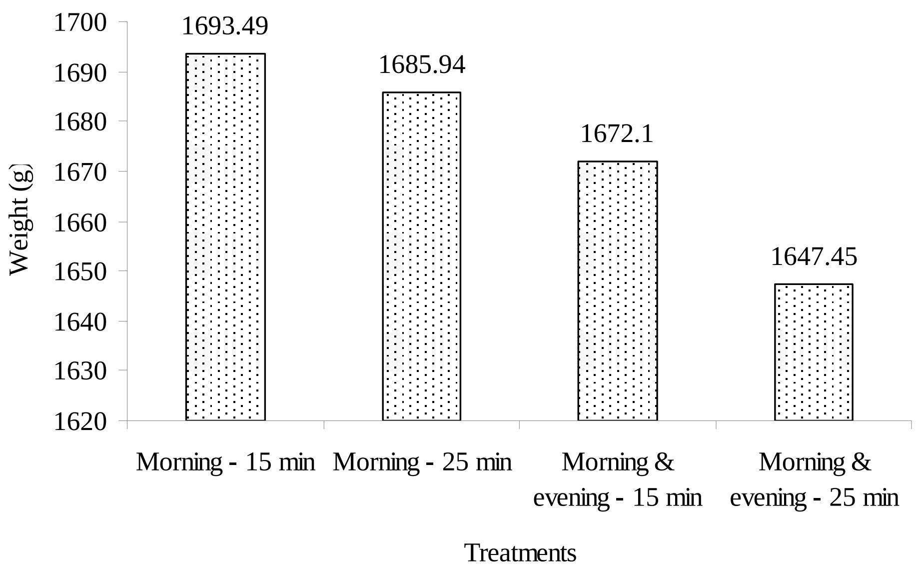



**Figure 2:** *Mean head weight in different irrigation durations and intervals under sprinkler irrigation systems.*

**Table 2:** *Mean head weight, mean plant height, mean head width, mean head height and mean root length of K – Y cross variety.*

| Irrigation type | Mean head weight (g) | Mean plant height (cm) | Mean head width (cm) | Mean head height (cm) | Mean root length (cm) |
|---|---|---|---|---|---|
| Morning sprinkler | 1564[a] | 28.922[a] | 20.651[a] | 13.693[a] | 19.783[a] |
| Morning & evening sprinkler | 1648[a] | 28.801[a] | 20.371[a] | 13.291[a] | 19.526[a] |
| Ridge & furrow | 1132[b] | 22.125[b] | 18.812[a] | 12.271[a] | 17.833[a] |

(Mean followed by the same letters is not significantly different at 5%level).

**Table 3:** *Mean head weight, mean plant height, mean head width, mean head height and mean root length of green coronet.*

| Irrigation type | Mean head weight (g) | Mean plant height (cm) | Mean head width (cm) | Mean head height (cm) | Mean root length (cm) |
|---|---|---|---|---|---|
| Morning sprinkler | 1686[a] | 34.718[a] | 16.906[a] | 15.651[a] | 21.687[a] |
| Morning & evening sprinkler | 1656[a] | 34.125[a] | 16.704[a] | 15.407[a] | 21.250[a] |
| Ridge & furrow | 961[b] | 30.687[b] | 14.406[b] | 13.042[b] | 18.958[b] |

(Mean followed by the same letters is not significantly different at 5%level).

### *3.4 Groundwater saving*

Table 4 shows the depth of water applied in single irrigation and total water used during entire crop growing season with mean head weight. The depth of water application in each 15 min irrigation was 3.17 mm, 5.29 mm of water in 25 min irrigation and 30.95 mm of water in ridge and furrow irrigation. While comparing the depth of irrigation applied and mean head weight, 15 min sprinkler irrigation was more economic than other irrigations. The saving of 69.31% of groundwater was accounted under sprinkler irrigation system with 15 min duration compared to ridge and furrow irrigation method. Adaptation of sprinkler irrigation is more economical and groundwater saving than the ridge and furrow irrigation.

**Table 4:** *Depth of water used during growing season*

| Treatment | Depth of water used (mm) | Total depth of water used (mm) | Mean head (g) |
|---|---|---|---|
| Morning – 15 min | 3.17 | 380.85 | 1693.49 |
| Morning – 25 min | 5.29 | 634.75 | 1685.94 |
| Morning & evening – 15 min | 6.34 | 761.70 | 1672.1 |
| Morning & evening – 25 min | 10.58 | 1269.51 | 1647.45 |
| Ridge & furrow | 30.95 | 1238 | 1046.5 |

## 4.0 CONCLUSION

In yield parameters, mean head width, mean head height, mean root length, mean head weight and mean plant height were significantly varies among sprinkler and ridge and furrow irrigation of Green coronet variety. Mean head weight and mean plant height were significantly varies among sprinkler and ridge and furrow irrigation of K – Y cross variety. Every day morning 15 minutes sprinkler irrigation was suitable to cabbage crop under calcic red yellow latosol considering the head weight. The saving of 69.31% of groundwater was accounted under sprinkler irrigation system with 15 min duration compared to ridge and furrow irrigation method. Adaptation of sprinkler irrigation is more economical and groundwater saving than the ridge and furrow irrigation.

## References

Aheeyar, M.M.M., Kumara, S.K., Samarasingha, L.W. (2005), “The application of micro irrigation technologies in the small farming sector in Srilanka”, Research Study No; 113, Hector Kobbekaduwa Agrarian Research and Training Institute, Colombo, SriLanka.

De Silva, C.S., Rushton, K.R. (1996), “Interpretation of the behavior of agro-well systems in Sri Lanka using radial flow models”, Journal of Hydrological Sciences, 41(6): 83 - 86

Dharmasena, P.B., Karunainathan, T. (2004), “Characteristics of regolith aquifers and their save exploitation through agro-wells”, Annals of the Sri Lanka department of Agriculture. 6: 79 – 87.

Gupta, K., Lantzke, N., Mcpharlin, I. (2001), “Evaluating sprinkler performance under windy conditions”, ‘WA Grower’, September, 2001.

Hassanli, A.M., Ahmadirad, S., Beecham, S. (2010), “ Evaluation of the influence of irrigation methods and water quality on sugar beet yield and water use efficiency”, Agriculture Water Management, 97: 357-362.

Jayapiratha, V., Thushyanthy, M. , S. Sivakumar (2010), “Performance evaluation of Okra under drip irrigation system”, Asian Journal of Agricultural Research. 2010: 1-9.

Li, J., Li, B., Rao, M. (2005), “ Spatial and temporal distributions of nitrogen and crop yield as affected by nonuniformity of sprinkler fertigation”, Agriculture Waste Management, 76: 160-180.

Rajasooriyar, L., Mathavan, V., Dharmagunawardhane, H.A., Nandakumar, V. (2002), “Groundwater quality in the Valigamam region of the Jaffna Peninsula, Sri Lanka. In: Sustainable groundwater management”, Hiscock, K.M., Rivett, M.O. and Davison, R.M. (Eds). Geological Society, Londan. 193: 181 – 197.

Sanchez, C. A., Roth, R. L., Gardener, B. R. (1994), “ Irrigation and nitrogen management for sprinkler-irrigated cabbage on sand”, Journal of the American Society for Horticultural Science, 119, 427–433.

Sivanappan, R.K. (1987), “Sprinkler Irrigation”, Oxford & IBH publishing company, New Delhi, India. Pp: 15-89.

Sivanappan, R.K. (1994), “Prospects of Micro-irrigation in India”, Irrigation and Drainage Systems 8:49 – 58.

Srimanne, C.H.L. (1967), “ Groundwater resources in the dry zone”, In studies on groundwater in Sri Lanka, C.H.L. Srimanne memorial volume. S. Arumugam. (Ed.) Pp. 117-126. Water Resource Board. Colombo.

# FARMER PARTICIPATION ON WATER MANAGEMENT IN THE TANK IRRIGATED SYSTEMS IN TAMIL NADU

Nanthakumaran. A[1] and Palanisami. K[2]

[1] Senior Lecturer, Dept. of Biological Sciences, Faculty of Applied Science, Vavuniya Campus, Vavuniya, Sri Lanka. n_ananthi@hotmail.com Tel:0094715364459 Fax:0094242222265

[2]Director, IWMI-TATA Policy Research Program, International Water Management Institute, South Asia Regional Office, Hyderabad, India. K.palanisami@cgiar.org

1- Corresponding author

**Abstract:**

Farmer participation is found to be a solution to arrest the deterioration of the tank irrigation system which is observed in Tamil Nadu state. The study was undertaken with the specific objective of identifying the determinants of farmer participation on water management and its impact on tank performance. The results of the tobit regression model and a production function analysis reveal that the contribution for farmer participation towards water management of Rs 1.00 at the mean level, *ceteris paribus*, would increase the rice yield by 2.7 kg/ha in Tank only typology whereas in Tank with wells typology by 2.2 kg/ha of land. The positive impact of farmer participation towards water management on rice yield indicates the importance of water management institutions in sustaining rice productivity and it has important policy implications for water management in tank commands. The water users association in the tank irrigated systems should be strengthened for water management which leads to better performance of the tanks.

Key words: Farmer participation, Water management, Tank irrigation system

## 1.0 Introduction

Tank irrigation systems are important sources of irrigation in South India. They account for more than one-third of the total irrigated area in Andhra Pradesh, Karnataka and Tamil Nadu states. The tank irrigation system has a special significance to the marginal and small scale farmers. Many studies in tank irrigation systems revealed that tank irrigation system is deteriorated because of negligence of tank management. Tank performance is determined by both tank management and water management. Water management includes allocation and distribution of water which is mainly done by water man (‘Neerkatti’). Field channel cleaning is also considered as water management activity. As water is the critical input for farming in tank irrigation systems, it is an important need to improve the water availability. Any action towards improvement in tank performance may improve the water availability in the tank command area. As water management directly influencing the tank performance it is necessary to identify the determinants of farmer participation in water management in different situations. Hence the objective of this study is to identify the determinants of farmer participation on water management and its impact on tank performance.

## 2.0 Materials and Method

### 2.1 Literature review

Sarma (1992) stated that the objective of increasing the irrigated area and agricultural production could be achieved only through improving the existing systems. Consequently equity and productivity of irrigation systems were thus a function of water distribution. Hence, determinants of water distribution were of primary concern to those interested in project performance; they formulated developmental strategies appropriate for specific agricultural production environment. Over years the area under tank irrigation had been declining and policy makers and planners were exploring the possibilities to revive the tank irrigation, as tank irrigation was the typical example of the water harvesting technique, and were mostly managed by the local communities as common property resource. Budget constraints and poor community participation made the tank performance unsustainable. The immediate solution was to identify the appropriate investment strategies and make the local Panchayat responsible for the operation and maintenance of the tanks. Resource mobilization by the local bodies was very essential (Palanisami and Easter, 2000) Balasubramanian and Selvaraj (2003) tried to understand the main causes for the degradation of tanks and the complex interrelationships among poverty, private coping mechanisms and community coping mechanisms that affected tank performance. Regression models such as a macro model on tank degradation, household-level models on collective action, and a production function incorporating collective action as an input were fitted and found out that poor people are more dependent on tanks for various livelihood needs and hence, they contributed more towards tank management compared to non-poor households. Collective action had a positive and significant impact on rice yields. The tank degradation showed that there had been a decline in the performance of the tanks. Narayanamoorthy (2007) made an attempt a) to study the growth pattern of tank irrigation across different periods both at the national as well as across states level b) to study the nexus between rainfall and area under tank irrigation at a specific state, which has relatively larger area under tank irrigation c) to find out the losers and gainers of tank irrigation among different size of farmers and d) to suggest policy measures to rejuvenate tank irrigation in India. He concluded that the reasons for the decline in area under tank irrigation might be different for different states. Maintenance works could not be carried out in a regular basis due to lack of financial allotment which resulted in an overall reduction in the storage capacity of many tanks. He added that since it was difficult to improve the performance of the tanks without users' participation, state agencies should make effort to revitalize age-old irrigation institutions, which had maintained the systems over centuries.

### 2.2 Methods

Two districts were purposively selected in Tamil Nadu, wherein Madurai and Sivagangai districts from southern part represent the Tanks only and Tanks with Wells typologies. Tanks are the main source of irrigation in these two districts.

#### 2.2.1 Sampling design

Ten tanks in each selected district were randomly selected for the study using the list of tanks in the districts. Then 25 households in each selected tank were randomly selected using the list of farmers available with the village administrative offices. Thus, the sample for this study consists of 20 tanks and 500 households which represent adequate distribution of sample households among the selected tanks.

As there were no tanks without wells, the 20 tanks selected randomly were categorized into two different typologies based on the farm households depending upon the source of water supplies, *viz*., Tank only and Tank with wells. Thus this categorization was primarily based on the percentages of households depending on the type of water source. If more than 80 per cent of the household in a tank, use tank as the only source for irrigation, then those tanks were categorized as typology I (Tank only situation) and the rest were grouped into typology II (Tank with wells situation). In the study area, eight tanks in Madurai district were categorized under typology I (*i.e.,* Tank only situation). It consists of 173 households using tanks as the only source of water for irrigation. There were 27 households under those particular tanks who use tanks with wells as the source of water for irrigation. Likewise 12 tanks in which 10 tanks in Sivagangai and two tanks in Madurai districts were categorized under typology II (*i.e.,* Tank with wells). It consists of 246 households using tank with wells as the source of water for irrigation. However, there were 54 households in the typology II who use only the tanks as the source of water for irrigation. Finally 27 households in the typology I and 54 households in the typology II were excluded from the analysis because these households could not fit into the above typologies due to their field locations and conflicts with other neighboring farmers in sharing the available water from tanks and wells. This exclusion was made to draw the conclusions and recommendations based on the results obtained under each typology. The details of the sample are given in Table 3.1

**Table 3.1 Sample household distribution in the study area**

| Typology | Tank only | Tank with wells | Total |
|---|---|---|---|
| Tank only | 173 (34.60) | 27 (5.40) | 200 (40.00) |
| Tank with wells | 54 (10.80) | 246 (49.20) | 300 (60.00) |
| Total | 227 (45.40) | 273 (54.60) | 500 (100.00) |

Figures in parentheses indicate percentage to the total

The field data from the sample respondents relating to agriculture year 2006-07 were collected with the help of pre-tested interview schedule through personal interview. The information regarding the age, education, occupation, family details, source of irrigation for wet and dry lands, well irrigation, water purchase and sales details, annual pumping hours, level of water in the wells, investment on wells, cost of cultivation details of crops under cultivation, household income, participation in tank and water management activities were obtained from sample respondents. Further the tank level information of the selected tanks like tank characteristics such as storage level, command area; number of wells in the tank command area, details of total extent of crops cultivated in each tank, details of livestock, tree resource in the tank bund was collected from the records maintained in the taluk offices for analysis. In addition to this, the block level data such as rainfall, geographical area, number of wells present were obtained from official records.

## 3.0 Theory

### 3.1 Tobit regression

Tobit model was used for identifying the determinants of farmer participation towards water management. The independent variables for the analysis were selected after a careful review of literature on factors affecting farmer participation. Group size is an important factor determining the extent of cooperation in the commons. Small groups are considered to be conducive for the emergence and stability of cooperative behavior in view of lower heterogeneity and transaction cost associated with organizing group action (Wade, 1988). As data is not available on the exact number of farmers in each of the sample tanks, tank size (command area) is used as a proxy for group size. Given the fact that the size of land owned under tanks does not show much variation across tanks, tank size provides a good proxy for group size. Participation in meetings is considered as the strength of that traditional organization and its effectiveness in its activities. It is hypothesized that it captures the extent of farmer participation (collective action) for water management. Thus, a dummy variable for institutional effectiveness that represents the active participation or not on water management is used.

Farm size, education as years of schooling of the household head are used as independent variables in this model. Number of wells included as a variable and it is hypothesized to have negative effects on farmer participation. Share of non farm income and age were also included in the model. The dependent variable is the total value of farmer participation (collective effort), which is calculated by summing up the monetary value of labor, materials such as gunny bags and money contributed for collective work. Since there was no contribution by some of the sample farmers, the dependent variable takes a zero value for all these observations and others take value more than zero. In view of the truncated nature of the dependent variable, the tobit regression was chosen and specified as follows:

The tobit model originally developed by Tobin is of the following form.

$Y_i = \beta_1 + \beta_2 X_i + \varepsilon_i$ If RHS>0

=0 otherwise

where RHS is right hand side. Additional X variables can be easily added to the model.

The model used in this study was modified from Balasubramaniam and Selvaraj (2003) to find out the amount of money contributed by way of farmer participation to tank and water management in relation to other socio economic and tank variables.

**3.1.1 Tank only situation:**

$Fpart = \beta_0 + \beta_1 Age + \beta_2 Yschl + \beta_3 WUA + \beta_4 Fsize + \beta_5 Tksize + \beta_6 NFIshare + \varepsilon$

**3.1.2 Tank with wells situation:**

$Fpart = \beta_0 + \beta_1 Age + \beta_2 Yschl + \beta_3 WUA + \beta_4 Wellden + \beta_5 Fsize + \beta_6 Tksize + \beta_7 NFIshare + \varepsilon$

Where,

| | | |
|---|---|---|
| Fpart | = | Farmer participation measured by contribution of money value (Rs / ha) |
| Age | = | Measured as number of years of household head |
| Yschl | = | Education measured as years of schooling of household head |
| WUA | = | Dummy for active participation in WUA meetings as a proxy for effectiveness of local institutional mechanism ( 1 if the WUA is active and 0 otherwise) |
| Wellden | = | Number of wells available/ ha. |
| Fsize | = | Farm size in ha. |
| Tksize | = | Command area of the tank in ha. |
| NFIshare | = | Share of non-farm income in the total household income |
| $\beta_0, \beta_1 \ldots \beta_7$ | = | Coefficients |
| ε | = | Error term |

**3.2 Multiple regression analysis**

The Cobb-Douglas model was fitted to capture the impact of farmer participation on rice yield in different scenarios of tank irrigation. As this study has two different typologies of tank irrigation for paddy, the regression models were specified separately as follows.

**3.2.1 For Tank only typology:**

$lnRiceyd = \beta_0 + \beta_1 lnSeed + \beta_2 lnFert + \beta_3 lnLabour + \beta_4 Npcide + \beta_5 lnFpart + \varepsilon$

**3.2.2 For Tank with wells typology:**

$lnRiceyd = \beta_0 + \beta_1 lnSeed + \beta_2 lnFert + \beta 3Labour + \beta_4 Npcide + \beta_5 Swirri + \beta_6 Fpart + \varepsilon$

Where,

| | | |
|---|---|---|
| Riceyd | = | Rice yield (kg/ha) |
| Seed | = | Value of Seeds used (Rs/ha) |
| Fert | = | Value of fertilizer NPK used (Rs/ha) |
| Labour | = | Value of human labor used (Rs/ha) |
| Npcide | = | Number of pesticides spray/ha |

Swirri = Number of supplemental irrigations/ha

Fpart = Monetary value of farmer participation for collective action (Rs/ha)

$\beta_0, \beta_1 .... B_6$ = Coefficients

$\varepsilon$ = Error term

## 4.0 Results

The mean values of the variables used and the results of the regression analysis for identifying the determinants of farmers' participation in water management are presented in the Tables 4.1 and 4.2 respectively.

**Table 4.1: Description and mean values of determinants of farmer participation on water management in Tank only and Tank with wells typology**

| Variables | Description | Mean values of Tank only typology | Mean values of Tank with wells typology |
|---|---|---|---|
| Age | Number of years of the household head | 48.54 | 49.68 |
| Yshcl | Education measured as years of schooling of household head | 6.97 | 7.81 |
| WUA | Dummy for active participation in WUA meetings as a proxy for effectiveness of local institutional mechanism | - | - |
| Fsize | Farm size in ha. | 0.83 | 1.22 |
| Wellden | Well density number/ha | | 0.27 |
| Tksize | Command area of the tank in ha | 147.76 | 214.76 |
| NFIshare | Share of non-farm income in the total household income | 0.27 | 0.24 |

**Table 4.2. Determinants of farmer participation on water management in Tank only and Tank with wells typologies**

| | Tank only typology | | Tank with wells typology | |
|---|---|---|---|---|
| Variable | Coefficient | Standard error | Coefficient | Standard error |
| Constant | -184.37 | 56.45 | -15.18 | 48.22 |
| Age | 0.3263 | 0.8126 | 0.297 | 0.734 |
| Yschl | 4.998* | 2.697 | 5.272* | 2.55 |
| WUA | 161.71*** | 0.0316 | 105.34*** | 16.36 |
| Fsize | 153.91*** | 22.82 | 25.78** | 9.81 |
| Wellden | | | -45.34*** | 8.246 |

| | | | | |
|---|---|---|---|---|
| Tksize | -0.146*** | 11.74 | -0.085*** | 2.028 |
| NFIshare | -63.47 | 31.82 | -275.53*** | 81.03 |
| Log likelihood function | -706.08 | | -563.71 | |
| Sigma | 78.57 | 4.901 | 66.1 | 4.99 |
| Sample size | 171 | | 184 | |

***, ** indicate significance at one and five per cent level

**Table 4.3. Description and mean values of variables used in rice production function analysis for water management in Tank only and Tank with wells typologies**

| Variable | Description | Mean values of Tank only typology | Mean values of Tank with wells typology |
|---|---|---|---|
| Riceyd | Rice yield (Kg/ha) | 4,123 | 4,789 |
| Seed | Value of seeds (Rs/ha) | 1,208 | 1,239 |
| Fert | Value of fertilizer (Rs/ha) | 3,645 | 3,875 |
| Labor | Value of labor (Rs/ha) | 6,178 | 6,278 |
| Swirri | Number of supplemental irrigation | - | 1.50 |
| Npcide | Number of pesticides spray | 1.40 | 2.50 |
| Fpart | Farmer participation (Rs/ha of command area) | 478 | 352 |

**Table 4.4 Impact of farmer participation on rice yield through water management in Tank only and Tank with wells typologies**

| | Tank only typology | | Tank with wells typology | |
|---|---|---|---|---|
| **Variable** | **Coefficient** | **Std.Error** | **Coefficient** | **Std.Error** |
| Constant | 1.73 | 0.643 | 3.787 | 0.666 |
| Seed | 0.143* | 0.086 | 0.11* | 0.0518 |
| Fert | .399*** | 0.082 | 0.155** | 0.053 |
| Labor | 0.145*** | 0.037 | 0.23*** | 0.014 |
| Npcide | 0.037 | 0.023 | 0.0038 | 0.0171 |
| Swirri | - | - | .129** | 0.0348 |
| Fpart | 0.202*** | 0.033 | 0.053** | 0.0188 |

| | | |
|---|---|---|
| Adjusted $R^2$ | 0.54 | 0.91 |
| F-value | 41.006 | 191.86 |

***, **, * indicate significance at one, five and 10 per cent level

## 5.0 Discussion

### 5.1 Determinants of farmers' participation on water management

From the Table 4.2 the coefficients of farm size and water users association were highly significant and positively contributing for farmer participation towards water management while tank size is negatively contributing for farmer participation and years of schooling positively contributing for farmer participation at 10 per cent significance level in Tank only situation whereas in Tank with wells situation the variables *viz.*, tank size, well density and non-farm income share showed negative contribution in farmer participation towards water management with one per cent significant level while years of schooling, farm size and water users' organization showed positive contribution in the extent of farmer participation towards water management. It could be interpreted that an increase in the well density by one from the mean level, *ceteris paribus,* would reduce the farmers' participation in water management by Rs. 45 per ha (Table 4.2). This result provides stronger evidence to the hypothesis which states that the increase in density of private wells in the tank command reduces the farmers' participation towards water management.

An increase in tank size by 100 ha from the mean level, *ceteris paribus,* would result in a reduction of farmer participation in water management by Rs. 14.60 whereas in Tank with wells typology by Rs. 8.50. In many cases these tanks serve more than one village thus increasing heterogeneity that discourages the cooperative action among the tank farmers. An increase in farm size by one ha from the mean level, *ceteris paribus,* would increase the farmer participation on water management by Rs. 154 in Tank only situation whereas in Tank with wells situation by Rs. 26. It indicates that even though the farmers own wells in Tank with wells situation they understand the importance of water management. An increase in the years of schooling by one year from the mean level, *ceteris paribus*, would result in an increase of farmer participation by Rs. 5.00 in Tank only situation and Rs. 5.20 in Tank with wells situations. This implies that the educated farmers understand the importance of water management on tank performance. By changing the attitude from poor participation to active participation in WUA's meetings keeping all other variables constant, contributes Rs. 161 for water management in Tank only typology whereas Rs. 105 in Tank with wells typology. An increase in non-farm income share by one, keeping all other variables constant, would reduce the farmer participation by Rs. 63 in Tank only typology whereas by Rs. 275 in Tank with wells typology.

### 5.2 Impact of farmer participation on water management

The $R^2$ value of 0.54 and 0.91 in Tank only and Tank with wells typologies (Table 4.4) indicated that about 54 per cent of the variation in the rice yield was explained by the independent variables (*viz.*, seed, fertilizer, labor, pesticide spray and farmer participation) selected for the analysis in Tank only typology whereas in Tank with wells typology, about 91 per cent of the variation in the rice yield was explained by the independent variables (*viz.*, seed, fertilizer, labor, pesticide spray, supplemental well irrigation and farmer participation) involved in this analysis.

The results shown in Table 4.4 indicate that all the independent variables included in the analysis showed positive impact on rice yield. Fertilizer, labor and the extent of farmer participation towards water management are statistically significant at one per cent level while seed is at 10 per cent significant level in Tank only typology whereas in Tank with wells typology labor was found to be highly significant in influencing the yield while the numbers of supplemental well irrigation from private wells, extent of farmer participation and fertilizer were significant at five per cent level. The positive impact of extent of farmer participation towards water management on rice yield indicates the importance of water management institutions in sustaining rice productivity. The significance of both the number of supplemental well irrigation and the extent of farmer participation in increasing rice productivity has important policy implications for water management and the regulation of private wells in tank commands.

The coefficients of farmer participation were 0.202 and .053 in Tank only and Tank with wells typologies (Table 4.4), could be interpreted that for one per cent increase in farmer participation towards water management from the mean level, *ceteris paribus,* would increase the rice yield by 0.202 per cent in Tank only typology whereas in Tank with wells typology one per cent increase in farmer participation towards water management from the mean level, *ceteris paribus,* would increase the rice yield by 0.053 per cent. It can be translated that for the contribution of Rs. 3.00 by farmer participation towards water management from the mean level, *ceteris paribus*, would increase the rice yield by 8.2 kg in Tank only typology whereas in Tank with wells typology Rs 1.12 increase from the mean level of contribution of farmer participation towards water management, *ceteris paribus*, would increase the rice yield by 2.25 kg. It can further be translated that for the contribution of Rs 1.00 at the mean level, *ceteris paribus,* would increase the rice yield by 2.7 kg in Tank only typology whereas in Tank with wells typology by 2.2 kg per ha of land. It indicates that the return to water management by farmers participation is more in Tank only typology than in Tank with wells typology. One per cent increase in supplemental irrigation from the mean level, *ceteris paribus*, would increase the rice yield by 0.129 per cent per ha in Tank with wells typology. It can be translated that an increase in supplemental irrigation by 0.025 from the mean level, *ceteris paribus*, would increase the rice yield by 6.17kg per ha which is same as that the increase the number of supplemental irrigation by one from the mean level, *ceteris paribus,* would increase the rice yield by 243 kg per ha of land in Tank with wells typology.

## 6.0 Conclusion

As water users association has significant role in farmer participation towards water management which in turn will increase the return from water management, action should be taken to strengthen the activities of water users association. Both capacity building initiatives and strengthening the social capital in the tanks are highly needed. Adequate efforts should be taken in this direction by the village panchayats and NGOs in the regions

### References

Balasubramanian, R. and K.N. Selvaraj, (2003), **"Poverty, Private Property and Common Pool Resource Management: The case of Irrigation Tanks in South India",** South Asian Network for Development and Environmental Economics, Nepal.

Narayanamoorthy, A. (2007), "Tank Irrigation in India: a Time Series Analysis" **Water Policy** 9: 193-216.

Palanisami, K. and K. W. Easter, (2000), **Tank Irrigation in the 21st Century – What Next?,** Discovery Publishing House, New Delhi.

Sarma, A.C. (1992), **"Modernization Programmes and Performance of Jamuna Irrigation System in Assam",** PhD thesis.

# EXPERIMENTAL INVESTIGATION OF HYPORHEIC INTERACTIONS

Upaka Rathnayake[1, 2] and Norihiro Izumi[2]

[1]Department of Civil Engineering, University of Strathclyde, Glasgow, United Kingdom,
[2]Graduate School of Engineering, Hokkaido University, Sapporo, Hokkaido, Japan

upaka.rathnayake@strath.ac.uk

**Abstract**
Research on hyporheic interactions is not new to the present world, but most of the previous research is in the environmental and ecological points of view. This study was to understand the hyporheic interactions by means of engineering perspectives. Several experiments were carried out at laboratory scale to identify the relationships between important non-dimensional river parameters and non-dimensional interaction parameters. Results can be concluded to show some clear relationships among the non-dimensional parameters.

**Keywords**: Hyporheic interactions; Hele-Shaw model; River Froude number

## 1. Introduction

River flow and seepage flow interactions are technically called Hyporheic interactions and frequently occurred in mountainous rivers. These interactions can occur either vertically or horizontally. Since these interactions are the governing force of the most biological activities in the vicinity of the river bed, it is very important to maintain the natural balance of river systems. However due to human activities, the natural balances of most of the nature have been disturbed.

The hyporheic zone can be simply defined as an active eco-tone between the surface stream and ground water, which facilitates to exchange water, nutrients and organic mater due to the variations in discharge and bed topography [1], [2], [3], [5], [12], [15]. Two interactions as “up-welling” and “down-welling” can be identified and Up-welling subsurface water supplies river organisms with nutrients while down-welling river water provides dissolved oxygen, inorganic ions and organic matter to microbes and invertebrates in the hyporheic zone [1], [2], [4], [10], [11], [13], [14], [17], [18], [23], [24].

These up-welling and down-welling interactions influence the biogeochemistry of stream ecosystems by increasing solute residence times and more specifically solute contact with substrates in environments with spatial gradients in dissolved oxygen and pH [8].

Literature shows some typical physical parameters about the hyporheic interactions and the hyporheic zone. The hyporheic flow paths can be centimetres to tens of meters in length [7] and the zone can be few centimetres to couple of meters of thick [16]. Some analysis shows that the hyporheic zone is facilitating to exchange at-least 10% of the river water flow [22].

## 2. Hele-Shaw model experiments

Several experiments were carried out under the laboratory scale at river and watershed engineering laboratory, Hokkaido University, in order to understand the hyporheic interactions and to find out the relationships among non-dimensional parameters. The experimental set up which was used to pursue this research is shown at Figure 1. Seepage layer was modelled using a Hele-Shaw model which is a longitudinal parallel plate model and is shown at Figure 2. Flow above the Hele-Shaw model was considered as the river flow in these experiments.

Hele-Shaw model is a parallel plate model forming a narrow channel and this viscous flow model was first used by Hele-Shaw to study the nature of flow around obstructions of various shapes [21]. In 1936 it was used for the first time in groundwater investigations by Dachler [3]. However later it has widely used to analyze the groundwater flow in a 2-dimensional cross section of an aquifer [3], [6], [9], [19], [20].

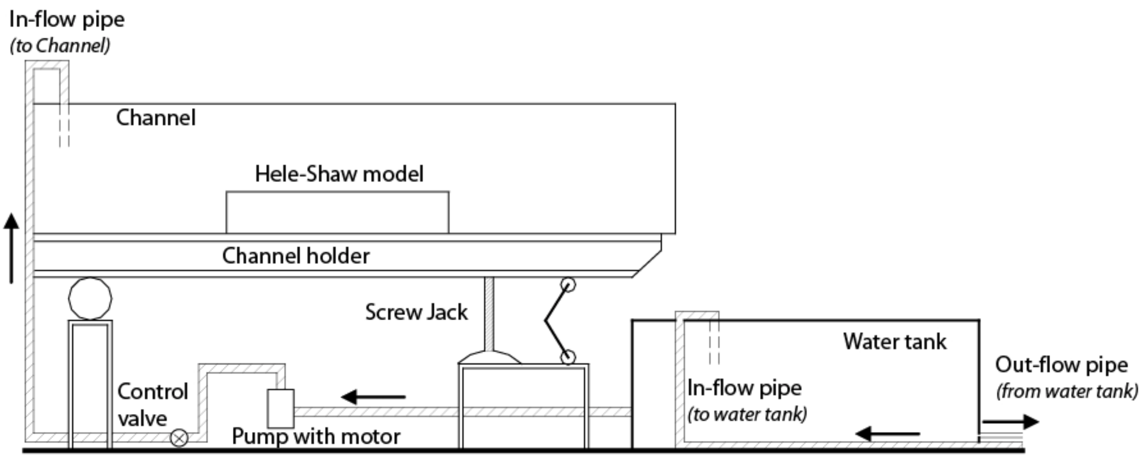



***Figure 1 Experimental setup***

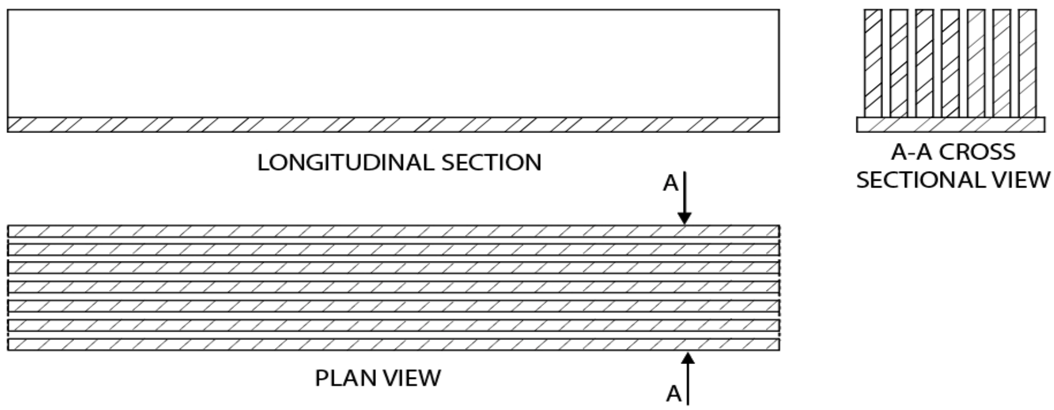



Figure 2 Sectional views of Hele-Shaw model

The re-circulating channel used in this work, has a section of 5 m in length and a channel width of 20 cm. The channel is 40 cm in depth and flexible to change its slope using the screw jack at right edge. It is made up of transparent materials, which allows direct observations of interactions and managed to have digital pictures and digital movies. Let us consider $x$ co-ordinate of the left edge of the channel is to be 0 m and the right edge of the channel is to be 5.0 m respectively. The Hele-Shaw model, where 2.0 m in length, 20 cm in width and 10 cm in height was placed at $x = 2.1$. The porosity; $\lambda$ of the Hele-Shaw model was 0.3 in the used set up.
Experiments were carried out for two slopes as 0.1%, and 0.2%. Combined channel flow height was controlled using a down-stream weir from 12 cm to 25 cm. Three trials of experiments with same conditions were carried out for the each river height and the discharge of the channel was measured for further calculations.
*Methylene blue (blue color dye) was injected just downwards and along the upper boundary of the Hele-Shaw model, in order to visualize the seepage flow and river flow interactions. Continuous pictures were taken at 5 s intervals in each and every experiment. In some cases few videos were taken. Best picture which has the clear interactions was digitized using the commercial package "Bytescout Graph Digitizer Scout 1.2.4". The corresponding wave lengths for S=0.1%, and 0.2% slopes were obtained using these digitized pictures.*
Digital pictures at Figure 3 and 4 are here to verify the hyporheic interactions.

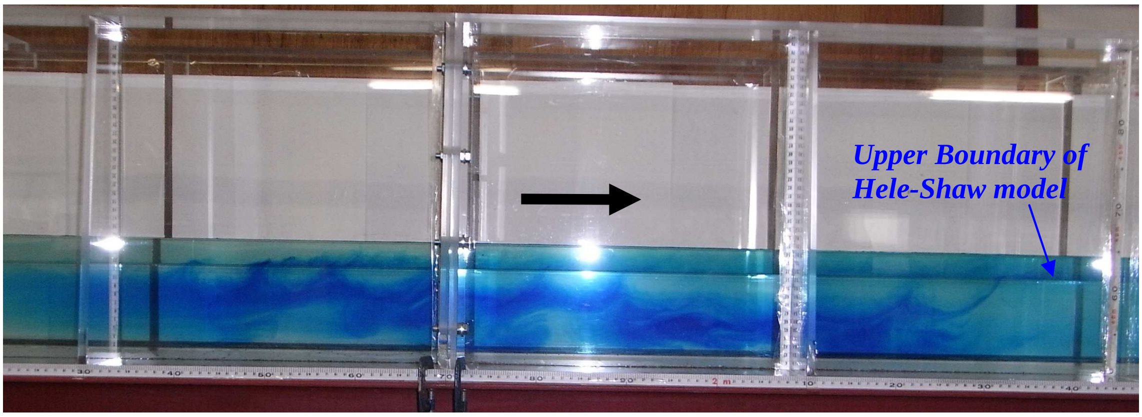



Figure 3 Interactions at 0.1% slope when the river height is 3 cm

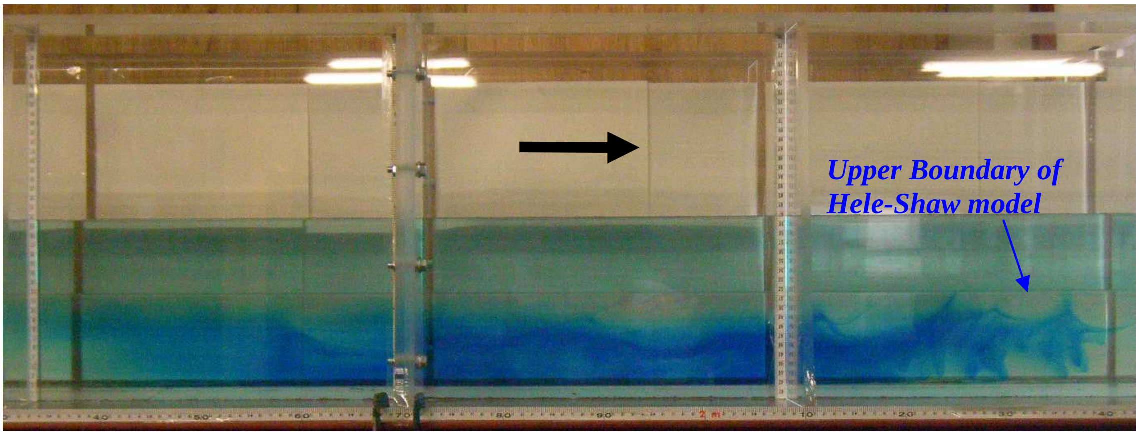



Figure 4 Interactions at 0.2% slope when the river height is 8.0 cm

## 3. Results of the experiments

Froude number for the river / open channel flow was calculated using the discharge measurements and obtained wave lengths are non-dimensionalized using the height of the river layer to obtain the non-dimensional wave numbers which are also know as the dominant wave numbers for interactions. The calculated Froude numbers are plotted against the dimensionless dominant wave number and shown in the Figure 5 and Figure 6. Also the same data are plotted in the one diagram in order to compare the results against the slope of the combined system in Figure 7.

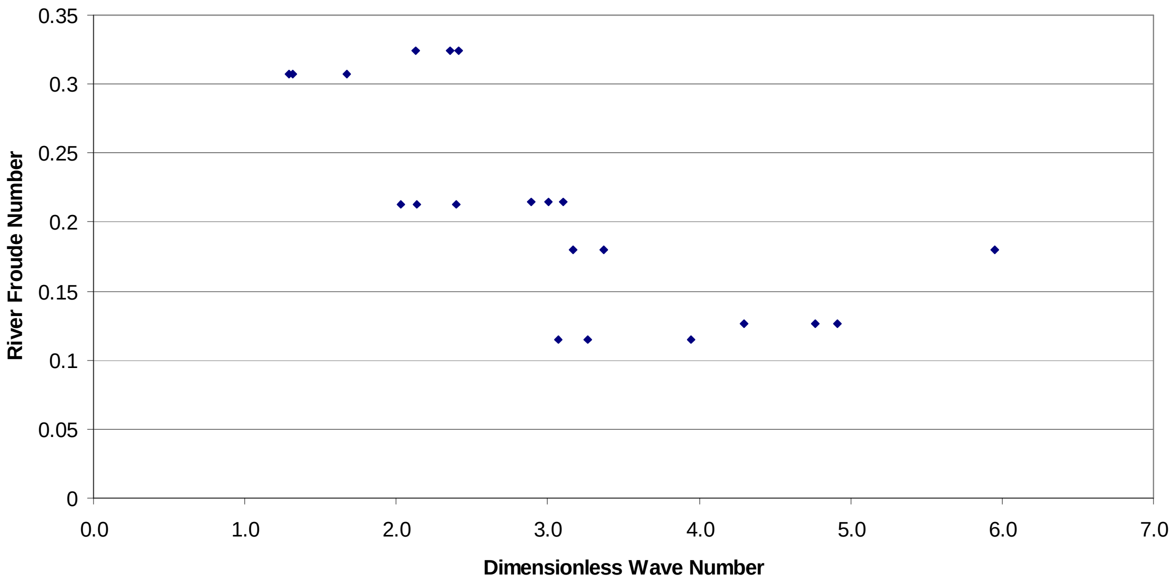



*Figure 5 Froude numbers vs. Dimensionless Dominant Wave Numbers for S=0.1%*

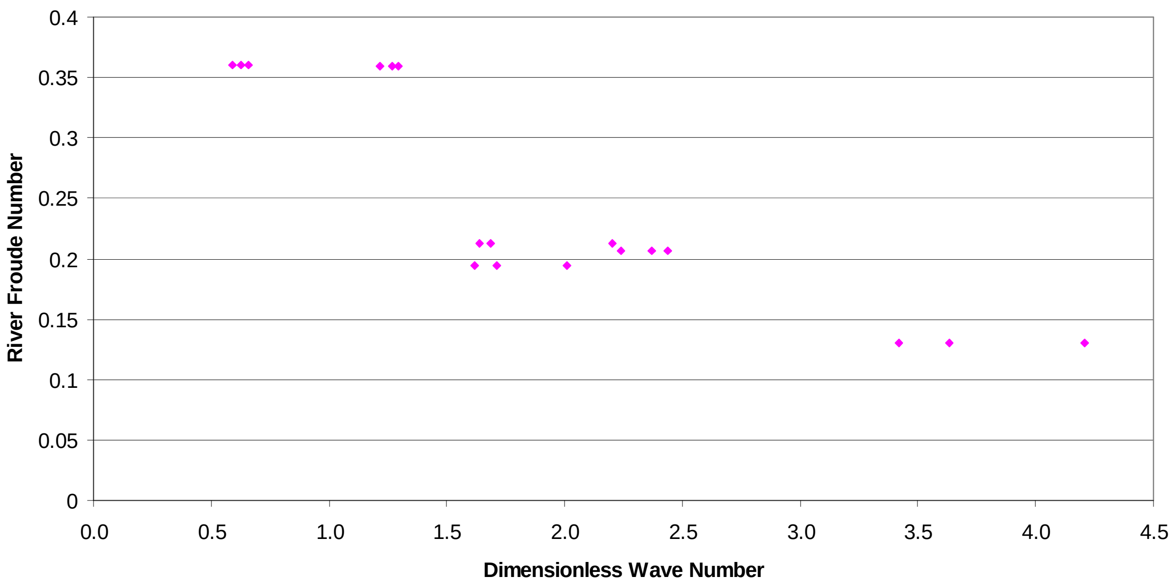



*Figure 6 Froude Numbers vs. Dimensionless Dominant Wave Numbers for S=0.2%*

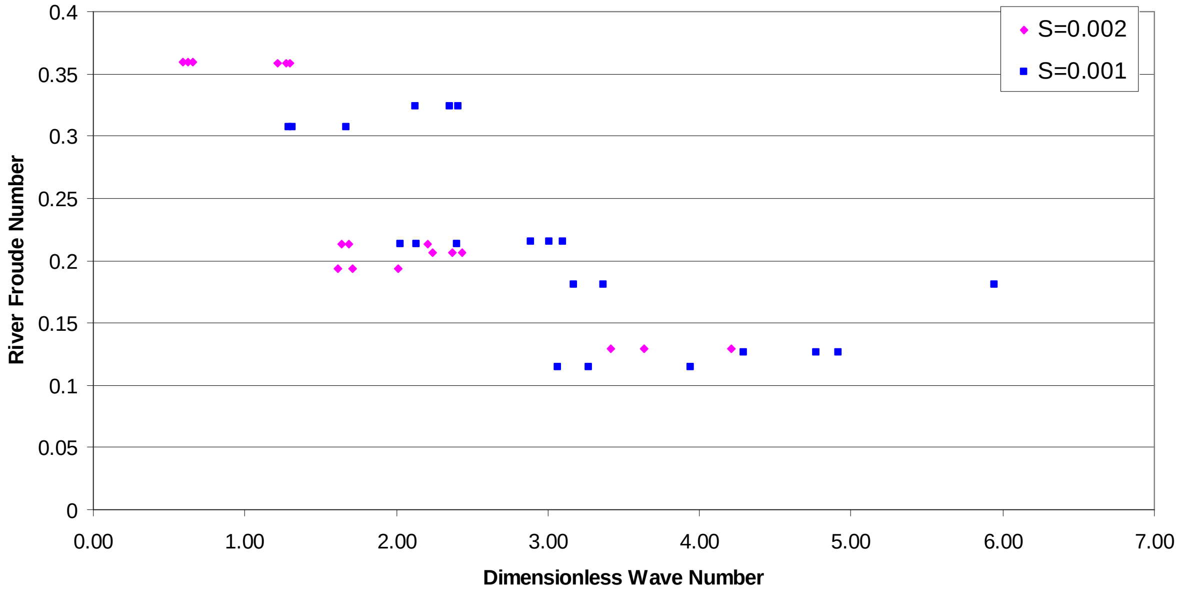



*Figure 7 Froude Number vs. Dimensionless Dominant Wave Number for both slopes*

## 4. Discussion and Conclusions

From the observations it can be clearly visualized the river flow and seepage flow interactions. However by examining Figures 5 and 6, it can be clearly understood a relationship between the Froude number of the river flow and the non-dimensional dominant wave numbers. This can be concluded that the non-dimensional dominant wave numbers for the interactions are increased with the decrease of the Froude number of the river flow. This means that the non-dimensional wave lengths of the interactions increase with the Froude number.

In addition, it can be seen that the dimensionless dominant wave numbers have an effect on the combined channel slope from the Figure 7. With the slope it can be concluded that the dimensionless dominant wave numbers are reached to the Froude number axis, or else the value of the dimensionless dominant wave numbers are decreased. This means that the non-dimensional wave lengths of the interactions increase with the combined slope of the river and the hyporheic zone.

From the experiments there are some observational conclusions and presented them as follows. Quick river flow and seepage flow interactions were occurred, when the height of the river layer is comparably small with the Hele-Shaw model, whereas slow river flow and seepage flow interactions were occurred, when the height of the river layer was comparably large with the Hele-Shaw model. With this observation, it can be concluded that the residence time of hyporheic interactions are increased with the height of the river layer.

When the river layer height was less than or equal to the Hele-Shaw model height, wavy form of the hyporheic interactions were clearly visualized, whereas the wavy form of hyporheic interactions were not clearly visualized, when the river layer height was higher than the Hele-Shaw model height. At the second stage the author was able to see the sudden pop up of dye as shown in Figures 8 and 9.

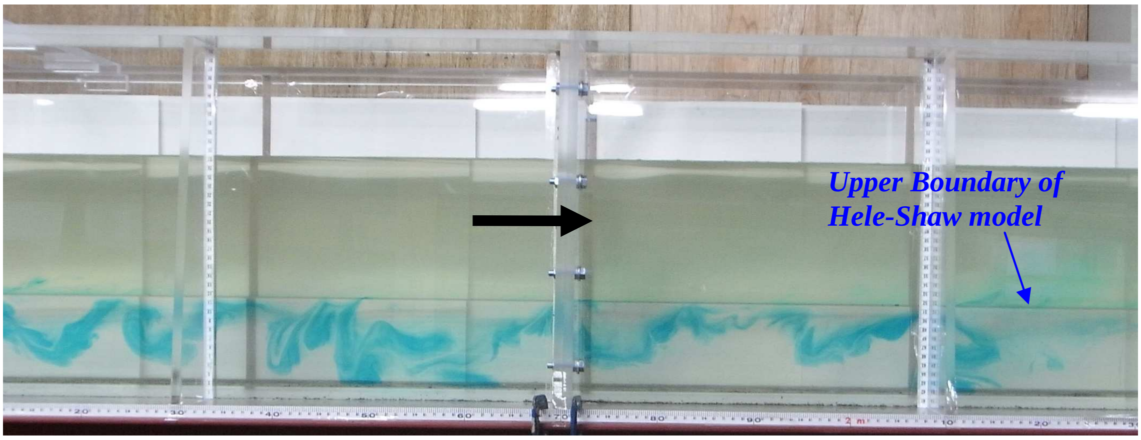



*Figure 8 Interactions at 0.1% slope when the river height is 15.9 cm*

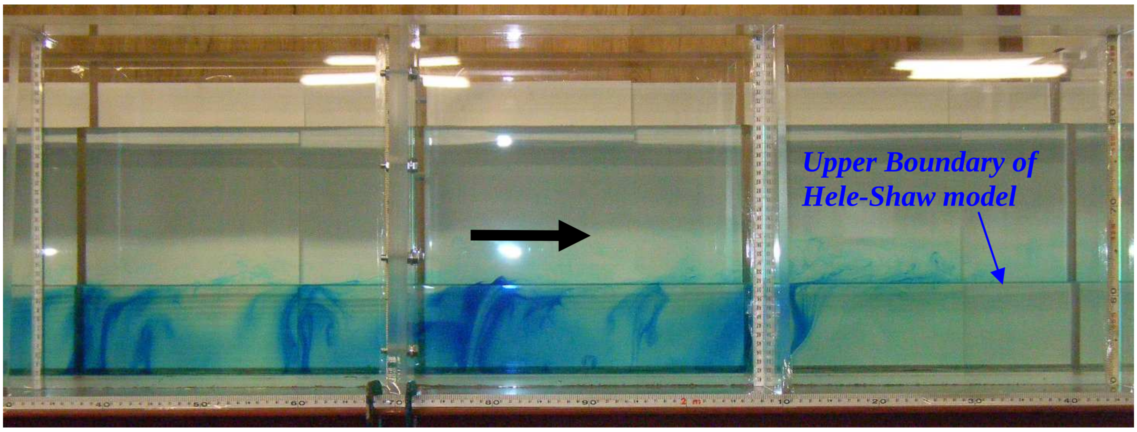



*Figure 9 Interactions at 0.2% slope when the river height is 17.6 cm*

## References

[1] Boulton A.J., Findlay S., Marmonier P., Stanley E.H., and Valett H.M., The functional significance of the hyporheic zone in streams and rivers, *Journal of Annual Review of Ecological Systems*, 29, pp 59-81, 1998

[2] Butturini A., Bernal S., Sabater S. and Sabater F., The influence of riparian-hyporheic zone on the hydrological responses in an intermittent stream, *Journal of Hydrology and Earth System Sciences*, 6(3), pp 515-525, 2002

[3] Columbus N., The design and construction of Hele-Shaw models, *Groundwater*, 4, pp 16-22, 1966

[4] Daniele T. and Buffington M., Hyporheic exchange in gravel bed rivers with pool-riffle morphology; Laboratory experiments and three dimensional modeling, *Journal of Water Resources Research*, 43, 2007

[5] Edwards R.T., The hyporheic zone, *River Ecology and Management*: *Lessons from the Pacific coastal eco-region*, Naiman R.J. and Bilby R.E. (editors), pp 399–429, Springer, New York. 1998

[6] Hartline B.K., An experiment to verify the permeability of Hele-Shaw cells, *Journal of Geophysical research letters*, 5, pp 225-227, 1978

[7] Harvey J.W. and Wagner B.J., Quantifying hydrologic interactions between streams and their subsurface hyporheic zones, *Streams and ground waters*, Jones J.B. and Mulholland P.J., (Editors), Academic Press, San Diego, USA, pp 197-220, 2000

[8] Kenneth E. B., Hyporheic zone hydrological processes, *Journal of Hydrological Processes*, 14, pp 2797–2798, 2000

[9] Marino M.A., Hele-Shaw model study of the growth and decay of groundwater ridges, *Journal of geophysical research*, 72(4), pp 1195-1205, 1957

[10] Miglio E., Quarteroni A. and Saleri F., Coupling of free surface and groundwater flows, *Journal of Computational Fluids*, 32, pp 73–83, 2003

[11] Nakamura Y. and Stefan H.G., Effect of flow velocity on sediment oxygen demand: theory, *Journal of Hydraulic Engineering*, 120(5), pp 996–1016, 1994

[12] Runkel R.L., McKnight D.M. and Rajaram H., Modeling hyporheic zone processes, *Journal of Advances in Water Resources*, 26, pp 901–905, 2003

[13] Ryan R.J. and Boufadel M.C., Lateral and longitudinal variation of hyporheic exchange in a piedmont stream pool, *Journal of Environmental Science and Technology*, 41, pp 4221-4226, 2007

[14] Salehin M., Packman A.I., and Paradis M., Hyporheic exchange with heterogeneous streambeds: Laboratory experiments and modeling, *Journal of Water Resources Research*, 40, 2004

[15] Smith J.W.N., Groundwater–surface water interactions in the hyporheic zone, *Science Report SC030155/SR1*, , Environment Agency (www.environment-agency.gov.uk), 2005

[16] Stanford J.A., Ward J.V. and Ellis B.K., Ecology of the alluvial aquifers of the Flathead River, Montana, Gibert J., Danielopol D.L., and Stanford J.A. (editors), *Groundwater ecology*, Academic Press, San Diego, California, pp 367–390, 1994

[17] Steinberger N. and Hondzo M., Diffusional mass transfer at sediment–water interface, *Journal of Environmental Engineering*, 125(2), pp 192–199, 1999

[18] Strayer D.L., May S.E., Nielsen P., Wolheim W. and Hausam S., Oxygen, organic matter, and sediment granulometry as controls on hyporheic animal communities, *Journal of* **Archiv** *fur* **Hydrobiology,** 140 pp 131–144, 1997

[19] Todd D.K., Unsteady flow in porous media by means of a Hele-Shaw viscous fluid model, *Journal of American geophysical union*, 35, pp 905-915, 1954

[20] Todd D.K., Laboratory research with groundwater models, *Proceedings of ASCE*, pp 199-206, 1955

[21] Todd D.K., *Groundwater hydrology*, John Wiley and Sons, New York, pp 336, 1959

[22] Triska F.J. , Kennedy V.C. , Avanzino R.J. , Zellweger G.W. , and Bencala K.E.**,** Retention and transport of nutrients in a third-order stream in Northwestern California: hyporheic processes, *Journal of Ecology*, 70 (6), pp 1893-1905, 1989

[23] Trussell R.R. and Chang M., Review of flow through porous media as applied to head loss in water filters, *Journal of Environmental Engineering*, 125(11), pp 998–1006, 1999

[24] Vollmera S., Ramos F.S., Daebel H. and Kuhn G., Micro scale exchange processes between surface and subsurface water, *Journal of Hydrology*, 269, pp 3–10, 2002

# WATER MARKET IN THE TANK IRRIGATION SYSTEMS IN TAMIL NADU

Nanthakumaran. A[1] and Palanisami. K[2]

[1] Senior Lecturer, Dept. of Biological Sciences, Faculty of Applied Science, Vavuniya Campus, Vavuniya, Sri Lanka. n_ananthi@hotmail.com Tel:0094715364459 Fax:0094242222265

[2]Director, IWMI-TATA Policy Research Program, International Water Management Institute, South Asia Regional Office, Hyderabad, India. K.palanisami@cgiar.org

1- Corresponding author

**Abstract**

Continued progress in water resources development in the future will depend upon the utilization of the existing irrigation potential. An irrigation tank is a small reservoir to catch and store water during rainy season and use it for irrigation during dry season. They recharge groundwater, which is not only a major source of drinking water for numerous rural and urban communities, but also serve as a supplementary source for tank water. Due to the loss in tank storage capacities, wells have become an important source of supplementary water. Since farmers initially use tank water for cultivation, the risk associated with getting adequate water, especially late in the season, has encouraged farmers to use wells for supplemental irrigation particularly later in the crop season. Since only a few farmers in the tank command area own wells, and there is a growing demand for well water, the well owners in most cases act like local monopolists. The study was undertaken with the objective to study the water market in the two districts of Tamil Nadu *viz.*, Sivagangai and Coimbatore. Inverse demand function, Output function and Cost function were used to study the monopolistic behavior of water market. The profit maximizing levels of well yield, price of water and hours of pumping are 4.6meters, Rs10 and 8.6 hours, respectively. Well owners maximize profits from water sales when the water level in the well is at about five meters and the price of pumping hour is Rs. 10 and this correspond to about nine hours of pumping per day from the well. Currently they pump only four hours per day and the water level in the well is about eight meter. Under these conditions, well water output can best be increased by having farmers install more wells and increased competition. With more wells, the demand for water from each individual well will fall, resulting in a lower well water price. Therefore there is a need to increase the number of wells in the tank command area in the study area up to threshold level.

***Key words:*** *Tank irrigation systems, Supplemental well irrigation, Water market*

## 1. Introduction

Supplemental well irrigation is a crucial factor which determined the rice yield in tank irrigated area. Since farmers transplant the rice immediately after the start of first tank filling the risk associated with getting adequate water, especially late in the season, has encouraged the farmers to use wells for supplementary irrigation particularly late in the crop season. The limited number of wells present in the tank command areas leads to the existence of water market in the tank command area. As there are only a few well owners, they act like monopolists. Each well owner may be the only supplier of groundwater, at least for the group of farmers located around the well. Since the number of wells is limited in most tanks, monopolistic behavior is quite common. Well interference during pumping and recharge rates is reflected in water availability and price. Well owners maximize their profits with respect to the water supplies available and likely demands. Well owners cannot set price and quantity independently since price is determined by the supply and demand for water. Reduction in pumping (up to a certain level) can increase the water price resulting in higher profit. However the marginal cost of pumping is very low (as the electricity is free of charge in Tamil Nadu) and it only pays to reduce pumping in the range where demand is inelastic.

## 2. Materials and Methods

### 2.1 Literature review

#### 2.1.1 Monopoly market

Monopoly is a market in which there is one seller of a product. The product has no close substitute. The cross-elasticity of demand with every other product is very low. He is a price-maker, who can set the price to his maximum advantage. In monopoly market, one firm controlled all the supply and set prices to suit it, at limited mainly by the availability of substitutes for its product (Roy *et al.*, 1971). In this study, a monopoly market is recognized as a situation where there is a single seller and many buyers.

#### 2.1.2 Groundwater utilization status and its market

Linsley *et al.* (1958) defined aquifer as a geological formation which contains water and transmits it from one point to another in quantities sufficient to permit economic development. Chow (1964) stated that usable ground water occurs in permeable geologic formation known as aquifers. According to Walton, (1990) ground water storage in deposits above aquifers permitted pumping for limited periods of time at rates greater than recharge. Many aquifers were limited in real extent and results in depletion of these aquifers. In a market, sellers were supposed to sell what they own or produce: in the case of water market, neither was the case. Water sellers neither owned nor produced the water they sell; all they sell were the services of well and also their irrigation equipments. The so called “water markets” were actually the lease markets for pumping equipment and a well. Ground water market were used to describe a localized, village level institutional arrangement through which owners of open or tube wells mounted with electric motor or diesel engine-supply irrigation service to other members of the community at a price. The sellers were typically private operators; but a state tube-well or a co-operatively owned tube well too may compete in water markets.

In this study groundwater market is perceived as an act of selling and buying of groundwater at a price and well owners are considered as local monopolists. And also the use of ground water as supplementation to tank water under different level of tank supply and an attempt was made to find out the price of water; hours of pumping and well yield.

### 2.2 Methods

Two districts were purposively selected in Tamil Nadu, wherein Sivagangai from southern part and Coimbatore district from North-western part represent the Tanks with Wells and Wells only typologies. 113 farm households and 27 farm households who involved in purchasing of water in Tank with wells typology and Wells only typology respectively were selected for this study.

#### 2.2.1. Estimation of inverse demand, cost and output functions

The limited number of wells present in the tank command area leads to the existence of water market in the tank command area. As there are only a few well owners, they act like monopolists. Each well owner may be the only supplier of groundwater, at least for the group of farmers located around the well. Since the number of wells is limited in most tanks, monopolistic behavior is quite common. Well interference during pumping and recharge rates is reflected in water availability and price. Well owners’ maximize their profits with respect to the water supplies available and likely demands. Well owners cannot set price and quantity independently since price is determined by the supply and demand for water. Reduction in pumping (up to a certain level) can increase the water price resulting in higher profit. However the marginal cost of pumping is very low (as the electricity is free of charge) and it only pays to reduce pumping in the range where demand is inelastic.

Henderson and Quant (1971) explained the basic principle used by considering a case of bilateral monopoly in the market for a produced good $Q_2$, the buyer uses $Q_2$ as an input to produce $Q_1$, according to his production function $q_1 = h(q_2)$. He sells $Q_1$ in a competitive market at the fixed price $p_1$. The seller uses a single input X for the production of $Q_2$. He buys X in a competitive market at the fixed price r. Assume that his production function can be expressed in inverse form as $x = H(q_2)$.

For this study the water is considered as a commodity in the market and solved for the equations of inverse demand function, output function and cost function derived from the field survey data specified as follows.

Inverse demand function: Pp = f (Qp)

Output function: Qp = g(WY)

Cost function: AC = h(Qp)

With derived inverse demand, output and cost functions, the profit function arrived as given below and equate its first derivative to zero will give the maximum profit level.

$$Л = (P_p * Q_p) – (AC * Q_p) - FC$$

$$= f(Q_p).\ Q_p – h(Q_p).\ Q_p – FC$$

$dЛ/dQ_p = f'\ Q_p + f – h'Q_p – h = 0$ and by substituting $Q_p$ in the equation , the value of well yield (Wy) can be arrived.

Where,

Л = Profit in Rs

$P_p$ =Price of pump water in Rs/hr

$Q_p$ –Quantity available for pumping in hrs

AC = average cost of pump water in Rs/hr

FC =fixed cost in Rs/hr.

## 3. Results

**Table 3.1 Water buyers in Tank with wells and in Wells only typologies**

| Farmers | Tank with wells | | Wells only | |
|---|---|---|---|---|
| | Number of farmers purchasing water | *Price Rs/hr | Number of farmers purchasing water | *Price Rs/hr |
| Marginal farmers | 53 | 17 | 0 | - |
| Small farmers | 57 | 18 | 13 | 15 |
| Large farmers | 3 | 35 | 14 | 22 |
| Total | 113 | *18 | 27 | *18.60 |

*weighted average of the price

**Table 3.2 Annualized cost and average cost of pumping hour in different typologies**

| Particulars | Typology | |
|---|---|---|
| | Tank with wells | Wells only |
| Average annualized cost (Rs) [10] | 11,560 | 14,750 |
| Average annual pumping hours* | 1,116 | 1,378 |
| Average cost/ pumping hour (Rs) | 10.35 | 10.70 |
| Average cost per irrigation per ha | 176 | 203 |
| Price of water in the water market (Rs) per irrigation per ha | 306 | 354 |

*Pumping hours was calculated from the survey data. During survey, the pumping hours per day frequency of irrigation in a week and months of irrigation were collected from the farmers. Based on this information month-wise pumping hours was calculated from January to December, 2006/07 cropping year and the average was taken for computation.

### 3.1 Price of water, pumping hours and well yield

For different level of water prices and varying pumping hours in the study area, it is important to know at what level of pumping ($Q_p$) and water price ($P_p$) well owners maximize their profit. Using the fitted inverse demand, and output and average cost (AC) functions, and solving the equations for well yield (WY),

---

[10] Groundwater cost at Tank with wells situation

Capital cost (C) = Rs 80000

Capital Recovery Factor (CRF) = $\frac{0.11(1.11)^{20}}{(1.11)^{20}-1}$ =0.125

Annualized cost (A) = CxCRF

= Rs 80000 x 0.125=10000

Repair and Labor cost = Rs 1560

Total cost = Rs 11560

Annual pumping hours = 1116

Average cost = Rs10.35/hour

For Well situation

Capital cost is 100000, as there are bore , tube wells;

Repair and labor cost =Rs 2250

annual pumping hours = 1378

Inverse demand function: $Pp = 25.24 – 1.655\ Qp^{**}$

(1.622) (0.62)

Output function: $Qp = - 0.237 + 2.19\ WY^{*}$

(.784) (1.07)

Cost function: $AC = 7.001^{*} – 0.591\ Qp^{***}$

(0.49) (0.193)

***, **, * indicate significance at one, five and 10 per cent level.

Figures in parenthesis are standard errors.

## 4. Discussion

### 4.1 Groundwater use in tank irrigation systems

Water purchase, sales and their price could show the scarcity and importance of water in the study area. It can also explain the details performing to the nature of water sales and the extent of water scarcity in the study region. The details of the water buyers and the price paid per pumping hour are given in Table 3.1. Out of the total farmers selected for the study, 113 and 27 farmers were water buyers in Tank with wells and Wells only typologies respectively.

Price per pumping hour differs with locations of the wells, its depth and the monopoly behavior of the well owner which ranged from Rs. 10 to Rs. 50 per pumping hour in the study area. Majority of the large farmers owned wells and a few of them do not own wells. As they are large farmers, the well owners might fix a higher rate for them and also due to the location of those wells, they paid a higher rate for a pumping hour in the study area. On an average a farmer from Tank with wells typology pays Rs. 18 per hour and in the case of Wells only typology, it is Rs. 18.60 per hour (Table 3.1).

### 4.2 Cost of pumping

The annualized cost of wells was computed to find out the average cost of irrigation in Tank with wells and Wells only typologies. The cost of irrigation depends on the type of well (dug well, dug cum bore well, tube well), current status of well, year of construction, average age or life of well and the discount rate. The value of electric motor and the annual repair charges were also included for the computation of annualized cost of irrigation.

The average annualized cost of wells was higher in Wells only typology than in Tank with wells typology (Table 3.2). Even though a higher annual pumping hours is observed in Wells only typology, the average cost of pumping was also higher than in the Tank with wells typology. This may be due to the depth of water table which is more in Wells only typology and most of the farmers have bore wells, dug cum bore wells and tube wells. The water table is very deep and the cost of construction is also high.

Seller of groundwater in the Tank with wells situation earns a profit[11] of Rs. 130 per irrigation per ha by providing one irrigation to the sugarcane crop (assuming one irrigation for a hectare takes 17 hours of pumping). In wells only situation, a profit of Rs. 151 per ha is earned by providing one irrigation to the sugarcane crop (assuming one irrigation for a hectare takes 19 hours of pumping). This

[11] Profit = (Price of irrigation per ha – Cost of irrigation per ha)

Price of irrigation per ha = Number of hours taken to irrigate per ha x price of water per pumping per hour.

Cost of irrigation per ha = Number of hours taken to irrigate per ha x Average cost per pumping.

higher charge for well irrigation is due to higher demand for groundwater in both Tank with wells and Wells only situations.

### 4.3 Price of water, pumping hours and well yield

The profit maximizing levels of WY, Pp and Qp are 4.6meters, Rs10 and 8.6 hours respectively.

Well owners maximize profits from water sales when the water level in the well is at about five meters and the price of pumping hour is Rs. 10 and this correspond to about nine hours of pumping per day from the well. Currently they pump only four hours per day and the water level in the well is about eight meter.

## 5. Conclusions

Well owners maximize profits from water sales when the water level in the well is at about five meters and the price of pumping hour is Rs. 10 and this corresponds to about nine hours of pumping per day from the well. Currently they pump only four hours per day and the water level in the well is about eight meters in the beginning of the tank season and fall drastically resulting in lesser pumping from the wells. Under these conditions, well water output can best be increased by installing more wells and the demand for water from each individual well will fall, resulting in a lower well water price. Therefore there is a need to increase the number of wells in the tank command area in the study area up to the threshold level.

## References


Chow, V.T. (1964), **"Hand book of Applied hydrology: A Compendium of Water Resources Technology",** New York: McGraw Hill Book Company, Inc: 13-14.

Henderson, J.M. and R.E. Quant, (1971), **Micro Economic Theory: A Mathematical Approach.** Mc Graw – Hill Inc. London.

Linsley, R.K., A. Kohler and J.L.H. Paulthus, (1958), **"Hydrology for Engineers",** New York: McGraw Hill Book Company, Inc:129.

Roy, P.E., L. F. Corty and D. G.Sullivan, (1971), **"Economics – Applications to Agriculture and Agri-business",** Illinois: The Interstate Printers and Publishers Inc. Danville.

Walton, W.C. (1990), **"Ground Water Resource Evaluation",** New Delhi: McGraw Hill Kogakusha Ltd: 361.

# TEMPORAL AND SPACIAL CHANGES IN BED-FORM DUE TO CHANGE IN FLOW IN A FLUME ENVIRONMENT

Chamika De Costa, A / Prof. Stephen Coleman

University of Auckland
Gregory.DeCosta@openpolytechnic.ac.nz

Prof Kawanishi
Hiroshima University

**ABSTRACT**
Although rates of bed-form growth for steady flows (Nikora and Hicks 1997), have been clarified, practical implications and models accessible to the engineer remain to be elaborated. For example, how and at what rates bed forms change for increasing and decreasing flows remains to be quantified. There has not been much progress since Julien and Klaassen 1995 in defining a relationship between flow and bed-form characteristics. Sediment transport engineers in the current era have a very good idea of the size and shape of a particular dune at various discharges and sediment types. But very little is known about the time it takes to change the dune when the flow was to experience an increase or decrease. Previous research in this specific area was done by (J.R.L. Allen, 1976) who did an earlier model for dune time-lag in periodically varying unidirectional flows.

The research undertook measurements of a river and data over discharge and dune wavelength over the year. This data was then computed on a monthly basis. Model showed that hydrograph shape could substantially influence dune behaviour in unsteady flows. For the same flow period and extreme discharge values, a reduction in the relative duration of the high-water stages causes an increase in the phase differences between dune dimensions and flow, and an increase in the dimensions averaged over the flow cycle as compared with the similarly averaged dimensions given no lag.

The relative range of dimensions over the flow cycle is little affected. This research is mainly about how the bed form reacts to the change in flow and specifically the time it requires for a specific bed form to adopt its new bed form in regards to increase/decrease in flow. This research takes an experimental form to develop a stochastic modal for the time required for the change in bed-form morphology in relation to the change in flow. This includes the dune shape and height. The experimental analysis is in a flume with controlled sediment type/density/size, water depth and also the flow rate. The analysis is for the flow in a uni-direction. The depth of the dunes, shape and the velocity of the flow is measured by an ADV, and analyzed later using matlab to include a 3D representation and analysis. Through which the temporal and special changes in bed form due to change in flow is made clear and presented herein.

## 1. INTRODUCTION

Dunes migrating along bars in a river are moving through a spatially changing sediment transport field that is associated with the larger-scale bed topography. Dunes respond to this change in their environment in three basic ways :( 1) by adjusting their shape,( 2) by adjusting in size, and (3) by adjusting their rate of downstream migration. The accommodation path the dunes take on any particular section of bar surface seems to be strongly dependent on the character of the bar topography forcing the change. Conversely the dynamics of the bar cannot be understood without taking into account the effects of dunes. For example, change in their shape as dunes move along point bars strongly affects the transport paths of sediment grains of different sizes, thereby affecting the sorting of bed material throughout meander bends and the equilibrium shape of point bars.

This stream wise change in dune shape is the consequence of a systematic cross-stream variation in dune migration rate. In another example, down current decrease in the average size of dunes has been linked directly to the deposition of sediment and growth of languid bars. In this case the average rate of migration remains constant as dunes become smaller by transferring sediment into underlying bar forms. Clearly, the ability to predict the migration rate of dunes is important to forecasting dune-bar interactions. Because dunes are themselves composed of transported sand, their rate of migration must be related to the local sediment transport rate. If all sediment moving over dune crests is captured on adjacent lee faces, then by conserving sediment as it is straight forward to relate the rate of dune advance to the volume flux of sediment and vice versa. Many trains of dunes are, however, imperfect sediment traps, and the behaviors of these dunes cannot be predicted from the sediment transport in a channel until the fraction of sediment that is bypassing dunes and therefore not contributing to their mass is known.

Accurate prediction of stage and flow developments for a flood must recognize the transient nature of erodible-boundary roughness, implying knowledge of bed-form generation and development processes as flows increase and decrease in intensity. From an experimental / measurement perspective and probably from a theoretical modeling perspective, the transient problem in which dune characteristics change over time poses *additional* severe difficulties beyond those of the equilibrium case. For sediment transport engineering, a minimum contribution desired of a theory ~model, understanding! for bed-form development would be a reliable means of determining which equilibrium would be established, i.e., delineating stability boundaries. Attempts have been made to base such boundaries on theoretical stability models, a` la Kennedy ~1969!, but engineering approaches ~e.g., van Rijn 1984a, b, c! have been primarily based on dimensional analysis and empiricism.

Recent experimental and theoretical works ~e.g., Coleman and Melville 1996; Coleman and Fenton 2000! have focused on the bed-form initiation process. Are there any implications of initiation and instability mechanisms for the finite-amplitude dune bed that is of most practical interest? Although turbulence may not be an essential feature of the initial instability of a sediment bed ~Coleman and Eling 2000!, does it play a more prominent role at later stages of bed evolution? While the mechanics of bed-form development ~Coleman and Melville 1994, and rates of bed-form growth for steady flows ~Nikora and Hicks 1997, have been clarified, practical implications and models accessible to the engineer remain to be elaborated. For example, how and at what rates bed forms change for increasing and decreasing flows remains to be quantified. The problem of transitions to a dune bed from a rippled or plane bed and from a dune bed to an upper-regime plane bed or antidune bed is also of much practical interest. Has recent work shed any light on this important aspect of non-equilibrium beds? Does turbulence modulation drive the dune upper regime plane bed transition?

## 2. PROCEDURE

The aim of this flume experiments is to see how the bed form reacts to the change in flow and specifically the time it requires for a specific bed form to adopt its new bed form in regards to increase/decrease in flow. This research takes an experimental form to develop a stochastic modal for the time required for the change in bed-form morphology in relation to the change in flow. This includes the dune shape and height. The experimental analysis is in a flume with controlled sediment type/density/size, water depth and also the flow rate. The analysis is for the flow in a uni-direction. The depth of the dunes, shape and the velocity of the flow is measured by an ADV, and analyzed later using matlab to include a 3D representation and analysis. Through which the temporal and special changes in bed form due to change in flow is made clear and presented herein.

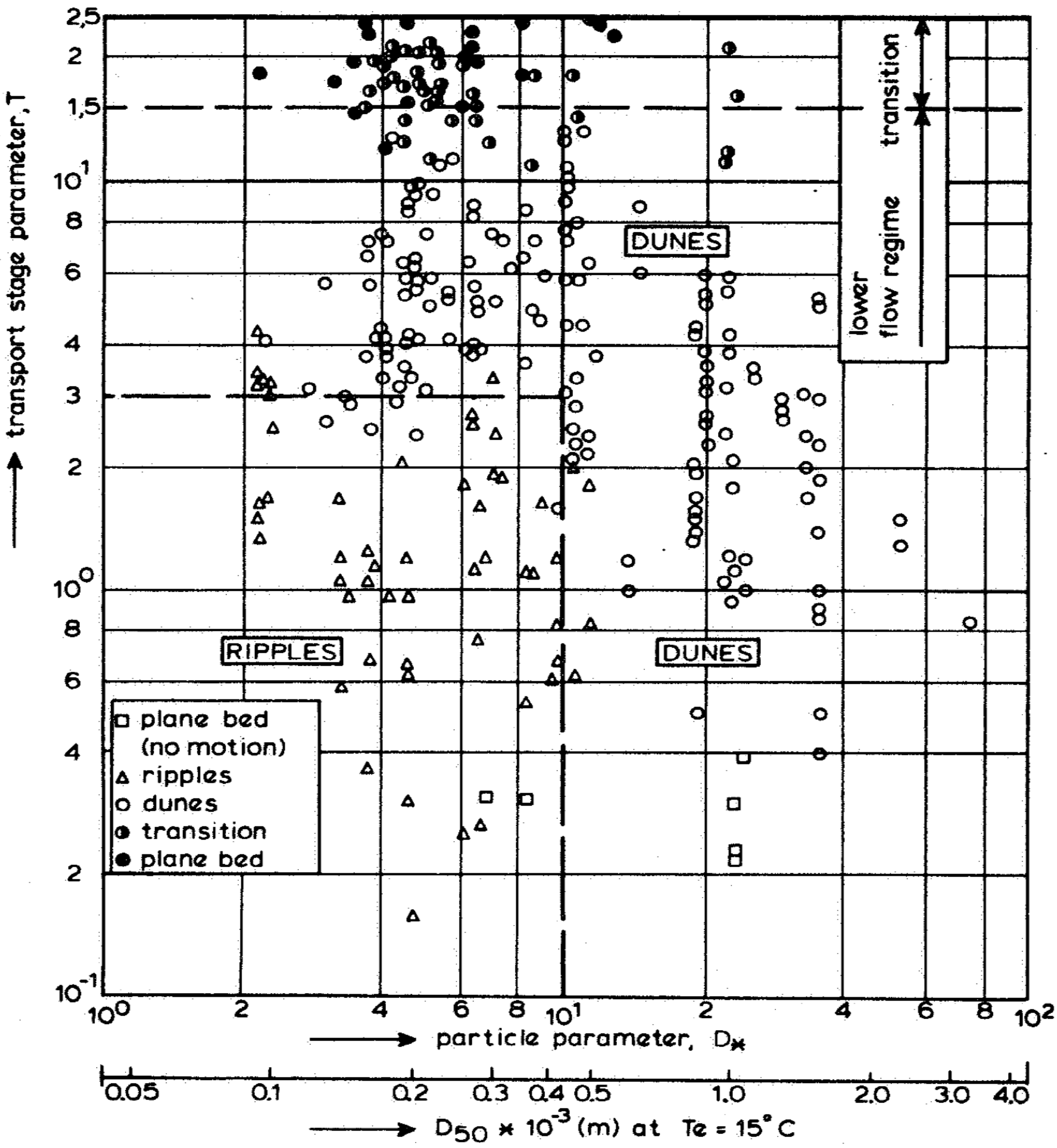
In defining dunes and ripples, the following figure is used. Bed form classification is performed accordingly. Additional Phase diagram formed by eliminating time explicitly between the variation with respect to time of the independent quantity discharge and the variation with respect to time of the chosen dune dimension, the dependent variable. Comparison with theoretical models, show that the dune dimensions vary on the same period as the discharge but on a different phase.

Although time is eliminated explicitly, each has only one correct trajectory, namely, anticlockwise in all the examples The loops differ sharply from the theoretical relationships between dune wavelength, height and discharge in the absence of lag, that is, had the dunes always responded perfectly to flow changes. The effect of increasing dune excursion is to make the dune assemblages of both series depart increasingly from this simple theoretical picture. At the smallest excursion, the range of mean actual wavelength is nearly identical with the theoretical range. At the largest excursion , however, the wavelength is virtually constant, although the discharge varies nearly six-fold. Mean actual dune height responds similarly to changing dune excursion, though the trend is weaker, because the dunes individually have some ability to respond in terms of height to the changes of flow, but no ability to vary in wavelength.

The two series differ most in terms of the shapes of the phase diagrams . Loops from Series tend to a smoothly oval form, closely resembling yielded by the earlier model for comparable excursions and the same simple-harmonic discharge variation (i.e. k--1). In contrast, graphs from Series B tend to be either pointed or flattened on the side representing low discharges. In these experiments, distinguished by a long low-water season, there are large reductions in dune dimensions over this extended period of almost constant flows.

***Equivalent phase differences***

A quantitative estimate of the phase difference between the variation of discharge and the variation of some dune dimension is obtainable using an earlier procedure. Briefly, the area of each loop is measured graphically, together with the area of the smallest escribed rectangle that has sides parallel with the ordinate and abscissa of the graph. The phase difference is estimated as an "equivalent" value by introducing the ratio of the two areas into the graphed function relating area ratio to phase difference in a doubly simple-harmonic theoretical model. It was earlier found that the equivalent phase difference generally increased with increasing excursion and time ratio, the latter a measure of the ratio of the long-term mean theoretical dune life-span to the flow period.




In each Series the equivalent wavelength phase difference increases steeply with the time ratio for small values of the ratio. At larger ratios, equivalent phase differences comparable with 7r/2rad are obtained. Some of the phase diagrams are ambiguous, however, affording a phase difference either somewhat smaller or a little larger than 7r/2 rad. A similar ambiguity was occasionally found earlier. At the larger time ratios, appear to yield the smaller wavelength phase differences.

The equivalent height phase difference also increases steeply with the time ratio for small values of the ratio. There are no ambiguous loops, however, the larger differences over the full range of experimental conditions. The generally smaller phase differences obtained for height as compared with wavelength may also be attributed to the effect of the non-zero coefficient of change, causing the dune assemblages to lag less in height than in wavelength.

***Instantaneous phase differences***

The phase difference as estimated above is merely a "characteristic" value, which could diminish in usefulness as the experimental system becomes more complex in behavior. Following an earlier discussion, when it was suggested that the life-span of a bed form was set partly by the prevailing environmental conditions, it seems likely that this characteristic difference is in truth a time-average, determined by the changing dune properties over the whole flow cycle. Because the equivalent phase difference increases with the time ratio, the difference must also increase with the long-term actual dune life-span. It follows that, although excursion is constant in each experiment, the life-span of a large dune created at high discharge will be substantially more than that of a small dune fashioned at low stage. Hence when large long-lived dunes typify the bed, we should expect to observe different values of an "instantaneous" phase difference (perhaps generally larger) than when small short lived forms predominate.

The practical estimation of the instantaneous difference may be illustrated by the case for dune wavelength. Dune wavelength in the model theoretically is linearly proportional to flow depth, which itself varies as the discharge to the power 2/3.

## 3. DISCUSSION

Ripples and dunes in many natural environments are subject to flows having high-frequency directional variations and can be expected to follow the same rule of alignment as the experimental wind ripples and subaqueous dunes. The dominant bedform trend parallels the resultant transport direction (upper right to lower left), but as the experimental conditions are near the transition to transverse bedforms, transverse bedforms are present also.

Although the vector resultant is the appropriate parameter for describing the net rate and direction of sediment transport, the problem of bedform genesis is so different that another parameter is needed to characterize flow conditions. When sediment is transported toward opposing directions, the opposing transport cancels out-a physical process that is accurately described when the resultant is calculated.

It can be argued that all transport should be considered to have a positive effect, because all transport may be involved in creating bedforms. For example, consider a wave-generated onshore-offshore flow combined with a small unidirectional alongshore flow. If the onshore and offshore components are equal, then they cancel out, and the resultant of the system is equal to the unidirectional vector. Regardless of the strength of the onshore-offshore flow, it has no effect on the resultant-yet it is typically this stronger wave generated oscillatory flow that is responsible for producing bedforms.

For problems of bedform alignment, a new parameter is needed to characterize a multidirectional flow in such a manner that flow toward opposing directions is represented rather than cancelled.
One such parameter is 'gross bedform-normal transport. Transport over any bedform can be resolved into two components, one normal to the bedform trend and one parallel to the bedform trend. In a purely unidirectional flow, all transport over perfectly transverse bedforms is bedform-normal, and no transport over longitudinal bedforms is bedform-normal.

Where a bedform is subject to two or more transport vectors, bedform-normal transport is defined as the sum of the bedform-normal components. Net bedform-normal transport is the sum of the bedform-normal components, considering forward transport across the bedforms to be positive and reverse transport to be negative. Gross bedform-normal transport is the sum of the bedform-normal components, considering all transport to be positive. By treating all transport as positive, no transport is lost to the cancellation of opposing vectors.

A complexity arises when determining bedform normal transport of a flow because the quantity cannot be determined independently of bedform orientation; a single multidirectional flow has different amounts of bedform-normal transport for different arbitrary bedform orientations.

Results of the present experiments with subaqueous dunes and the previous experiments with wind ripples indicate that the bedforms take the orientation that for the given pair of flow vectors has the 'maximum gross bedform-normal transport'

## Bibliography


1. Day, J. W. (2007). *Restoration of the Mississippi Delta: Lessons from Hurricanes Katrina and Rita.* Science 315.
2. Dean, C. (1999). *Against the Tide: The Battle for America's Beaches.* New York: Columbia University Press.
3. Engineers, USA, (2010). *Coastal and hydraulics lab*. Retrieved 2010, from http://chl.erdc.usace.army.mil
4. Kirk, R. M. (1991). River-beach interaction on mixed sand and gravel coasts: a. *Applied Geography* .
5. Shaghude, V. (2004). Shore morphology and sediment characteristics south of pangani river, coastal tanzania. *Western indian ocean J Mar Sci Vol3* , 93-104.
6. Survey, U. G. (2010). *usgs*. Retrieved 2010, from http://www.usgs.gov/
7. W.B. Wilkinson", ". G. (1997). Rivers and coastal research in the Land. *The science of the total environment* , 5-14.

# NON-LINEAR STATISTICAL MODEL FOR THE DAILY STREAM FLOW PREDICTION IN THE KALU RIVER CATCHMENT IN SRI LANKA

H.A.N.S. Hewavitharana[1], K.Perera[2] and K.D.W. Nandalal[3]

[1]Postgraduate Institute of Science, University of Peradeniya, Sri Lanka.
(hewavitharana.09@gamil.com)

[2] Department of Engineering Mathematics, University of Peradeniya, Sri Lanka.
(kanthip@pdn.ac.lk)

[3]Department of Civil Engineering, University of Peradeniya, Sri Lanka.
(kdwn@pdn.ac.lk)

**Abstract**
Having a long record of stream flow is very valuable in planning water resources development projects. However, in many occasions, stream flow records are available for very short periods though very long rainfall records are available. Therefore, possibility to relate rainfall over a catchment to the stream flow at its outlet will enable having a long record of stream flow. Besides prediction of stream flow using already available predicted rainfall will permit taking precautionary measures in water related disaster situations such as floods and droughts. This paper presents a research carried out to find a model to predict daily stream flow of Kalu River at Ratnapura. The model, a non-linear regression model based on Marquardt's procedure, was developed using measured daily stream flow at Kalu River at Ratnapura and daily rainfall at eight rainfall gauging stations within the catchment above Ratnapura. Data for the period 1987-1994 were used for the calibration of the model while data for the period 1995-2000 were used for verifying it. The model was validated using Nush-Sutcliffe efficiency and pseudo $R^2$. Nush-Sutcliffe efficiency (78%) and pseudo $R^2$ (85%) show the possibility of the fitted model in predicting daily stream flow of Kalu River at Ratnapura.

Keywords: Marquardt's procedure, Non-linear regression, Nash-Sutcliffe efficiency, pseudo $R^2$

## 1. Introduction

The determination of the amount of water that would be carried by stream in the future is of crucial importance since it directly affects the design and operation of many water resource structures. Learning the hydrological behavior of a water structure is the first step of design. Because of this reason stream flow data are very important for many areas of water engineering such as dam planning, flood mitigation, operation of water reservoir, distribution of drinking water and drainage water, hydropower generation in dry periods and planning of river transport.

Kalu River is the second largest river in Sri Lanka. It drains an area about 2690 $km^2$. Magnitude of the annual flow is over 7300 million $m^3$. Rainfall occurs in Sri Lanka during the North-east, South-west, First inter-monsoon and Second inter-monsoons. During the Southwest monsoon rainfall is mainly confined to the Southwest of the Island, whereas during the Northeast monsoon rainfall occurs in the North and East of the Island. Kalu River catchment receives rain during both of these monsoons. The average annual rainfall of the overall catchment is around 4000 mm and it ranges from 2750 mm in coastal areas to 5000 mm in mountainous areas. Since the catchment is entirely situated in wet zone, it has a high rainfall to runoff response.

The objective of this research is to fit a nonlinear statistical model for average daily streamflow prediction in Kalu River upper cachtment using average daily rainfall. The Theory of regression and correlation is a classical frame work to describe relation between two or more variables. Rainfall-stream flow relationship is a very typical example of the application area of the regression analysis in the science of hydrology in the generation of stream flows using rainfall. This study uses a totally different method, the non-linear regression analysis using marquardt's procedure, to develop a model to compute stream flow. The model is estimated directly using input output data. The resulting model, which was developed with the application of marquardt's procedure in non linear regression is an algorithm for least square estimation of nonlinear parameters.

## 2. Methodology

Daily stream flow prediction has been done by the several scholars in different locations all over the world. Tabrizi et al (2005) found modeling of Non-linear stochastic systems using effective rainfall by considering non-linear autoregressive moving average with exogenous input. Kluppelberg et al (2008) used electricity spot price modeling with a view towards extreme spike risk by non-linear regression analysis. Farahmand et al (2006) studied daily stream flow prediction using impulse response approach. Amisigo et al (2006) predicted monthly stream flow by considering Single-Input-Single-Output (SISO) non-linear system identification techniques. They employed it to model monthly catchment runoff at selected gauging sites in the Volta Basin of West Africa.

### *2.1 Study area and data used*

Ratnapura is located in the south-western part of Sri Lanka within the wet zone. Ratnapura is the main city of Sabaragamuwa Province and it is located in a valley, which is approximately 21 m above mean sea level and it is surrounded by mountain ranges.

Daily average rainfall and daily average stream flow are required as input to the model. Data for the period 1987-2000 at the stations given in Table 1 were used in this study.

Table 1. Rainfall and stream flow gauging stations

| | Gauging Station | Data Period | Latitude (N) | Longitude(E) |
|---|---|---|---|---|
| Rainfall (mm) | Ratnapura | 1987-2000 | 6.68 | 80.40 |
| | Pinnawala | 1987-2000 | 6.73 | 80.86 |
| | Kuruwita | 1985-2000 | 6.80 | 80.38 |
| | Lellopitiya Estate | 1987-2000 | 6.68 | 80.50 |
| | Alupola Group | 1987-2000 | 6.72 | 80.58 |
| | Balangoda (Post office) | 1987-2000 | 6.65 | 80.70 |
| | Wellandura Estate | 1987-2000 | 6.53 | 80.57 |
| | Hapugastanna | 1987-2000 | 6.72 | 80.52 |
| Steam flow ($m^3/s$) | Ratnapura | 1987-2000 | 6.68 | 80.40 |

### *2.2 Thiessen polygon method*

Thiessen polygon network was drawn by connecting eight rainfall stations and drawing perpendicular lines through mid points of each line segment. The method assumes that the area covered by a polygon gets an average rainfall similar to the rainfall received at the station inside the polygon. Average rainfall for the catchment was calculated as weighted average rainfall using following equation.

$$\overline{R} = \frac{(\sum_{i=1}^{n} R_i A_i)}{A}$$

Where, $Ri$ is the daily average rainfall of the i$^{th}$ station in mm

$\overline{R}$ is the average daily rainfall over the catchment in mm

$Ai$ is the area covering i$^{th}$ station in km$^2$

$A$ is the total area of the catchment in km$^2$

### *2.3 Daily average rainfall and daily average stream flow modeling*

Many daily average rainfall and daily average stream flow were studied using various approaches available for modeling.

**Non-Linear regression model**

General form of the non-linear regression model is Farahmand et al (2006), Neter,4th editon:

$$y_i = f(x_i, \gamma) + \epsilon_i$$

Where;

$$x_i = \begin{pmatrix} x_{i1} \\ .. \\ x_{iq} \end{pmatrix} \qquad \gamma = \begin{pmatrix} \gamma_0 \\ .. \\ \gamma_p \end{pmatrix} \qquad \varepsilon_i = \text{Random error} \qquad Y_i = \text{Response variable}$$

### *2.4 Estimation of regression parameters*

Data from 1987 to 1995 have used to estimate the model parameters of the average daily stream flow model and 1996 to 2000 data have used to check the validation of the predicted model.

In many nonlinear regression problems, it is more practical to find the least estimates by direct numerical search procedures. Direct numerical search procedures are Gauss-Newton, steepest descent and Marquardt's procedures.

**Marquardt's procedure**

The Marquardt procedure is an iterative procedure. To start a minimization, the user has to provide an initial guess for the parameter vector. This procedure seeks to utilize the best features of the Gauss-Newton and steepest descent procedures and occupies a middle ground between these two procedures.

In one hand uninformed initial values will work on the other hand the procedure converge only initial guess close to the final solution. In each step the parameter vector is replaced by a new estimator. If reduction of sum of square is rapid, a smaller value can be used for the iterative procedure. The reduction of the sum of squares from the least parameter vector, then iteration stops and the last parameter vector is to be the solution Neter et al 4th edition.

### *2.5 Model validation*

**Nash-Sutcliffe efficiency**

The efficiency provide by the Nash and Sutcliffe et al (2005) is defined the one minus sum of the absolute squared difference between the predicted and observed values normalized by the variance.

$$E = 1 - \frac{\sum_{i=1}^{n}(Oi - Pi)^2}{\sum_{i=1}^{n}\left(Oi - \bar{O}\right)^2}$$

Where,
$E$ = the Nash-Sutcliffe efficiency.
Oi =Observed value
Pi =Predicted value
$\bar{O}$ =Average value of the observed value

The normalization of the variance of the observation series results in relatively higher value of $E$ in catchment with high dynamics. The range of Nash-Sutcliffe efficiency lies between 1 and –infinity. If the value is closer to 1 model, it would make perfect fit model.

**Pseudo-$R^2$**

In nonlinear regression, such a measure is not readily defined. One of the problems with the $R^2$ definition is that it requires the presence of an intercept, which most nonlinear models do not have. A measure relatively closely corresponding to $R^2$ in the nonlinear case is:

$$Pseudo - R^2 = 1 - \left( SSE / SST_{Corrected} \right)$$

where;

SSE= Sum of square error
SST corrected= Total sum of square corrected

## 3. Analysis of the system

### 3.1 System description

The system under investigation consists of eight rainfall stations and one stream flow station. Eight rainfall stations are Allupola group, Balangoda, Kuruwita, Pinnawala, Rathnapura, Lellopitiya Estate, Wellandura Estate, Hpugastanna and steam flow station is Ratnapura.

### 3.2 Data analysis

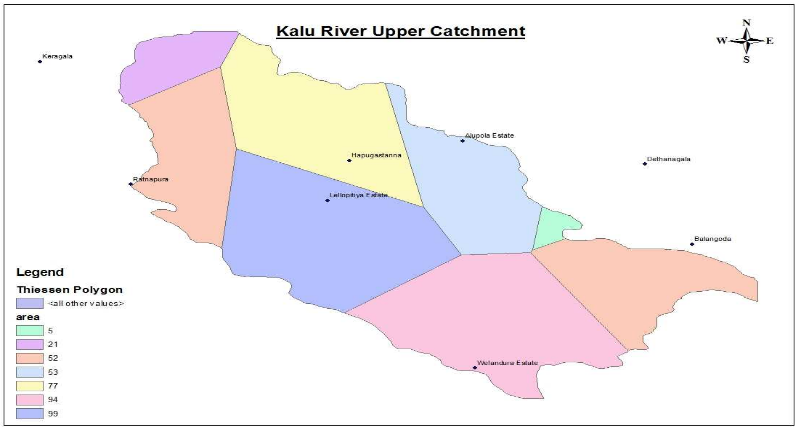
The data analysis began by investigating the characteristics of the data. The procedure included analyzing the statistical properties of data, normality of data, mean analysis, non stationary analysis and Thiessen polygon method to calculate average rainfall over the catchment.

The above statistical analysis suggested that the rainfall data set is not well correlated and that there may be significant non stationarity in the data. Therefore, it is essential to pre-process by shifting the data, removing the mean, transformation or use of proper rainfall filter. Logarithmic transformation is used to minimize the variation of the average daily stream flow.

The data during the period from 1987 to 1994 were used in the calibration of the model while data from 1995 to 2000 were used to validate the model.

## 4. Results and discussion

Daily rainfall of eight rainfall stations were used to calculate average daily rainfall over the catchment based on the Thiessen polygon. The Thiessen polygon network and the polygon areas are given in Figure 1.




| Rainfall Station | Area(km$^2$) |
|---|---|
| Ratnapura | 51.55 |
| Pinnawala | 4.63 |
| Kuruwita | 21.21 |
| Hapugastanna | 76.76 |
| Lellopitiya | 99.45 |
| Alupola | 53.48 |
| Balangoda | 51.77 |
| Wellandura | 94.00 |

Figure 2. Thiessen polygon Network

The modeling of average daily rainfall and average daily stream flow processes is a major task for hydrologist who requires such model applications such as flood forecasting, waste water flow rate forecasting and water table evaluation and such models received considerable attention in the hydrological literature. However, the average daily rainfall and average stream flow are clearly dynamic with non-linear characteristics.

Since the antecedent average daily rainfall condition clearly affects the subsequent flow behavior, the physical process should be non-linear. If the prior average daily rainfall has been sufficient to thoroughly wet soil in the catchment area then the average daily stream flow in the river will be significantly higher than the condition when the soil had been dried out due to the lack of rainfall.

Since there is seasonal variation, two cosine terms with time trend were used to catch the seasonality of the data set. Also lag 1, lag 2, lag3 of average daily rainfall ($r_{t-1}$, $r_{t-2}$ and $r_{t-3}$ ), and lag 1 of logarithmic transformation of average daily stream flow ($z_{t-1}$) were used.

$$Z(t) = 0.0239*\cos(3*(22/7)*t/950) + 0.0239*\cos(2*(22/7)*t/3) + 0.9316*Z(t-1)$$
$$+ 0.00704*r(t-1) - 0.00093*r(t-2) - 0.00279*r(t-3)$$

The number of previous days average daily rainfall and logarithmic transformation of average daily stream flow were decided based on the improvement on the model statistics. The predicted and the observed average daily stream flow of the Kalu River at Ratnapura for the calibration period is given in Figure 2.

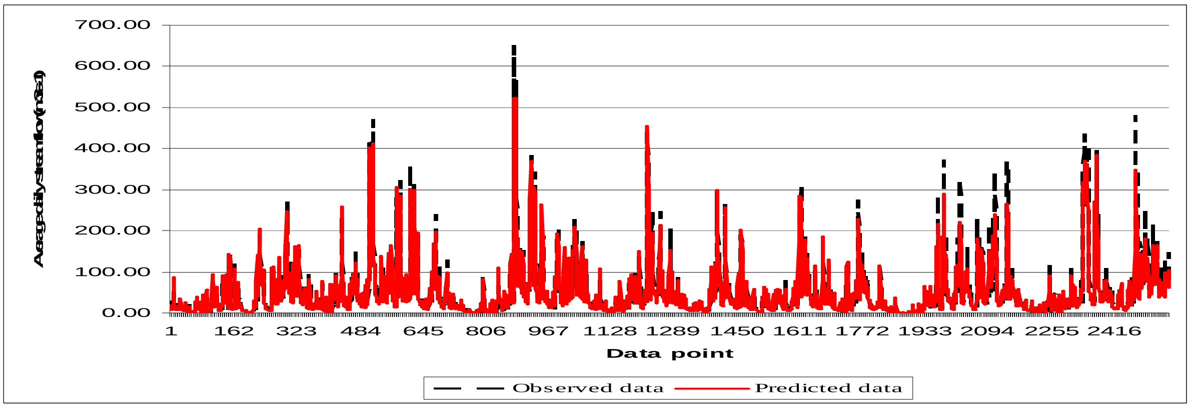



Figure 2. Plot of observed and predicted stream flow for the calibration period

**Testing of Hypothesis**

According to the analysis of variance (Table 2), p-value less than 5% significant level it can be conclude that the model is significant. Table 4 presents the significance of parameters.

Table 2. Analysis of variance

| Source of Variation | DF | Sum of Square | Mean Square | F-value | P-value |
|---|---|---|---|---|---|
| Regression | 6 | 7567.7 | 1261.3 | 4770.05 | <.0001 |
| Error | 2547 | 155.5 | 0.0454 | | |
| Uncorrected total | 2553 | 7683.2 | | | |
| Corrected total | 2552 | 1197.4 | | | |

Table 3. Significance of the parameters

| Parameters | Estimates | Approximate Standard error | Approximate 95% confidence interval | |
|---|---|---|---|---|
| b0 | 0.02390 | 0.00534 | 0.01340 | 0.0344 |
| b1 | 0.9316 | 0.00516 | 0.9215 | 0.9417 |
| b2 | -0.00093 | 0.00036 | -0.00164 | -0.00023 |
| b3 | 0.00704 | 0.00035 | 0.00635 | 0.00773 |
| b4 | 0.00407 | 0.00032 | 0.00344 | 0.00407 |
| b5 | -0.00279 | 0.00034 | -0.00346 | -0.00213 |

According to the Table 3, seven parameters do not have confidence interval including zero. Therefore, all parameters are significant.

**Model validation**

Table 4 shows test statistics for the calibration period and validation period of the model.

Table 4. Measure of predictability of the model

| | Calibration | Validation |
|---|---|---|
| $R^2$-Pseudo | 0.90 | 0.90 |
| Nash-Sutcliffe efficiency | 0.95 | 0.84 |
| Press statistic | 40.68 | 49.89 |
| Root MSE | 0.21 | 0.35 |

Figure 3 is the plot of daily average stream flow for the validation period of 1996-2000. It shows that the deviations of the two curves are not significant and the distribution patterns of the two data sets are almost similar. Figure 4 shows that the plot of monthly total of the stream flows for the validation period. Both observed data set and predicted data set follow same distribution.

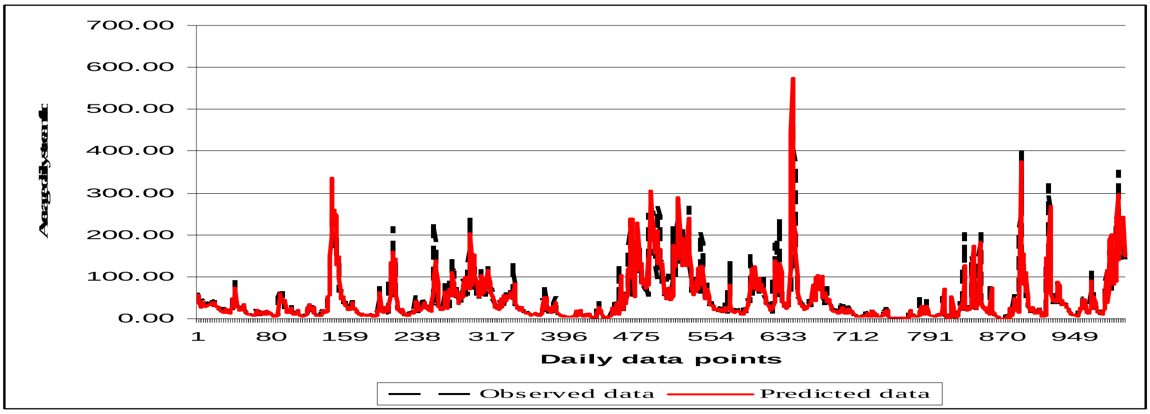


Figure 3. Average daily stream flow for the validation period

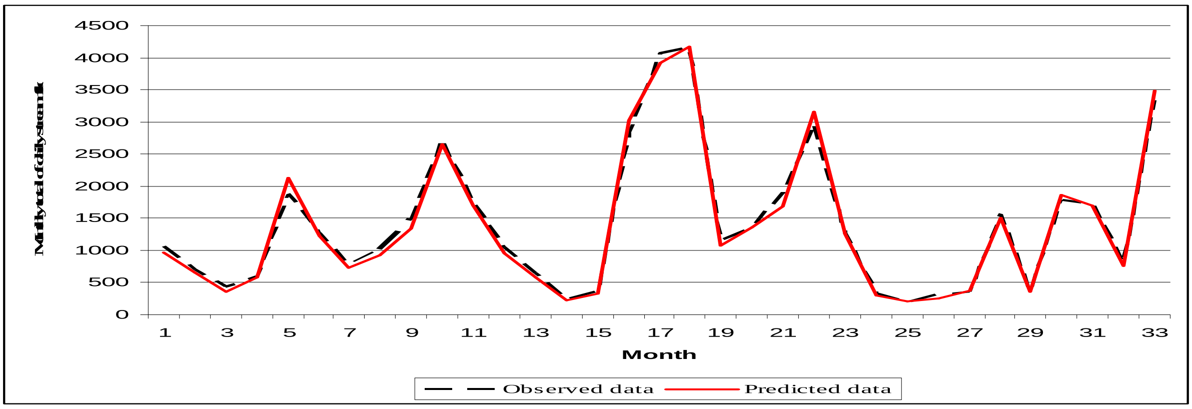



Figure 4. Monthly total of the stream flow for the validation period

## 5. Conclusion

Model has been estimated using marquardt's procedure. The comparison of the model predicted output with the actual output over both the estimation and test data set were used to measure the model performance.

However, the model validity test suggested that there were missing model terms. This may be due to the difficulty involved in fitting models to rainfall and stream flow data which are affected by many unknown factors, such as soil moisture, temperature, etc.

Finally, non-linear regression model was developed by using marquardt's procedure for the Kalu River upper catchment outlet for the calibration period 1987-1995. Since the model is significant according to the p-value, the fitted model can be suggested as suitable for the generation of daily average stream flow.

## 6. References

1. Tabrizi, H.N., Mohammad, J. and A.S. Billing (2004) Modeling of Non-linear stochastic systems.
2. Kluppelberg, C., Brandis, T.M. and A. Schmidt (2008) Electricity spot price modeling with a view towards extreme spike risk.
3. Amisigo, B.A., Rogers, C., Andah, W.E.I. and J. Friesen (2006) Monthly Streamflow Prediction in the Volta Basin of West Africa: A SISO NARMAX Polynomial Modelling
4. P.Krause, D.P. Boyle, F. Base (2005), "Comparison of different efficiency criteria for hydrological model assessment".
5. Gordon K. Smyth Edited by Abdel H. El-Shaarawi and Walter W. Piegorsch(2002),"Nonlinear regression", Volume 3.
6. Prajneshu, Indian Agriculture Statistical Research Institute, Library Avenue, New Dilhi, "Non-linear regression models and their application".
7. Neter, Katner, Nachtsheim, Wasserman, "Applied Linear Statistical Model", 4th Edition, 531-559.
8. Farahmand, T., Fleming, S.W., and E.J. Quilty(2006) Detection and visualization of storm hydrograph changes under urbanization: An impulse response approach

# FIELD MAPPING OF WETTING FRONT USING GROUND PENETRATING RADAR UNDER UNIFORM AND NON-UNIFORM WETTING

D.M.C.S. Mimrose[1], L.W. Galagedara[2*], and G.W. Parkin[3]

[1]Postgraduate Institute of Agriculture, University of Peradeniya, Peradeniya, Sri Lanka
mimrose124@gmail.com

[2]Department of Agricultural Engineering, University of Peradeniya, Peradeniya, Sri Lanka
lgalaged@pdn.ac.lk

[3]School of Environmental Sciences, University of Guelph, Guelph, ON, Canada
gparkin@uoguelph.ca

**Abstract:** This research was carried out to develop 2D and 3D maps of a wetting front and to identify potential preferential flow areas using Ground Penetrating Radar (GPR). GPR grid data were collected during uniform and non-uniform wetting experiments. Maps were prepared for different depth profiles for each data set, collected at different time intervals after starting water application. The wetting front had reached a maximum depth of 0.45–0.50 m within 25 hours of continuous wetting based on 2D and 3D GPR images. In the uniform wetting experiments, potential preferential flow zones could be identified in 2D and 3D maps.

*Key Words: Ground penetrating radar, wetting front, preferential flow, water content*

## 1. INTRODUCTION

A wetting front can be defined as a thin transition zone where the soil water content changes from its initial low value to a higher value at the leading edge of an infiltration event. Usually the process of water infiltration in soil from a point source creates an onion-shaped wetting front that slowly propagates vertically and horizontally (Gvirtzman *et al*., 2008). Instability in the wetting front may lead to the creation of preferential flow pathways (Raats, 1973). Preferential flow refers to the uneven and often rapid movement of water and solute through porous media, while matrix flow is a relatively slow and even movement of water and solute through soil (Singh, 1995). Once preferential flow paths have formed, the soil no longer impedes infiltration of water; additional precipitation tends to infiltrate through the pre-existing preferential paths, which have been wetted before (Dekker *et al.*, 2001). Thus, dry zones tend to persist due to their water repellent character and their low hydraulic conductivity. Research conducted by Mowjood *et al.* (2005) using crack measurements and water advance front sensors shows that water can move rapidly through a subsurface crack network, with the cracks acting as preferential flow pathways. Most models which are used to simulate water and solute transport through the unsaturated zone assume uniform downward movement of the wetting front parallel to the soil surface during an infiltration event. But this assumption is not valid for most of the cases described above when there is preferential flow.

Preferential flow is often considered to facilitate groundwater contamination. Identifying and mapping of preferential flow areas can help in minimizing groundwater contamination by adopting appropriate soil and water management strategies in the identified areas. t is necessary to understand how preferential flow pathways develop in the soil subsurface both vertically and horizontally (Galagedara, 2003). Densely sampled soil moisture data required for mapping cannot be obtained through conventional methods such as gravimetric, time domain reflectometry (TDR), capacitance-based sensors or neutron scattering. On the other hand, the GPR method has been identified as an efficient method to measure soil moisture variability over large areas (Grote *et al*., 2003; Hubbard *et al*., 2002; Huisman *et al*., 2001) with larger sampling volume, non-intrusive sampling and less time consumption than other methods. In addition, researchers have used GPR to characterize hydrological processes in the vadose zone including mapping of wetting front movements and identify potential preferential flow pathways (Daniels *et al*., 1994; Galagedara *et al*., 2005; Gish *et al*., 2002; Rucker and Ferre, 2002; Saintenoy *et al*., 2007; Vellidis *et al*., 1990).

The main objective of this research was to develop 2D and 3D maps of a wetting front during infiltration from a combined line and point source and identify the potential preferential flow zones within the wetted soil under field conditions using GPR. A second objective was to compare wetting front configurations for two different methods of applying water to the soil surface. As explained below, 2D and 3D maps of soil water distribution were prepared using GPR data collected in a grid of 2.0 m x 2.5 m with two different methods of water application with different water application rates and durations. Finally, to assess the importance of characterizing the 2D and 3D nature of the infiltration, the patterns of wetting front advance measured by GPR were compared against estimates of the advance of the wetting front for similar experimental conditions using a 1D infiltration model (HYDRUS 1D).

## 2. MATERIALS AND METHODS

### 2.2 The GPR Method

Researchers have put forth GPR as a sensitive shallow earth mapping technique that can be potentially used to identify paths for preferential flow by monitoring soil water movement in the vadose zone (Daniels *et al*., 1994; Freeland, 2006; Kishel and Gerla, 2002; Vellidis *et al*., 1990). In a GPR survey conducted in a uniformly wetted land by Vellidis *et al*. (1990), it was evident that the shapes of the wetting front and water application uniformity curves are mirror images of each other. In other studies, high water content zones (possibly preferential flow zones) were identified by a borehole GPR survey conducted during wetting and drying conditions in a well drained sandy loam soil (Galagedara *et al*., 2003; Parkin *et al.*, 2000). Research conducted by Saintenoy *et al*. (2007) showed that surface-based GPR data provide valuable information to study the evolution of a water bulb with hydrodynamic modeling. Research done in sandy soils using GPR have visualized discrete wetting front and preferential flow paths (Harari, 1996). Simulated wetting fronts were observed in sand tanks in the laboratory using GPR (Seung-Yeup *et al*., 2007). For this study, the PulseEKKO PRO GPR system was used and data collection procedure, survey method, equipment settings etc., were done based on the operation manual (Sensor and Software Inc., 2006). Site selection was done through a background survey conducted using both 100 MHz and 200 MHz GPR antennas.

## 2.2 Location

Research was carried out at the Meewathura research station of the Department of Agricultural Engineering, University of Peradeniya, Sri Lanka ($7^0$ 15' 10.97" N, $80^0$ 35' 42.57" E, and Elevation 475 m). A study area of 2.5 m x 2.0 m was selected (Fig. 1) having nearly uniform, flat ground with nearly homogeneous soils in the subsurface down to about 1.5 m depth.

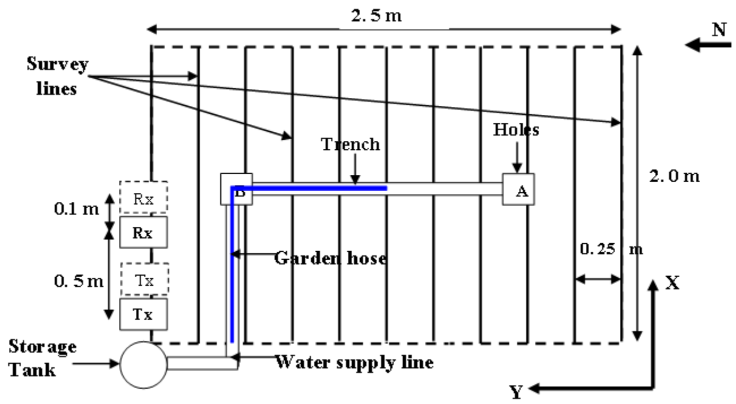



Fig. 1: Schematic diagram of the experimental area showing water supply system and GPR survey lines
Tx: Transmitter antenna; Rx: Receiver antenna.

Having uniform subsurface conditions is potentially important to help observe the wetting front clearly with the GPR method as the number of wave reflections of soil horizon boundaries is minimized. The major soil type in the area is sandy clay loam originating from alluvial deposits. Undisturbed soil samples were collected from the study site at three different depths with two replicates. These samples were used to estimate the soil physical properties using standard laboratory procedures. Physical properties of the soil in the study area are given in Table 1.

**Table 1: Measured soil physical and hydraulic properties of the field site**

| Soil Type and Depth (cm) | Sand (%) | Silt (%) | Clay (%) | OM† (%) | Porosity (%) | BD‡ ($g/cm^3$) | Ks§ (m/sec) |
|---|---|---|---|---|---|---|---|
| 0-15 (SCL) # | 59.4 | 8.6 | 32.0 | 3.14 | 43.2 | 1.54 | 3.5E-07 |
| 15-30 (SC) # | 60.0 | 5.0 | 35.0 | n/a | 46.3 | 1.49 | 1.2E-05 |
| 30-40 (SC) # | 47.0 | 16.0 | 37.0 | 1.54 | 45.3 | 1.45 | 1.1E-06 |

†Organic Matter; ‡Bulk Density; §Saturated Hydraulic Conductivity; ¶Sandy Clay Loam; #Sandy Clay

### 2.3 Experiment 1

Water application to the study area was carried out from two square-shaped of 15-cm in size and 20-cm in depth holes (A and B) and a connecting trench of 5-cm width and 10-cm depth (Fig. 1). A garden hose (1.27 cm diameter) was laid over half of the trench as a water source for the trench and holes A and B. (Fig. 1). Water discharged from the end of the garden hose to fill the trench and holes and expected that the holes would generate preferential flow beneath them. The trench and the two holes (A and B) were back filled with stone chips (5-10 mm diameter) in order to obtain good ground coupling of GPR antennas. Fresh water was applied at an average rate of 4.6 mL/sec for 21.50 h of total time duration. Water application rate was obtained by measuring the volume of water collected at a known time period. A tightly-controlled water application rate was not possible since the water supply hose was directly connected to a garden tap and water pressure was varying during the day.

### 2.4 Experiment 2

In this case, a perforated PVC pipe of 1.27 cm diameter and 1.5 m length was placed in the trench to supply water more uniformly than the garden hose. Two sets of holes were made on both sides along the horizontal axis of the pipe. Holes were placed keeping a constant interval distance of 5.0 cm to ensure uniform water supply along the trench. Under experiment 1, uniform water application was not possible because water was applied from one location of the trench. A PVC pipe having the same diameter as the perforated pipe was buried in the soil at 10-cm depth to supply water from the tank to the perforated pipe. The trench and two holes (A and B) were filled with coarse sand after placing the pipe. Water was applied at a rate of 24 mL/sec for 25 h of total time duration. A much more uniform water application rate could be obtained during this experiment compared to experiment 1 by having a constant water head tank (Fig. 1) as the source of water instead of the tap.

In both experiments, GPR grid surveys were carried out using 200-MHz antenna employing the reflection survey method (Annan, 2005; Smith *et al*., 1992; Hunaidi *et al*., 1998). GPR surveys were carried out having 0.25-m line spacing, 0.5-m antenna separation and 0.1-m step size as shown in Fig. 1. GPR grid survey lines (eleven lines) were oriented from West to East (X direction) and each survey was carried out beginning from the survey line in the South to North (Y direction) as shown in Fig. 1. Data collection began 30 min. after starting water application in both experiments. Four data sets were collected during experiment 1, while six data sets were collected during experiment 2. Background surveys were carried out before starting the water application in both experiments in order to assess the effect of water application and changes in the wetting patterns with depth. EKKO Delux, EKKO View and EKKO Mapper (Sensors and Software Inc.) and Voxler 3D (Golden Software Inc) computer softwares were used to develop and interpret 2D and 3D maps of the subsurface soil water content distribution collected at the field.

### 2.5 Simulation of wetting front advance using HYDRUS 1D

Wetting front advancement was simulated using HYDRUS 1D (Šimůnek *et al*., 2005) for a three layer (Table 1) soil profile of 120-cm thickness. During this simulation, data from Table 1 were used for three different soil layers. The thickness of the third layer was considered from 30 to 120 cm since soil properties below 40 cm depth was not obtained. During the simulation, water flow was considered as one dimensional (vertical). Calculations were done for 25 h of total time period starting from 0.5 h. Changes in wetting front depth in the profile were simulated 9 times at different elapsed times from starting of water application. Soil textural values and bulk density values for three different layers were used to predict soil hydraulic parameters in HYDRUS 1D (Table 1). Van-Genuchten- Mualem model was used in parameter estimation for unsaturated water flow simulation. Simulation was done using two upper boundary condition values. For the first run, the initial upper boundary conditions (matrix potential) were set as 0 kPa for 0-1.0 cm layer and -1000 kPa for 1.0-120.0 cm layer assuming the saturation condition at the trench. For the second run, the initial upper boundary conditions were set as -10 kPa for 0-1.0 cm layer and -1000 kPa for 1.0-120.0 cm layer assuming the near saturation condition at the trench. This second run was done because the real saturation condition is not generally achieved during field conditions. And also, due to comparatively lower hydraulic conductivity of the first layer, the water application rate had to be kept at a lower rate in order to avoid flooding conditions which can affect GPR antennas. So that, we assumed the second run was more closely representing the actual field situation. As for both runs, the lower boundary condition was kept as free drainage.

## 3. RESULTS AND DISCUSSIONS

### 3.1 Experiment 1

Two-dimensional images showing relative strength of reflected signal at different soil depths at different time intervals beneath the grid area were prepared using EKKO Mapper (SSI) software. The velocity of the radar wave could not be estimated accurately due to difficulty in separating the direct ground wave (DGW) from other waves collected using common mid-point (CMP) survey data, during this experiment. The average radar wave velocity of 0.10 m/ns was assumed in developing these wetting from maps. The disturbances masking the DGW could potentially be due to environmental noise from nearby power lines and or metals presence in the vicinity. Fig. 2 shows variation of reflected wave strengths at different times at 0.20 – 0.30 m and 0.25 – 0.30 m depths. The expected preferential flow areas in holes A and B could not be observed, potentially due to uneven water application to the trench. However, according to Fig. 2d, three preferential wettings could be observed. The most dominant wetting can be seen at 1.25 mN line (just south of B hole) which is just below the water application point (Fig. 1). It is clear that the wetting front had reached at 0.25 – 0.30 m depth after 7 hours of wetting (Fig. 2e) and the signal strength increases after 7 hours of wetting (Fig. 2d) due to increase of moisture content with continued infiltration. However, as shown in Fig. 2b, the wetting is more dominant north of hole B after 40 min of wetting compared to other areas including the infiltrating point.

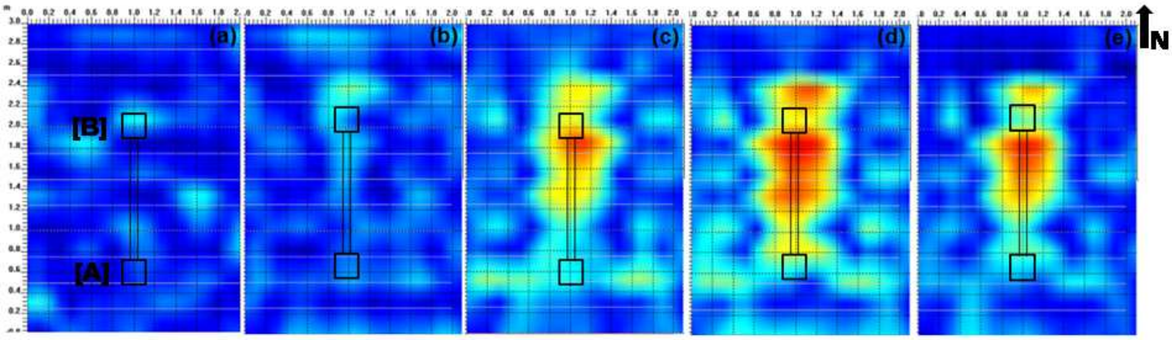



Fig. 2: Maps of GPR reflected signal strength showing changes in wetting pattern in 0.20 - 0.30 m depth slice under experiment 1 conditions. (a): background; (b) after 40 min; (c) after 5 h; (d) after 7 h; (e) 0.25 – 0.30 m depth slice after 7 h.
(Strength of the reflected signal increases from blue→green→yellow→orange→red).

According to Fig. 3, the wetting pattern has developed horizontally (horizontal seepage) as well, when compared with the same depth slice (Figs. 3b and 3c). However, the horizontal seepage is less in Fig. 3d compared to Fig. 3c. This variation could be due to the variability of wetting due to the effect of gravity and negative pressure potential. When the soil is dry, negative pressure dominates the water flow where wetting font advancement can be expected both horizontally and vertically. With continued wetting negative pressure decreases and gravity dominates in wetting where vertical movement of water is prominent at the given depth compared to horizontal movement. In addition, water application method and rate directly connecting to a garden tap during the experiment 1 could also have created this variation. Overall, it is clear that both vertical and horizontal wetting had occurred during this experiment and sharp wetting fronts become less well defined with time when comparing Figs. 3d and 3e.

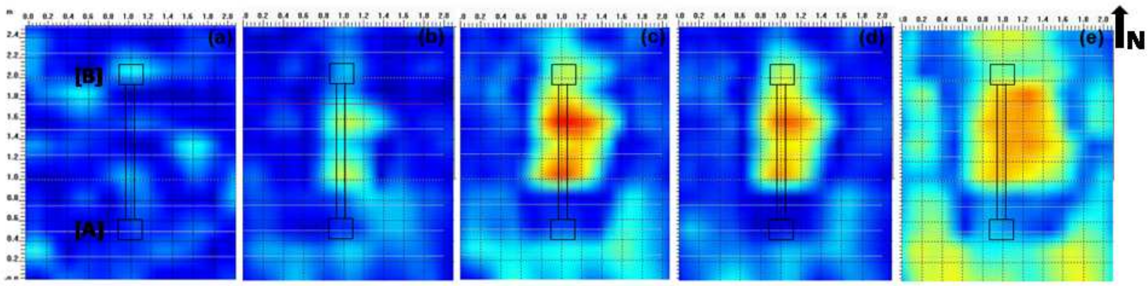



Fig. 3: Maps of GPR reflected signal strength showing changes in wetting pattern in 0.20- 0.30 m depth slice under experiment 1 conditions. (a): background; (b) after 30 min; (c) after 1 h 30 min; (d) after 2 h 30 min (e) after 21 h.
(Strength of the reflected signal increases from blue→green→yellow→orange→red).

In the 3D images shown in Fig. 4, the middle vertical plane is located at the same location as the water application trench in the Y direction of the grid (Fig. 1). Changes in wetting pattern can be observed in the middle vertical plane with time in comparison with the 3D image produced for the background survey (Fig. 4). Two prominent wetted areas can be observed in the 3D image after 30 min. representing the two potential preferential flow areas found in 2D images at the 1.0 and 1.6 m N lines (Fig. 3c). After 21.0 h, applied water has concentrated in a depth range about 0.20 – 0.30 m, and has not reached a depth below 0.50 m. Accumulation of water in 0.20 – 0.30 m soil profile was observed in 2D images as well (Figs. 2d, 3c and 2d).

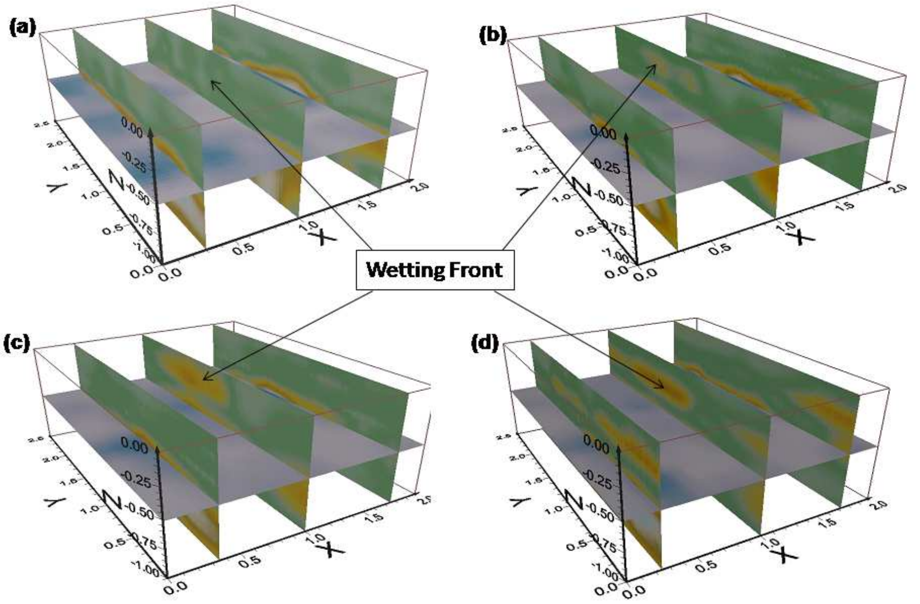



Fig. 4: 3D images of the GPR reflected signal strength beneath the study area at the indicated elapsed times after starting water application under experiment 1 condition. Three vertical slices are at 0.5, 1.0 and 1.5 m North lines where the water was applied at 1.0 m North line (Arrows show the development of the wetting front with time at the centre line). (a) background; (b) after 30 min; (d) after 1 h and 30 min; (d) after 21 hr.

### 3.2 Experiment 2

In Fig. 5, nearly uniform wetting could be observed along the trench when compared with images produced in the previous experiment. A stronger reflected signal indicating the wetting pattern with red color is visible than in figures obtained in the previous experiment. It is probably due to the increased volume of applied water. A preferential flow area is visible near the North end of the trench (Fig. 5). This clear preferential flow area is prominent at the early stage of water application and reached down to about 0.35 - 0.40 m depth within 30 min. (Fig. 5c). The wetting pattern could be clearly observed up to the 0.45 - 0.50 m depth slice with 0.100 m/ns velocity.

The wetting pattern is prominent in the middle area of the trench in experiment 1 as water was applied from the middle area. In experiment 2, the wetting pattern has spread throughout the whole length of the trench. The maximum depth of water infiltration of 0.45 - 0.50 m was achieved within 1 h at a water application rate of 24 mL/sec with an assumed velocity of 0.100 m/ns. Actual velocity could not be estimated due to difficulty of producing a velocity profile for the study area using the DGW method. This depth may slightly change with the actual wave velocity.

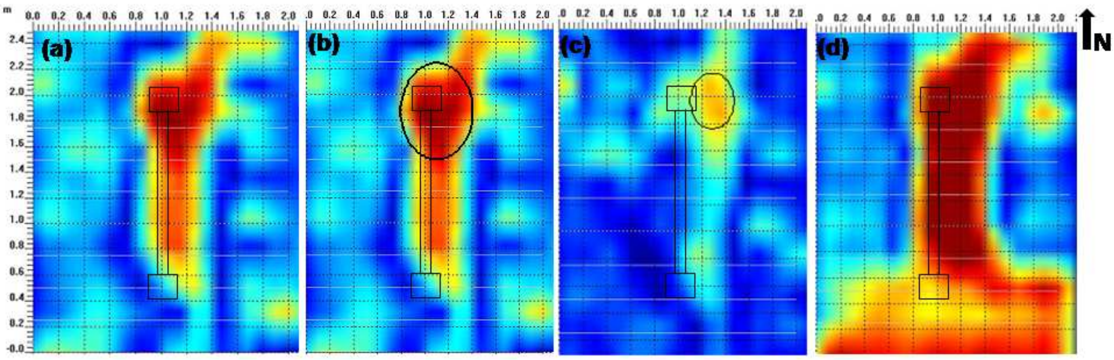



Fig. 5: Maps of GPR reflected signal strength showing changes of wetting pattern and potential preferential flow areas under experiment 2 conditions. (a) after 30 min at 0.20-0.30 m depth; (b) after 30 min at 0.30-0.40 m depth; (c) after 30 min at 0.35-0.40 m depth; (d) after 2 h at 0.20-0.30 m depth.
(Strength of the reflected signal increases from blue→green→yellow→orange→red)

### 3.3 Simulated depths of wetting front using HYDRUS 1D

According to derived hydraulic properties during the HYDRUS 1D simulation, the third layer has the highest saturated soil water content (porosity) and the lowest saturated hydraulic conductivity (Ks). However, according to the measured values, the highest Ks value is found at the middle layer followed by the lower layer and upper layers, respectively (Table 1). As for the saturated water content values (porosity), all the values derived by HYDRUS 1D were lower compared to measured values. The effect of different hydraulic properties at different layers on the simulated wetting front can be seen in Fig. 6. It is clear that none of the layers reached the saturated water content under the second run potentially due to the lower boundary of free drainage and lower water application rate. However, it can be seen in Fig. 6b that the first layer water content had reached to steady state water content of 0.38-0.39 $m^3/m^3$ of soil (the saturated water content is 0.406 $m^3/m^3$ of soil).

Applying the HYDRUS 1D model to the experiment 2 conditions, the wetting front has reached up to about 35 cm depth in 10 h and about 80 cm depth in 25 h for the first model run (Fig. 6a). The wetting front has reached up to about 15 cm depth in 10 h and about 30 cm depth in 25 h for the second model run (Fig. 6b). When comparing the results of the first model run with wetting front depth observed using GPR,

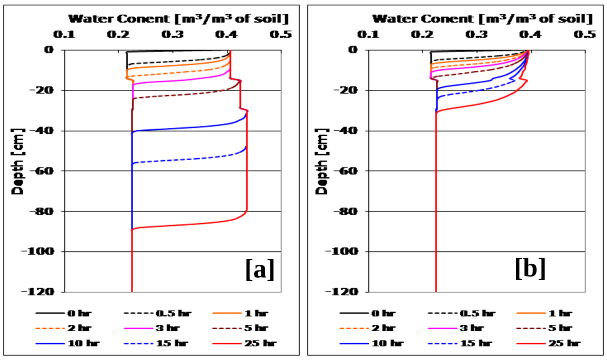



Fig. 6: Simulated depths of wetting front using Hydrus 1D under conditions of experiment 2.
(a): Constant pressure of the upper boundary = 0 kPa
(b): Constant pressure of the upper boundary = -10

It is clear that the first model run resulted in an overestimation of the wetting front advancement. However, the second model run gives comparatively better prediction of the wetting front advancement with the GPR method. In the second experiment, the average wetting front obtained from the GPR method is around 0.25-0.30 m even though the maximum depth observed was 0.40 m (Fig. 5c). This maximum depth is reached after 25 hours of wetting potentially due to preferential flow and the average depth of wetting along the entire trench could be considerably lower. Variation in physical properties along the soil profile govern actual wetting front advancement while the assumed homogeneous properties along the trench length in the simulation might have affected the slight differences in the simulated wetting front advancement and maximum wetting depth observed in the GPR method. It can be revealed that the GPR observed average wetting depth could also be simulated using HYDRUS 1D. However, slight differences between the observed wetting front and simulated wetting front could be due to the differences in soil hydraulic properties as well as variation of the GPR wave velocity with increasing water content which was not considered in preparation of wetting front advancement maps.

## 4. CONCLUSIONS

A maximum wetting front infiltration depth of 0.40 - 0.45 m was achieved at a water application rate of 24 mL/sec within 1.0 h based on the observation made by the GPR method. This depth of penetration was much greater than the depth predicted by HYDRUS 1D, demonstrating the importance of preferential flow under the site and experimental conditions of this study. Simulation exercise revealed that upper soil layer reached steady state water content within 25 hours of water application. This study has shown that the surface GPR method can be satisfactorily used to identify wetting patterns and potential preferential flow areas under field conditions. Both 2D and 3D maps of the pattern of wetting beneath an infiltration trench were successfully developed. Preferential flow zones were visible in both map configurations as zones of relatively high wave reflection. Preferential flow was observed under both experimental designs, but a uniform wetting is recommended instead of wetting from one location as used in this study. A second recommendation based on this work is that the GPR grid data should be collected at shorter time intervals during the early stage of water application to observe a clear wetting pattern. Similar study under more controlled condition can also be recommended and validation should be done using a 2D simulation (HYDRUS 2D) model to catch the horizontal variation of the soil hydraulic properties.

**ACKNOWLEDGEMENTS**

Authors wish to acknowledge the National Research Council (NRC) of Sri Lanka for financial support given to purchase the GPR equipment and conduct field research (Grant Number NRC/05-38). Dr. van Genuchten, M. Th. for providing HYDRUS 1D free of charge. Senani gave an excellent support during field works.

**REFERENCES**

Annan A.P. (2005), "GPR Methods for Hydrogeological Studies in Hydrogeophysics", Springer Netherlands.

Daniels, J.J., Robrts R., Vendi, M. (1994), "Ground penetrating radar for the detection of liquid contaminants", *Journal of Applied Geophysics,* 33: 195-207.

Dekker, L.W., Ritsema, C.J., Oostindie K. (2001), "Wetting patterns and preferential flow in soils". *In: Proc. 2nd Int. Symp on Preferential Flow, Water Movement and Chemical Transport in the Environment,* pp. 85-88 D.D. Bosch and K.W. King. St. Joseph (Eds), Michigan 3-5 January 2001, Honolulu, USA.

Freeland, R.S., Odiambo, L.O., Tyner, J.S., Ammons, J.T., Write, W.C. (2006), "Non-intrusive mapping of near surface preferential flow", *Applied Engineering in Agriculture,* 22(2): 315- 319. American Society of Agriculture and Biological Engineers. ISSN. 0883-8542.

Galagedara, L.W. (2003), "The GPR direct ground wave method for soil moisture content estimation: Field experimental modeling (*Unpublished PhD thesis*)", University of Guelph, Canada.

Galagedara, L.W., Parkin, G.W., Redman, J.D., Endres, A.L. (2003). "Assessment of soil moisture content measured by borehole GPR under transient irrigation and drainage", *Journal of Environmental and Engineering Geophysics,* 8(2): 77-86.

Galagedara, L.W., Parkin, G.W., Redman, J.D., von Bertoldi P., Endres, A.L. (2005), "Field studies of the GPR ground wave method for estimating soil water content during irrigation and drainage", *Journal of Hydrology,* 301: 182-197.

Gish, D.W.P., Kung, K.J.S., Daughtry, C.S., Doolittle, J.A., Miller, P.T. (2002), "Evaluating use of ground penetrating radar for identifying subsurface flow pathways", *Soil Science Society of America,* 66: 1620-1629.

Grote, K., Hubbard, S., Rubin, Y. (2003), "Field-scale estimation of volumetric water content using ground-penetrating radar ground wave techniques", *Water Resources Research,* 39(11): 1321-1335.

Gvirtzman, H., Shalve, E., Dahan, O., Hazor, Y.H. (2008), "Large-scale infiltration experiments into unsaturated stratified loess sediments: monitoring and modeling", *Journal of Hydrology,* 394: 214-229.

Harari, Z. (1996), "Ground penetrating radar for imaging stratigraphic features and ground water sand dunes", *Journal of Applied Geophysics,* 36(1): 43-52.

Hubbard, S., Grote, K., Rubin, Y. (2002), "Mapping the volumetric soil water content of a California vineyard using high-frequency GPR ground wave data", *The Leading Edge,* June: 552-559.

Huisman, J.A., Sperl, C., Bouten, W., Verstraten, J.M. (2001), "Soil water content measurements at different scales: accuracy of time domain reflectometry and ground-penetrating radar", *Journal of Hydrology*, 245: 48-58.

Hunaidi, O., Giamou, P. (1998), "Ground penetrating radar for detection of leaks in buried plastic water distribution pipes", *In: Seventh International Conference in Ground Penetrating Radar*, Lawrence, Kansas, USA,

Mowjood, M.I.M., Pathmarajah, S., Rambanda, M. (2005), "Effect of cracks on water requirement for land soaking in lowland paddy cultivation", *Journal of Soil Science Society of Sri Lanka*, 17: 1-10.

Parkin, G., Redman, D., von Bertoldi, P., Zhang, Z. (2000), "Measurement of soil water content below a wastewater trench using ground penetrating radar", *Water Resources Research*, 36(8): 2147-2154.

Raats, P.A.C. (1973), "Unstable wetting fronts in uniform and non-uniform soils". *Soil Science Society America Journal*, 1973; 37: 681-685.

Rucker, D.F., Ferre, T.P. (2002), "Measuring the advance of a wetting front using cross-borehole GPR", *In: Ninth International Conference on Ground Penetrating Radar*, Koppenjan SK, Lee H (eds). Proceedings of SPIE 4758, 2002; 176-179. 176-180.

Saintenoy, A., Schneider, S., Tucholka, P. (2007), "Evaluating ground penetrating radar use for infiltration monitoring", *Vadose Zone Journal*, 7: 208-214.

SSI. (2006), PulseEKKO PRO users' guide, Sensors and Software Inc., Mississauga, ON, Canada.

Seung-Yeup, H., Yu-Sun, Jo, Heon-Cheol, Oh, Se- Yun, K., Young- Sik, K. (2007), "The laboratory scaled-down model of a ground-penetrating radar for leak detection of water pipes", .*Measurement Science and Technology*, 18: 2791-2799.

Šimůnek, J., van Genuchten, M.Th., Šejna, M. (2005), "The HYDRUS-1D Software Package for Simulating the One-Dimensional Movement of Water, Heat, and Multiple Solutes in Variably-Saturated Media", Department of Environmental Sciences, University of California Riverside, Riverside, California.

Singh, V.P. (1995), "Environmental Hydrology", *Springer*. pp 194-199.

Smith, D.G., Harry, M.J. (1992), "Ground-penetrating radar investigation of a Lake Bonneville deita, Provo level, Brigham City, Utah", *Geology*, 20: 1083-1086,

Vellidis, G., Smith, M.C., Thomas, D.L., Asmussen, L.E. (1990), "Detecting wetting front movement in a sandy soil with ground penetrating radar", *American Society of Agricultural Engineers*, 33(06):1867-1873.

# ASSESSMENT OF RAIN WATER HARVESTING POTENTIAL – A CASE STUDY FOR IDAMELANDA

Bandara, H.R.S*[a], Yahiya, Z.[a], Zubair, L.[b]

[a]Foundation for Environment, Climate and Technology, c/o MASL, Digana Village, Rajawella.
[b]The Earth Institute at Columbia University, P.O. Box 1000, (Street: 61 9W), Palisades, NY 10964-8000, U.S.A.

Corresponding author – Bandara H.R.S
*ranmalee11@gmail.com
Foundation for Environment, Climate and Technology, c/o MASL, Digana Village, Rajawella
Tel: +94-81-2376746

**Abstract**

This project assessed the water scarcity and the potential for rain water harvesting of a rural village vulnerable to severe droughts.

Meteorological data was used to calculate the Weighted Average Standardized Precipitation (WASP) Index and Palmer's Drought Severity Index (PDSI. This was then cross referenced with the drought relief data for the region. The lowest and highest correlation values between the two methods were 0.57 and 0.77 respectively.

The feasibility of RWH for both domestic and agricultural activities was then studied and was found to be practical to collect the rainwater using the roof and the land surface as catchments.

**Key Words:** *Rain Water Harvesting (RWH), WASP, PDSI*

## 1. Introduction

Drought is the most frequent disaster in Sri Lanka and the expenditure on drought relief has been the dominant except for the recent Tsunami. Rising population and intensification of water use by domestic, industrial and municipal sector along and use changes can lead to higher frequency of drought incidence. Drought leads to agricultural losses, hardships for those in drought affected areas, and also in a loss of hydroelectricity generation leading to planned electricity outages [1] and can be characterized with different measures: there is the meteorological drought – due to deficiency of rainfall; hydrological drought – due to lack of water in the land surface; and agricultural drought when crops and animals lack water. There are several indicators to assess the severity [2] of a meteorological and hydrological drought and out of these; two methods – namely the PDSI and WASP Index – were used in this work.

In accordance to the type of catchment being used, RWH can be categorized into four groups – roof catchment systems, rock catchment systems, ground catchment systems and check and sand dams, hafirs [3]. Depending on the reliability of the system, there are four more categories; (a) Occasional - water is stored for only a few days in a small container. Suitable when there is a uniform rainfall pattern with very few days without rain and there is a reliable alternative water source nearby (b) Intermittent - in situations with one long rainy season when all water demands are met by rainwater; however, during the dry season water is collected from non-rainwater sources (c) Partial - rainwater is used throughout the year but the 'harvest' is not sufficient for all domestic demands. For instance, rainwater is used for drinking and cooking, while for other domestic uses (e.g. bathing and laundry) water from other sources is used and (d) Full - for the whole year, all water for all domestic purposes is rainwater. In such cases, there is usually no alternative water source other than rainwater, and the available water should be well managed, with enough storage to bridge the dry period [4]. The goal of this study is to use the roof area as the main catchment for domestic consumption with a partial reliability. It will also utilize the ground catchment system as much as possible.

Water scarcity is mainly the physical deficit of water in an area. We focused on the Idamelanda Grama Niladari Division located within the Hanguranketa Divisional Secretarial area in Nuwara Eliya District. It is situated at an elevation of 500-600 m above mean sea level with an annual rainfall of 1530 mm with the mean temperature ranging from 20-25$^0$C. Low annual rainfall, occurring from October - December (175 mm) and January and April (100 mm), coupled with high elevation with dry air currents, has resulted in deficits of water. This does not mean that the shortage of water must lead to droughts. This is because if potential capacity to undertake mitigatory measures such as pumping water, transportation to water available areas, introducing drought sustainable crops, rain water harvesting etc. – and in addition the capability of adapt by the people of that area. Both the scarcity of water and vulnerability to this scarcity must combine to lead to a disaster. The vulnerability can be considered for four principal categories – people, economic activity, infrastructure and networks. In the case of droughts, the effect is more on people and economic activities while it has a rare effect on infrastructure or networks in Idamelanda.

At Idamelanda, water is required both for domestic purposes, as well as for agricultural activities. Home gardens are important for the peoples' welfare. In addition, a source of income is from rain fed cultivation in Chena (slash, burn and fallow) land due to non availability of a dependable source of water for agriculture. Slash and burn agriculture is unsustainable as the available forests are being constantly lost. In addition, it affects the water table as water that percolates to the ground during the rainy season prevalent is lost due to flash floods during the wet period. At present, some people use water from deep wells to meet their domestic water needs.

Although drought has a relationship with climate, little use has been made of meteorological information to characterize the characteristics of drought by region and seasons and to make use of this information to inform the design of rain water harvesting units. Here, we investigate the viability of:

(1) Generation of regional, seasonal and long term indicators for drought risk at fine scale
(2) Design Guidelines of a Tank for RWH

## 2. Methodology

This study utilizes two independent methods, (a) Palmer's Drought Severity Index (PDSI) and (b) Weighted Average Standardized Precipitation (WASP), of assessing the drought.

### *2.1 Meteorological Data*

The meteorological information, rainfall, temperature, solar radiation, wind speed, maximum and minimum relative humidity as well as the soil moisture of the area were needed for the assessment of the severity of the drought and the design for RWH. Since both the rainfall and temperature were not available for Idamelanda itself, there being no established climate station, the data from proximate stations were used for interpolation. The Inverse Distance Method was used for the interpolation [5]. Using this information, all calculations were done. The selected data duration was from 1960-2000.

### *2.2 Drought Assessment Methodology*

Among the indices used to assess drought, we have made use of WASP and PDSI methodologies for our assessment process. The WASP is normalized to a range of values from -4 to +4 while the PDSI ranges from -6 to +6. The negative values indicate less rainfall while positive values indicate excess in rainfall. The PDSI [6] is a measure of drought with due consideration to the soil type, water retaining capacity of the soil and watershed information while WASP concentrates on contextualizing the current rainfall in terms of recent rainfall and what may be expected.

### *2.3 Comparison of Drought Assessment Methods – WASP and PDSI – with Drought Relief Data*

The WASP Index and the Palmer Drought Severity Index (PDSI) were calculated. The severity of the drought was then compared. The WASP index for the Western Region of Sri Lanka was compared with the record of droughts.

## 3 Results & Analysis

The WASP (Weighted Anomaly Standardized Precipitation) Index and PDSI (Palmer Drought Severity Index) were obtained. The correlation for each month (Table 1) was then calculated to assess the strength of the relationship before graphically comparing the two methods with the recorded drought relief data.

| **Month** | **January** | **February** | **March** | **April** | **May** | **June** | **July** | **August** | **September** | **October** | **November** | **December** |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Correlation** | 0.74 | 0.59 | 0.57 | 0.59 | 0.61 | 0.73 | 0.77 | 0.74 | 0.75 | 0.72 | 0.67 | 0.66 |

**Table 1**: *Correlation between PDSI and WASP for each month*

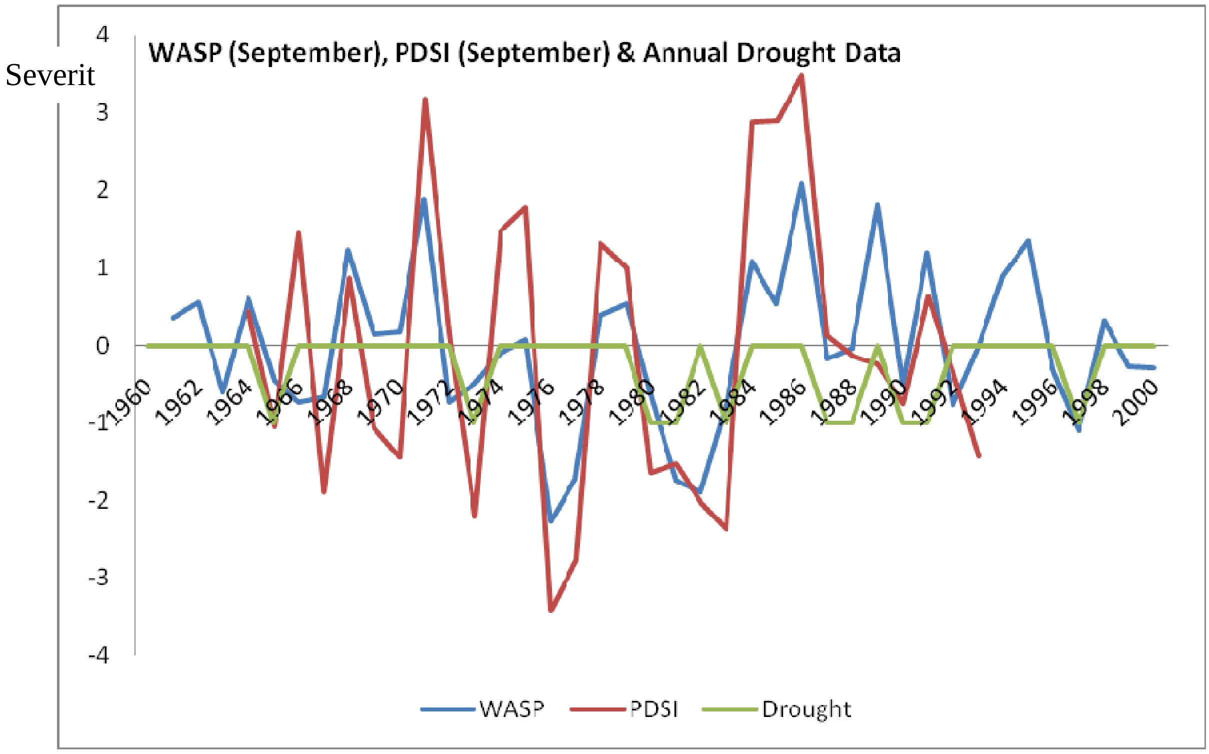



**Figure 1**: *12 Month WASP and PDSI for the month of September against the Annual Drought Relief Data for the Western Slopes*

By making use of records of relief payments by the government, it is possible to get an idea of how the drought had affected the region of interest. For this, the relief payments in the Western Region (inclusive of Nuwaraeliya) were graphed inversely (so that it is easy to identify the years for which relief had been allocated) with the regional WASP and PDSI values obtained via calculations (Figure 1).

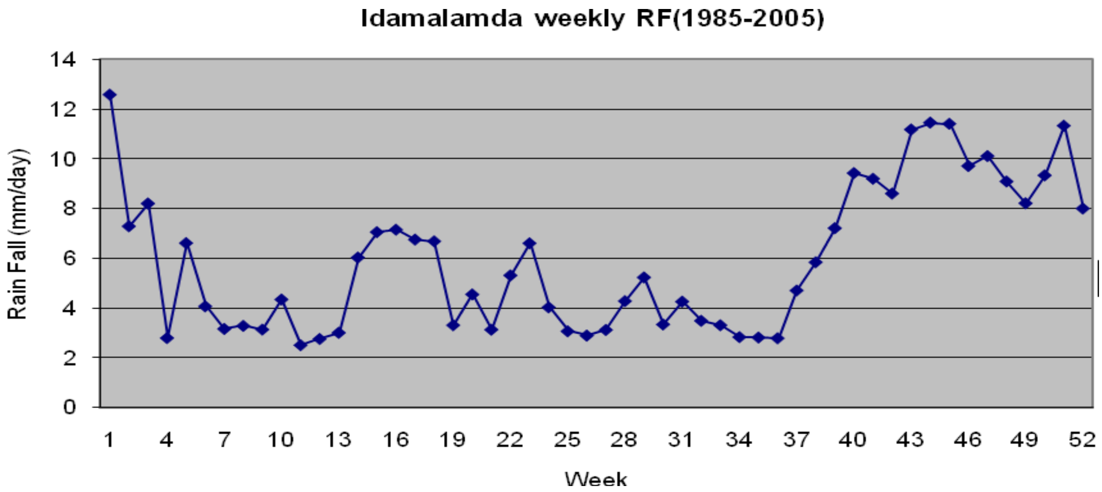



**Figure 2**: *30 year Averaged Rainfall for Idamelanda area during a year. This data was estimated using 7 neighboring stations – Katugastota, Galpihilla, Dackwari, Woodside, Kurundu Oya, Hope Estate, New Forest – using the Inverse Distance Calculation Method.*

According to Figure 2, Idamelanda gets a rainfall over 8 mm per day for 15 weeks while the rest of the time, it is lesser than that. The period where the rainfall is lower than 8 mm was considered as the dry period [7] for this area and the RWH system design was done to fulfill the water requirements of the people of the Idamelanda area during this 15 dry weeks.

## 4 Discussion

*Design Guidelines of a Tank for RWH*

The amount of water that can be collected from a unit area (1 $m^2$) of roof with Asbestos sheeting (runoff coefficient is taken as 0.84) [8] within the 15 week duration is around 850 liters. As an average, the houses in this area have an approximate roof area of 70 $m^2$. Using a horizontal area of 50 $m^2$ from the roof (leaving out 20 $m^2$ for the chimney, slope of roof etc.) as a catchment during the rainy season, one household can collect about 45 000 liters of water.

Taking the average consumption (drinking purposes only) of a person as 20 liters/day [7, 9], the number of members in a family as five (05), the amount of water needed by this particular family to see the dry period through would be approximately 26 000 liters. Even though they can collect up to 45 000 liters and what they need for their consumption during the dry period is about 25 000 liters, the maximum volume of the tank that is proposed is 8000 l. This is because anything more than this would have an extremely high building cost. At the same time, it is assumed that the water collected in the tank is mainly used for drinking purposes.

It is taken that a home garden has an area of 1 000 $m^2$ (¼ an acre) and that the area that can be used to collect the runoff is 500 $m^2$. Taking that 10% of the rainfall goes off as the runoff [10], what is collected is approximately 16 000 liters. The runoff will be stored in a tank below surface level and the volume of this tank was designed as 15 000 liters. It is assumed that most of the runoff is collected and stored so as to be used in irrigation purposes. The irrigation water requirement changes with the crop type and irrigation system to be used, but, in this case, the tank capacity was calculated to store as much water as possible to be collected within the rainy period.

In this context, our work provides a methodology for the design of mitigation for drought through the use of RWH. We proposed an above-the-surface tank (Figure 3a) for collecting rainwater for domestic purposes and a surface level tank (Figure 3b) [11] for collecting the runoff water and showed how climate information should be included in tailoring these units. Through the explicit relationship with climate, we were then able to go on to assess the impact of climate change on both water scarcity and the demands for mitigation.

**Figure 3a:** *Above-the-surface tank (Idamelanda) under construction (Credit: Janaki Chandimala)* **Figure 3b:** *Polythene tank (Moonasinghe, V)*

## 5 Conclusions

The close correspondence between drought disasters and the hazard index confirms the utility of the WASP methodology to identify drought risk. The PDSI has comparatively lesser correspondence with the relief data and this may be due to the fact that it considers many other factors – temperature, soil moisture, and location – in the calculation in addition to the precipitation. Yet, it is possible to identify a similarity in trends. The correlations among the two methods (Table 1) and the graphic representation (Figure 1) are good proofs for this.

The designs proposed under the study require low installation and maintenance cost, so that the people could achieve the sustainability of their lives and agricultural activities. Investing on RWH tanks is economical in the long run as it could reduce the relief payments.

## References


[1] L. Zubair, Ralapanawe V., Tennakone, U., Yahiya, Z., Perera, R., "Natural Disaster Risks in Sri Lanka: Mapping Hazards and Risk Hotspots," in *Natural Disaster Hotspots Case Studies*, R. C. Margaret Arnold, Uwe Deichmann, Maxx Dilleyand Arthur Lerner-Lam, Randolph E. Pullen, Zoe Trohanis, Ed., ed Washington DC: World Bank, 2006.

[2] M. Hayes, "Drought Indices," N. D. M. Center, Ed., ed. Alaska.

[3] J. Gould, Nissen-Petersen, E., *Rainwater Catchment Systems for Domestic Supply: Design, Construction and Implementation.*: Practical Action, 1999.

[4] J. Smet. (2003, August 10, 2010). Domestic Rainwater Harvesting. *WELL factsheet*. Available: http://www.lboro.ac.uk/well/resources/fact-sheets/fact-sheets-htm/drh.htm

[5] P. Goovaerts, "Geostatistical approaches for incorporating elevation into the spatial interpolation of rainfall," *Journal of Hydrology*, vol. 228, pp. 113-129 February 21, 2000 2000.

[6] W. C. Palmer, "Meteorological Drought," U. D. o. Commerce, Ed., ed. Washington DC: U.S. Government Printing Office, 1965, p. 58.

[7] S. G. J. Rajkumar, "Holistic approach in managing waste fluxes in a drainage basin – a case study from Vavunia – Sri Lanka," *Water Science and Technology,* vol. 49, pp. 7-221, 2003.

[8] C. M. Navaratne, "Low Cost Auto Irrigation System for Home Garden Intensification to Minimize the Agro-climatic Risk: A Case Study in Matara District," PhD, Irrigation and Water Management University of Ruhuna, Matara, 2003.

[9] H. A. H. Jayasena, Selker, J.H., "A Need for Efficient Water Management in The Deduru Oya Basin in Sri Lanka – A Case Study Based on Minimum Daily Water Requirements," in *World Water Week* Stockholm, 2007.

[10] J. Rockstrom, "Water Resources Management in Smallholder Farms in Eastern and Southern Africa: An Overview," *Physics and Chemistry of the Earth*, vol. 25, pp. 275-283, September 24 2000.

[11] V. Moonasinghe, "Rain Water Harvesting Technologies in Sri Lanka," ed. Sri Lanka: South Asia Research Institute for Policy and Development 2004.

# MULTI TANK MODEL FOR ENERGY EFFICIENT RAIN WATER HARVESTING

[1]S. Sendanayake, [2]M. T. R Jayasinghe

[1]Postgraduate Student, Department of Civil Engineering, University of Moratuwa, Moratuwa, Sri Lanka.
[1]E-mail: sisuru@civil.mrt.ac.lk
[1]Telephone: +94-11-2650567-8 (Ext.2021) ; Fax: + 94-11-2651216

[2]Professor, Department of Civil Engineering, University of Moratuwa, Moratuwa, Sri Lanka.
[2]E-mail: thishan@civil.mrt.ac.lk ; mtrjayasinghe@yahoo.com
[2]Telephone: +94-11-2650567-8 (Ext.2006 - 2001) ; Fax: + 94-11-2651216

**Abstract**: Rain Water Harvesting (RWH) to supplement service water is an important aspect of sustainable development. As such, much research has been carried out on optimizing the system components of RWH systems so that a maximum water saving efficiency (WSE) can be reached with a minimum storage capacity, enabling the outlay on capital minimized. However, if RWH is to proliferate it should be able to operate in par with centralized service water, supplying collected rain water to user points reliably. This is usually achieved by pumping the collected rain water to service points utilizing electricity, which in fact could negate the positive gains of RWH with regard to principles of sustainability.

By introducing a multi tank model, where a smaller tank is installed for each floor at its roof level in addition to the main storage tank, a solution can be reached with superior system performance. In the model, the roof collection enters the top most tank first and then cascades through multiple tanks in multi story situations, before being collected in the main storage tank, vastly improving the overall energy efficiency of the system. The model, not only addresses the space and structural issues of the building but also ensures that the aesthetics of the building envelop is not disturbed due to smaller sizes of the upper tanks.

***Key words***: *Rainfall, Cascading, Multi tank, Sustainability*

## 1. Introduction

Any conventional RWH system consists of a rain water collector (usually the roof), a storage tank and a piping network to convey the collected rain water to the storage as the main components. Of the three main components mentioned above, the storage tank has drawn most of the attention due the size, positioning and the capital outlay required on it thus subjecting it to the bulk of research on RWH. Though various studies have been carried out to optimize the size of the tank (Fewkes)(9) with regard to the water saving efficiency (WSE) of the composite system, no alternative has been found to position the tank other than at or below the ground level, requiring energy consuming pumping to feed the collected rain water to service points. Positioning the tank in between the roof and the service points is a partial solution but the space requirement and the structural support required hinders the proliferation of RWH. This study introduces a methodology where the main storage capacity is distributed among the floors in a multi storey building so that the roof collection cascades down to a parent tank through a series of tanks, each collecting and feeding individual floors while contributing to the composite system. As the tanks at upper levels feed the corresponding floors under gravity, the energy required for pumping is minimized.

## 2. Energy requirement in multi tank RWH systems

For a given service water demand D (in $m^3$/year), roof collection area A (in $m^2$), annual rainfall R (in m) and storage capacity S (in $m^3$), the water saving efficiency (WSE) $\eta$ can be found from generalized curves for WSE, developed by Fewkes (9).

For cascading multi tank situations, the following algorithms are valid.

For each floor, If the yield is $Y_i$, for i = 1 to n

Pumping requirement $Q_i$ ;

$Q_i = D_i - Y_i = D_i(1- \eta_i)$ [1]

Then for the i[th] floor (i[th] tank),

When the demand is $D_i$, supply is $(AR)_i$

But, $(AR)_i = (AR)_{i+1} – Y_{i+1}$

Since $Y_{i+1} = D_{i+1} * \eta_{i+1}$

$(AR)_i = (AR)_{i+1} – D_{i+1} * \eta_{i+1}$ [2]

Further, if the total demand is D,

$$D = \sum_{i=1}^{n} Di \qquad [3]$$

The overall WSE for the system is denoted as $\eta_o$

Therefore, if the number of floors are n and the ground floor is taken as i=0, it can be shown that;

The amount of water that can be pumped up in CMTRWH system, Q,

$$Q = \sum_{i=1}^{n} Qi - \sum_{i=1}^{n} Qi\,(1- \eta_P) = \sum_{i=1}^{n} Qi * \eta_P$$

From Equation 3.2,

$$Q = \eta_P \{ \sum_{i=1}^{n} Di - \sum_{i=1}^{n} Di\eta i \} \qquad [4]$$

When,

$$(AR)i = AR - \sum_{i=i+1}^{n} Di * \eta i \qquad [5]$$

When the demand at each floor level is taken as $D_i$, and the total system demand is taken as D, for i = 1 to n;

Since $\sum D_i = D$,

$D_1 = D_2 =$………..$= D_n = D/n$

Therefore, from equations 3.5 and 3.6,

$$Q = \eta_P \{ \sum_{i=1}^{n} Di - \sum_{i=1}^{n} Di\eta i \}$$

$$Q = \eta_P D\{1 - 1/n \sum_{i=i+1}^{n} \eta i \} \qquad [6]$$

$$(AR)_i = AR - D/n \sum_{i=i+1}^{n} \eta i \qquad [7]$$

Energy required in pumping collected rain water in two types of houses, namely single story and two story houses, are analyzed for daily demands of 200 L, 300 L, 400 L and 600 L. In the single story house, two tanks are employed with the upper tank of 1 $m^3$ capacity located at the eve level, just below the roof collection area. Three tanks are employed in the 2 story house with the upper tanks located at eve and first floor levels and the parent tank at ground level. In the two story house, the demand is taken as equally divided between the two floors. The energy required is shown as a percentage of energy required to pump collected rain water from a single tank at ground level against D/AR, where A is the collector area in $m^2$ and R is the annual average rainfall in m for a particular geographical region. Use of the parameter D/AR will give more flexibility to use any combination of A and R, for a given constant AR value. Fewkes (1999) generalized curves validated for Sri Lanka (Sendanayake & Jayasinghe, 2006), is used to determine WSE for a given demand and storage volume. All storage tanks located at upper levels are of 1 $m^3$ capacity.

The roof collection area is taken as 50 $m^2$ in the wet climatic region of Sri Lanka, where the annual average rainfall is 2500 mm (Meteorological Department of Sri Lanka). Therefore, AR is calculated as 125 $m^3$ and for maximum WSE, $S_p$ is taken as 0.1 AR, i.e. 12.5 $m^3$. As the generic curves for WSE is valid for $0.25(AR)_i \leq D_i \leq 2.00$, the maximum possible demand is calculated as 600 L/day. The amount of rain water that can be pumped up when only the parent tank is employed is denoted as $Q_o$. The value Q/ $Q_o$ is representative of the energy requirement in pumping as a percentage. Q/ $Q_o$ values are plotted against D/AR to determine the operating characteristics of CMTRWH systems, where D is the total daily demand. This will effectively compare the CMTRWH situations for two and three tank models with conventional single tank RWH systems under the same A, R and D.

Table 3.1: *Energy requirement % vs. Demand in Two Tank model*

| D L/day | D $m^3$/yr | D/AR | $\eta_0$ | $\eta_1$ | Q | Qo | Q/Qo% |
|---|---|---|---|---|---|---|---|
| 200 | 73 | 0.58 | 100 | 67.5 | 23.73 | 73 | 33 |
| 300 | 109.5 | 0.87 | 90 | 50 | 49.28 | 101.28 | 48 |
| 400 | 146 | 1.17 | 65 | 45 | 52.2 | 116.8 | 44 |
| 600 | 219 | 1.74 | 35 | 32 | 44 | 122.64 | 36 |

Table 3.2: *Energy requirement % vs. Demand in Three Tank model*

| D L/day | D $m^3$/yr | D/AR | $\eta_0$ | $\eta_1$ | $\eta_2$ | Q | Qo | Q/Qo% |
|---|---|---|---|---|---|---|---|---|
| 200 | 73 | 0.58 | 100 | 92.5 | 77.5 | 10.95 | 73 | 15 |
| 300 | 109.5 | 0.87 | 92.5 | 77.5 | 55 | 36.96 | 101.29 | 36 |
| 400 | 146 | 1.17 | 80 | 67.5 | 52.5 | 35.04 | 116.8 | 26 |
| 600 | 219 | 1.74 | 56 | 50 | 42.5 | 28.06 | 122.64 | 23 |

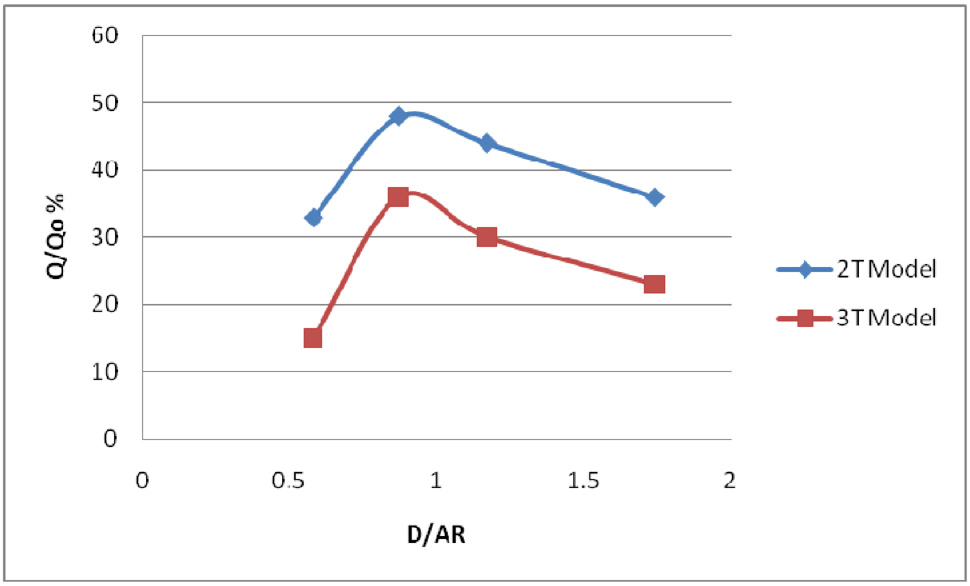



Chart 3.4: *Energy requirement % vs. Demand in Two and Three Tank models*

## 3. Discussion

From the Chart 1 it can be seen that for a desired total demand D- equally distributed among the floor levels- and for a given storage capacities, the energy requirement of the system as a percentage of the energy requirement when the same storage is used in a conventional rain water harvesting (RWH) system increases and maximizes when D/AR = 1.00. This implies that when D/AR increases, the quantity of collected rain water that can be pumped up (Q) increases maximizing at D/AR = 1.00. However, when D/AR > 1.00, Q decreases, indicating that the system is under-performing. The behavior of the system with regard to increase of D/AR can be further explained referring to equations 1-4 and Fewkes generalized curves for WSE. It can be seen that when D/AR increases, WSE of upper tanks as well as the composite system drops with the latter dropping at a lesser rate. This can be clearly seen by referring to the generalized curves of WSE, where the storage capacity of the composite system $S > S_i$ and $S_i/AR$ falling in the critical zone of the WSE chart, i.e. when S/AR < 0.01. Further, when D/AR > 1.00, though both $\eta_i$ and $\eta_o$ decreases, the WSE chart indicates that $\eta_o$ decreasing rapidly hence dropping the overall performance of the system at a much higher rate. From equations 3 and 4 and Fewkes generalized curves for WSE it can be deduced that for any number of floor levels Q maximizes when D/AR = 1.00 implying that the shape of the percentage pumping energy curve is governed by the characteristic curves of WSE. Therefore, it indicates that for a given set of system parameters D, A, R and S, the maximum yield that can be extracted from the system occurs when D = AR. In other words, it is clear that when D < AR the full potential of the system is not harnessed and when D > AR the system is underperforming. Hence it is important to focus on the energy efficiency of the RWH system when D = AR, i.e. when the system is operating at optimum conditions, to introduce energy efficient pumping so that the overall system performance is optimized.

## References


1. ABS 2000, Australian Bureau of Statistics, Aust-Stats, Environmental Issues: Peoples' Views and Practices, Water use and Conservation. www.abs.gov.au/auststats.

2. ABS 2000, Australian Bureau of Statistics, Aust-Stats, Water Account for Australia, 1993-94 to 1996-97.

3. Adger, W.N., (2006), "Vulnerability," Global Environmental Change, Volume 16, Issue 3, Pages 268-281.

4. ADWG 1996, Australian Drinking Water Guidelines, (1996), National Water Quality Management Strategy, NHMRC & ARMCANZ.,www.health.gov.au/nhmrc/publications.

5. Appan, A. (1999), "A dual mode system for harnessing roof water for non-potable uses", Urban Water, 1(4): 345-54.

6. Chilton, J.C., Midment, G.G., Marriott, D., Francis, A., Tobias, G. (1999), "Case study of a rainwater recovery system in a commercial building with a large roof", Urban Water; 1(4): 345-54.

7. Coombes, P. (2002), "Rainwater tanks revisited. New opportunities for urban water cycle management." PhD Thesis, University of NewCastle, Australia.

8. Edwards, D., Keller, K. (1984), "A workshop design for rainwater catchment systems (Training Guide) & appendix. "Rainwater Harvesting for Domestic Water Supply in Developing Countries," La Paz, UNESCO/ROSTLAC, UNICEF.

9. Fewkes, A. (1999a),"Modeling the performance of rainwater collection systems towards a generalized approach", Urban water, 1, 323-333.

10. Fewkes, A. (1999b), "The use of rainwater for WC flushing: The field test of a collection system", Building & Environment, 34, 765-772.

11. Gould, J., Nissen-Peterson, E. (1999), "Rainwater catchment implementation", London, Intermediate technology publications.

12. Hermann, T., Schmida, U. (1999),"Rainwater utilization in Germany: efficiency, dimensioning, hydraulic and environmental aspects", Urban water, 1, 307-316.

13. Jayasinghe, M.T.R., Jayawardene, A. I., Attalage, R. A. (2001) "Concepts for sustainable residential developments for urban and suburban areas in Sri Lanka", Engineer, Journal of Institution of Engineers, Sri Lanka, vol xxxiv, No.2, pp 63-77.

14. Jewitt, G., (2002), "Can Integrated Water Resources Management sustain the provision of ecosystem goods and services?**"** Physics and Chemistry of the Earth, Parts A/B/C, Volume 27, Issue 11-22, Pages 887-895.

15. Miller, G., Yu, Bofu., Gardner, T. (2003), "Rainfall catch efficiency for domestic water supply," 28$^{th}$ International Hydrology and water resources Symposium, Wollongong, NSW, Australia, Institution of Engineers, Australia.

16. Mwenge Kahinda, J.m., Taigbenu, A.E., Boroto, J.R.,(2007), "Domestic rainwater harvesting to improve water supply in rural South Africa**,"** Physics and Chemistry of the Earth, Parts A/B/C, Volume 32, Issue 15-18, Pages 1050-1057.
17. Villareal, E.L., Dixon, A. (2004), "Analysis of a rainwater collection system for domestic water supply in Ringdansen, Norrkoping, Sweden", Building & Environment, 40, Elsevier Ltd.

18. Zobrist, J., Muller, S.R., Amman, A., Bucheli, T. A., Mottier, V., Ochs, M., Shoenenberger R., Engster, J., Boller, M. (2000), "Quality of roof run-off for Ground water infiltration", Water Resources, vol.34, No.5, pp 1455-1462.

**Acknowledgements**
The technical staff of the Department of Civil Engineering, Messers S.P.Madanayake, S.L Kapuruge assisted this project at the concept development stage.

**About the Authors**

**S. SENDANAYAKE,** B.Sc. Hons. Moratuwa., M.Sc. Moratuwa is currently reading for his Ph.D. at the Department of Civil Engineering, University of Moratuwa.

**M. T. R. JAYASINGHE,** B.Sc. Hons. Moratuwa., Ph.D. Cambridge., MIE (SL), C.Eng. is a Professor at the Department of Civil Engineering, University of Moratuwa. His research interests are in the areas of tall buildings, energy efficiency, sustainable development and disaster resistant structures.

# STATISTICAL MODELING OF DAILY EXTREME RAINFALL IN COLOMBO

N.Varathan[1], K.Perera[2] and N.Wikramanayake[3]

[1]Lecturer, Department of Mathematics and Statistics, University of Jaffna, Jaffna, Sri Lanka.
[1]E-mail: varathan10@gmail.com
[1]Telephone: +94-77-9352990

[2]Senior Lecturer, Department of Engineering Mathematics, University of Peradeniya, Peradeniya, Sri Lanka.
[2]E-mail: kanthip@pdn.ac.lk
[2]Telephone: +94-81-2393346

[3]Senior Lecturer, Department of Civil Engineering, Open University, Navala, Sri Lanka.
[3]E-mail: tomwiks@yahoo.com
[3]Telephone: +94-77-7763123

**Abstract:** The occurrence of heavy rainfalls in Sri Lanka results in significant damage to agriculture, ecology, infrastructure systems, disruption of human activities, injuries and the loss of life. The modelling of extreme rainfall has to be developed to manage the natural resources and the built environment to face the impacts of climate change. The main goal of this study is to find the best fitting distribution to the extreme daily rainfalls measured over the Colombo region for the years 1900-2009 by using the maximum likelihood approach. The study also predicts the extreme rainfalls for return periods and their confidence bands. In this study extreme rainfall events are defined by two different methods based on (1) the annual maximums of the daily rainfalls and (2) the daily rainfalls exceeds some specific threshold value. The Generalized Extreme Value distribution and the Generalized Pareto distribution are fitted to data corresponding to the methods 1 and 2 to describe the extremes of rainfall and to predict its future behaviour. Finally we find the evidence to suggest that the Gumbel distribution provides the most appropriate model for the annual maximums of daily rainfall and the Exponential distribution gives the reasonable model for the daily rainfall data over the threshold value of 100mm for the Colombo location. We derive estimates of 5, 10, 20, 50 and 100 years return levels and its corresponding confidence intervals for extreme daily rainfalls.

**Keywords:** Annual maximum, Threshold value, Generalized Extreme Value distribution, Pareto distribution, Maximum likelihood estimation

## 1 Introduction

Extreme rainfall events cause significant damage to agriculture, ecology and infrastructure, disruption of human activities, injuries and loss of lives. In Sri Lanka many areas are affected by the heavy rainfall and the associated floods and land slides. In particular Colombo, the capital of the country faces serious flooding problems in low lying areas due to extreme rainfalls. In order to design measures to reduce the threat of flooding it is necessary to carry out statistical modelling of extreme rainfalls and develop design rainfalls of different return periods.

The statistical analysis of extreme rainfall has been done by the scholars in different locations all over the world. In Sri Lanka, Baheerathan and Shaw(1978) have analyzed Rainfall depth duration frequency studies for Sri Lanka using the annual maximum rainfall depths with 3-,6-,12- and 24-h durations for 19 stations spread over the country. They have analysed data from 8 to 24 years in different stations by fitting Gumbel distribution with maximum likelihood parameter estimation. Dharmasena and Premasiri (1990) studied the same concept but the regionalization technique and linear interpolation of intensities for short durations adopted by Baheerathan and Shaw are not adopted in their study. They used 25 years of data of five regions and considered Gumbel distribution with maximum likelihood estimation technique to fit the data.

In this study, we find the best fitting distribution to extreme daily rainfall by using all available past data from 1900-2009 in Colombo station. We use two techniques to select the sample: one is considering the annual maximums of daily rainfall and the other is selecting exceedances over a specific threshold value. The Generalized Extreme Value distribution (GEV) and the Generalized

Pareto Distribution (GPD) are used to find the best fitting distribution for the above two techniques respectively. The parameters are estimated by maximum likelihood method. Moreover, the outliers are considered and the confidence intervals for predicted extreme rainfalls also developed in our study.

## 2 Theoretical Framework

### *2.1 The Extreme Value Distributions*

There are three models that are commonly used for extreme value analysis. These are the Gumbel, Frechet, and Weibull distribution functions. The Gumbel is easier to work with since it requires only location and scale parameters, while the Weibull and Frechet require location, scale, and shape parameters. The GEV distribution function is, *H(x)* = exp { $-(1+\xi(x-\mu)/\psi)^{-1/\xi}$ } ;where $\psi > 0$ – scale, $\xi$ - shape and $\mu$ - location parameter.

According to the value of $\xi$ , H(x) can be divided into following three standard types of distributions:

1. If $\xi \to 0$ (Gumbel Distribution)

   $H(x) = \exp(-e^{-x})$ , all $x$

2. If $\xi > 0$ ( Frechet Distribution with $\alpha = 1/\xi$ )

   $$H(x) = \begin{cases} 0 & ; x < 0 \\ \exp(-x^{-\alpha}) & ; x > 0 \end{cases}$$

3. If $\xi < 0$ ( Weibull Distribution $\alpha = -1/\xi$ )

   $$H(x) = \begin{cases} \exp(-|x|^{\alpha}) & ; x < 0 \\ 1 & ; x > 0 \end{cases}$$

### *2.2 The Exceedances over Threshold*

In this technique the data are collected over some specific threshold (cut-off) value. Modelling the extremes under this method enables a more efficient usage of extreme value information than that given by an analysis of annual maxima data, which excludes from inference many extreme events that did not happen to be the largest annual event. As a statistical modeling technique this procedure was popularized by Davison and Smith (1990).

Assuming the daily data to be independent with common distribution function F, the conditional distribution of excesses of a threshold u is determined by, $\Pr(X \le u + y \mid X > u) = 1 - \dfrac{1 - F(u+y)}{1 - F(u)}, y > 0$ .Renormalizing and letting $u \to \infty$ leads to an approximate family of distributions given by, $G(y) = 1 - (1 + \xi(y-u)/\sigma)^{-1/\xi}$ is the Generalized Pareto family. This family describes all non-degenerate limiting distributions of the scaled excess-of-threshold distributions. When $\xi \to 0$ the above generalized Pareto family converges to an Exponential family.

### *2.3 Return Periods*

Briefly, the return period (occurrence interval) can be defined as the average time until the next occurrence of a defined event. When the time to the next occurrence has a geometric distribution, the return period is equal to the inverse of probability of the event occurring in the next time period, that is, $T = 1/P$, where $T$ is the return period, in number of time intervals, and $P$ is the probability of the next event's occurrence in a given time interval.

## 3 Materials and Methods

The data consists of daily rainfall for the years from 1900 to 2009 for the Colombo location. The data was obtained from the Department of Meteorology, Colombo, which lists the daily rainfalls in millimetres.

We have applied the Univariate Extreme Value Theory to fit the distribution and estimate the return periods for the 110-years (1900-2009) of daily extreme rainfall in Colombo by using the statistical software “GenStat”. The GEV distribution to annual maximums and GPD to rainfall over some specified cutt-off value are considered first, and then by testing the shape parameter the best fitting distribution is identified. Thereafter 95% approximate confidence intervals for return periods are found using the identified model.
By examining the mean residual life plot and the parameter stability plot of sigma, it was decided that a value of 100mm seemed reasonable, as the mean residual life plot was approximately linear for a threshold > 100mm and sigma was stable for values of a threshold > 100mm.

## 4 Results and Discussion

### *4.1 Fitting distribution to Annual Maximums of Daily Rainfall*

**Fitting Generalized Extreme Value Distribution (GEV)**

Table 4.1 *Maximum Likelihood Parameter Estimation*

| Parameter | Estimate | Standard Error |
|---|---|---|
| $\mu$ | 114.9 | 5.839 |
| $\psi$ | 38.52 | 4.449 |
| $\xi$ | 0.1235 | 0.1008 |

After fitting the GEV distribution, we check whether the shape parameter ($\xi$) is zero or not. (P-value = 0.049<0.05), so the data do not fit the Gumbel distribution. The data fits Frechet distribution (since $\xi > 0$).

The Table 4.2 gives the return values of the annual maximum rainfall daily and their 95% confidence levels for the return periods 5, 10, 20, 50 and 100 years.

Table 4.2 *Return periods and its 95% Confidence bands*

| Probability | Return Period | Return Level | Lower | Upper |
|---|---|---|---|---|
| 0.2000 | 05 | 178.4 | 156.2 | 200.5 |
| 0.1000 | 10 | 214.8 | 181.0 | 248.6 |
| 0.0500 | 20 | 253.1 | 201.4 | 304.8 |
| 0.0200 | 50 | 308.0 | 221.2 | 394.8 |
| 0.0100 | 100 | 353.5 | 229.9 | 477.0 |

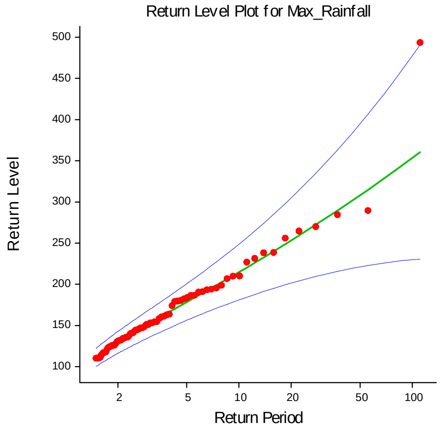



Figure 4.1 *Return Level Plot*

According to the Figure 4.1, it can be seen all the data points are lie within the confidence bands except one point. So we test whether this data point is significant outlier or not, using the Box plot and Grubb's test we found one point was an outlier. This is for the year-1992 annual maximum.

### *4.2 Fitting GEV Distribution to Outlier Removed Data*

After removing the outlier, again we fit the GEV distribution for the annual maximums of daily rainfall data.

Table 4.3 *Maximum Likelihood Parameter Estimation for GEV*

| Parameter | Estimate | Standard Error |
|---|---|---|
| $\mu$ | 115.5 | 5.870 |
| $\psi$ | 37.94 | 4.371 |
| $\xi$ | 0.0441 | 0.1126 |

For testing the shape parameter $\xi = 0$, Since the P-value=0.572 > 0.05, there is no evidence to reject the null hypothesis $\xi = 0$. That is, after removing the outlier annual maximums of daily rainfall data fits Gumbel distribution well.

Table 4.4 *Maximum Likelihood Parameter Estimation for Gumbel*

| Parameter | Estimate | Standard Error |
|---|---|---|
| $\mu$ | 116.4 | 5.496 |
| $\psi$ | 38.59 | 4.165 |

Table 4.5 *Return periods and its 95% Confidence bands under Gumbel*

| Probability | Return Period | Return Level | Lower | Upper |
|---|---|---|---|---|
| 0.2000 | 05 | 174.3 | 155.7 | 192.9 |
| 0.1000 | 10 | 203.3 | 179.3 | 227.3 |
| 0.0500 | 20 | 231.0 | 201.6 | 260.5 |
| 0.0200 | 50 | 267.0 | 230.3 | 303.6 |
| 0.0100 | 100 | 293.9 | 251.8 | 336.1 |

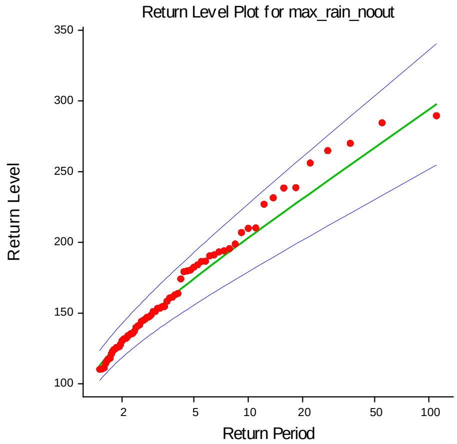



Figure 4.2 *Return Level Plot under Gumbel distribution*

We can observe that the return values of the daily extreme rainfall after removing the outlier is smaller than that of the original data set and the confidence bands width also narrow (Table 4.2 & 4.5). Therefore, it can be said that the Gumbel distribution is the best fit for the annual maximums of daily rain data for the Colombo location.

### *4.3 Fitting Distribution to Daily Rainfall over a Specified Threshold*

**Fitting Generalized Pareto Distribution (GPD)**

The threshold value of 100mm was found using the Mean Residual life plot and the Stability plot. After removing the outlier, 174 data points were collected using the threshold value of 100mm. By using this collected data, first we fit the Generalized Pareto Distribution (GPD).

Table 4.6 *Maximum Likelihood Parameter Estimation for GPD*

| Parameter | Estimate | Standard Error |
|---|---|---|
| $\mu$ | 35.04 | 5.944 |
| $\psi$ | 0.06553 | 0.1317 |

The Table 4.6 gives the estimates of the parameters of the GPD distribution using maximum likelihood method. After fitting the GPD distribution, we check the whether the shape parameter ($\xi$) is zero or not (ie: the data fits the Exponential distribution or not). Since the P-value = 0.3977 >0.05, we don't have evidence to reject the null hypothesis at 5% level of significance. That is, the data fits the Exponential distribution.

Exponential Distribution:

= 1 - exp(-(y-u)/ 37.47) ; for y > u - the threshold

Threshold u = 100

Based on the identified Exponential distribution we find the return values of the daily rainfall and their 95% confidence levels for the return periods 5, 10, 20, 50 and 100 years. From the Table 4.2.1.2, the 20 year return period is 229.4, which means every 20 year we can expect in average 229.4 mm or more daily extreme rainfall with the probability 0.05.

Table 4.7 *Return periods and its 95% Confidence bands*

| Probability | Return Period | Return Level | Lower | Upper |
|---|---|---|---|---|
| 0.2000 | 05 | 177.5 | 160.3 | 194.7 |
| 0.1000 | 10 | 203.4 | 181.0 | 225.9 |
| 0.0500 | 20 | 229.4 | 201.7 | 257.2 |
| 0.0200 | 50 | 263.7 | 228.9 | 298.6 |
| 0.0100 | 100 | 289.7 | 249.5 | 330.0 |

Figure 4.3 gives the return periods in years (as the period length was given as 365). Approximate confidence limits for the return periods can be read off the bands.

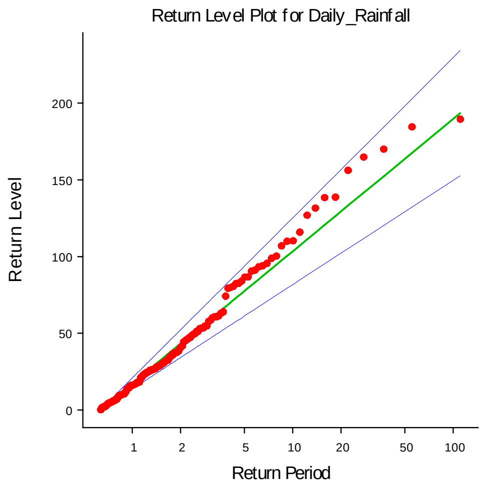



Figure 4.3 *Return Level Plot under Exponential distribution*

Based on the threshold technique the best fitting distribution of the daily rainfall is Exponential with parameter $\psi = 37.47$ .

From the Tables 4.5 and 4.7 we can notice, the predicted return values and the confidence levels are very similar in both sampling techniques. When we consider the return period of the outlier 493.7 is nearly 3000 years. So we can't predict this return value using the above identified results shown in Tables 4.5 and 4.7. Therefore more sophisticated analysis is needed to establish its true return period.

### *4.4 Goodness of fit Test*

In order to test the fitness of the fitted distributions Gumbel and Exponential, the goodness of fit test was carried out. It was observed that empirical distributions agree with the theoretical distributions at the 5% level of significance. Summary of this analysis is given below.

Table 4.8 *Goodness of fit Test Results for Gumbel*

| Test | Test Statistic | Critical Value (5%) | Decision |
|---|---|---|---|
| Anderson-Darling | 0.237 | 0.787 | Not Significant |
| Cramer-von Mises | 0.033 | 0.126 | Not Significant |
| Watson | 0.031 | 0.116 | Not Significant |

Table 4.9 *Goodness of fit Test Results for Exponential*

| Test | Test Statistic | Critical Value (5%) | Decision |
|---|---|---|---|
| Anderson-Darling | 0.351 | 0.787 | Not Significant |
| Cramer-von Mises | 0.047 | 0.126 | Not Significant |
| Watson | 0.042 | 0.116 | Not Significant |

## Conclusions

In this study we have performed a statistical modelling of extreme daily rainfall over 110 years in Colombo, Sri Lanka using extreme value distributions under two sampling techniques. Even though the original series of annual maximum daily rainfall data fits the Frechet distribution, the distribution converges to the Gumbel distribution and the predicted values for different return periods and their confidence levels decrease following the removal of the single outlier identified using Grubb's test. Therefore the outlier is more important in this analysis.

We have established the Gumbel and Exponential distributions are suitable models for extreme daily rainfall by considering annual maximums of daily rainfall and daily rainfalls greater than 100mm and checked the adequacy of the models using the goodness of fit test. Finally, we have provided estimates of the return level of daily rainfall and the corresponding 95% confidence intervals for Colombo location in Sri Lanka. These estimates could be used as measures of flood protection.

This paper only provides an initial study of extreme daily rainfall in Colombo. This study can be extended in several ways. One way is to use distributions that are more flexible than the GEV and GPD, such as four parameter Lamda distribution. The other is a more sophisticated analysis of the actual return period of the identified outlier in order to assess its relevance for design.

## References


1. Vanniyasingam R. Baheerathan and Elizabeth M.Shaw , Rainfall Depth-Duration-Frequency Studies for Sri Lanka, Journal of Hydrology, 37 , 223-239, 1978.
2. G.T. Dharmasena and S.M. Premasiri, Rainfall Intensity Studies for Sri Lanka, Engineer, December, 1990.
3. Davison A.C and Smith R.L, Models for Exceedances over High Thresholds, Journal of the Royal Statistical Society, 52, No:3, 393-442, 1990.
4. Barbel Finkenstadt and Holger Rootzen, Extreme Values in Finance, Telecommunications, and the Environment, 5th Edition, 2001.
5. Enrique Castillo, Extreme Value Theory in Engineering, Statistical Modeling and Decision Science, Gerald L.Lieberman and Ingram Olkin, editors, Stanford University, Stanford, California.

## About the Authors

**N.VARATHAN,** B.Sc. Hons., Jaffna, M.Sc. Peradeniya, is a Lecturer at the Department of Mathematics and Statistics, University of Jaffna. His research interests are in the areas of Time Series and Regression Analysis.

**K.PERERA,** B.Sc. Hons., Sri Jeyawardenapura., M.Sc. New York, Albany, Ph.D. New York, Albany is a Senior Lecturer at the Department of Engineering Mathematics, University of Peradeniya. Her research interests are in the areas of Estimation, Regression Analysis and probability Theory.

**N.WIKRAMANAYAKE,** B.Sc Eng. Hons., Peradeniya, M.Sc. MIT, Ph.D. MIT, MIE (SL), CEng, is a Senior Lecturer at the Department of Civil Engineering, Open University of Sri Lanka. His research interests are in the areas of Hydrology and Coastal Engineering.

# RAIN WATER HARVESTING FOR WATER EFFICIENCY AND MANAGEMENT

[1]Ariyananda T, [2]Wickramasuriya S S, [3]Wijeyesekera D S

[1]Director, Lanka Rain Water Harvesting Forum, 28/3a, Subadrarama Lane, Nugegoda, Sri Lanka.
[1]E-mail: tanuja@sltnet.lk

[2]Department of Civil Engineering, University of Moratuwa.
Department of Civil Engineering, University of Moratuwa. Katubedda, Moratuwa, Sri Lanka.
[2]E-mail:sunilw@civil.mrt.ac.lk

[3]former Vice Chancellor, University of Moratuwa, former Vice Chancellor, Open University of Sri Lanka, Former Chairman, Institute of Construction Training and Development, Past President, Institution of Engineers, Sri Lanka. No. 7, Barnes Avenue, Mount Lavinia, Sri Lanka.
[3]Tel: +94 777312238, Fax: +94 112735596 ; E-mail: dayantha@sltnet.lk

**Abstract:** Water being one of the world's basic resources and one of the most essential needs to life, could be considered the nature's precious gift to the living being. Although this resource was available in plenty till recently, due to urbanization, increase in population, industrialization and for large scale enterprises the shortage of water in the world has become evident. Furthermore, due to implications of climate change on rainfall patterns, with extremities of weather giving rise to floods and droughts there is growing concern globally about appropriate strategies to be adopted as far as the built environment is concerned for proper management and harvesting of rain water. Thus globally a Millennium Development Goal has also been set and rain water harvesting has emerged as an important issue in the international scenario. It has been highlighted at the third World Water Forum held in Kyoto, Japan, in the context of Millennium Development Goals and the issue of sustainability, at the Global Ministerial Environmental Forum in Korea, which has led to formulation of many networks and policies.

The famous proclamation by King Parakramabahu the Great in (1153-1186 AD) could be considered as one of earliest policy statements, on water resources development and management in Sri Lanka, which highlights rain water harvesting. "Let not allow a single drop of water falling as rain flow into the sea without being used for the benefit of mankind". This shows the wisdom and commitment of ancient kings and people to conserve and efficiently manage water resources by building tanks specially in the dry zone and the design and construction of complex water collection and distribution systems such as in the Sigiriya rock fortress.

The Government of Sri Lanka in June 2005 accepted a "National Policy on Rain Water Harvesting & Strategies". Sri Lanka has used rain water for both domestic and agricultural purposes for many centuries and the institutionalized rain water harvesting became a practice in Sri Lanka in 1995, under the World Bank funded Community Water Supply and Sanitation Project. This project initiated the emergence of the Lanka Rain Water Harvesting Forum (LRWHF).

A major challenge is the need to have a delivery of the stored rain water, for which gravity flow and hand/manual pumping has been the economic option the use of Solar Energy is being promoted specially in rural areas where there is no main grid power available. Another challenge is the public Health concern due to the comparatively stagnant nature of rain water harvesting.

There has been a significant increase in the use of rain water harvesting in Sri Lanka, which has proved to be a boon to rural people, particularly for domestic water supplies in water scarce situations. An estimated thirty thousand systems are presently in operation, scattered over a large number of districts. Interestingly, several large scale projects have also been implemented in the urban context, and this too is likely to increase in the future. With a National policy on Rain Water Harvesting and other legislation in effect, Sri Lanka stands to benefit significantly by the appropriate use of this technology.

***Keywords:*** *Water efficiency and management*

## Abbreviations[12]:

12

| | | |
|---|---|---|
| ADRA | - | Adventist Development & Relief Agency Sri Lanka |
| Asia Onlus | - | ASIA ONLUS |
| BLIA | - | Buddha's Light International Association |
| CI | - | Care International |
| CWSSP | - | Community Water Supply & Sanitation Project (under the Ministry of Urban Development and Water Supply (**www.urbanlanka.lk/Agencies.htm#cwwsp**) |
| EC | - | Ekamuthu Cultivators |
| GTZ | - | GTZ |
| HKLM | - | HKL Menike |
| IOM | - | International Organization for Migration |
| ITDG | - | Intermediate Technology Development Group (Practical Action) |
| KOPBMO | - | Kala Oya River Basin Management Office |
| LRWHF | - | Lanka Rain Water Harvesting Forum |
| NCC | - | National Christian Council |
| NGOWSSDS | - | GO Water Supply and Sanitation Decade Service |
| NWS&DB | - | National Water Supply & Drainage Board (under the Ministry of Urban Development and Water Supply) (www.waterboard.lk) |
| ORDE | - | Organization for Resource Development and Environment |
| OXFAM | - | OXFAM |
| PALM | - | PALM Foundation |
| Plan | - | Plan Sri Lanka |
| PRDA | - | People's Rural Development Association |
| Sarvodaya | - | Lanka Jathika Sarvodaya Shramadana Sangamaya |
| SDA | - | Southern Development Authority |
| USIP | - | Urban settlement Improvement Project |
| WV | - | World Vision |

## 1. Introduction

Many water problems in the world can be attributed to the uneven distribution of rainfall in time and space. Extremities of weather give rise to floods and droughts, often causing considerable damage to life and property. Countries subject to monsoonal weather patterns such as Sri Lanka, can experience flooding after a prolonged dry spell or a period of drought. There is also a growing concern globally about the implications of climate change on rainfall patterns, and about appropriate strategies to be adopted as far as infrastructure is concerned for proper management of rain water. In addition, globally a Millennium Development Goal has also been set, which aims to halve the number of people who do not have access to safe drinking water by the year 2015. Thus rain water harvesting has emerged as an important issue in the international scenario and was highlighted at the third World Water Forum held in Kyoto, Japan in 2003. In the context of Millennium Development Goals and the issue of sustainability, the following conclusions are noteworthy:

- Harvested rain water is a major water supply option , as important as surface and groundwater
- That decentralized water utilization and resource management uses rain water harvesting for the sake of the people and the Earth

Furthermore, at the Global Ministerial Environmental Forum held in Korea in 2004, it was concluded that alternative and cost effective technologies, such as rain water harvesting, should be explored and promoted and the transfer of appropriate technology increased. Suggesting rain water harvesting as a new paradigm, Han (2004) suggests the building of a worldwide network to promote rain water harvesting.

## 2. Rain Water Harvesting Policy & Legislative Support

One of the earliest policy statements on water resources development and management in Sri Lanka, mentions rain water harvesting. The famous proclamation by King Parakrambahu the Great (1153-1186 AD), "......let not even a small quantity of water obtained by rain, go to the sea without benefiting man" (Arumugam, 1969, quoted from Mahawansa), shows the wisdom and commitment of ancient kings and people to conserve and efficiently manage water resources. The ancient tanks of the dry zone and the complex water collection and distribution system of the Sigiriya rock fortress bear ample testimony to this fact.

In June 2005, the government of Sri Lanka accepted a "National Policy on Rain Water Harvesting & Strategies" presented to the Parliament by Minister for Urban Development and Water Supply. The policy objective is aimed at encouraging communities to control water near its source by harvesting rain water. This results in, minimizing the use of treated water for secondary purposes, reduction of flooding, improving soil conservation and groundwater recharge, providing water for domestic use with adequate treatment, agricultural benefits and reduce energy consumption.

## 3. Rain Water Harvesting – Current Situation

Sri Lanka has used rain water for both domestic and agricultural purposes for many centuries. However, institutionalized rain water harvesting became a practice in Sri Lanka in 1995, under the World Bank funded Community Water Supply and Sanitation Project (CWSSP) which introduced rain water harvesting as a water supply option in two districts Badulla and Matara. (Heijnen and Mansur (1998).

This project initiated the emergence of the Lanka Rain Water Harvesting Forum (LRWHF), which is an NGO actively engaged in promoting rain water harvesting in the country. Since the CWSSP, a number of other organizations and institutions have adopted rain water harvesting as a means of supplying water to water scarce households in both the wet and dry zones. Some of the noteworthy contributions in rain water harvesting for domestic use have been made by , the Southern

Development Authority, the Dry Zone Development Project funded by IFAD (International Fund for Agricultural Development), Kandy, by National Water Supply and Drainage Board, 3rd and 4th ADB Water and Sanitation projects and the 2nd CWSSP project. During the last 13 years, LRWHF has actively promoted the concept of rain water harvesting in all districts through demonstration projects, awareness programs, training, research and development, and networking.

Presently, there are more than 30,000 domestic rain water harvesting systems recorded in 25 districts (Table 1). Most of these rain water harvesting systems are in rural areas and have been implemented through government projects or by NGO's.

Table 1: *Distribution of Rain Water Harvesting Systems in Sri Lanka*

| Province | District | No. of total rain water tanks | | |
|---|---|---|---|---|
| | | By LRWHF | By other organizations | |
| | | | No. of tanks | Organizations |
| Central Province | Kandy | 10 | 2663 | CWSSP |
| | Matale | | 994 | CWSSP |
| | Nuwara Eliya | 5 | 964 | CWSSP, PALM |
| Eastern Province | Ampara | 652 | 31 | CI |
| | Baticoloa | 11 | 36 | Asia Onlus |
| | Trincomalee | 19 | - | - |
| North Central Province | Anuradhapura | 13 | 3483 | Plan, KOPBMO, ITDG, BLIA, NWS&DB |
| | Polonnaruwa | - | 1096 | NWS&DB, NCC |
| North Western Province | Kurunegala | 51 | 577 | GTZ, NWS&DB, Sarvodaya, Plan |
| | Puttalam | 14 | 1652 | ORDE, PRDA, NWS&DB |
| Northern Province | Jaffna | 14 | - | - |
| | Kilinochchi | 09 | - | - |
| | Mannar | 11 | 98 | IOM |
| | Mulathivu | 03 | - | - |
| | Vavunia | 48 | 66 | WV, IOM |
| Sabaragamuwa Province | Rathnapura | - | 111 | EC, HKLM |
| | Kegalle | 8 | 1664 | NWS&DB |
| Southern Province | Galle | 1397 | - | - |
| | Hambanthota | 1107 | 2811 | Sarvodaya, WV, ADRA, OXFAM, NWS&DB, ITDG, SDA |
| | Matara | 629 | 1089 | CWSSP |
| Uva Province | Badulla | 1 | 5488 | CWSSP |
| | Moneragala | 40 | 1904 | Sarvodaya, NWSDB, SDA, ITDG |
| Western Province | Colombo | 5 | 41 | USIP |
| | Gampaha | 1 | 23 | EC, CWSSP |
| | Kalutara | - | 1443 | NGOWSSDS, Asia Onlus, NWS&DB |
| Total | | 4048 | 26234 | |
| Grand total | | | 30282 | |

## 4. Water Quality Issues

Rain water is one of the purest sources of water available as it does not come into contact with many of the pollutants often discharged into local surface waters. It comes free and can be used to supply potable (drinkable) water and non-potable water. If collected properly, it can be used for all domestic purposes including drinking.

Rain water from well managed roof catchment sources is generally safe to drink without treatment. Except in heavily urbanized and industrialized areas or regions adjacent to the volcanoes, atmospheric rain water is pure. Any contamination of the water usually occurs after contact with the catchment system (roof). Regular cleaning and inspection of the catchment area and gutters are important to ensure good quality water (Heijnen and Pathak, 2006). Further treatment through boiling, exposure to sunlight and chlorination can be undertaken if there are concerns about the water quality. Insects breeding inside the tank can be prevented, by keeping the storage tanks and other openings sealed. Awareness and education are the two most important strategies to prevent water pollution.

## 5. Rain Water Harvesting in Urban Buildings

Households in urban areas use pipe borne water not only for drinking and cooking, but also for gardening, car washing and also other all activities. A close examination of Table 2 shows that apart from drinking and cooking, there is immense potential to utilize rain water to supplement household water supply for non-potable requirements, thereby reducing the use of treated pipe borne water. Thus authorities will be able to supply pipe borne water to more households.

In view of the existing constraints faced by the authorities in meeting the increasing demand for water, it is vital that rain water harvesting be used as a new source of water in urban areas in which pipe borne water consumption is very high.

A study indicates that on average in low income households in Sri Lanka, if **30%** of the monthly water requirement was met by rain water then a **34%** reduction in water bills can be obtained (Ariyananda & Gunasekara, 2004). In middle income households, if **30%** of the monthly water requirement is met by rain water, then the monthly water bill can be reduced by **61%** at the present water rates.

The potential of rain water harvesting for large housing projects as a supplementary source and the required structural measures to be adopted have been studied and presented by Jayasinghe (2004).

Table 2: *Average amount of water used per family per day (Ariyananda & Gunasekara, 2004)*

| Water use Activities | Low Income | | High Income | |
|---|---|---|---|---|
| | Ave. Liters per day per family | % | Ave. Liters per day per family | % |
| Drinking | 24 | 4 | 20 | 2 |
| Cooking and Washing pans | 90 | 15 | 100 | 11 |
| Washing clothes | 127 | 21 | 147 | 17 |
| Toilet use | 140 | 23 | 150 | 17 |
| Bathing and Washing face | 163 | 26 | 257 | 29 |
| Gardening | 43 | 7 | 117 | 13 |
| Other | 30 | 5 | 90 | 10 |
| **Total** | **617** | **100** | **880** | **100** |

Rain water harvesting in urban areas has many functions. It can supplement pipe borne water for non-drinking purposes thus conserving pipe borne water; reduce energy cost of pumping, and also reduce flooding. Rain water collection in commercial and public buildings has particular advantages resulting from large roof areas.

Various technical problems faced were overcome with the assistance of the University of Moratuwa and the Institute for Construction Training and Development (ICTAD).

## 6. Rain Water for Drought Mitigation and Recharging Groundwater

### *6.1 Domestic Use*

Rain water harvesting has brought much relief to people during times of drought, water scarcity and recently to those affected by the devastating tsunami of 2004.

Even though Sri Lanka has a relatively high rainfall, it varies both temporally and spatially. Some areas can experience extreme dry spells between monsoons or on occasions a total failure of the monsoons. Several dry zone districts of Sri Lanka experience prolonged drought, causing tremendous hardships to people. Rain water harvesting systems constructed in Hambantota, Moneragala and Anuradhapura were able to use rain water stored in the tanks for as long as 5 -6 months during this period. (Ariyanbandu & Aheeyar, 2000)

Research done by Kumari (2008) on rain water harvesting for rural water supply shows that a 5 cu.m. tank with a roof area between 75 to 100 sq.m. can supply 300 liters/ day to a household, with an overall probability of success of 50%, in the districts of Anuradhapura, Hambantota and Puttalam. However this figure will reduce depending on the season. Substantially higher degrees of success can be obtained within the wet and intermediate zones (Ranasinghe, 2008). Further work on appropriate tank capacities for rain water harvesting in the Jaffna district has been done by Gamage (2006).

### *6.2 Agricultural Use*

The rural sector in Sri Lanka constitutes around 80% of the population and most of those in the rural sector depend on rainfall-based sources of income, such as agriculture, livestock production and inland fisheries. Freshwater availability is a key limiting factor in food production and improvement of livelihood.

Lack of a dependable water supply is a major limiting factor in our attempts to develop the rural sector. From the total rainfall, on average around 50% of rain water is lost in the form of surface run-off and conserving this water will promote crop growth in areas where water is limited. The most effective and economical method of conserving this water is by storing it in surface tanks which are abundant in the Dry Zone. However, many small tanks are dilapidated and/or silted and need rehabilitation.

If the run-off water is stored in the land itself, it would be available to plants when there is a water shortage. In some parts of the dry zone, small ponds called "Pathahas" have been used to collect and store rain water. Such a water collecting system on farm would enable farmers to cultivate crops during the dry seasons.

***Fig. 1: Run off tank at Kurundamkulama***

A study was carried out in Kurundamkulama (a village in Mihintale in Anuradhapura District) to harvest/collect run-off rain water in tanks. The maha rains were collected in 5 m3 run off tank (Figure 1). Water collected was used during Yala for crop production. As a result the incomes of the families in the study increased substantially (Weerasinghe et al. 2005). Collection of run- off rain water not only conserves water but also reduces soil erosion and degradation of the land.

### *6.3 Recharging Groundwater*

Water lost from the ground by way of evaporation, tube wells etc, needs to be replenished. Collecting rain water in ponds and pools in a manner where water percolates in to the ground raises the water table. A study conducted in Nikaweratiya on the use of *pathahas* (Figure 2) (Shanthi de Silva, 2005), shows that these elevate the ground water level, thus increasing the quantity of water available for both domestic and agricultural use even during the dry season.

***Fig. 2: "Pathaha" at Nikawaratiya***

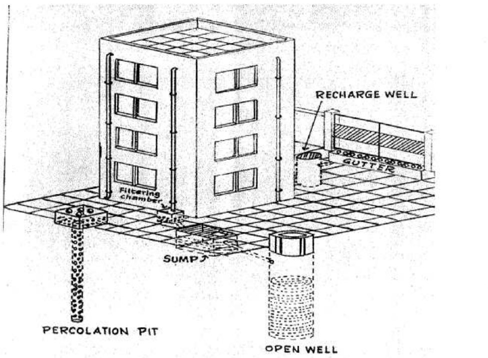



***Fig. 3: Recharging structure (Raghavan 2006)***

In several Indian states, ground water recharging in urban areas through recharging structures (Figure 3) is encouraged and legalized to increase the exploitable quantity of groundwater, improve the quality of groundwater and to mitigate flooding.

## 7. Social & Economic Aspects

As with any new technological intervention, rain water harvesting too needs changes in the attitudes, perceptions and behavior of the community if the new technique is to be successful in terms of social, economic, cultural and environmental factors. Training and awareness are key factors to ensure quality construction, proper operation and maintenance, management of harvested water, and to change myths, attitudes and wrong perceptions of the concept of rain water harvesting.

Community mobilization and training of beneficiaries are vital components, since rain water tanks will ultimately be managed at private cost at the household level. Community contribution towards the project is recommended in order to increase the sense of ownership and motivate people towards sustainable management. The beneficiaries can supply unskilled labour and local materials easily towards the project. The contribution of unskilled labour alone provides almost 15% of the total value of the system.

Institutional and commercial level rain water harvesting in schools, government offices, hospitals and other public places is highly recommended due to their large roof areas, but proper institutional arrangements are vital for sustainable operation and maintenance.
One of the major disadvantages of roof rain water harvesting technology is that it requires a higher capital investment initially for the construction of storage cisterns and other supplementary components. The cost is much higher when the rainfall is low and there is a longer dry period, which results in the need for a larger cistern to ensure water security.

The success of any technological intervention depends on the cost and the affordability of the users. Therefore the use of an appropriate tank size and use of less and cheaper materials, less labour and simple construction aids are important factors to reduce the cost of construction.

Table 3: *Cost estimates for different sizes of Ferro cement tanks (Aheeyar, 2009)*

| **Cost of cistern** | **5 $m^3$** | **7 $m^3$** | **8 $m^3$** | **10 $m^3$** |
|---|---|---|---|---|
| Material | 24,305.50 | 26,919.50 | 29,719.50 | 35,370.50 |
| Skilled labour | 4,500.00 | 5,400.00 | 5,580.00 | 8,000.00 |
| Unskilled labour | 5,000.00 | 5,500.00 | 6,000.00 | 10,000.00 |
| Transport of materials | 2,500.00 | 2,500.00 | 2,500.00 | 2500 |
| Reusable Frame/Miscellaneous | 1427.5 | 1,500.00 | 1595 | 1750 |
| | **37,733.00** | **41,819.50** | **45,394.50** | **57,620.50** |
| Pipes, First flush, Gutters (26 feet), other accessories & fixing charges | 5,000.00 | 5,000.00 | 5,000.00 | 5,000.00 |
| **Total** | **42733.00** | **46,819.50** | **50,394.50** | **62,620.00** |

***Fig. 4: Under ground tank***

***Fig. 5: Partial under ground tank***

***Fig. 6: Above ground Ferrocement tank***

Rain water harvesting system tanks can be placed above ground, underground (Figure 4-6) or partially underground.

According to the past research (Thomas and Rees, 2001), the unit cost of construction of rain water tanks shows a negative relationship with increasing size of the cistern. It is cheaper to go for a larger tank and to avoid using two smaller tanks and larger communal systems compared to small individual units.

High initial cost has been a prohibitive factor for many poor households in adopting rain water harvesting systems, though they are willing to collect rain water for their household needs. Therefore, some supportive mechanisms such as loans and subsides can be effectively used to promote the technology among poor families. Use of subsides in the past has shown positive results in introducing rain water harvesting systems among rural poor (Gould and Petersen, 1999).

Some of the social and economic benefits identified by households using rain water harvesting systems are;

- Easy access to clean drinking water
- Less time spent on collecting water
- Time saved ( average 1.5 hrs per day) is used for social and economic activities
- Skills enhancement in the village
- Less reliance on external water providers
- More water security at household level
- Better sanitation due to more water availability
- Enhanced income through use of rain water for home gardening, animal rearing and brick making etc.
- Reduction in diarrheal disease
- Better quality water, especially in areas with high levels of Fluoride in ground water, saline water (after the tsunami) and brackish water.

## 8. Energy and Efficiency

One of the major advantages of rain water harvesting systems is their minimal energy consumption in operation. No system is viable and sustainable if it is not energy efficient in the context of the global energy crisis. i.e., escalation of energy costs as well as global warming and its consequences as a result of fossil fuel burning. Therefore, for RWH to be viable and to fall within the context of sustainable development, harvested water should be utilized spending minimum energy and avoiding energy derived from fossil fuel burning as much as possible.

Research studies undertaken by Sendanayake (2009) indicate that providing a means of low cost, low energy consuming method of supplying potable water from a natural phenomenon not only will enhance the quality of life of the population, but also will integrate them to the development on a sustainable scale more readily.

In considering parallels and absorbing new technological advances from global experiences, it is possible to supply and/or supplement existing water supply systems giving considerations to simple, practical and low cost RWH systems having optimum capacities and high water saving efficiencies (WSE), which can provide reasonable quality water.

By positioning the storage facility of the RTRWH system at a higher elevation to the end user point, water can be drawn-off by gravity thereby eliminating the need for pumping, hence the energy issues. While gravity operated systems do not consume energy and are suitable primarily for external use such as garden watering, many limitations hinder wide use of such systems in more sophisticated situations.

Of course there are limitations of gravity operated tanks such as the additional cost involved in constructing support structures. Extra space required for such structures, in certain instances restricting land use, dependence of end user point water pressure on the height of the supporting structure and poor aesthetic value for the location.

When gravity flow is not feasible, various pumping options of Water from storage facility would be necessary, such as, hand pumps, centrifugal pumps and positive displacement pumps.

While hand pumps are the most widely used in rural Sri Lanka, it can also be classified as positive displacement pumps (see figure 7), but are operated manually thereby limited to small scale draw-offs. Centrifugal pumps, working on the principle of creating a vacuum for suction by rotating an impeller at high speed, are the most widely used pumping option. However, the high starting torques required, low pumping heads and low pump efficiencies are the main draw backs of centrifugal pumps thereby needing higher energy input.

By considering the alternative energy sources, such as solar power, wind power and bio gas, solar power seems the most suitable for tropical climates, given the abundance of sun throughout the year as well as the relative low cost of components compared to wind turbines, apart from the durability and the viability in domestic usage. Hence, for RWH systems to be of self-sustaining and eco friendly nature, solar pumping of harvested water is important and development of viable, low cost solar pumping devices are vital.

Unlike in centrifugal pumps where a number of variables dictate the overall efficiency, only the volumetric efficiency (which is governed by the cylinder-piston assembly design), indicate the overall efficiency of a PD pump. As such PD pumps are typically 15% to 20% more efficient than a centrifugal pump of similar power rating. The capability of delivering at low pump speeds is one of the major advantages displayed by the PD pump.

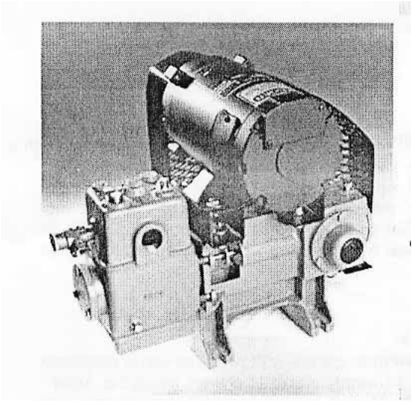


***Fig. 7: An advanced positive displacement pump***

This is in contrast to a centrifugal pump which needs a minimum rotational speed for the impeller output head to overcome the static head . Therefore, PD pumps can be used in low power input situations as well as when the power input is variable.

As such delivery reliability of PD pumps are much higher in Photo Voltaic (PV) pumping situations. Although centrifugal pumps are superior in higher output flow rates, PD pumps are much superior in achieving higher delivery heads at low output rates. Therefore, in overcoming higher heads, such as that found in multi-story building situations, multi-staging is not required compared to the case of using centrifugal pumps.

By considering the alternative energy sources, such as solar power, wind power and bio gas, solar power seems the most suitable for tropical climates, given the abundance of sun throughout the year as well as the relative low cost of components compared to wind turbines, apart from the durability and the viability in domestic usage. Hence, for RWH systems to be of self-sustaining and eco friendly nature, solar pumping of harvested water is important and development of viable, low cost solar pumping devices are vital. The best type of pumps for PV powered RWH situations are PD pumps due to their higher efficiency and low starting torque. These characteristics allow down-sizing of the PV array and hence the cost. Since RWH systems in households require a limited quantity of collected rain water to be pumped up per day, a pump with a low delivery rate (say 1-2 L/min) would be sufficient. Such pumps are available at 80 – 100 W rating, usually double acting and piston type PD pumps. The total cost of a complete system inclusive of the battery can be around Rs.100,000 - 120,000.

In sizing solar pumps, PV technology can be effectively used to pump harvested rain water to service points. The selection of a suitable PV array for pumping of collected rain water is in the following manner.

The hydraulic energy required (kWh/day)

$$= \text{volume required } (m^3/\text{day}) \times \text{head } (m) \times \text{water density} \times \text{gravity}/\ (3.6\text{x}10^6)$$

$$= 0.002725 \times \text{volume } (m^3/\text{day}) \times \text{head } (m)$$

$$\text{The solar array power required } (kW_p) = \frac{\text{Hydraulic energy required (kWh/day)}}{\text{Av. daily solar irradiation } (kWh/m^2/\text{day} \times F \times E)}$$

Where, F = array mismatch factor = 0.85 on average and
E = daily subsystem efficiency = 0.25 – 0.40 typically

To ensure uninterrupted power supply a deep cycle battery (Usually a lead-acid type) can be integrated to the system via a charge controller thereafter the battery size can be determined according to the load. The load, in this case the pump, has to be of high efficiency type to reduce the cost on PV array. PD pump with a DC motor is ideal for its overall efficiency and the requirement of low starting current can be operated under low light conditions.

## 9. Conclusions and Recommendations

Even though Sri Lanka presently has no water scarcity except in some areas during the dry season, due to increase in population, urbanization, pollution of water sources and climate change issues, it may face water problems in the future. Adopting rain water harvesting and utilizing it to the maximum will help the Country to overcome either all or some of these problems.

- Using rain water for drinking purposes should be encouraged in dry zone districts where the groundwater is both mineralized and contaminated, especially in areas where a high incidence of kidney problems due to polluted ground water has been reported.
- Rain water harvesting should be encouraged as a supplementary water source in urban areas to reduce water bills, save on energy, save on water treatment costs and to reduce flooding in some areas.
- To encourage householders to adopt rain water harvesting by offering incentives such as tax rebates.
- Incorporate rain water harvesting in all public and commercial buildings with large scale use of pipe borne water.
- Potential areas for recharging should be identified and encouraged.
- Rain water harvesting system components for urban houses should be made available locally.
- Professionals should use innovative designs incorporate rain water harvesting in new buildings.
- The delivery of harvested water would have to be made depending on the energy sources available and in particular in rural areas where mains grid power is not currently available, alternative energy sources specially Solar Energy should be harnessed with appropriate pumping mechanisms.

**Acknowledgements**

The compilation of this paper has been mainly a collective effort of Dr Tanuja Ariyananda and Prof Sunil Wickramasuirya, while Prof Dayantha Wijeyesekera collated most of the information. The contributions made by Prof Thishan Jayasinghe, Prof Rahula Attalage and Mr. Susuri Sendanayake are gratefully acknowledged. The work done in this field and the data made available from the Lanka Rain Water Harvesting Forum are greatly valued not only in the compiling of this paper but also by the general public in Sri Lanka.

**References**

Aheeyar M.M.M (2009) Economic and Social Aspects of RWH Systems. In Ariyananda T. (Ed). Rain Water Harvesting Practitioners Guide for Sri Lanka. *Lanka Rain Water Harvesting Forum.*

Ariyanbandu R. De S. and Aheeyar M.M.M. (2000) Evaluation of the Effectiveness of Rain Water as Domestic Water source in Rural Sri Lanka. *Lanka Rain Water Harvesting Forum*

Ariyananda T. and Gunasekara J. ( 2004) Study of potential Feasibility of Roof Rain Water Harvesting in Urban Area. *Proceeding of 7th Symposium of Lanka Rain Water Harvesting Forum, Colombo, Sri Lanka.*

Ariyananda T. and Wickramasuriya S.S.(2009) Rain Water Harvesting In Sri Lanka. Economic Review: Water & Development. Vol. 35, Nos 3&4 (June/July)

Arumugam, S. (1969). Water resources of Ceylon. *Water Resources Board, Sri Lanka.*

Gamage, N.P.D. (2006). Guidance on use of rain water tanks for the Jaffna Peninsula. *Annual Transactions, Institute of Engineers, Sri Lanka.*

Gould, J. and Petersen, E.N. (1999). Rain water catchment systems for domestic supply: Design, construction and implementation. *Intermediate Technology Publications Ltd., London.*

Han, M. (2004). Rain water harvesting: A new paradigm to meet MDG and sustainability. Rain Water Harvesting Workshop, *IWA 4th World Water Congress and Exhibition, Morocco.*

Heijnen, H and Mansur U. (1998) Rain water harvesting in the community water supply and sanitation project. Proceeding of the Symposium on *Rain Water Harvesting for Water Security Feb. 1998.* Lanka Rain Water Harvesting Forum. *Publication* Sri Lanka. P53-56

Heijnen, H. and Pathank N. (2006) Rain Water Quality Health and Hygiene Aspects. Proceeding of International Workshop in Rain Water Harvesting, Kandy, Sri Lanka

Jayasinghe, M.T.R. ( 2004). Rain water harvesting to supplement water for large housing projects. *Annual Transactions, Institute of Engineers, Sri Lanka.*

Kumari, D.M.C.S. ( 2008). Drought mitigation using rain water harvesting in the dry zone of Sri Lanka. *M.Eng. dissertation.* University of Moratuwa.

National Rain Water Harvesting Policy & Strategy, Ministry of Urban Development and Water Supply, Government of Sri Lanka.

Ranasinghe, R.K.W.K. (2008). Improved water conservation practices using rain water harvesting in the wet and intermediate zones of Sri Lanka. *M.Eng. dissertation.* University of Moratuwa.

Raghavan S. (2006) Rain Water Harvesting – Need, Relevance And Importance Of Groundwater Recharge In Urban Areas With Particular Reference To Coastal Cities. Proceeding of International Workshop in Rain Water Harvesting, Kandy, Sri Lanka

Sendanayake, S. (2007). *Development of solar powered pumping for the integration of rain water harvesting in multistorey houses.*

Shanthi de Silva (2005) Impact of Artificial Recharging of Groundwater with Rain Water: a case Study in Kotewehera of Sri Lanka, *Lanka Rain Water Harvesting Forum, Sri Lanka*

Thomas, T. and Rees, D. (2001) New technology for roof rain water harvesting. *Rain water harvesting Workshop, Indian Institute of Technology New Delhi, India*

Weerasinghe P.A, Ariyananda T.N, Weeraratna C.S. (2005) Rain Water Harvesting for Home Gardens in Dry Zone of Sri Lanka. *Proceeding of XII IRSCA Conference, New Delhi, India*

## Author Names and Affiliations

Dr Tanuja Ariyananda, Director, Lanka Rain Water Harvesting Forum,
28/3A, Subadrarama Lane, Nugegoda, Sri Lanka Tel: +94 115 524612 email: **tanuja@sltnet.lk**

Prof. S.S. Wickramasuriya, Department of Civil Engineering, University of Moratuwa. Katubedda, Moratuwa, Sri Lanka + 94 11 2650 567/8 ext 2015 email:sunilw@civil.mrt.ac.lk

and Corresponding Author

Prof. Dayantha Wijeyesekera, Chairman, Tertiary & Vocational Education Commission,
No 7 Barnes Av Mt Lavinia, Sri Lanka.+94 777312238 email: **dayantha@sltnet.lk**

# SUSTAINABLE GREEN BUILDING RATING SYSTEMS FOR ENERGY EFFICIENCY FOR NEW RESIDENTIAL BUILDINGS IN US

R. Purasinghe[1], E. Maguino[2]

[1]Professor, Department of Civil Engineering, California State University, Los Angeles.California,
[1]E-mail: rpurasi@calstatela.edu
Telephone: (1) 323-343-4459; Fax: (1)323-343-6316

[2]Graduate Student, Department of Civil Engineering, California State University, LA, .California.

**Abstract:** The Building Industry places a high demand on world's resources and there by impacting the environment. Currently Engineers are paying attention to sustainability based design concepts rather than traditional cost effective design concepts for buildings. In US there are about seventy five million residential buildings as opposed to mere five million commercial buildings. As such this paper addresses residential buildings as opposed to commercial buildings. In US there are several metrics to measure eco friendliness for homes. Three such systems are National Association of Home Builders Model Green Building (MGB) Guidelines, National Association of Home Builders Green Building Standard (ICC 700-2008) and United States Green Building Council's LEED (Leadership in Energy and Environmental Design). The Model green building guidelines address, seven criterions, lot design, resource/water/energy efficiency, indoor environmental quality, homeowner education, and global impact in determining sustainability. The ICC 700 uses very similar six criterions, lot design, resource/energy/water efficiencies, indoor environmental quality, operation, maintenance, and building owner education. The LEED recognizes sustainability with eight criterions of Innovative design process, location and linkage, sustainable sites, water efficiency, energy efficiency, materials selection , indoor environmental quality, and awareness and education. This paper discusses two rating systems used in US for eco friendly building design for **homes** with special emphasis on **effects of energy efficiency in rating values.** For energy efficiency requirements of International Energy Conservation Code is also addressed. Using a new home in California as an example the paper describes how to evaluate green building ratings for energy efficiency using three software tools for the MGB and GBS for homes in US.

***Keywords:*** *Energy Efficiency, Green Building Ratings, National Association of Home Builders, Green Building Standard*

## 1. Introduction

The Building Industry places a high demand on world's resources and there by impacting the environment. According to the Environmental Protection Agency (EPA), the buildings in US accounts for about 40 % of annual energy use, 12 % of water consumption, and 88 % of electricity consumption [1]. Also buildings are known to be responsible for about 38 % of carbon dioxide emissions and 20-30 % of landfill deposits in US [1]. Austin, Texas is considered the birthplace of green buildings in US having developed a green building program in '90s towards conserving energy [2]. Since economic benefits were readily apparent, other cities, states and the US Government realized the economic and environmental benefits of developing green building technology. Currently there are many programs in US which can rate a building project for sustainability and environmental impact [1]. Many of them have a rating tool with multiple thresholds for commercial and/or residential new and/or retrofit construction. Many of them also address the land, water, materials, and air quality and energy issues. These programs [1] are ASHRE, Green Global Design, ICC 700 NGBS, NAHB Model Green Home Building Guidelines, Green Globes online, DOE Energy Star Qualified Home, and USGBC LEED (for new construction, commercial interiors, existing buildings, homes, schools, retail, healthcare, and neighborhood development).

The basic principles of the following programs related to residential buildings were described in an earlier paper [3].

i) Model Green Building (MGB) Guidelines by National Association of Home Builders(NAHB)
ii) National Green Building Standard (NGBS) by National Association of Home Builders(NAHB)
iii) Leadership in Energy and Environmental Design (LEED) Programs by The United States Green Building Council (USGBC)

The current paper describes **the analysis of Energy Efficiency on the green building rating systems of MGB Guidelines and NGBS using an example building.** Both of these require Energy efficiency compliance with International Energy Conservation Code (IECC) [4] or State approved similar standard in addition to other requirements. For this study IECC 2009 is used.

## 2. The Example Building

The example building is a new two story single residence with attached two car garage. The project site is situated on a hill side in Monterey Park, a suburban of Los Angeles, California. The first floor living area is 1,638 square feet which includes the living room, family room, dining room, kitchen, and bathroom. The second floor living area is 1,358 square feet which includes three bedrooms and two bathrooms. The total living area is 2,996 square feet on a 7,504 square feet lot. The square footage of the house is above the typical average for this area.

## 3. International Energy Conservation Code (IECC®) 2009 Compliance Study

The IECC® deals with new construction, remodeling, window replacement etc. The code addresses both residential and commercial buildings. The code ensures energy efficient building envelopes and energy efficiency of elements of buildings that are not part of building envelope. The Residential portion of the Code deals with buildings that are three stories or less. Currently the State of California requires the buildings to be in compliance with IECC. For checking the IECC compliance the code refers to tools such as ResCheck TM [5] a software tool for detailed plan analysis of the building for energy efficiency.

RESCheck considers the energy ratings of windows, doors, walls, ceiling, floors etc. RESCheck examines the effect of different levels of insulation (R value), window U-values and solar heat gain coefficient (SHGC) factors, and space conditioning equipment efficiencies to identify a cost-effective system. R value is an indication of thermal resistance, higher values indicating higher thermal resistance, thus energy savings. The U value measures the conduction of heat and is the reciprocal of the R value. These R and U values are set by the American Society of Heating, Refrigerating, and Air-Conditioning Engineers. The SHGC factor is a measure of effectiveness of window glazing in minimizing solar heat. SHGC is used by the National Fenestration Rating Council (NFRC). REScheck program calculates an overall UA (U*A) of a building by first multiplying U factor of each element (such as a window) by its area and then summing U*A for all the components in the building. The program then compares this value against code given maximum values. If the total heat loss (represented as a UA) of your building does not exceed the total heat loss allowed for the same building per the code, then the building confirms to the IECC code. A partial view of the RESCheck results for the example building is shown in Figures, 1, 2 and 3. After running RESCheck software the example residence is 24% above IECC 2009 requirements and hence achieves compliance.

**Compliance: Passes using UA trade-off**

**Compliance:** **Maximum UA: 776** Your UA: **590** Maximum SHGC: **0.30** Your SHGC: **0.30**

| Assembly | Gross Area or Perimeter | Cavity R-Value | Cont. R-Value | Glazing or Door U-Factor | UA |
|---|---|---|---|---|---|
| **Ceiling: Flat or Scissor Truss** | 2996 | 22.5 | 22.5 | | 66 |
| **Wall - 1st Floor: Wood Frame, 16in. o.c.**<br>**Orientation: Front** | 369 | 10.0 | 10.0 | | 18 |
| **Window: Vinyl Frame, Double Pane**<br>**SHGC: 0.30**<br>**Orientation: Front** | 45 | | | 0.300 | 14 |
| **Wall - 1st Floor: Wood Frame, 16in. o.c.**<br>**Orientation: Back** | 428 | 10.0 | 10.0 | | 19 |
| **Window: Vinyl Frame, Double Pane**<br>**SHGC: 0.30**<br>**Orientation: Back** | 54 | | | 0.300 | 16 |
| **Door: Solid**<br>**Orientation: Back** | 40 | | | 0.500 | 20 |
| **Wall - 1st Floor: Wood Frame, 16in. o.c.**<br>**Orientation: Right Side** | 504 | 10.0 | 10.0 | | 22 |
| **Window: Vinyl Frame, Double Pane**<br>**SHGC: 0.30**<br>**Orientation: Right Side** | 72 | | | 0.300 | 22 |
| | 40 | | | 0.500 | 20 |

*Figure 1: Partial View of REScheck Software Analysis for Energy Efficiency*

Passes UA +24.0% *Your UA:* 590.0 *Max. UA:* 776.0

*Figure 2: Partial View of Recheck Energy Efficiency Certification*

**2009 IECC Energy Efficiency Certificate**

| Insulation Rating | R-Value |
|---|---|
| Ceiling / Roof | 45.00 |
| Wall | 20.00 |
| Floor / Foundation | 20.00 |
| Ductwork (unconditioned spaces): | _____ |

| Glass & Door Rating | U-Factor | SHGC |
|---|---|---|
| Window | 0.30 | 0.30 |
| Door | 0.50 | 0.30 |

*Figure 3: View of REScheck Energy Efficiency certificate*

## 4. Model Green Building (MGB) Guidelines Compliance Study

First published in 2005, National Association of Home Builders (NAHB) model green building guidelines [6] were intended to be a baseline so that members could develop local green building programs. The MGB guidelines were written only for single family new construction. The guidelines consider seven factors in Table 1, to determine three levels of ratings Bronze, Silver, and Gold.

For each category of seven, a score is assigned based on satisfying several requirements. For the above seven categories, there is a minimum number of points required for the three levels of ratings mentioned above. This is to ensure that the all aspects of green building principles are addressed to some extent. In addition there is a minimum total point score required for each level of achievement which is depicted in Table 1.

The National Association of Home Builders has made available to public a user friendly online scoring tool at http://www.nahbgreen.org/ScoringTool/ free of charge. A partial view of the MGB online scoring tool is shown in Figure 4.

*Table 1: Threshold Point Ratings for Green Buildings per Model Building Guidelines Applicable for Single Family Homes*

| Threshold Point Ratings for Green Buildings | | | | |
|---|---|---|---|---|
| Green Building Categories | | Performance Point Levels | | |
| | | BRONZE | SILVER | GOLD |
| 1. | Lot Design, Preparation, and Development | 8 | 10 | 12 |
| 2. | Resource Efficiency | 44 | 60 | 77 |
| 3. | **Energy Efficiency** | **37** | **62** | **100** |
| 4. | Water Efficiency | 6 | 13 | 19 |
| 5. | Indoor Environmental Quality | 32 | 54 | 72 |
| 6. | Operation, Maintenance, and Home Owner Education | 7 | 7 | 9 |
| 7. | Global Impact | 3 | 5 | 6 |
| 8. | Additional Points from any category | 100 | 100 | 100 |
| Total Points | | 237 | 311 | 395 |

Source: Per information at http://www.nahbgreen.org/ScoringTool/

The Energy Efficiency ratings are detailed in Chapter 3 of the guidelines. The tool queries many aspects of the building including quality of duck system work, exhaust system of bathrooms, water heater, insulations, energy star labeled appliances, energy star lighting packages, and compliance with International Energy Conservation Code (IECC) [4].

As seen in the Figure 5, the section 3.1.1 of the MGB tool queries if the building is equivalent to IECC or local energy code. As depicted similar compliance checks are performed as part of MGB guideline checks.

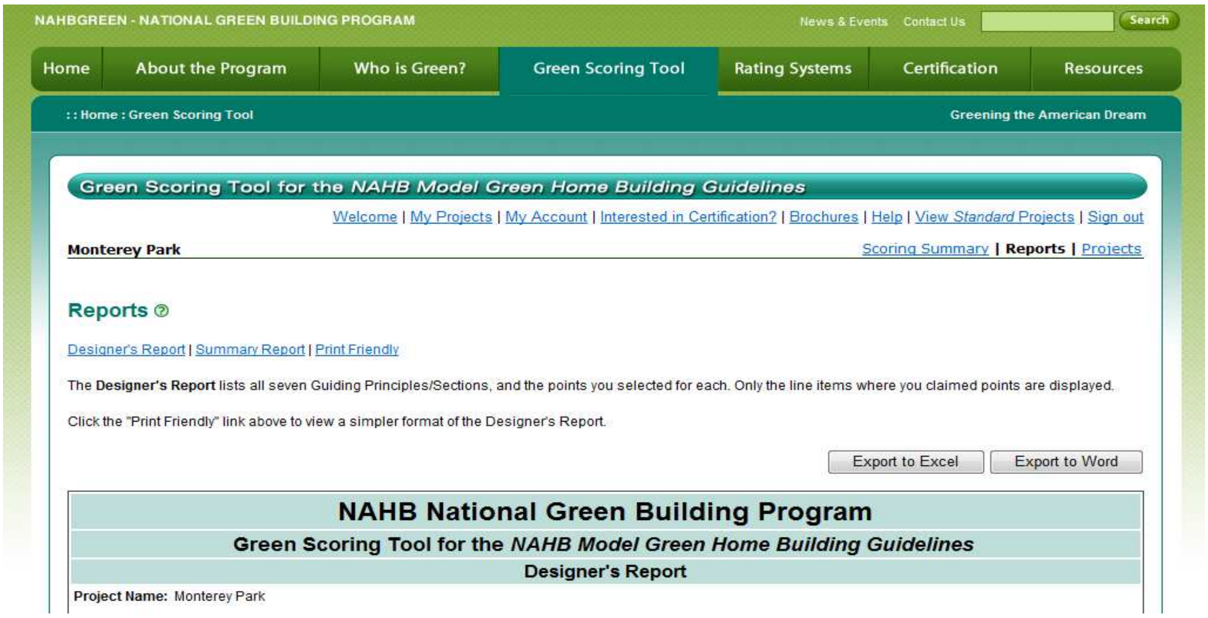



*Figure 4: Partial View of Green Building Scoring Tool per MGB Guidelines*
*(Per information at http://www.nahbgreen.org/ScoringTool/)*

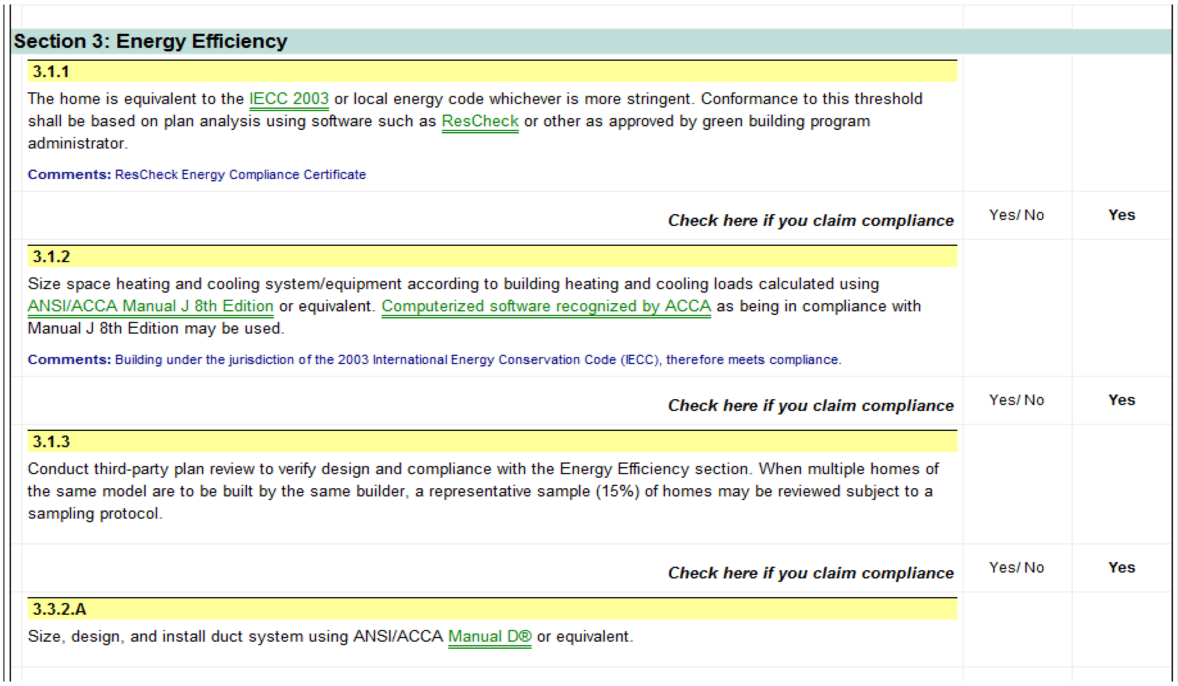



*Figure 5: Partial View of Green Building Scoring Tool per MGB Guidelines*
*(Per information at http://www.nahbgreen.org/ScoringTool/)*

After analysis of many other factors such as insulation of hot water lines, heat traps between hot and cold water lines, use of ENERGY STAR® advanced lighting packages, and ENERGY STAR® appliances, and allocating points the tool come up with a total score of 75 points for Energy Efficiency of the building. Based on total points claimed for Section 3 (Energy Efficiency) the building qualifies as a Silver level rating (minimum 62 points) for Energy efficiency of the building.

| Total Points Claimed in Section 3: Energy Efficiency | 75 | | Bronze | Silver | Gold |
|---|---|---|---|---|---|
| | | Required Points | 37 | 62 | 100 |
| | | Additional Points | 96 | 71 | 33 |

*Figure 6: View of Total Score for Energy Efficiency per MGB Guideline Tool*

## 5. National Green Building Standard (NGBS ICC 700-2008) Compliance Study

This standard was developed by expanding the Model Building guidelines to include single family/multifamily homes, residential remodeling, and site development. The American National Standards Institute (ANSI) approved this standard as International Code Council (ICC) 700-2008 [4] in 2009. Thus this becomes a consensus developed option. The standard considers the six factors in Table 2 to come up with four levels of ratings **Bronze, Silver, Gold, and Emerald**.

Similar to the MGB guidelines there is a minimum number of points required in each of the above categories for each level of ratings. As shown on the NAHB web site (www.nahbgreen.org/Guidelines/ansistandard.aspx ), the threshold values are as follows

*Table 2: Threshold Point Ratings for Green Buildings per National Green Building Standard (ICC 700-2008) applicable to Single and Multi Family Homes*
*Source: http://www.nahbgreen.org/Guidelines/ansistandard.aspx*

| **Threshold Point Ratings for Green Buildings** | | | | | | |
|---|---|---|---|---|---|---|
| **Green Building Categories** | | | **Performance Point Levels (1) (2)** | | | |
| | | | **BRONZE** | **SILVER** | **GOLD** | **EMERALD** |
| 1. | Chapter 5 | Lot Design, Preparation, and Development | 39 | 66 | 93 | 119 |
| 2. | Chapter 6 | Resource Efficiency | 45 | 79 | 113 | 146 |
| **3.** | **Chapter 7** | **Energy Efficiency** | **30** | **60** | **100** | **120** |
| 4. | Chapter 8 | Water Efficiency | 14 | 26 | 41 | 60 |
| 5. | Chapter 9 | Indoor Environmental Quality | 36 | 65 | 100 | 140 |
| 6. | Chapter 10 | Operation, Maintenance, and Building Owner Education | 8 | 10 | 11 | 12 |
| 7. | | Additional Points from any category | 50 | 100 | 100 | 100 |
| **Total Points** | | | 222 | 406 | 558 | 697 |
| (1) In addition to the threshold number of points in each category, all mandatory provisions of each category shall be implemented. | | | | | | |
| (2) For dwelling units greater than 4,000 square feet (372 square meters), the number of points in Category 7 (Additional Points from any category) shall be increased in accordance with Section 601.1. The "Total Points" shall be increased by the same number of points. | | | | | | |

The National Association of Home Builders web site also has a user friendly online **scoring tool at** http://www.nahbgreen.org/ScoringTool/ free of charge for National Green Building Standard (ICC 700-2008) based scoring for green homes. A partial view of the scoring tool is shown in Figure7. As can be seen the standard has some mandatory practices clauses (section 701), the example building complied.

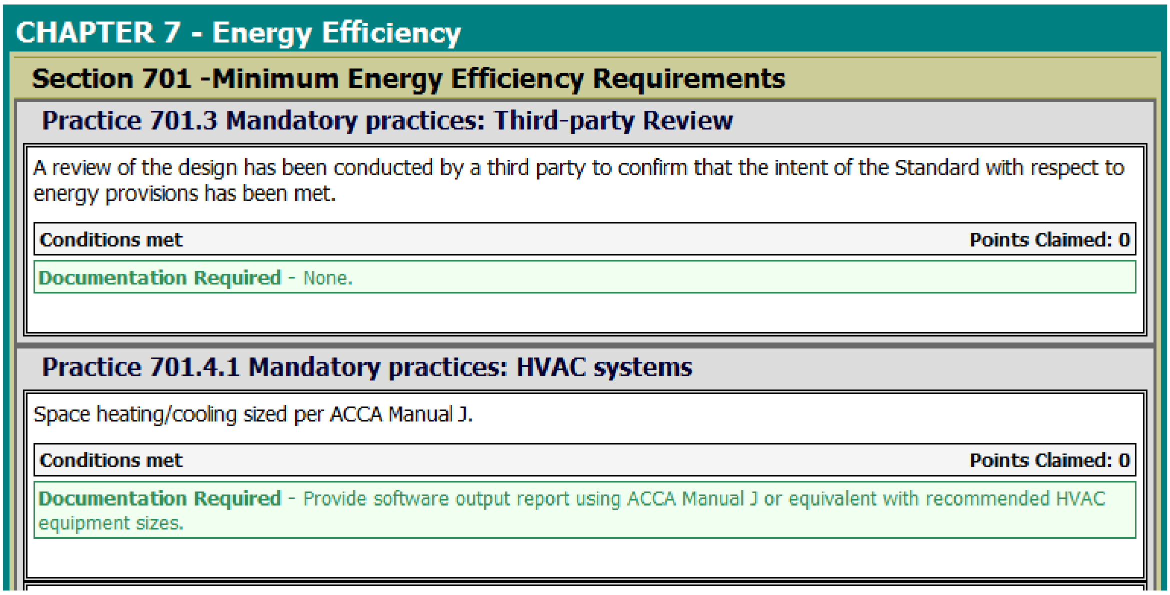



*Figure 7: Partial View of Green Building Scoring Tool per ICC 700-2008. (http://www.nahbgreen.org/Guidelines/ansistandard.aspx)*

The standard also requires through the Performance path (Sec. 702) that Energy Efficiency performance be more than at least 15% of IECC minimum requirements. By selecting the performance path the standard allows flexibility in that some part of the building may be less efficient than expected of IECC as long as the overall building meets the required performance. The performance path allows the designers to consider variety of factors such as roof reflectivity, shading devices etc. Since our IECC 2009 analysis through REScheck software has a performance of 24 % more than the IECC baseline, this building receives 30 points (minimum required 15% more than IECC performance.) for this section as shown in Figure 8.

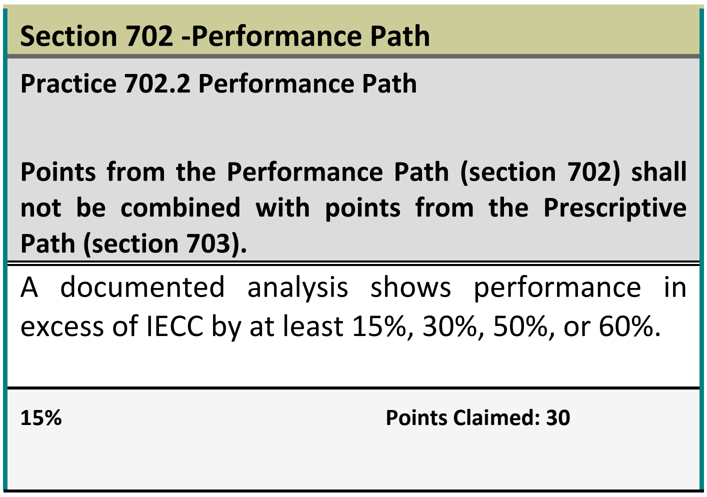



*Figure 7: Partial View of Green Building Scoring Tool per ICC 700-2008. (http://www.nahbgreen.org/Guidelines/ansistandard.aspx)*

With all the points accounted for in Chapter 7 (Energy Efficiency) of the standard the building received 73 points and qualified to be Silver (minimum 60 points) rating as shown in Figure 8.

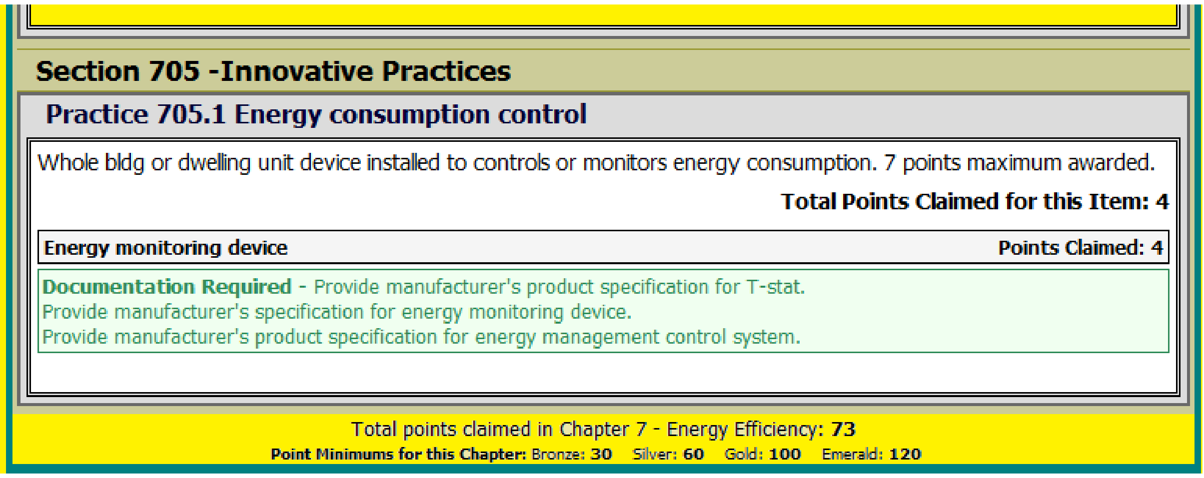



*Figure 8: View of Total Score for Energy Efficiency per ICC 700-2008 Tool (http://www.nahbgreen.org/Guidelines/ansistandard.aspx)*

## 6. Discussion

The paper surveys two popular methods for rating green residential buildings in the United States. The two methods are very similar. As such the MGB guideline tool is planned to be discontinues beginning 2011 and the Green building standard tool to continue. The builders and designers of homes may find the user friendly free online tool based on ICC-700 and REScheck useful. The ICC 700-2009 is an accepted standard by the American Standard Institute (ANSI) which makes it a consensus developed option.

### References

1. International Code Council, Washington, DC, “An Introduction to the ICC 700-2008 National Green Building Standard,”2008.
2. Bilka, A. “The Current Status of Green and Sustainable Building Program, Standard and Code Development in the United States,” ASCE 5th International Engineering and Construction Conference (IECC’5), August, 2008.
3. Purasinghe, R., “Current Sustainable Green Building Technology Rating Practices for Homes in US,” Proceedings of the International Conference on Environmental Sustainability with Green Building Technology (ICESGBT’ 10) pages 43 to 45, Meenakshi Sundararajan Engineering College, Chennai, India, March 15-17, 2010.
4. US Department of Energy, “2009 International Energy Conservation Code,” 2009.
5. http://energycode.pnl.gov/REScheckWeb/
6. National Association of Home Builders ,(NAHB) http://www.nahb.org/generic.aspx?genericContentID=56077
7. ANSI, NAHB, and ICC, “National Green Building Standard,” ICC-700-2008.
8. US Green Building Council, “LEED for Homes,” V3, ,2009 http://www.usgbc.org/DisplayPage.aspx?CMSPageID=147

### Acknowledgements

Technical help from Ravi Bhattaram, Vincent Chui and Hugh Wong are greatly appreciated.

### About the Authors

**R. PURASINGHE,** B.Sc. Peradeniya, Ph.D., P.E., is a Professor at the Department of Civil Engineering, California State University, Los Angeles. He is also a registered Professional Civil Engineer in California. His research interests are in the areas of earthquake resistance structures, progressive collapse analysis, energy efficiency and sustainable design.

**E. MAGUINO,** B.S., is currently a graduate student at the Department of Civil Engineering, California State University, and Los Angeles.

# WASTE MANAGEMENT STRATEGEIS: MUNICIPAL WASTE VS DISASTER WASTE

G. Karunasena[1], D. Amarathunga[2]

[1]Lecturer, Department of Building Economics, University of Moratuwa, Moratuwa, Sri Lanka.
[1]E-mail: gayani@becon.mrt.ac.lk
[1]Telephone: +94-11-2650738; Fax: + 94-11-2650738

[2]Senior Lecturer, School of the Built Environment, University of Salford, United Kingdom.
[2]E-mail: r.d.g.amaratunga@salford.ac.uk
[2]Telephone: +44-161-295 4471; Fax: + 44-161-295 5011

**Abstract:** Waste has become a global issue with rising population, urbanization, economic activities and consumerism. Further, this is becoming more critical due to waste generated through frequent disasters. This is evident with increased number of environmental, social, economical and health issues such as epidemics. Thus, this paper intends to critically review waste management practices, of both municipal and disaster waste to identify prevailing gaps. Lack of physical, human and financial resources, less enthusiasm among community groups and legal loopholes are identified as major gaps. Community involvement in planning, development and implementation of waste strategies, enhancing strategic level capacities, raising public awareness and establishing supportive authorities are proposed to eliminate identified gaps.

***Keywords:*** *Municipal waste, Disaster waste, waste, Gaps, Waste management*

## 1. Introduction

Waste is defined as any losses produced by activities that generate direct or indirect costs but do not add any value to the product from the point of view of the client (Formoso *et al*, 1999) or any substance or object which the holder intends or is required to discard. Hoornweg *et al*, (1999) further refined that waste arises from human and animal activities which are normally solid referred as solid waste. Tchobanoglous *et al*, (1993) categorized it into two as municipal solid waste (MSW) and industrial waste. According to the business dictionary (2009), MSW means all types of solid waste generated by household and commercial establishments and collected usually by local government bodies and includes residential, commercial, institutional and construction and demolition waste. According to Environment Protection Agency in USA, disaster waste also comprised with similar items such as soil and sediments, building rubble, vegetation, personal effects, hazardous materials, mixed domestic and clinical wastes and often, human and animal remains representing a risk to human health from biological, chemical and physical sources (EPA, 2008). Zon (2000) stated, people do not seem to be much aware of possible environmental problems caused by the disposal of household waste where it is only seen as a problem when practical issues occur at storage or disposal. Further, Damgghani *et al* (2007) stated that poor waste management practices may result in several problems such as unpleasant odour and the risk of explosion in landfill areas, as well as ground water contamination. Kobayashi (1995) indicated that managing disaster waste become further critical unlike ordinary waste as it is mixed and difficult to separate. Peterson (2004) added that this become further critical in disasters as it differs from the normal situation which generates waste in a more or less stable quantities and composition which may contain or be contaminated with certain toxic or hazardous constituents. In Sri Lanka, solid waste management become a environmental, social as well as a political issue due to scarcity of vacant lands, collection and disposal issues (Jayaratna, 1996) and dengue epidemic (Anji, 2009). This was evident during the Asian Tsunami in 2004. Thus this burning problem should be eliminated for betterment of the nation. Though community and the government seek a solution through conducting various solid waste management projects, still issues are visible which has become a researchable issue in Sri Lanka. Accordingly, this study intends to identify gaps existing in solid waste management in Sri Lanka, with special emphasize on municipal and disaster waste. Forthcoming sections of the paper illustrate literature findings, methodology adopted, survey findings and conclusions drawn.

## 2. Literature findings

### *Solid Waste Management*

Solid Waste Management (SWM) is a major part of the social system (Rahardyan *et al*, 2004). In early days, the issues related to solid waste management was at its lowest level with each taking care of his own by dumping at the back of his cave (Wilson, 1977). Today it is increasing rapidly and composition is also changing with urbanization, change in life styles and food habits of people (Poopor *et al,* 2004; Ogbonna, 2007; Agdag, 2008). Key reason for solid waste becoming an issue is the rapid increase of population rather than developing waste management systems. According to Damghani *et al* (2007), it can be classified into four groups as, municipal, hospital, industrial and construction and demolition. As previously stated municipal waste means all types of solid waste generated by household and commercial establishments and collected usually by local government bodies (Business Dictionary 2009). Cader (2001) indicates that SWM involves managing activities associated with generation, collection, transport and disposal of solid waste in an environmentally compatible manner, adopting principles of economy, energy and conservation. Kum *et al,* (2004) highlighted major challenges associated with waste collection services and disposal facilities. This becomes a challenge with waste generated by frequent disasters due to volume and composition. Brown *et al* (2010) indicate that following a disaster in addition to above another three waste streams may get generated such as disaster generated debris, emergency and relief services generated waste and surplus donations. Further, authors highlighted that it is likely in a large scale event, that municipal and industrial waste streams will also be altered due to disruptions and displaced persons. Thus, these are evident for complexness of solid waste management system. Many have introduced various strategies, models and projects for management of waste such as Three R concepts (Reduce, Reuse and Recycle), The Nova Scotia MSW strategy (Wagner and Arnold, 2006), Unit Charging Programs-Pay As You Throw (PAYT programs) (Chakrabarti, 2008), community based solid waste management programs and community awareness programs. Next section of the paper reveals the literature findings on solid waste management practices in Sri Lanka.

### *Solid waste management in Sri Lanka*

In Sri Lanka, the basic legal framework required for solid waste management is provided under Government, Provincial Council (PC) and Local Authorities (LA) regulations and legislations. Rameezdeen (2009) indicated that there are three levels of legislation related to SWM in Sri Lanka. Those are the National Environmental Act (NEA), local governmental laws and the Police Ordinance. Further, he mentioned that according to NEA (Amendment) No.56 of 1988, Central Environmental Authority (CEA) can request any local authority to comply with and give effect to any recommendation related to environmental protection and any recommendation relating to some aspect of environmental pollution such as to prohibit unauthorized discharge, emission or deposit of litter, waste, garbage and sewerage. With respect to disaster waste, in-depth review on national level polices for disaster management (Refer Disaster Management Act no 13 of 2005) revealed that there are no provisions for disaster waste management. Disaster Management Act only states that disaster management council shall provide protection for environment and maintain and develop affected areas (Disaster Management Act, 2005). Thus, disaster waste is also classified within municipal solid waste as there are no other regulations specifically dealing with them.

In addition to that, the national policy is build on the ‘polluter pays’ principle. Reduction of consumption and maximization of recycling and reuse were initiated through various projects (Rameezdeen (2009). The “National Strategy for Solid Waste Management” is based on the premise of waste management from generation to final disposal (Chandana *et al*., 2006). Further, “Waste Management Zonal Concept” is the strategy, which has been identified by the Waste Management

Authority of Sri Lanka to overcome present short comings in the administration of waste management in the Western Province. This concept facilitates sharing of available resources among local authorities of each zone and working as groups in waste management (Waste Management Authority of Sri Lanka, 2005). Under the public awareness programmes, promotions are conducted encouraging the public to segregate waste at generation points. Waste collection points called "*Sampath piyasa*" are built to store the waste until they are subjected to proper disposal. In addition, public awareness programmes titled *Pivituru paasel* project, *parisara mituro* project, *pivituru suva piyasa* project, *parisara kekulu* project and *pivituru ayatana* are conducted along with various other media campaigns. Information material related to public awareness includes posters to be displayed at schools, government institutions, community centres and in public buses (Zon and Siriwardena, 2000). Though there are many initiatives, issues related to solid waste management are still prevailing in Sri Lanka as evidenced by the Dengue epidemic. The next section of the paper illustrates the research approach used to identify the gaps in solid waste management with a special emphasis on municipal and disaster waste.

## 3. Methodology

Case study was selected as the research approach as it provides an opportunity for in-depth analysis of existing solid waste management practices to identify gaps. According to Yin (2003) it is "*an empirical inquiry that investigates contemporary phenomena within its real life context; especially where boundaries between phenomena and context are not clearly evident*". Three waste management projects are selected as cases which are currently conducted in Sri Lanka as illustrated at table 01. All projects mentioned below are coordinated by the government institutions at national level targeting management of municipal solid waste in short term period as three to five years. None of the projects identifies disaster waste except the COWAM project which was initiated with the intension of management of construction waste generated by the Asian Tsunami in 2004.

Table 1: *Profile of waste management*

| **Project** | **Description** |
|---|---|
| Project A & B | Provide supportive services to local authorities on SWM. |
| Project C | To create awareness and provide infrastructure to conduct SWM projects |

Semi-structured interviews were conducted to gather data as it facilitated in depth analysis and gather different views and opinions of respondents within scope of the study. Three interviews were conducted to collect data from each case, where one was conducted with the particular project managers to gatherer general information on each project and other two with the beneficiaries to identify real benefits received. Content analysis was used to analyze collected data. Content analysis is a method that compresses many words into a fewer content categories. According to Silverman (2006) this involves establishing categories and then counting the number of instances that fall into each category. This method pays particular attention to reliability of its measures and to the validity of its findings. Nvivo software was used for easier and speedy content analysis. Relevant coding structures were prepared using software and analysed in order to determine gaps in solid waste management as illustrated at figure 1.

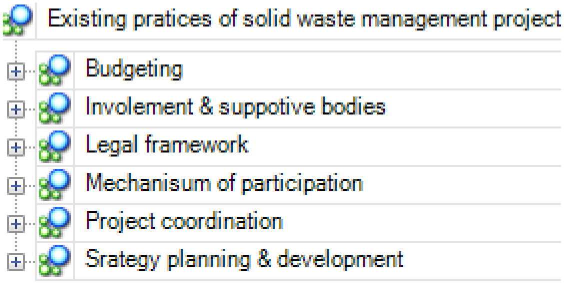



Figure 1: *Coding structure*

## 4. Case findings

As already mentioned, data gathered through case studies revealed information in following six areas: budgeting (funding and cost management), involvement of supportive bodies (tools and equipments), legal framework (regulations and legal development), participation (contribution and target groups), project coordination(committee involvement) and strategy planning and development (requirements identification, strategy development etc) as follows.

***Budgeting***

All three projects are mainly government funded projects. Project A & B were partly funded by non-governmental organizations. Budgetary support was mainly aimed at enhancing technical capacities to conduct the project but not to uplift local authorities' support services such as physical resources. It is an identified weakness of projects A and B.

In terms of cost management, each project has an annual budget based on an action plan for the entire project matching with the total predetermined budget. Also, all projects promoted waste collection by separation at generation points and collecting recyclable waste as a cost management strategy. Further, public awareness strategies are used for minimizing costs of per person for waste management.

Findings revealed that projects A and B are at satisfactory levels of recovering the project costs by promoting large composting projects while the project C was more concentrated towards cost saving at strategy implementation stage by allowing participants to use available resources as supportive equipments of the project.

***Supportive bodies***

All accept that involvement of many parties can achieve successful decision making. Community and other committee level involvement can be seen at decision making process of the project A while projects B and C do not identify the importance of community involvement in preparing project action plans. Further, all project coordinators accepted that sound knowledge and attitudes of project staff is essential in proper project handling. Less dedication of employees in local authorities is also considered as a major weakness of projects A and B. In addition inadequate machinery, collection and transportation equipments and suitable lands have further aggravated the issues in these projects. Thus, having supportive bodies as recyclable waste collectors is strengthening these projects. Further, in project B, labourers are promoted to use manual systems instead of highly technological and complex systems which raised health issues. Project C revealed that there are no issues regarding handling of tools and equipment to conduct awareness programmes. However, inadequate resources to collect waste by separation, collection and transport are identified as major obstacles.

***Legal framework***

All projects are coordinated by government organisations; hence there are fever obstructions when working with other organizations. In case of projects A and B, national policy on SWM is followed to ensure environmentally sound solid waste management practices. Although having such a corporate policy is for betterment of the project, it is a weakness noted in that policy it has no clear sources of funding. Hence, programs initiated by projects A and B such as “unit charging” and “polluter has to pay” programmes are malfunctioning due inadequate regulatory support.

In term of regulations, bodies responsible for project B have failed to implement a licensing system, regulations, standards or guidelines for solid waste disposal except for some hazardous materials. However, bodies responsible for project C have enacted a provincial legislation within the Western province provide proper regulatory framework for SWM.
According to interviewees of projects A and B, there are lesser opportunities to develop by-laws with local authorities.

***Participation***
It is revealed that projects A, B and C obtain adequate contributions from several parties. While projects A and B are having public, private and community involvement, project C is involved only with the public sector and the community.

Project A is to obtain highest participation in project implementation stage by getting a higher level of community participation in strategy planning. Also, attention has been paid to special target groups such as labours of local authorities and students. Project C has selected students as their main target group of the project and project B targeted students when conducting awareness programmes. Further, in project A general public are getting a real experience on SWM by contributing to prepare action plans whereas in projects B and C they are not obtain any such assistance from the general public.

Projects A and C believe that target groups concept enhances community contribution in projects. Students have a higher participatory level in project C. The views of beneficiaries of projects A and B differ by considering that communities do not have enough time to participate at awareness programmes such as workshops, training programme, etc.

***Project coordination***
Projects A and B have satisfactory national level project coordination and indicated on importance of involvement of coordinating committees. In both projects, coordination committees are involved in providing technical guidance and financial support. Project A also identified the importance of coordination in community level of the project. Interviewees’ point of view, community level coordination is identified as critical factor. In case of project C, it does not perform activities through proper coordination committees.

***Strategy planning and development***
All projects have realised the importance of identifying real needs that shall be addressed by the projects. Project A identified technological and financial assistance as the real need of local authorities and project B identified development of capacities of local authorities as the timely requirement. Project C identified community awareness as the key requirement. In addition to above, all projects are giving consideration to environmental, economical, technological and social factors in strategic planning. However, project C is providing less interest on economical factor since it promotes reuse to minimize initial costs.

Projects A and B are directed towards enhancement of capacities of local authorities that have close relationships with communities whereas the project C is targeting to change the mind set of the future generation.

In term of strategy development, the project C is more concentrated on the target groups concept in strategy planning accepting that through selected target groups (such as students) relevant messages can be given to a larger number of people within a short period of time through awareness programmes. Further, the project C is more concerned about positive attitudes and motivation of the community. Both A and B projects are indirect community conjuncture projects but having direct connections with local authorities.

## 5. Discussion

Case studies carried out led to the identification of following gaps which can be summarised as illustrated in table 2:

Table 2: *Gaps identified in SWM projects*

| | Gaps identified |
|---|---|
| Participation | ▪ No proper way to reach all group of people<br>▪ Community has less positive attitudes towards waste management<br>▪ Less attention to increase public awareness<br>▪ Projects conducted only for selected groups of people |
| Strategy planning and development | ▪ Less consideration on air and water pollution<br>▪ Have not developed proper system for recycling waste transportation<br>▪ Political interferences |
| Project coordination | ▪ Inadequate coordination with related communities |
| Legal framework | ▪ Inflexible legal framework<br>▪ Inadequate legal solutions<br>▪ Policies are not clear on matters of funding. |
| Supportive environment | ▪ Collection and transportation equipments' shortage<br>▪ Inadequacy of available lands for composting, dumping and land filling<br>▪ Accountability is less among of employees of local authorities<br>▪ Inadequate resources<br>▪ Less enthusiasm of private sector organizations |
| Budgeting | ▪ Less budgetary allocation for equipments<br>▪ Polluter *pay* concept is not well functioning |

Accordingly, findings revealed the importance of committee involvement in strategy planning, development and project implementation stages. However, it is identified that there is lesser community participation. Therefore, through establishing community based committees it will be possible to create proper links between project activities and communities. It will be helpful to achieve community participation in project implementation stage hence get their involvement in strategy planning level in SWM projects.

Most SWM project failures occur due to improper management at project coordination, handling of legal issues, financial management, handling of equipment and personnel and strategy management. Although having adequate resources such as funds and equipment, unless a project implements proper management strategies, it can end as a failure. Hence, it is important that any critical project activity be linked with strategic management.

Further, the research study shows that gaps arise due to less participation of target groups. Successful project participation can be achieved through awareness of every stakeholder regarding requirements of proper SWM strategies which motivate involvement with positive attitudes. It can be achieved through awareness or training programmes to establish public - private community participation.
As mentioned in section 3, there are no waste management projects targeting disaster waste management in Sri Lanka, other than the one initiated after the Asian Tsunami in 2004, called Construction Waste Management (COWAM) project. It offers consultations on sustainable management of construction and demolition waste within the targeted region. Major gaps identified, according to respondents' are, public unawareness, less enthusiasm among the public, legal issues, value and ownership issues, access to private property, safety of workers, unavailability of single point responsibility and lack of resources such as labour and machineries.

Accordingly, in respect of both municipal and disaster waste, similar gaps of waste management such as less participation of community, legal issues, inadequate resources, etc are prevalent.

## 6. Conclusions

Solid waste becomes a global challenge due to limited resources, an exponentially increasing population, rapid urbanization and worldwide industrialization. In developing countries like Sri Lanka, these factors are further affected by inadequate financial resources, inadequate management and technical skills within municipalities and government authorities. Therefore, many solid waste management projects were introduced to avoid these drawbacks. However, environmental, social and health impacts are still visible as a result of poor waste management practices. Thus, this became a researchable problem to further investigate to identify gaps existing in solid waste management (SWM) projects in Sri Lanka.

The aim was achieved through in depth investigation of selected three SWM projects (cases) at national level. Unavailability of proper procedure to reach all groups of people, less positive attitudes of the community, less attention to increase community awareness, political interferences, lesser consideration on air and water pollution, unavailability of proper systems for recycling waste and transportation, absence of community participation in strategy planning and development, inflexible legal frameworks, inadequate legal solutions, less budgetary allocations and inadequate resources (collection and transportation equipment and lands for final disposal) are identified as gaps in solid waste management. Public unawareness, less enthusiasm among public, legal issues, value and ownership issues, access to private property, safety of workers, unavailability of single point responsibility and lack of resources are identified as major gaps prevalent in the case of disaster waste. Establishment of community committees with access to strategy planning, development and implementation, adopting strategic management to critical activities of project to minimize failures related to financial resources, project coordination, handling of legal issues, handling of physical and human resource and enhancing public awareness can be proposed as ways to minimize prevailing gaps.

## References


20. Agdag, O.N., (2008), "Comparison of old and new municipal solid waste management systems in Denizli, Turkey", Waste Management, 29, 456-464.
21. Anji, A., (2009), "Battle against dengue mosquito", *Daily news*, 25 June, p.18b.
22. Brown, C., Mike, M. and Seville, E., (2010), "Waste management as a "life line?'A New Zealand case study analysis", *International Journal of Disaster Resilience in the Built Environment* 1(2):192-207
23. Business dictionary, (2009), "Municipal solid waste" [online]. Available from: http://www.businessdictionaty.com/definition/solid-waste.html [Accessed 27 October].
24. Cader, A.A., (2001), "A public perception study on garbage and related issues in Sri Lanka" [online]. Centre for policy alternatives, media unit. Available from: http://www.cpalanka.org/page.php [Accessed 20 July 2009].

25. Chakrabarti, S., Majumder,A. and Chakrabarti, S., (2008), "Public-community participation in household waste management in India: An operational approach," *Habitat International,* 33, 125-130.
26. Damghani, A.M., Savartpour, G., Zand, E. and Deihimfard, R., (2007), "Municipal solid waste management in Tehran: Current practices, opportunities and challenges", *Waste management,* 28, 929-934.
27. Disaster Management Act No 13 of 2005, (2005), Published as a Supplement to Part II of the Gazette of the Democratic Socialist Republic of Sri Lanka of May 13 2005 Colombo: Government publications bureau
28. Environmental Protection Agency, (2008), "Planning for Natural Disaster Debris" (Available online http://wwwepagov/CDmaterials/pubs/pnddpdf [accessed on 5/8/2008])
29. Formoso, C.T., Isatto, E.L. and Hirota, E.H. (1999), "Method for waste control in the building industry", Proceedings IGLC-7, 7th Conference of the International Group for Lean Construction, Berkeley, CA, 26-28 July.
30. Jayaratne, K.A., (1996), "Community Participation in Urban Solid Waste Management Case Study of Siddharthapura Low Income Settlement, Colombo, Sri Lanka "[online]. Sri Lanka, Availavle from: http://www.globenet.org/preceup/pages/fr/chapitre/partenai/h/wast.htm [Accessed 12 November 2009].
31. Kobayashi, Y., (1995) "Disasters and the problems of wastes" *International Symposium on Earthquake Waste*, 12-13 June, Osaka Shiga: Japan,6–13
32. Kum, V., Sharp, A. and Harnpornchai, N., (2004), "Improving the solid waste management in Phnom Penh city: a strategic approach", *Waste management*, 25, 101-109.
33. Petersen, M., (2004), "waste management following disasters", *International conference on post disaster reconstruction*, 22-23 April, UK. Coventry: IF Research group.
34. Popov, V., Ltoh, H., Brebbia, C.A. and Kungolos, S., (2004), "Waste management and the environment", 1st ed. UK: Wit press.
35. Rahardyan, B., Matsuto, T., Kakuta, Y. and Tanaka, N., (2004), "Resident concerns and attitudes towards solid waste management facilities", *Waste management,* 24 (5), 2-14.
36. Rameezdeen, R., (2009), "Construction Waste Management" 1st ed. EuropeAid co-operation office: COWAM project.
37. Silverman, D. (2006), Interpreting Qualitative research: meanings or practices?, *Information systems journal*, 8.3, 3-20
38. Wagner, T. and Arnold, P., (2006)," A new model for solid waste management: an analysis of the Nova Scotia MSW strategy", *Journal of Cleaner Production,* 16, 410-421.
39. Waste Management Authority, (2005), "Draft rules for public comments" Sri Lanka: Western Provincial Council & USAID/USAEP.
40. Wilson, D.G., (1977), "Hand book of solid waste management" 1st Ed. New York: Van Nostrand Reinhold Company.
41. Yin, K. (2003), Case study research: Design and methods, 3rd edition, SAGE publications, London
42. Zon, L.V., and Siriwardena, N., (2000), "Garbage in Sri Lanka" [online]. Available from: http://environmental.scum.org/slwaste [Accessed 29 July 2009].



## Acknowledgement

The authors wish to acknowledge contributions of Miss. Anuradha S.D.S., an undergraduate of Department of Building Economics, University of Moratuwa, who took part in field survey in gathering data on municipal solid waste management practices in Sri Lanka.


## About Author

Gayani Karunasena graduated with a BSc (Hons) Degree in Quantity Surveying from the University of Moratuwa, Sri Lanka. Thereafter, she obtained a Master of Philosophy in Construction Information Technology from the same University. Currently, she is employed as a lecturer at the same University and reading for her split site PhD on the area of disaster waste management.

# DEVELOPMENT OF A SUSTAINABLE ENVIRONMENTAL PRESERVATION CENTRE (EPC) AT NAWALAPITIYA FOR URBAN SOLID WASTE MANAGEMENT

Ariyawansha R.T.K.[1], Chandrasena A.S.H., Senevirathne, S.A.D.N., Basnayake B.F.A.[2]

Solid Waste Management Research Unit (SWMRU), Department of Agricultural Engineering, Faculty of Agriculture, University of Peradeniya, Sri Lanka 20400
Tel: +94-772291235[1], +94-812395475 ;
Fax: +94-812395454
E-mail: chathunisha@gmail.com[1], nri.srilanka@gmail.com[2]

**Abstract:**

An Environmental Preservation Centre (EPC) was established to resolve the solid waste management crisis of Nawalapitiya Urban Council. Material Recovery Facility (MRF), Inclined Step Grate (ISG) composting system of 5.5 tonnes/ day capacity, screening, storage and residual landfill, the total area covered by the fence including buffer zone are the major components of the EPC. After separating recyclables at the MRF, the remaining biodegradable is fed in to ISG system. Residual landfill is designed for inert materials that are remaining after the MRF and composting facility operations. MRF and ISG system are already constructed and currently initiated the operations. This is technically and financially feasible sustainable approach for protecting the environment and posterity.

***Key words:*** *Environmental Preservation Centre, Material Recovery Facility, ISG composting unit, Residual Landfill*

## 1. Introduction

Nawalapitiya Urban Council (NUC) is situated in Kandy District in the Central Province home to 14,254 habitants spread over 261.3 ha of land area. Total waste generation in NUC is 12 tonnes and about 8 tonnes of waste is being collected by the NUC and directly dumped on the bank of Mahaweli River as the main disposal method until lately. It has had some influence on health, environmental, socioeconomic and political issues in detrimental ways [1, 2]. Lack of suitable and adequate lands for safe disposal of Municipal Solid Waste (MSW) has been one of the major problems. Hence, composting like landfill pre-treatment measures is an appropriate low cost management system and its application would definitely help to reduce the burden of MSW management to a certain extent while generating income and employment within the Local Authority (LA).

As a collaboration work of Japan International Cooperation Agency (JICA) with National Solid Waste Management Support Centre (NSWMSC) and Ministry of Local Government and Provincial Council, an Environmental Preservation Centre (EPC) was established to resolve this dilemma. In this effort, the vision of the UC is “to develop better and beautiful future for whole community in Nawalapitiya”. To fulfill this vision, the missions of this project are to develop a zero waste city, less waste generation rate, environmentally friendly, economically viable, socially just waste disposal methods which should be viable to treat MSW for more than twenty five years at the selected land. Thus, this paper is aimed to examine the design and development approach of the sustainable EPC at Nawalapitiya.

## 2. Material and Methods

### *2.1 Selection of the optional technical system*

The reuse and recycling of wastes were the main purposes for establishing and operating an EPC. Geographical characteristics of the site, climate, wind velocities, composition of wastes, quantity of wastes, quality of wastes, aeration of composting systems, machinery used and essential services were major criteria used in selecting the best process and wastes disposal system. The total MSW collected by NUC has a high content of perishable organic matter of around 49 % by wet weight with a moderate amount of garden waste (20 %) and paper (18 %), and low content of plastic, metal and glass. Hence, around 5.5 tons of waste could be used for composting per day [3]. To design the components of the EPC, the types and composition of wastes were considered. Also to increase the efficiency, adequate space was provided both manual and mechanical operations. Double handling was reduced too.

Safety and other factors that are needed for the workers were considered for their well being such as rest rooms, toilets, dinning (away from the waste processing area) and office space. In addition, uninterrupted water and electricity supply was ensured. Besides, landscaping of the environment guarantees agreeable and pleasing visual environs for the workers as well as reduced environmental impacts beyond the perimeter of EPC.

### *2.2 Site selection*

The selected project site of 2.02 ha is located 5 km from the town which is ideal in considering social and other environmental aspects. All of these aspects were considered in conducting feasibility and IEE studies [3, 4]. Out of which 0.8331 ha has been selected as the EPC demarcations.

### *2.3 Components of the EPC*

By considering all the factors, Material Recovery Facility (MRF), Inclined Step Grate (ISG) composting system of 5.5 tonnes/ day capacity, screening, storage, residual landfill and fencing of the total area, including buffer zone was selected as the major components of the EPC. The layout plan of the EPC is given in Figure 1. The land was developed considering all of the constructions, including the road network. The storm water drains were constructed first followed by cut and fill of the site and then the road network, including the side drains of the roads as indicated in Figure 1. The infrastructure facilities such as site office, workers rest room and sanitary facilities, screening & storage facility, maturing facility, security room, site services and fencing were constructed parallel to other major construction activities. MRF and ISG system are already constructed and currently initiated the operations.

Trees were planted within 4 m distance from the fence as buffer while, 2 m wide buffer was established for the front of the facility as a visual barrier. The total area covered by the buffer zone is 1306 $m^2$. In the front, some of the flowering trees were planted to give additional value to frontage.

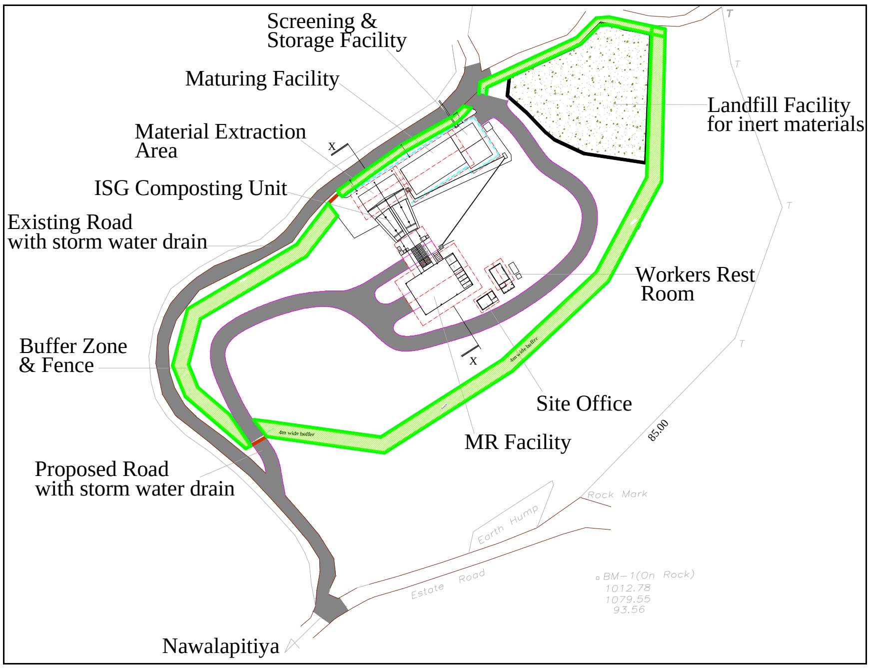



Figure 1: *The layout plan of the EPC*

### *2.3.1 Material recovery facility*

The establishment of a MRF in a SWM system should be a feasible alternative to achieve sustainable development goals in urban areas if current household and curbside recycling can not prove successful in the long run [5]. The main function of the MRF is to maximize the quantity of recyclables processed, while producing materials that will generate the highest possible revenues in the market. The stages involved in designing a MRF system to process commingled recyclables were receiving of the materials in the hopper, the primary sorting of the materials in the conveyer, manual sorting of the individual materials and baling and separation of materials for transport. The functional space requirements for different categories of wastes, number of workers, ergonomics and storage space were taken into account in designing the MRF (221m$^2$).

As shown in Figure 2, incoming mix wastes to the MRF should be directly emptied from compacter trucks (tractor trailer) to unloading area where possible recyclables should be separated as much as possible to different categories such as polythene, papers, plastic, metal and glass, and others. After that, all the remaining materials should be pushed to the sloppy mesh area. Workers from both sides of the mesh will rake the materials where debagging process is happening, since it is easy to separate recyclables and rapid biodegradable. Manual raking method could be done at this area to debag and separate recyclables such as papers, polythene, glass, etc. Heavy particles which will roll down at high speeds through the sloppy mesh can be collected to a separate bin that is fixed to the edge of the mesh.

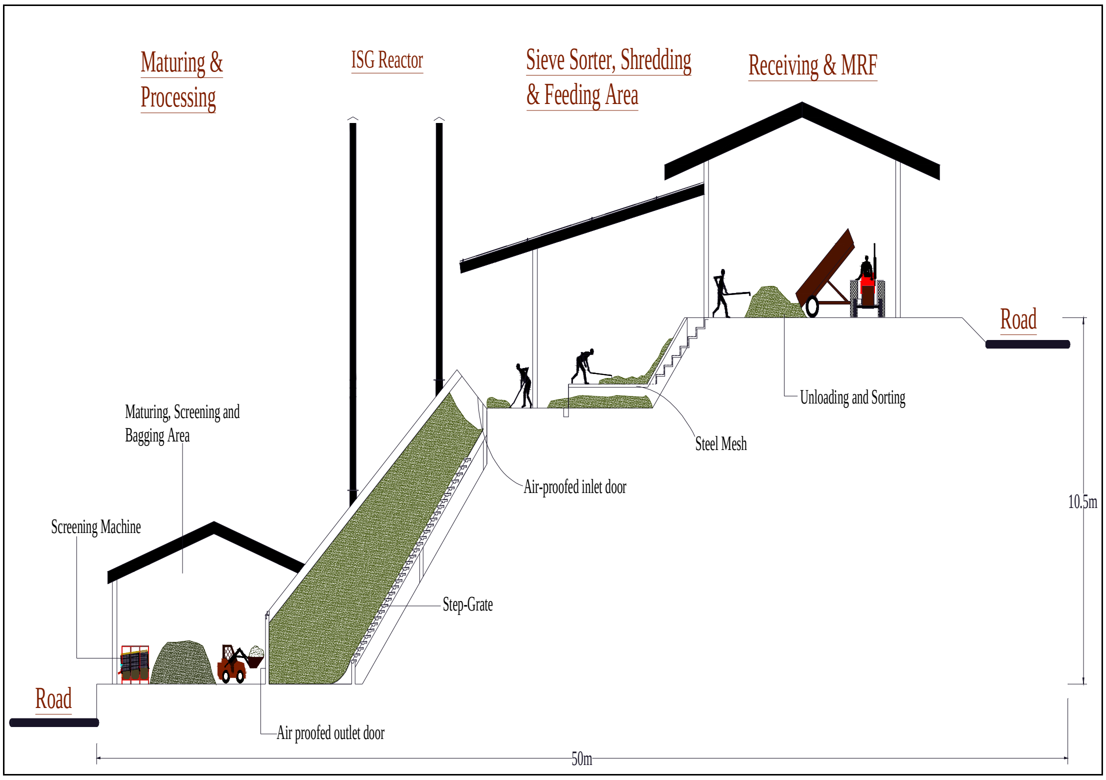



Figure 2: *Operational activities of MRF and ISG unit*

These particles are removed considering the types of wastes and collected in containers for eventual storage, size reduction of large biodegradable materials that will be composted along with small ones that was originally sieved. The recyclable materials like plastics, paper, etc. will be bailed using a press and transported to re-processing centers while the compost will be sold locally to tea estates. The remaining materials that cannot be composted and recycled will be disposed in a landfill meant only for inert materials.

### *2.3.2 The composting facility*

Out of all available compositing systems, considering the terrain, the ISG composting system which was developed at the University of Peradeniya is the best and most suited to the type of wastes, climatic conditions and ease of operation. In fact, it is the least cost in terms of plant size and operational costs. This system consists of a manual or mechanical de-bagging unit, a concrete and stainless steel composting unit and a backend-sorting unit (manual or mechanical sorting and sieving). The compost unit has step-grates and vertical chimneys to facilitate aeration, where the top and chimney surfaces are blackened to create a draught at the bottom of the chimney, permitting air to enter the unit through the step [6]. The chimney effect is most advantageous than any other system.This system is unique since the operational costs are considerably low, and it has minimum moving parts. The system is virtually free from odour and any type of organic wastes could be treated.

Retention time of 28 days, bulk density variation over 28 days , repose angle of $45^o$ and 5 % slopes on sides and roof, dimensions; length, height, width, chimney design ( the chimney height based on temperature variations between inside and outside to obtain an aeration rate of 0.36 to 0.39 $m^3$ of air per day per kg of volatile solids in a mixed urban waste), inlet and outlet openings, number of concrete steps, size and angle, total loads, foundations, columns, beams, side walls, roof structure, leachate pit and maturity shed for minimum of 30 days were the parameters and criteria used in the design of a 5.5 tonnes/day ISG plant. It is expected to have a minimum physical life of 25 years. However, regular maintenance is required of the steel fabrications, but the major part of the structure will be made from concrete, which will require yearly rehabilitation works for esthetic reasons. The Inclined step grate

Unit was located close to the entrance to get the maximum advantage of the existing slope. The inclined floor under the step grate was concreted to protect from erosion.

At the stage of removal from the back-end of the ISG, the wastes with or without recycled materials, have been stabilized but not matured to be termed “compost”. The removal can be done manually or else a small loader could be used. Employing a loader is a better alternative since it is safer as well as very efficient. After that, unloaded stabilized wastes will be made into piles for maturity over a period of 30-45 days. Much higher piles can be made with a loader and they could be as much as 3 meters. Moisture retention is higher with larger piles. If the need arise, moisture can be added to the piles and allow for maturing process. Finally, each pile will be sieved by motorized rotary 5 mm sieve for producing quality compost to meet market demand with the required quality assurance. Leachate collection drains were constructed from the ISG unit, MRF and sorting & feeding area to flow leachate and wastewater into one collecting tank. The collected leachate will be pumped and sprayed inside the ISG units, since the moisture requirement is higher than leachate formations. However, it is important to have adequate stocks of dried recyclable materials to absorb excess leachate formed during the rainy seasons. However, the wastewater from washings will be used as irrigation for buffer zone.

The type of compost produced from this process is unique since the waste materials undergo a degradation phase at high temperatures ($70^o$ - $75^oC$) for a period of 7 days. This type of compost is now termed "Thermophilic Compost". The recent research at the University of Peradeniya indicates that this compost is much better than inorganic fertilisers, producing much higher sustained crop yields since Thermophilic Compost lasts much longer in soil than conventional compost made at low temperatures.

### *2.3.3 Residual landfill*

The objective of the designing of final disposal site is to prevent or reduce as far as possible negative effects to the environment from the waste arriving at EPC. Based on other facilities and treatment methods at the EPC, about 12.5 % of incoming waste would remain for eventual land filling.

Residual materials after composting process or materials that were sorted prior to composting operation will be placed in the landfill. Then, it will be compacted to a density of 800 $kg/m^3$. Hence the required landfill volume for ten years is 4927.4 $m^3$.

The land filling operation will be done every seven days or depending on accumulated ‘compacted’ volume to dispose it in a cell which will be covered with a 6 " soil layer. Temporary drains will be installed in order to divert storm water accumulated in the cell, to existing drainage system in rainy seasons as shown in Figure 3. Base liners and leachate collection system are not essential to be constructed in the landfill facility, since it is an inert landfill. However surface and subsurface drains will be established to divert runoff water.

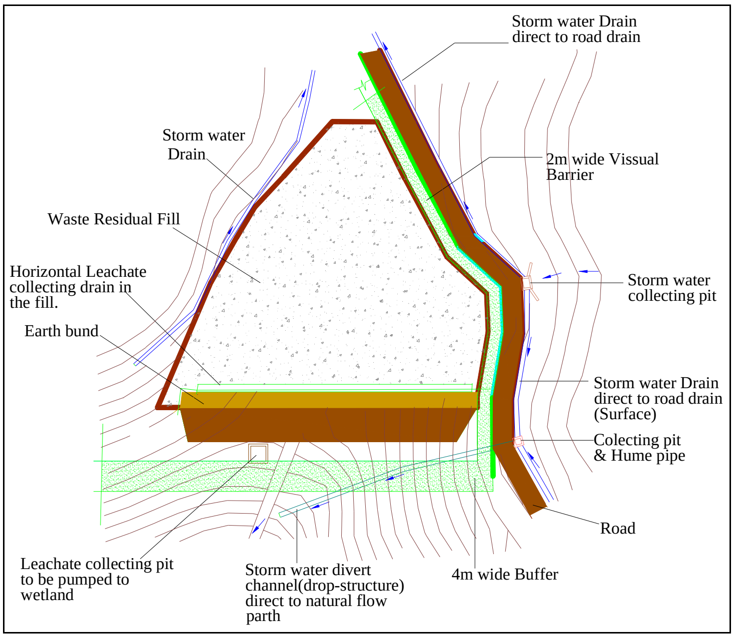



Figure 03: *Layout plan of the Inert Landfill Facility*

## 3. Discussion

The selection of a suitable site was a priority for the LA and it is more than likely for public acceptance of the new site. The most important feature of the project was to move away from the River and locate EPC in an abandoned tea estate. The buffer zone of trees, shrubs and fence reduce the effects of pollution from the EPC. It will have negligible level of pollution from the technologies that will be applied and daily operations carried out in the EPC. However, it is best to provide a leachate management facility, since the rainfall is high and constructed wetland is ideal to treat dilute concentrations of leachate [7]. Also, it is important to have an effective and efficient incinerator for hospital waste disposal at the hospital premises itself, thus healthcare wastes at the landfill site can be avoided.

It is likely that considering the enthusiasm of the UC officials, the EPC will be an attractive site for visitors or even an outing for the locals as an educational and relaxing site. An additional landmass given for landscaping and for the cultivation project that the UC is envisaging might become an eye opener for reclaiming such lands. In fact, funds could be made available if these lands could be given for the project so that compost could be used for rejuvenating the lands to replant tea or convert back to rain forest.

This will not only be a contribution to combat climate change and reduce the influence on global warming, but also provide immediate environmental benefits offsetting escalating environmental costs of poor health of the population. In one of the studies,[8] a value of Rs 60 per capita was derived for costs incurred due to poor solid waste management. The evaluation was done for Colombo and it could be more than applicable to Nawelapitiya. Therefore, health sector and the population will save more than Rs 800,000 annually. The health authorities spend as much as Rs 4.4 billion as recurrent expenditure on public health services. It can be expressed in terms of per capita and it amounts to Rs 231. Also it may be possible to approximate 10 to 50 % of this value on account of poor solid waste management. Thus, based on a percentage 20 %, a value of Rs 46.2 per capita may be the recurrent expenditure on vector control caused by solid wastes. It will amount to Rs 660,000 for Nawelapitiya.

In considering direct benefits, a point source separation programme should be introduced such that the quality of recyclables will be more, thus fetching higher prices which will augment revenue. In fact, the cost of collection will also reduce, saving on fuel and other resources. Inevitably, the environmental benefits will be much more. Hence, awareness programmes, should be conducted to change attitudes of the communities and leadership qualities of individuals enhanced so that grass root level point source separation programmes will be successful.

The awareness programmes should then be coupled with efficient collection system to prevent fly infestations. Enclosed bins are required to stop the breeding grounds, thus eliminating the problem at source. The emission of leachate is very less in the high temperature composting system of ISG. In the event of poor operations, there will be small quantities that will be recycled. In addition long term fibrous biodegradable materials will be mixed with the raw wastes to reduce leachate generations. The wastewater stemming from washing of the MRF will be used for irrigating the buffer zone of the environmental protection centre. It will be aesthetically landscaped to reduce if there will be any odour emissions and to protect the surrounding natural environment. It is also possible to incorporate biofilters to the chimneys [9] if overloading of the reactors will take place causing high polluting gas emissions. The made compost can be improved depending on the market demand for different crop management practices. Thus, inputs such as nitrogen, phosphorous and potassium can be mixed in the form of inorganic or organic into compost piles after the maturity phase is completed or just before bagging the compost. The materials that cannot be composted and there is no other market for non-biodegradable materials will be transported and disposed in the inert landfill. At present, it is the major drawback for efficient management of the facility, since the construction of the landfill is yet to be completed.

The revenue from these processed materials will not be sufficient to meet all of the expenditure if the venture capital will have to be repaid with an interest that is prevailing today. Therefore, the UC will be compelled to impose a ‘green tax’ and the level of taxation can be reduced if the UC is able to secure subsidy or grant to meet some of the capital requirements. The UC also has the option of providing the cost of wages to negate the ‘green tax’. However, it will be very risky as a solution, since there will not be the possibility to buffer losses in the event of market failures, particularly so of selling plastic based materials. The best alternative is to find partners for investing as forward contracts for compost and other recyclable materials. This will reduce the risk. Thus, benefits of banking at high interest rates will be transferred to enterprises. Further remunerations could be attained if collective production systems are developed within the same region to claim for carbon credits. It is technically and financially feasible sustainable integrated solid waste management system for protecting the environment and future generations.

## 4. Conclusion

The EPC will prevent the present pollution loads caused by dumping of wastes. In contrast, the application of mitigation technologies will minimize and enhance the present environment. However, the environmental parameters should be monitored and reported. In the period of construction, adequate protection will be provided to minimize soil erosion. The compost plant is made operational, however hampered with the lacking of landfill facility that requires completion in the near future. Another important addition is a wastewater treatment facility and it is recommended to establish a constructed wetland. The facility will function efficiently, since the Central Government provided the funds to establish the facility and now awaiting for the remaining capital to be injected to the project. The capital is vital, since the facility cannot function viably based on the income from sale of reusable and recyclable materials. Indeed, the facility will no doubt support the development of the city for future generations as a sustainable system.

## 5. Acknowledgement

We wish to express our sincere gratitude to the Chairman Nawelapitiya Urban Council and his staff for their support in conducting this study. We are very thankful to CUP-NSWMSC experts Mr. Nafumi SATO, Mr. Akira Doi, Ms. Mangalika, the Director NSWMSC her staff members for assisting us to accomplish this task.

## References

[1] Gunawardana, E.G.W., Shimada, S., Basnayake, B.F.A., Iwata, T. "Influence of Biological Pre-treatment of Municipal Solid Waste on Landfil Behaviour in Sri Lanka", *J. waste Manage. Res.* 2009; x:xx-xx.

[2] Nagendran, R., Selvam, A., Joseph, K. and Chiemchaisri, C. "Phytoremediation and rehabilitation of municipal solid waste landfills and dumpsites: A brief review", *J. Waste Mgt* 2006; 26: 1357-1369.

[3] Basnayake B.F.A. "Technical Specification of Feasibility Study of Environmental Preservation Centre at Nawalapitiya Urban Council on CUP-NSWMSC'', Solid Waste Management Research Unit, Department of Agricultural Engineering, University of Peradeniya, Sri Lanka, 2008.

[4] Nawalapitiya Urban Council (NUC). "The initial environmental impact examination report for the Proposed Environmental Preservation Centre in Nawalapitiya'', Nawalapitiya Urban Council, Nawalapitiya, 2008.

[5] Davila, E and Chang, N.B. "Sustainable pattern analysis of a publicly owned material recovery facility in a fast-growing urban setting under uncertainty'', *Journal of Environmental Management*, 2005; 75(4): 337-35.

[6] Basnayake**,** B.F.A and Ekanayake, K.M. "Evaluation of different municipal solid waste landfill pretreatment (composting) systems in Sri Lanka'', Proceedings on the Tenth International Waste Management and Landfill Symposium – Sardinia 3-7 October 2005 at Santa Margherita di Pula (Cagliari) Sardinia – Italy, 2005.

[7] Sasikala,S., Mowjood, M.I.M. and Basnayake, B.F.A.2005. "Formulation of Harvesting Regimes from Bulrush (*Scirpus grossus L.f.*)Growt Responses to MSW Leachate in Constructed Wetlands with Coir-Based Media'', *Tropical Agricultural Research* , 17.

[8] Cleantech (Pvt.) Limited. 2006. "Environmental Impact Assessment for Proposed Engineered Landfill Site, Kaduwela'', Central Environmental Authority, Baththaramulla, Sri Lanka.

[9] Ariyawansha, R.T.K., Basnayake, B.F.A., Kariyawasam, H.K.P.P., Jayasighe, I.H. "Applications of double layer compost biofilter for odor control in industrial and waste management facilities in Sri Lanka''. In 9th Asia Pacific Roundtable for Sustainable consumption and Production, 10th -12th June 2010, Colombo, Sri Lanka.

# REMOVAL OF AMETRYN USING MEMBRANE BIOREACTOR PROCESS & ITS INFLUENCE ON CRITICAL FLUX

Dimuth Navaratna[1*] and Veeriah Jegatheesan[2]

School of Engineering and Physical Sciences, James Cook University, Townsville, QLD 4811, Australia
*Corresponding Author; [1]Email: dimuth.navaratna@jcu.edu.au ; [1]Telephone: +61-7-47815535
[2]Email: jega.jegatheesan@jcu.edu.au ; [2]Telephone: +61-7-47814871

**Abstract:** Compared to the Conventional Activated Sludge Process (*ASP*), Membrane Bioreactors (*MBRs*) have proven their superior performance in wastewater treatment and reuse during the past two decades. Further, MBRs have wide array of applications such as the removal of nutrients, toxic and persistent organic pollutants (*POPs*), which are impossible or difficult to remove using ASP. However, fouling of membrane is one of the main drawbacks to the widespread application of MBR technology and Extra-cellular Polymeric Substances (*EPS*) secreted by microbes are considered as one of the major foulants, which will reduce the flux ($L/m^2/h$) through the membrane. Critical flux is defined as the flux above which membrane cake or gel layer formation due to deposition of EPS and other colloids on the membrane surface occurs. Thus, one of the operating strategies to control the fouling of MBRs is to operate those systems below the critical flux (at *Sub-Critical flux*). This paper discusses the critical flux results, which were obtained from short-term common flux step method, for a lab-scale MBR system treating Ametryn. This study compares the critical flux values that were obtained by operating the MBR system (*consisting of a submerged Hollow-Fibre membrane with pore size of 0.4µm and effective area of 0.2m²*) at different operating conditions and mixed liquor properties. This study revealed that the critical flux values found after the introduction of Ametryn were significantly lower than those of obtained before adding Ametryn to the synthetic wastewater. It was also revealed that the production of carbohydrates (*in SMP*) is greater than proteins, subsequent to the introduction of Ametryn and this may have influenced the membrane to foul more. It was also observed that a significant removal (*40-60%*) of Ametryn from this MBR during the critical flux determination experiments with 40 minutes flux-step duration.

**Key Works:** Membrane Bioreactors (MBRs), Critical Flux, Mixed Liquor Suspended solids (MLSS), Extra-cellular Polymeric Substances (EPS), Ametryn

## 1 Introduction

Membrane Bioreactor (MBR) process, which is a combination of biological treatment and membrane filtration for separation of biomass, is one of the most novel wastewater treatment processes available at present. Bioreactor and membrane filtration cannot be considered as individual unit operations in MBRs, as these processes interact in many different ways. For the past two decades, many MBR plants have been installed in the treatment of domestic and industrial wastewater in the world. MBR technology is now becoming very popular at an approximate market value of US$217 million and a growth rate of 10.9% in 2005 (Simon Judd, 2007) due to its wide array of advantages over conventional treatment technologies, such as the production of superior quality of treated effluent, confining to smaller footprints, higher efficiency in removal of micro-pollutants and persistent organic pollutants and its ability to produce higher quality effluent even when the sludge is bulked. The demand for MBR systems increases steadily because they are now becoming more cost-effective, due to continuous fall in the costs of membrane module and related accessories that could be associated with high competition and advances in technology as well as the imposition of more stringent environmental laws and regulations in every state and region in the world. Due to fast-growing industry applications of MBR technology in wastewater treatment, the number of related research studies continued to increase for finding solutions to the presently identified drawbacks of MBR systems (mainly fouling of membrane) and for optimization of their performance (especially in nutrient removal, the treatment of micropollutants such as pesticides, herbicides, pharmaceuticals, etc.), to use them as a reliable treatment process.

MBRs mainly comprises of either microfiltration or ultrafiltration and as shown in Figure 1; in the submerged MBR systems the membranes are placed inside (Flat-Sheet or Hollow Fibre membranes) bioreactors and in the side-stream MBR systems the membranes (multi-tube/ tubular) are placed outside the bioreactor (Simon Judd, 2007 and Le-Clech et al., 2006). Presently, most of the MBRs are operated aerobically (98%) and the rest are anaerobically (Mulligan and Gibbs, 2003). In submerged MBRs, air is supplied for biodegradation and membrane cleaning (coarse bubbling).

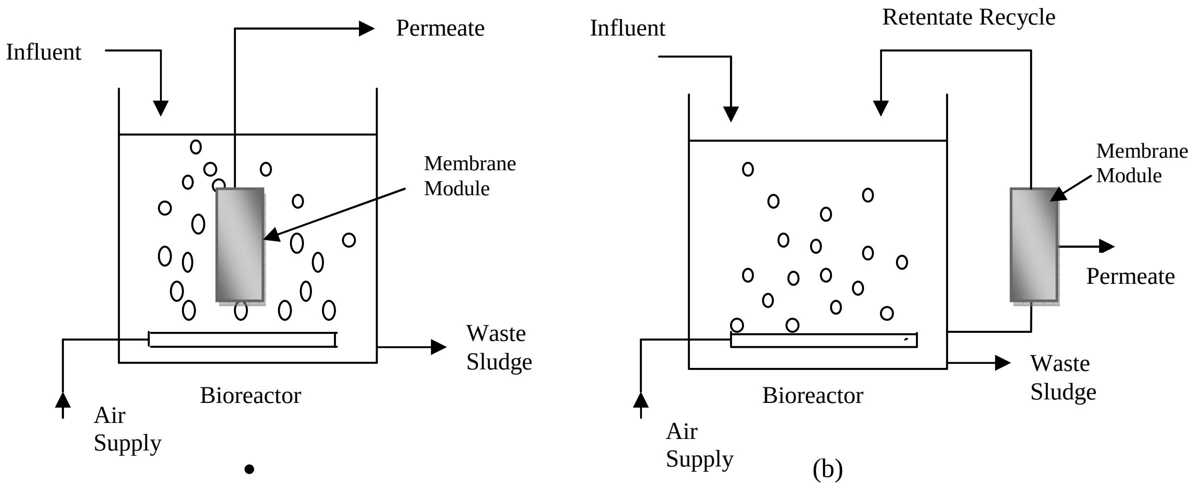



Figure 1 – *Configurations of MBR Systems: (a) Submerged MBR; (b) Side-Stream MBR*

Membrane fouling, which is caused due to the restriction, occlusion or blocking of membrane pores (Simon Judd, 2007) at the surface of the membrane, reduces the permeate flux (volumetric flow rate per unit membrane area) through the membrane. Thus, fouling is considered as the main obstacle to the widespread application of MBR. Fouling of membrane is mainly caused due to physical (nominal particle size of microbial flocs), chemical (hydrophobicity) and biological (extra-cellular polymeric substances (EPS) and viscosity) factors related to biomass. According to Meng et al. (2009), fouling mechanisms in a MBR are: (a) adsorption of solutes and colloids within or on membrane surface; (b) deposition of sludge flocs onto the membrane surface; (c) formation of cake layer on the membrane surface; (d) detachment of foulants attributed mainly to shear forces; (e) the spatial and temporal changes of the foulant composition such as the change of microbial community and biopolymer components in the cake layer during the long term operation. Most of the previous research work (Laspidou and Rittmann, 2002; Jang et al., 2006; Le-Clech et al., 2006; and Rosenberger et al., 2006) confirmed that Soluble Microbial Products (SMP which is referred to as free EPS) and bound EPS (eEPS), which are secreted by microorganisms, are the main organic compounds that cause fouling of membrane. Free and bound EPS mainly consist of polysaccharides (carbohydrates) and protein, and they play a major role in the formation of cake and gel layers on the membrane.

Operating MBRs at subcritical flux (below the "critical flux", where the flux starts to form the cake or gel layer on the membrane surface) is considered as one of the most practical strategies to control the fouling of membranes in MBR. In addition to this, subcritical flux operation reduces the consumption of energy and hence minimizes the operational cost of MBR. Field, et al. (1995) originally introduced the concept of the critical flux in microfiltration using an empirical approach and they defined the "critical flux" as "a flux below which a decline of flux with time does not occur (that is at subcritical flux, where $\frac{dTMP}{dt} = TMP' = 0$) and above which (supercritical flux) fouling is observed". However subsequent to that, Le-Clech et al. (2003) showed that a zero rate of TMP increase may never be obtained ($TMP' \neq 0$) during their short-term (common flux step method) critical flux determination tests carried out for synthetic and real sewage. Since then, different types of short-term critical flux determination and long-term sub-critical flux operational studies have been carried out under different feed-wastewater characteristics, biomass/sludge conditions and operating operations (Bouchot et al., 2006; Defrance and Jaffrin, 1999; Fan et al., 2006; Fane et al., 2002; Ndinisa et al., 2006; Torre et al., 2009; Van der Marel et al., 2009;

Le-Clech et al., 2003; Ognier et al., 2004; Saroj et al., 2008; Guglielmi et at., 2007a and b; Jinsong et al., 2006).

Feed-wastewater characteristics influence the mixed liquor/ sludge conditions (mixed liquor suspended solids (MLSS), SMP and eEPS) of MBRs. Therefore, mixing micropollutants such as herbicides to the feed wastewater would have an impact on the production of SMP and eEPS, and hence to the membrane filterability and fouling of membrane. The value of critical flux is a measure of fouling of membrane and the critical flux values for the laboratory-scale MBR system is evaluated and compared in this study to identify the influence of herbicides in fouling of membrane. This paper discusses the results obtained during the critical flux tests, which were carried before and after introduction of Ametryn to the MBR system.

Ametryn, which is a herbicide, is commonly used for controlling weeds (Table 1) in farmlands located in the Great Barrier Reef (GBR) Catchments in North Queensland (Australia). Ametryn falls to the category of second generation herbicides (Photosystem II) and it is fairly persistent and bio-accumulated in the environment. Therefore, Ametryn that is found in very low concentrations (a micropollutant having a concentration of µg/L or ng/L) is also considered as a Persistent Organic Pollutant (POP). A comprehensive review on impacts, existence, transport and treatment of these herbicides found in GBR catchments has been carried out elsewhere (Navaratna et al., 2010). As a broad objective of this overall research study, the laboratory-scale MBR is researched to optimise the removal of Ametryn from wastewater, while studying critical flux determination and subcritical operations of this MBR system. This paper also describes the early performance of Ametryn removal from this MBR system during the critical flux determination studies.

## 2 Material and Methods

### *2.1 Experimental setup*

Figure 2 shows the laboratory-scale MBR system installed at the hydraulics laboratory at School of Engineering, James Cook University, Townsville, Australia. The reactors are made out of Perspex and the maximum hydraulic capacities of the feed tank and the MBR are 50 and 15L respectively. A hollow fibre polyethylene (PE) membrane module (pore size 0.4µm, effective area 0.2m$^2$) is submerged in the MBR reactor. Air to the MBR is supplied from the central compressed air system via air regulators and valves, an air flow meter and perforated PVC manifold approximately with 20 holes (diameter around 1.5mm for providing coarse bubbling aeration) and installed at the base of the MBR. As a backup air supply, a portable compressor is also used. Peristaltic pumps are used to feed the MBR tank at a uniform feed rate and to pump out permeate (treated effluent) from the MBR through the membrane. A vacuum pressure gauge is fitted to measure TMP. Peristaltic pumps are connected to an electronically controlled timer to operate them intermittently (12 minutes “on” and 3 minutes “off”). One of these pumps is used when required for backwashing the membrane with treated water, which has very low turbidity.

The recipe of synthetic wastewater fed to the MBR system during this study consists of Glucose ($C_6H_{12}O_6$ – 710mg/L), Ammonium Acetate ($CH_3COONH_4$ – 200mg/L), Sodium Hydrogen Carbonate ($NaHCO_3$ – 750mg/L), Ammonium Chloride ($NH_4Cl$ – 30mg/L), Potassium Di-Hydrogen Phosphate ($KH_2PO_4$ – 30mg/L), Potassium Hydrogen Phosphate ($K_2HPO_4$ – 60mg/L), Magnesium Sulphate ($MgSO_4.7H_2O$ – 50mg/L), Calcium Chloride ($CaCl_2.2H_2O$ – 30mg/L) and Sodium Chloride (NaCl – 30mg/L). In addition to these chemical compounds, Ametryn was added 1 mg/L approximately. In order to prepare the stock solution, a precisely measured quantity of Ametryn was dissolved in methanol, mixed with distilled water and then methanol was evaporated. The COD concentration of synthetic feed wastewater was maintained around 700±50mg/L.

Activated sludge (approximately 8,000 mg/L) was brought from the Cleveland Bay Wastewater Purification Plant in Townsville (QLD, Australia) and acclimatized in the bioreactor. The laboratory-scale MBR system has been operated for over 400 days continuously adjusting influent, sludge and operating parameters.

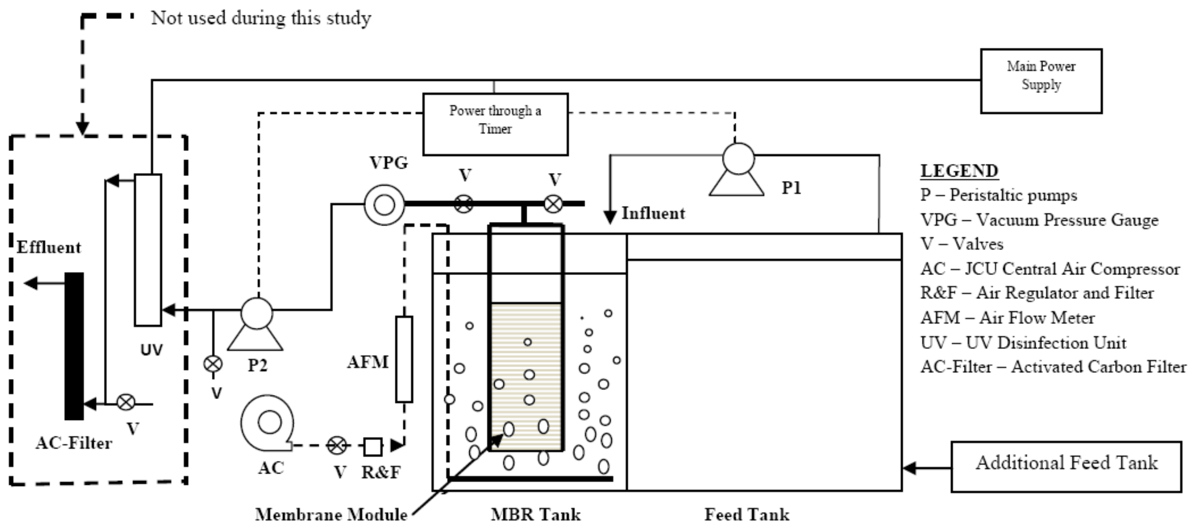
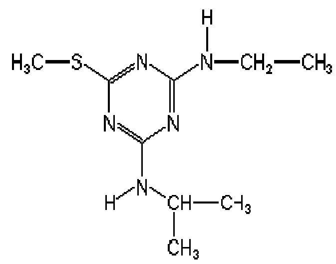



Figure 2: *Schematic of the Experimental Setup*

Table 1: *Characteristics of Ametryn*

| | | |
|---|---|---|
| **Molecular Weight (g/mol)** | 227.33 |  |
| **Molecular Formula** | $C_9H_{17}N_5S$ | |
| **Melting Point ($^oC$)** | 84-85 | |
| **Appearance** | White Powder | |
| **Solubility** | 185 mg/L (water 20$^o$C) and readily dissolves in solvents (acetone) | |
| **Purpose** | methyl-thio-triazine herbicide to control grass | |
| **IUPAC Name** | N2-ethyl-N4-isopropyl-6-methylthio-1,3,5-triazine-2,4-diamine | |

## *2.2 Laboratory analysis*

During these studies, dissolved oxygen (DO), pH and turbidity were measured using YSI DO 200 dissolve oxygen meter, WP-80 TPS pH and temperature meter and HACH 2100P turbidimeter respectively. Mixed liquor suspended solids (MLSS) concentration was analysed using the standard methods (1985). COD measurements were carried out adopting Photometric method using Spectroquant COD cell test kits and Thermo-reactor TR-320. EPS extraction was carried out using the method stated by Bin et al. (2008) with a slight modification. Initially, a 100ml of mixed liquor sample was allowed to settle for 45 minutes to 1 hour and the supernatant was removed. The settled sediment/sludge was then diluted with 40ml of distilled water and mixed in a mechanical shaker for 5 minutes at 150 rpm. Then the diluted sludge mixture was centrifuged at 8000 rpm for 10 minutes and the supernatant was collected, which is considered as soluble microbial products (SMP) or free EPS. Subsequent to that the remaining sludge was re-suspended with 40 mL of 0.1N NaOH solution allowing it to mix thoroughly in the same mechanical shaker at 150 rpm for 120 minutes before it was centrifuged again at 13,000 rpm for 15 minutes at 4$^o$C. Finally, the supernatant (eEPS or bound EPS) was extracted. Both SMP and eEPS samples were neutralised separately with diluted HCl. SMP and eEPS Protein and Carbohydrate concentrations were determined by using Lowry method (Lowry et al., 1951) with bovine serum albumin as reference and Dubois et al. (1956) method with glucose as standards respectively. Diluted Sludge Volume Index (DSVI) was estimated by diluting the mixed liquor by four folds, allowing solids to settle for 30 minutes in a 1L measuring cylinder. High performance liquid chromatography (HPLC) method was used to analyse the feed and permeate Ametryn concentrations.

### *2.3 Critical flux determination methods*

The critical flux was determined in different occasions in MBR operation by changing the controlling parameters of MBR. Several short term critical flux determination experiments were carried out using the common flux step method, which was described by Le Clech et al. (2003). The flux step durations were chosen as 20 and 40 minutes for the experiments discussed in this paper. Flux step height was kept as a constant throughout these studies at 3 $L/m^2/h$. The tests were carried out with and without intermittent permeate suction for above flux step durations. Experiments were conducted before and after introduction of Ametryn to the MBR system. The membrane module was cleaned chemically using 3g/L NaOCl solution as per the procedure described by the manufacturer before every experiment.

## 3 Theory/ Calculations

The flux through the membrane $J$ ($m^3m^{-2}s^{-1}$) can be related to the applied trans-membrane pressure $\Delta TMP$ (Pa), viscosity of the fluid $\mu$ (Pa s) and the membrane resistance $R$ ($m^{-1}$) according to Darcy's Law:

$$J = \frac{\Delta TMP}{\mu R} \quad (1)$$

$$R = R_m + R_n + R_c + R_p \quad (2)$$

$$R = R_m + R_f \quad (3)$$

Where, $R_m$ is the hydraulic resistance of the clean membrane, $R_n$ is the irreversible resistance due to fouling, $R_c$ is the membrane resistance due to cake or gel layer formed by concentration polarization (mainly in ultrafiltration), deposition of suspended solids, colloids and solutes, and $R_p$ is the membrane resistance due to pore blocking occurred by deposition of soluble and colloidal substances. $R_f$ is the sum of $R_m$, $R_n$ and $R_p$ and depends on applied trans-membrane pressure and the system mass transfer properties. For microfiltration, the fouling by concentration polarization could be ignored due to the large size of particles retained in the reactor (Lim and Bai, 2003).

During these short-term critical flux determination experiments, pressure of the mixed liquor in the reactor has to be kept constant and the TMP assumed to vary only with changes in permeate pressure due to fouling. For each flux step, three TMP values were recorded (initial TMP=$TMP_i$, intermediate TMP=$TMP_{im}$ and final TMP=$TMP_f$). Then the following parameters were estimated;

Initial TMP increase, $\Delta TMP_0 = TMP_i^n - TMP_f^{n-1}$ (4)

Rate of increase of TMP, $\dfrac{dTMP}{dt} = \dfrac{TMP_f^n - TMP_i^n}{t_f^n - t_i^n}$ (5)

Average TMP, $TMP_{ave} = \dfrac{TMP_f^n + TMP_i^n}{2}$ (6)

In the above expressions, "$n$", "$i$" and "$f$" are denoted the flux step number, initial and final observations made for each run, respectively.

## 4 Results and Discussion

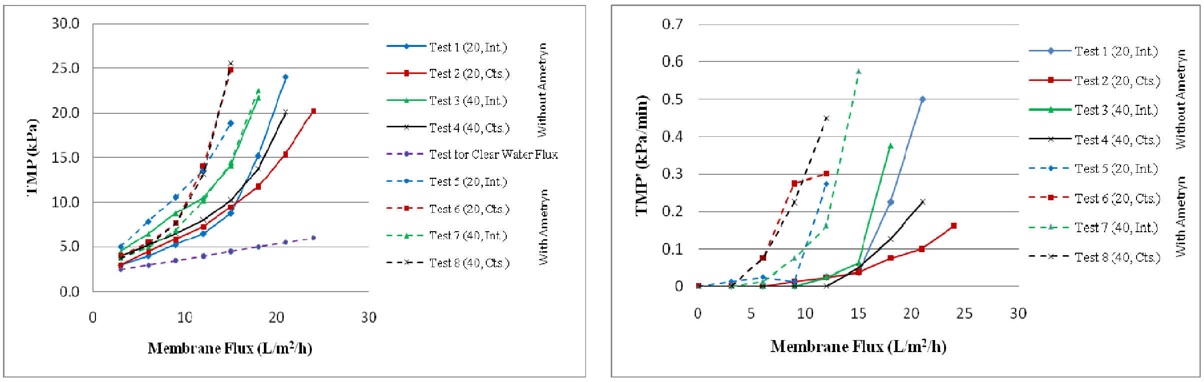
Table 2 shows the results obtained for the eight short-term (common flux step method) critical flux determination tests (Test 1 through 8) for before and after the introduction of Ametryn. When comparing the critical flux values obtained from tests carried out before and after the introduction of Ametryn, it can be seen that there is a significant reduction of Ametryn in MBR permeate (40-60%) in the tests carried out after introducing Ametryn. On the other hand, by observing the critical flux values obtained for Tests 5 through 8, the tests carried out with intermittent permeate suction (12 minutes "on" and 3 minutes "off") show higher values of critical flux, compared to that of the tests carried out with continuous permeate suction mode. However, this pattern was not observed for Tests 1 through 4, probably due to the differences in the way the cake layer formed during the two different wastewater and MBR mixed liquor conditions before and after the addition of Ametryn.

Table 2: *Operating conditions and results during critical flux determination tests*

| **Parameter** | | **← Before Ametryn →** | | | | **← After Ametryn →** | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | **Test 1** | **Test 2** | **Test 3** | **Test 4** | **Test 5** | **Test 6** | **Test 7** | **Test 8** |
| Suction Mode | | INT | CTS | INT | CTS | INT | CTS | INT | CTS |
| Flux step duration (minutes) | | 20 | 20 | 40 | 40 | 20 | 20 | 40 | 40 |
| Average MLSS (mg/L) | | 7478 | 7478 | 10383 | 10383 | 7962 | 7962 | 9195 | 9195 |
| DSVI (mL/g-MLSS) | | 123 | 123 | 150 | 150 | 156 | 156 | 126 | 126 |
| Average SMP (Soluble EPS)/ (mg/L) | Protein | 138.53 | 138.53 | 146.70 | 146.70 | 76.87 | 76.87 | 112.24 | 112.24 |
| | Carbohydrates | 39.43 | 39.43 | 50.99 | 50.99 | 64.59 | 64.59 | 77.66 | 77.66 |
| Average eEPS (Bound EPS)/ (mg/L) | Protein | 913.09 | 913.09 | 959.64 | 959.64 | 815.76 | 815.76 | 712.99 | 712.99 |
| | Carbohydrates | 228.65 | 228.65 | 270.31 | 270.31 | 210.87 | 210.87 | 253.69 | 253.69 |
| Estimated Critical Flux ($L/m^2/h$) – when dP/dt (TMP')>0.075kPa/min | | 15-18 | 18-21 | 15-18 | 15-18 | 9-12 | 6-9 | 9-12 | 6-9 |

INT – Intermittent Permeate flux (12 minutes "ON" and 03 minutes "OFF")
CTS – Continuous Permeate Flux

The components of EPS (protein and carbohydrates of soluble EPS-SMP and bound EPS-eEPS) in mixed liquor of a MBR system is considered as the most influential organic substances that cause fouling of membrane. According to EPS results shown in Table 2, it can be seen that the concentrations of protein in SMP and bound EPS are less in Tests 5 through 8 compared to that of Tests 1 through 4. This describes that this reduction of protein in SMP and bound EPS have not been contributed significantly to increase the critical flux values in this study. However, it can be seen that more concentration of carbohydrates in SMP (52-64%) for the tests, which were carried out after introducing Ametryn. It was found that the critical flux values are significantly smaller when Ametryn was introduced, compared to that of tests carried out before introducing Ametryn. Thus, concentration of carbohydrates in SMP of mixed liquor is the main organic foulant that could be causing the fouling of membrane.

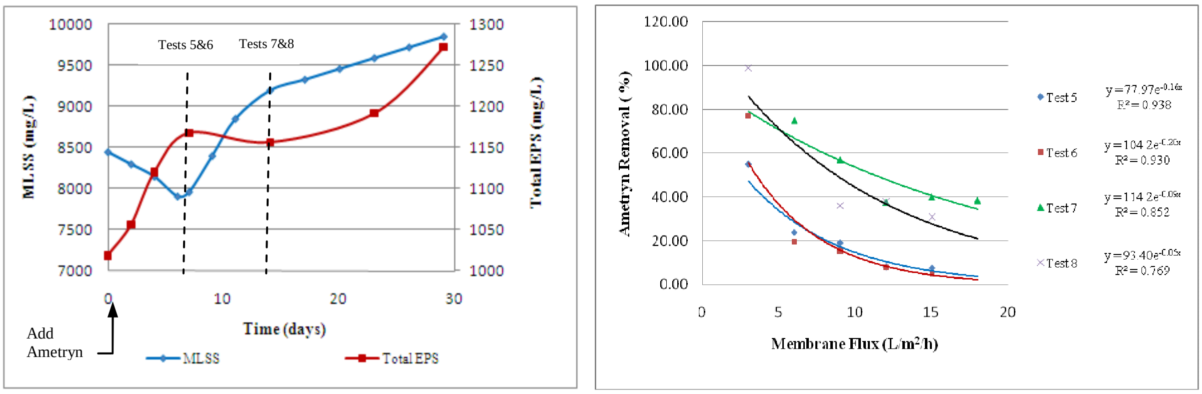



(a) (b)

Figure 3: *Short-term flux-step test results: (a) Average TMP and (b) TMP' versus membrane flux*

Figure 3(a) shows the average TMP variations with membrane flux during the short-term flux step tests that were carried out before and after introduction of Ametryn to the MBR system. Field et al. (1995) defined two distinct forms of critical flux values namely strong and weak. The strong form is the flux at which the TMP starts to deviate (exponentially) from the clear water flux curve, which is linear as shown in Figure 3(a). On the other hand, the weak form is the flux that shows a significant fouling of membrane from the start-up of the filtration and therefore, the trend curves for TMP against flux of Tests 1 through 8 are above that of the clear water flux curve.

Figure 3(b) shows the variation in the rate of fouling of membrane TMP' with membrane flux for Tests 1 through 8. These trend curves are used to estimate the critical flux values (Table 2) of each test. In this study, the critical flux values were determined for the flux value corresponding to $TMP' > 0.075$ kPa/min and from Table 2 it can be seen that the critical flux decreased significantly after the introduction of Ametryn irrespective of the type of test conducted.




(a) (b)

Figure 4: *(a) MLSS and Total EPS variation during the first 29 days after the introduction of Ametryn (b) Ametryn removal by the MBR during short-term critical flux tests*

Figure 4(a) shows the variation of the concentrations of MLSS and total EPS (soluble and bound EPS) of mixed liquor of the MBR during the first 29 days of operation after the introduction of Ametryn. During this period, MBR was operated at a flux of 5.1L/m$^2$/h with intermittent permeate suction (12 minutes “on” and 3 minutes “off”) and an infinite sludge retention time (SRT) as there was no sludge disposal carried out intentionally. From Figure 4(a), it can be seen that the concentrations of MLSS and total EPS show opposite and different trends (total EPS increases, when MLSS decreases). This confirms that the concentration of EPS does not fluctuate always with MLSS positively or negatively in MBR operation.

Table 3: *Variation of Protein and Carbohydrates in SMP and eEPS from the day that Ametryn was introduced to the laboratory-scale MBR system*

| **Days elapsed from the introduction of Ametryn** | **MLSS/ (mg/L)** | **SMP** | | **eEPS** | |
|---|---|---|---|---|---|
| | | **Protein/ (%)** | **Carbohydrates/ (%)** | **Protein/ (%)** | **Carbohydrates/ (%)** |
| 7 | 7962 | -7.80 | -11.46 | 26.66 | -3.35 |
| 14 | 9195 | 34.63 | 6.46 | 10.70 | 16.27 |
| 29 | 9847 | -18.29 | -3.70 | 40.72 | 4.69 |

Negative values indicate "reduced % of concentration" compared to that of the day Ametryn was introduced to the MBR system

By analysing the results illustrated in Table 3, it can be seen that protein in eEPS is the only EPS component that has been increased after adding Ametryn to the system. However, this production of protein in eEPS is reduced after the day 7, but showed an increase of protein in SMP and carbohydrates of eEPS. However, this change in the production of EPS components during the day 7 and 14 has resulted to maintain the total EPS at a stable level. Subsequent to this period, it again shows a higher production of protein in eEPS and that contributes the total EPS in MBR to depict greater rate of increase as shown in Figure 4(a). Although, reason/s for these fluctuations of EPS components are not confirmed in this paper, the studies are being continued to analyse the impact of herbicides and pesticides such as Ametryn on the production of EPS in MBR systems.

Figure 4(b) shows the variation of Ametryn removal % with membrane flux during the critical flux determination experiments carried out after the introduction of Ametryn to the synthetic feed of the laboratory-scale MBR system. The percentage of Ametryn removal declines exponentially with the increase in membrane flux. Tests 5 and 6, which were carried out with shorter flux-step duration (20 minutes) and lower MLSS (7962mg/L), show a greater decrease in Ametryn removal with membrane flux compared to that of Tests 7 and 8, which had longer flux step duration of 40 minutes and higher MLSS (9195mg/L). Further, both Tests 7 and 8 show higher removal of Ametryn (about 50-60% for the critical flux of those tests) compared to the removal observed in Tests 5 and 6. When comparing Tests 7 and 8, it can be observed that Test 7, which was operated under intermittent permeate suction mode, gives a better removal of Ametryn compared to Test 8, which was studied under continuous permeate suction mode at similar MLSS. This study is being continued to observe the improvement in the removal of Ametryn the MBR system used in this study.

## 5 Conclusions

In this study, critical flux values for a laboratory-scale MBR (PE membrane - 0.4μm and 0.2 $m^2$) were obtained using short-term (common flux-step method) tests under different hydrodynamic and sludge environments. Synthetic solutions with and without Ametryn were used as the feed for MBR. It could be seen that carbohydrate in SMP was higher (52-64%) in tests that were carried out after Ametryn was added, and this could have probably caused higher fouling propensity. However, on the other hand, it was found that production of protein in eEPS had been increased significantly after adding Ametryn to the MBR feed. Further, at early stages of operation (within the first month), it was seen that a removal of 50-60% of Ametryn by the MBR for a feed solution that contained 1mg/L of Ametryn.

### References

Bin, Z., Baosheng, S., Min, J., Taishi, G., Zhenghong, G., 2008. Extraction and analysis of extracellular polymeric substance in membrane fouling in submerged MBR. Desalination. 227, 286 – 294

Bouchot, M. E., Espinasse, B., Cabassud, C., 2006. Fouling effects and critical flux in relation with module design and aeration conditions for a side stream outside/in filtration system. Desalination. 199, 487-489.

Defrance, L., Jaffrin, M.Y., 1999. Comparison between filtration at fixed transmembrane pressure and fixed: application to a membrane bioreactor used for wastewater treatment. Journal of Membrane Science. 152. 203-210.

Dubois, M., Gilles, K. A., Hamilton, J. K., Rebers, P. A., Smith, F., 1956. Colorimetric method for determination of sugars and related substances. Analytical Chemistry. 28, 350-356

Fan, F., Zhou, H., Hussain, H., 2006. Identification of wastewater sludge characteristics to predict critical flux for membrane bioreactor processes. Water Research. 40, 205-212.

Fane, A. G., Chang, S., Chardon, E., 2002. Submerged hollow fibre membrane module – design options and optional considerations. Desalination. 146, 231-236.

Field, R.W., Wu, D., Howell, J.A., Gupta, B.B., 1995. Critical flux concept for microfiltration fouling. Journal of Membrane Science. 100, 259-272.

Guglielmi, G., Chiarani, D., Judd, S. J., Andreottola, G., 2007a. Flux criticality and sustainability in a hollow fibre submerged membrane bioreactor for municipal wastewater treatment. Journal of Membrane Science. 289, 241-248.

Guglielmi, G., Saroj, D. P., Chiarani, D., Andreottola, G., 2007b. Sub-critical fouling in a membrane bioreactor for municipal wastewater treatment: Experimental investigation and mathematical modelling. Water Research. 41, 3903-3914.

Jang, N., Shon, H., Ren, X., Vigneswaran, S., Kim, I.S., 2006. Characteristics of bio-foulants in the membrane bioreactor, Desalination 200 (2006) 201-202

Jinsong, Z., Chuan, C. H., Jiti, Z., Fane, A. G., 2006. Effect of Sludge Retention Time on Membrane Bio-Fouling Intensity in a Submerged Membrane Bioreactor. Separation Sci. and Tech. 41: 7, 1313-1329.

Judd, S. (2007). The status of membrane bioreactor technology. Trends Biotechnology. 26(2) 109-116.

Laspidou, C.S., Rittmann, B.E., 2002. A unified theory for extracellular polymeric substances, soluble microbial products, and active and inert biomass, Water Research 36 (2002) 2711-2720

Le-Clech, P., Chen, V., Fane, T.A.G., 2006. Fouling in membrane bioreactors used in wastewater treatment, journal of Membrane Science 284 (2006) 17-53

Le-Clech, P., Jefferson, B., Chang, I.S., Judd, S.J., 2003. Critical flux determination by the flux-step method in a submerged membrane bioreactor. Journal of Membrane Science. 227, 81-93.

Lim, A.L., Bai, R., 2003. Membrane fouling and cleaning in microfiltration of activated sludge wastewater, Journal of Membrane Science 216 (2003) 279-290

Lowry, O. H., Rosebrough, J., Farr, A. L., Randall, R. J., 1951. Protein measurement with Folin Phenol reagent. Journal of biological Chemistry. 193, 265-275

Meng., F., Chae, S., Drews, A., Kraume, M., Shin, H., Yang., F., 2009. Recent advances in membrane bioreactors (MBR): Membrane fouling and membrane material, Water Research 43 (2009) 1489-1512

Mulligan, C.N., Gibbs, B.F., 2003. Innovative Biological Treatment Process for water in Canada, Water Qual. Res. J. Canada, 200 Volume 38, No. 2, 243-265

Navaratna, D., Shu, L., Jegatheesan, V., 2010. Existence, impacts, transport and treatments of herbicides in Great Barrier Reef catchments in Australia, in: Virkutyte, J., Varma, R. S., Jegatheesan, V. (Eds.), Treatment of Micropollutants in Water and Wastewater. IWA Publishing, London, UK, Chapter 11, pp. 425-463.

Ndinisa, N. V., Fane, A. G., Wiley, D. E., 2006. Fouling control in a submerged flat sheet membrane system: Part I – Bubbling and hydrodynamic effects. Separation Science and Technology. 41: 7, 1383-1409.

Ognier, S., Wisniewski, C., Grasmick, A., 2004. Membrane bioreactor fouling in sub-critical filtration conditions: a local critical flux concept. Journal of Membrane Science. 229, 171-177.

Rosenberger, S., Laabs, C., Lesjean, B., Gnirss, R., Amy, Jekel, M., Schrotter, J.C., 2006. Impact of colloidal and soluble organic material on membrane performance in membrane bioreactors for municipal wastewater treatment, Water research 40 (2006) 710

Saroj, D.P., Guglielmi, G., Chiarani, D., Andreottola, G., 2008. Subcritical fouling behaviour modelling of membrane bioreactors for municipal wastewater treatment: The prediction of the time to reach critical operating condition. Desalination. 231, 175-181.

Standard Methods for the examination of water and wastewater(1985), 16th edition. APHA. AWWA. WPCF (ISBN 0-87553-131-8)

Torre, T. D., Iversen, V., Moreau, A., Stüber, J., 2009. Filtration characterization methods in MBR systems: A practical comparison. Desalination and Water Treatment. 9, 15-21.

Van der Marel, P., Zwijnenburg, A., Kemperman, A., Wessling, M., Temmink, H., Van der Meer, W., 2009. An improved flux-step method to determine the critical flux and the critical flux for irreversibility in a membrane bioreactor. Journal of Membrane Science. 332, 24-29.

## Acknowledgements

We thank Dr. Shane Askew of Advanced Analytical Center of James Cook University for analysing the concentration of Ametryn in permeate and feed samples using HPLC, Dr. Joy Morgan for releasing the Chemistry Laboratory for analysing COD and preparation of Ametryn stock solution and Dr. Jennifer Elliman for releasing the Bacteriology Laboratory of Veterinary and Biomedical Sciences for extracting and analysing of EPS.

# STONEWORTS (CHARA, NITELLA) AS A TOOL FOR SUSTAINABLE TREATMENT OF LOW STRENGTH WASTEWATERS

P.I.A. Gomes[1], T. U. L. Liyanage[2], N.M.S. Wasanthi[3]

[1]Senior Lecturer, Department of Civil and Environmental Engineering, University of Ruhuna, Galle, Sri Lanka.
[1]E-mail: ishan_gomes@yahoo.com; ayantha@cee.ruh.ac.lk
[1]Telephone: +94-77-1380742; Fax: + 94-11-2651216

[2]Civil Engineer, SMEC International (Pty) Ltd, No. 26, Jawaththa Road, Colombo 5
[2]E-mail: thiruniudeshi@gmail.com
[2]Telephone: +94-77-3029428

[3]Instuctor, Department of Civil and Environmental Engineering, University of Ruhuna, Galle, Sri Lanka.
[3]E-mail: wasanthin_m_s@yahoo.com
[3]Telephone: +94-77-6301881

**Abstract:** Lack of sustainability is evident in many existing wastewater treatment technologies. Wetlands and/or plant mediated remediation (phytoremediation) seemed to be a good answer in this regard. However greatest problem with this technology is the fate of plants that going to be rich in a particular contaminant, as upon senescence the contaminant it accumulated will get released to water. Thus as an answer to this shortcoming we have tested the applicability of stoneworts for the phytoremediation of low polluted wastewaters. Two independent experimentations were conducted to check the remediation of phosphorous (P) and heavy metals (chromium and cadmium). Results suggest stoneworts managed to store considerable portion of all these contaminants in redox insensitive forms. Thus we conclude stonewort mediated phytoremediation to be promising sustainable technique within the ranges and conditions investigated.

***Keywords:*** *Heavy metals, Phosphorous, Phytoremediation, Stoneworts*

## 1. Introduction

The increasing shortage of water in the world along with rapid population increase and development gives reason for concern and the need for appropriate water management practices. Many present day systems are a “disposal-based linear system”, where the treated wastewater is disposed to waterbodies (e.g. river) or land (e.g. agricultural land). Even though the effluent meet the statutory requirement frequent disposal will ultimately make the ecosystem of the receiving body polluted in the long run due to processes like bioaccumulation. Thus the problem with the current treatment technologies is they lack sustainability. As such sustainability concepts like reuse, recycle etc., are not been practiced. It should be noted 99% of the wastewater is composed of water and the remaining contain vital nutrients like nitrogen, phosphorous (P), potassium and sulphur [1]. Disposal-based linear systems not only omit sustainable goals but also not cost effective.

Phytoremediation is the use of plants and their associated microbes for environmental cleanup [2]. This technology is an effective cleanup technology for variety of organic and inorganic pollutants. Inorganic pollutants that can be phytoremediated include plant macronutrients (nitrate and phosphate), plant trace elements [chromium (Cr), copper, iron, and manganese (Mn)) nonessential elements (cadmium (Cd), cobalt, fluorine, mercury). In case of aquatic plants, uptake of heavy metals to produce an internal concentration greater than in the external environment appeared widespread [3].

Phytoremediation generally takes place in a wetland system (either constructed or natural). In wetland treatment, natural forces (chemical, physical, and solar) act together to purify the wastewater [1]. Wetland treatment technology in developing countries offers a comparative advantage over conventional, mechanized treatment systems because the level of self-sufficiency, ecological balance, and economic viability is greater. The system allows for total resource recovery [4]. Thus in the first glance it can be concluded wetlands as a feasible option and green building rating systems such as LEED™ [5] explicitly recommend it as a potential technology. However greatest problem of phytoremediation would eventually be the fate of plants used for the purpose which become rich in that particular contaminant. Thus harvesting the plants at regular intervals is necessary if not upon senescence and decomposition, accumulated heavy metals or similar contaminants will get re-enter to the water column. Thus to make a wetland system and/or phytoremediation more sustainable, considerable attention need to be given to the above issue. Thus here we are going to check the applicability of the aquatic macrophyte stoneworts (also known as charophytes) for the phytoremediation of nutrients and heavy metals.

Charophytes the growth form of characean algae are an obvious form of aquatic vegetation in many quiescent water bodies: fresh to brackish and temporary to permanent with a worldwide distribution [6]. Many forms of charophytes are subject to calcification, which in the form of $CaCO_3$, takes place on stems, branchlets and on the surface of oogonia [7]. Calcification accompanies the photosynthetic utilization of bicarbonate [7].

## 2. Objectives and Methodology

The objective of this study is to assess the advantages of stoneworts, the growth form of characean algae in a wetland system. We specifically investigated the following:

1) To study the applicability of locally found stoneworts to accumulate P and selected metals and heavy metals
2) To discuss the suitability of stoneworts colonized shallow wetlands for ground water recharge

***Experiment 1***

Two sets of microcosms (5 L laboratory fish tanks) were maintained for a period of one year planted with: two stoneworts (*Chara* sp. and *Nitella* sp.). After one year plants were analyzed for total P and carbonate bound P as described by Siong and Asaeda [6] and, Gomes and Asaeda [7]. Plants were grown in a basic water sand combination: several times washed commercially available river sand and city tap water. The results will be compared with the results of two vascular angiosperm herbs (*Najas marina* and *Vallisneria gigantea*) as reported by Siong and Asaeda [6].

***Experiment 2***

Five sets of microcosms (1 L beakers) were maintained for a period of one year planted with *Nitella* sp. Three microcosms with plants; no heavy metals (control), 0.2 mg/L $Cr^{6+}$ and 0.01 mg/L Cd. The other two without plants were given the same heavy metal treatments; 0.2 mg/L $Cr^{6+}$ and 0.01 mg/L Cd. All units contained 40 mg/L calcium (Ca). After one year, plants and sediments were sampled. Plants were analyzed for the relevant heavy metal in alkaline and acidic regions of the main thalli. A sequential fractionation procedure to determine Cr or Cd speciation as exchangeable, carbonate bound, organic bound and residual was carried out according to Tessier et al. [8] for sediment. For other water analyses of both experiments were carried out according to APHA [9].

## 3. Phosphorous fractionation of plants

The total P and carbonate bound P fraction of plants of experiment 1 are shown in Table 1. The total P of *V. gigantea* and *N. marina* was significantly higher than *Chara* sp. and *Nitella* sp. (*ANOVA; $P<0.05$*). It should be noted even when compared on ash-free dry weight basis the total P of *V. gigantea* and *N. marina* observed to be significantly higher than the *Chara* sp. and *Nitella* sp. (data not shown). However the carbonate bound P content of *Chara* sp. and *Nitella* sp. observed to be significantly higher than *V. gigantea* and *N. marina* (*ANOVA; $P<0.05$*). The carbonate bound P fraction of *V. gigantea* and *N. marina* was less than 1 %. Thus the remaining 99% of the total P should contain water soluble P and/or organic P. Many water soluble and organic P compounds are bio-available. For example water soluble P contains inorganic P forms, which are immediately available for planktonic microorganism uptake. Unlike the two angiosperm species discussed the carbonate bound P fraction of stoneworts was notably high, 10% and 8% for *Chara* sp. and *Nitella* sp. respectively.

Table 1: *The total phosphorous (TP) and carbonate bound phosphorous (P) fraction of plants of experiment 1.*

| Sample | TP (mg/g) | Carbonate bound P fraction (%) |
|---|---|---|
| *Chara* sp. | 1.0 (0.05) | 10.1 (3) |
| *Nitella* sp. | 0.8 (0.01) | 8.2 (2) |
| *V. gigantea*[1] | 4.0 (0.12) | 0.9 (0.01) |
| *N. marina*[1] | 4.5 (0.06) | 0.3 (0.00) |

[1] source: Siong and Asaeda [6]

Rhizoid-bearing stoneworts are known to acquire P and also other nutrients primarily from the water column [45]. In contrast, vascular plants acquire P mainly from sediment via their roots and this result in high P content in plant biomass [7]. This is the main reason for the high P values observed in vascular plant tissues relative to charophytes [7]. Thus, vascular plants assimilation can result in reduction of nutrients in the water column. However, vascular submerged plants, upon senescence release the accumulated P again to the water column, making net P accumulation (in the long run) zero [9].

Decalcification, followed by co-precipitation of phosphate with $CaCO_3$ is an important process in the reduction of the bio-available P in the water column [10]. Calcium bound P in stoneworts has been discussed by Kufel and Kufel [11] referencing to sediment of lake that has been dominated by charophytes.

## 4. Hyperaccumulation of heavy metals by stoneworts

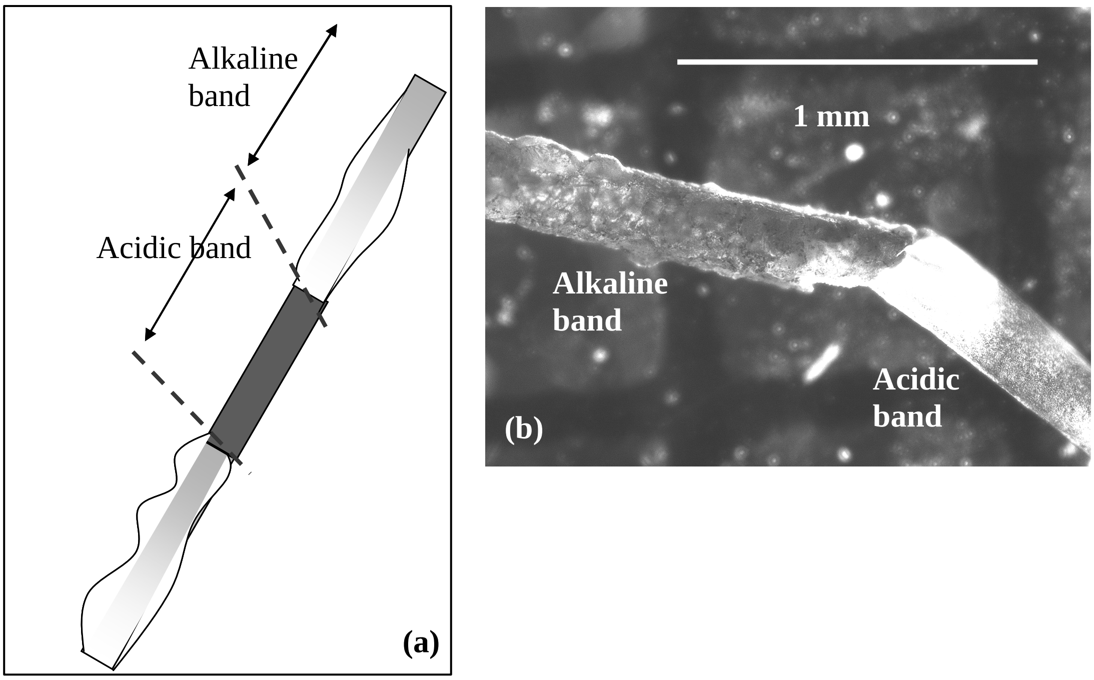



Figure 1: *(a) Conceptual layout and (b) microscopic view (Olympus, Japan) of acidic and alkaline bands, respectively.*

Table 2 illustrates the levels of Cr and Ca observed in alkaline and acidic areas of plants of the experiment 2. Alkaline areas contained 0.75 mg/g Cr, whereas 0.61 mg/g Cr in acidic areas. Thus alkaline areas contained about 55% of the total Cr of plants. It should be noted the alkaline areas had an ash content of > 90 % from its dry weight, compared to < 1 % in acidic areas. The ash content is comparable to the Ca levels of the respective regions and they can be used as alternatives when only one is available [12]. Thus after correcting for ash alkaline regions will give significantly high (*ANOVA*, $P < 0.05$) Cr content relative to the acidic regions. Similar results were obtained for Cd treated units (data not shown).

Table 2: *Chromium (Cr) and calcium (Ca) levels measured in alkaline and acidic areas of experiment 2. Parentheses give standard deviation for mean.*

| Fraction | Alkaline band | | Acidic band | |
|---|---|---|---|---|
| | Cr | Ca | Cr | Ca |
| Fraction (%) | 55.1 | 99.8 | 44.9 | > 0.2 |
| Concentration (mg/g) | 0.75 (0.09) | 74.8 (1.1) | 0.61 (0.13) | 0.1 (0.0) |

***Sequential fractionation of sediment for heavy metals***

Table 3 shows sequential fractionation of sediments of Cr treated units (experiment 2). The carbonate bound Cr observed to be the highest fraction. Next highest fractionation was organic bound. When compared with the results of microcosoms without plants but treated with heavy metals it was evident that these two fractions were indeed from plant detritus. Similar results were obtained for Cd treated units (data not shown).

Table 3: *Sequential fractionation of sediments carried out for the chromium treated units of the experiment 2. Parentheses give standard deviation for mean.*

| Fraction | % | mg/g |
|---|---|---|
| Exchangeable | 19.3 | 0.013(0.001) |
| Carbonate bound | 35.4 | 0.025(0.009) |
| Organic bound | 34.5 | 0.024(0.010) |
| Residual | 10.8 | 0.007(0.000) |

A thick marl bottom sediment layer frequently found beneath charophytes meadows is an evident that charophytes calcite can function as long term storage of Ca. Subsequent analysis conducted for these sediments found it contain not only Ca but also other elements in high levels. Alkaline areas to have extremely high levels of Ca are a well documented observation and similar results were obtained with this experiment. Apart from Ca, Mg is also known to get precipitated [7]. Alkaline areas to have precipitations of strontium (Sr) and Mn was reported by McConnaughey [13].

High Cr content in alkaline areas and carbonate bound fraction suggest Cr can get accumulated in processes associated with calcite. Two processes are possible: absorption/adsorption and coprecipitation. As significant percentage of Cr are in redox insensitive forms makes frequent harvesting unnecessary.

## 5. Advantages of using stoneworts in a wetland designed to treat low polluted wastewaters

The intention of sustainable design is to "eliminate negative environmental impact completely through skillful, sensitive design". Often domestic wastewaters after the secondary treatment could be regarded as low polluted. In many cases the effluent might have already met the standards stipulated by the authorities. However the effluent still contains pollutants at trace levels. Thus such an effluent might not be good for sustainable activities like groundwater recharge or irrigation. Freshwater wetlands can, in some circumstances, renovate added secondarily treated wastewater, thus providing an alternative to land or water disposal or expensive physical-chemical treatment processes [14]. Successful remediation of wastewater in wetlands is advantages in many ways. Wetlands can be used for groundwater recharge. This makes the wetland a closed loop treatment. Researchers have discovered ground water recharge of up to 20% of wetland volume per season [15]. However this needs extreme caution considering the possible contamination of groundwater if the recharge is meant by wetlands used for wastewater treatment. The plants we are proposing such as stoneworts are advantages in many ways. It not only accumulates nutrients like P, but also in redox insensitive forms. After the plant senescence most of the accumulated P bound with calcite will get precipitated in the wetland bottom. Thus in the long run resource recovery will be possible. It should be noted that P is an especially important nutrient to recycle, as the P in chemical fertilizer comes from limited fossil sources. Ability to accumulate heavy metals when present in trace levels will make this plant ideal for the treatment of industrial wastewaters. It should be noted treatment of industrial and domestic wastewaters should be done in separate wetlands. It is recommended to carryout pilot scale experimentation in this regard.

## References


1 Volkman, S., Sustainable Wastewater Treatment and Reuse in Urban Areas of the Developing World, M.S. thesis, Department of Civil and Environmental Engineering, Michigan Technological University, 2003

2 Salt D.E., Smith R.D., Raskin, I., Phytoremediation. *Annual Review of Plant Physiology and Plant Molecular Biology* 49:643-668,1998

3 Vymazal, J., 1995. Algae and element cycling in wetlands. Lewis Publishers, Chelsea

4 Rose, G.D., Community-Based Technologies for Domestic Wastewater Treatment and Reuse: Options for Urban Agriculture, N.C. Division of Pollution Prevention and Environmental Assistance, CFP Report Series: Report 27, 1999.

5 LEED (Leadership in Energy & Environmental Design) Green Building Rating System for new Construction & major Renovations, version 2.1, U.S. Green building council, 2002

6 Siong, K., Asaeda, T., Does Calcite Encrustation in *Chara* Provide a Phosphorus Nutrient Sink? *Journal of Environmental Quality* 35: 490-494, 2006

7 Gomes, P. I. A., Asaeda, T., Impact of calcium and magnesium on growth and morphological acclimations of *Nitella*: implications for calcification and nutrient dynamics, *Chemistry and Ecology (accepted), 2010.*

8 Tessier, A., Campbell, P. G. C., Bisson, M., Sequential extraction procedure for the speciation of particulate trace metals, *Analytical Chemistry* 51: 844–851, 1979.

9 APHA (American Public Health Association)/American Water Works Association/Water Environment Federation,. Standard Methods for the Examination of Water and Wastewater, 20th ed. Washington, D.C.,1998

10 Otsuki, A., Wetzel, R. G., Coprecipitation of phosphate with carbonates in a marl lake, *Limnology and Oceanography* 17:763-767, 1972.

11 Kufel, I., Kufel, L., Eutrophication process in a shallow, macrophytes-dominated lake: nutrient loading to and flow through Lake Łuknajno (Poland), *Hydrobiologia* 342-343: 387-394, 1997.

12 Kufel, I., Kufel, L., Chara beds acting as nutrient sinks in shallow lakes- a review, *Aquatic Botany* 72: 249-260, 2002.

13 McConnaughey, T., Calcification in *Chara corallina*: $CO_2$ hydroxylation generates protons for bicarbonate assimilation, *Limnology and Oceanography* 36: 619–628, 1991.

14 Kadlec, R. H., Tilton, D. L., Ewel, K. C., The use of freshwater wetlands as a tertiary wastewater treatment alternative. *Critical Reviews in Environmental Science and Technology,* 9: 185 – 212, 1979

15 Weller, M.W., Freshwater Marshes University of Minnesota Press: Minneapolis, Minnesota, 1981

# ECO FRIENDLY RESORT FOR TOURISM – A CASE STUDY AT ULAGALLA RESORT

[1]Mr. Ranjan Dissabandara, [2]Eng. Namal Peiris

[1] Business Promotional Manager, International Construction Consortium, Kohuwala, Sri Lanka.
[1]E-mail: kamal@icc-construct.com
[1]Telephone: +94-114-768600 (Ext: 8647); Fax: +94-114-768666

[2] Chief Executive Officer, International Construction Consortium, Kohuwala, Sri Lanka.
[1]E-mail: namal@icc-construct.com
[1]Telephone: +94-114-768600 (Ext: 8647); Fax: +94-114-768666

**Abstract:** Tourism can be considered as an industry that can create considerable economic activity. However, tourism related activities can also contribute to a significant level of carbon emission due to electricity used for creating comfortable conditions. In tropical climates, air conditioning is considered as an essential item in tourist resorts. The resources used for construction and maintenance of resort also could have significant impact on the environment. Therefore, tourism can offer many opportunities for a greater degree of sustainability where innovative thinking and strategic use of new materials and systems could make a significant change.

## 1. Introduction

With significant number of tourists arriving in Sri Lanka, it can make some impact on the carbon foot print per person. At present, Sri Lanka is considered to have a very low carbon foot print per person, estimated to be only about 700 kg per annum. This compares very favourbly with 2200 kg per annum in India or 5600 kg in China or about 24500 kg in USA [1]. Though, it is a significant achievement, maintaining such low values will be of a considerable challenge especially in view of rising electricity demand. For example, the carbon foot print of Sri Lanka is expected to increase up to 1100kg per person with commissioning of the proposed coal power plants. In this context, the promotion of the eco friendly resorts would be a promising idea. This paper addresses the creation of such a resort and various sustainable features included with respect to construction of buildings, energy efficiency, water efficiency and food production.

## 2. The Ulagalla Resort

Ulagalla resort is based on a large bungalow of over 100 years old of an extended family who used to hold significant positions in the government. It is located at Tirappane, 40 km north of Dambulla – Anuradhapura road. The property has an area of 58 acres. It borders two ancient reservoirs. The Ulaglla resort is a luxury boutique hotel that has utilized the bungalow as the reception/ lobby area, coffee lounge at lower level and a fine dining restaurant at the upper floor level. It also consists of 25 elegantly laid out luxurious Chalets spread over the 58 acres. Most of them overlook the reservoir while others overlook the paddy fields that also belong to the resort. This resort has many features purposely included to improve the utilization of sustainability concepts while ensuring a reduced carbon foot print.

Keeping with the “Go Green” concept of the owning company of the resort, 50% of the energy will is generated through solar power; the resort has the largest solar farm in the Sri Lanka at present with a capacity of 125 KVA. This has been coupled with the use of renewable materials for Chalets and various water and energy saving measures. In addition, a total of 1600 trees also have been planted to restore the natural habitat. Thus, it can be a good candidate for LEED certification [2].

## 3. The restoration of the adobe

One important principle of sustainable construction is repair and reuse of existing facilities. In this context, a decision was taken to completely restore the existing large adobe with minimum use of new materials. Figure 1 indicates the structure before restoration and Figure 2 indicates the fully restored building where careful attention has been placed to intricate details.


Figure 1: The bungalow prior to restoration

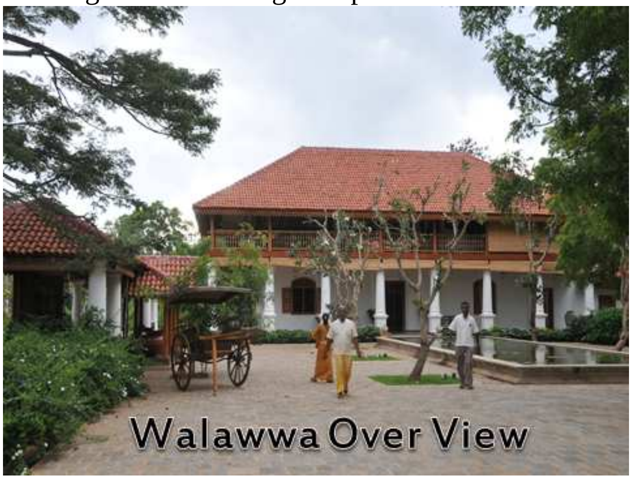



Figure 2: The bungalow after restoration

## 4. The Chalets

Each Chalet is of 80 $m^2$. This includes a separate dining area furnished with elegance and comfort combined with high end finishes and elegant architecture. Another special feature is the varandhas and the plunge pool. Each Chalet is designed so that they could be easily used by differently abled persons. Figures 3 and 4 indicate some details of Chalets.

The chalets have been provided with many useful features. The walls have been constructed with straw bonded solid panels called "Dura". These walls with lower conductivity are expected to reduce the air conditioning load due to lower heat transmittance. The chalets are fitted with LED bulbs that reduce the carbon footprint of the resort by 80%. The AC system fitted into each chalet utilizes an environmentally friendly gas and the heat recovery system provides the entire requirement of hot water. Durra straw panels reduce the transmission of sound. It is also a 100% recyclable. It can provide a strong load bearing wall [3].


Figure 3: A view from the front side


Figure 4: A bedroom arranged with elegant features

Chalets include a separate living area. The chalets are furnished with elegance and refinement while combining state-of-the-art amenities. Each chalet also offers a tranquil courtyard and private plunge pool.

### *4.1 The elevated structure*

One of the key features promoted with new construction under LEED certification is storm water quality control. In this context, a decision has been taken to minimize the earthwork by having the chalets as elevated houses constructed on columns. Although this almost eliminated the earthwork, a good solution was required with ground floor. The elevated ground floor was completed with straw bonded solid panels supported on a timber framework.

### *4.2 The roof*

One of the main sources of inward flow of heat is solar radiation absorbed by the roof. In order to minimize this, a decision was taken to use thatch laid on the top of straw bonded solid panels as shown in Figure 5. The structural robustness of straw bonded solid panels gave a strong roof that would be able to withstand strong winds.


Figure 5: The chalet with thatch as roof covering

## 5. Renewable energy & rainwater Harvesting

One of the most important contributing factors to Ulagalla Resort's eco-friendly policy is the solar farm. It is the largest solar farm in Sri Lanka at present and supplies over 50% of the hotel's energy requirement when in full operation. The rain falling on solar panels could be directed to rain water harvesting tanks. This water could be used for various purposes including the maintenance of vegetation. Another view is given in Figure 7.

Figure 6: Solar panels arranged to facilitate rain water harvesting

Figure 7: Another view of the solar panel arrangement

## 6. The wastewater treatment

Another feature well recognized for LEED certification is waste water minimization and treatment. The fixtures have been provided with water efficiency features. In order to re-use the waste-water, a very efficient treatment plant has been installed. It is a Sequencing Batch reactor (SBR) type biological treatment plant that contain the following operations

i. Equalizing
ii. Filling
iii. Aeration
iv. Settling
v. Decanting
vi. Sludge Removal

This process can be illustrated as shown in Figure 8. The methodology is as follows:

i. Raw sewerage, waste water & Laundry effluent will initially will gravitated to an underground Equalization tank
ii. Effluent is then feed to the SBR tank by means of a submersible pump for reduction of BOD.
iii. The reactor operating sequence will comprise Filling, Aeration, Settling, and Sludge Removal & Decanting with a pre determined time spans. Decanting pump will withdraw treated water from top and the sludge will be withdrawn with a submersible pump from bottom of the reactor after settling phase. Sludge will be collected to sludge storage tank combined with drying beds for dewatering.
iv. Treated water from decanter pump of SBR tank will be fed through a perforated feed piping system to Gravity Sand filter consist with 4 different layers of filtering media to entrap the remaining particles.
v. Treated water from the gravity sand filter will be then collected in to an Intermediate collection tank while feeding liquid chlorine to disinfect the treated effluent.
vi. Then the disinfected treated water will be fed by means of dry mounted centrifugal feed pump to /activated Carbon Filter(ACF) followed by a Cartridge filter Array(CFA). Filter array consists of 3 parallel lines of which each consisting with 20 micron, 10 micron, 5 micron cartridges respectively to ensure effective entrapping very fine remaining particles; finally will give clear treated effluent

To facilitate high quality treated water from secondary treatment step with SBR, a pre aeration step has been installed in the equalization tank prior to commence SBR operation.

**Diagrams of Treatment Process**

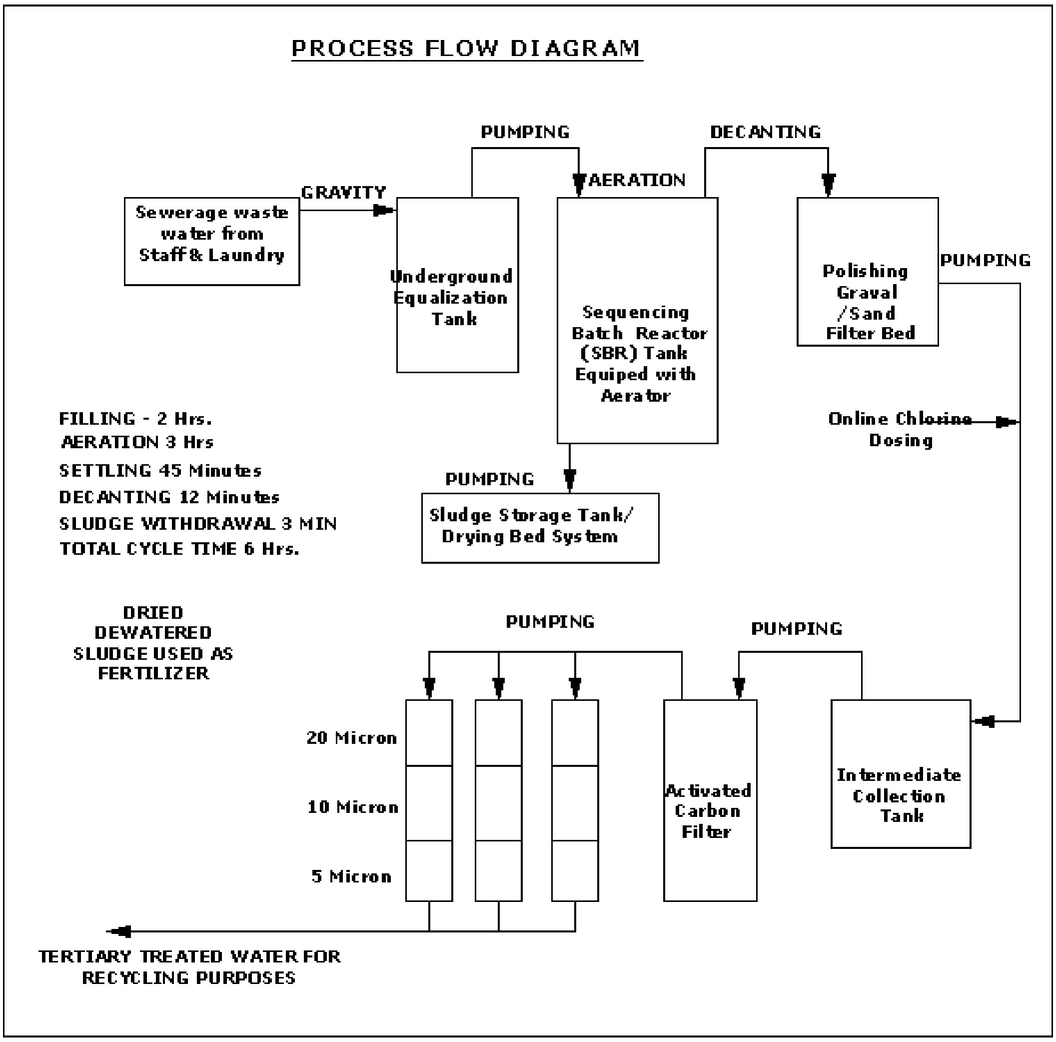



Figure 8: The flow-chart that indicates the process used at the waste water treatment plant

## 7. Solid waste management

Solid Waste at Ulagalla resort is segregated at the point of generation, all recyclable solid waste such as polythene, plastic, glass, paper and metal being collected separately and dispatched to recycling centers periodically.
Efforts are made to reduce the use of polyphone, plastic water bottles and cans, etc. The staff of the resort is well briefed to create a polythene and plastic free environment.
Biodegradable waste is being composted and used as organic fertilizer for paddy cultivation, and also for organic vegetable and fruit gardens.

## 8. Organic Paddy , Vegetable and Fruit Garden

While selecting areas to erect chalets, areas that were without trees and a minimum amount of shrub were selected and special attention was given to ensure that no trees were cuts down during the construction period. The resort includes over 20 acres of lush paddy fields, vegetable and fruit gardens. The paddy fields produce the entire resort's rice requirement while the vegetable and fruit gardens make substantial contributions as well. Figure 9 and 10 indicate the plots used for organic farming.


Figure 8: Plots with organic farming


Figure 9: Mixed cultivation with organic farming

## 9. Use of Electric Cars to avoid pollution

Another feature that is actively promoted at this resort is the use of electric cars to transport guest and also to provide other services.

Figure 10: Electric car service at the resort

## 10. Conclusion

Ulagalla Resort is a well planned project where many innovative features have been included that would give economic benefits either short or long term. This project that has been successfully completed and made operational in June of 2010 can be an important example that indicates many opportunities that would be available for those who value sustainable built environments and development. Its operation in coming years would be a very good learning exercise on sustainability.

### References:

[1] hewage, I., "Norochcholai Power Plant to be commissioned on January 1st ", Daily News, Friday, 3rd September, 2010

[2] Leadership in Energy and Environmental Design (LEED) Developed by the U.S. Green Building Council (USGBC)

[3] Munasinghe, K., Jayakody, S., Jayasinhe, M. T. R., "Straw bonded solid panels – The constructability, thermal performance and structural behavior" to be presented on ICSBE-2010, International Conference on Sustainable Built Environment, Kandy, Sri Lanka, December, 2010.

# SUSTAINABLE USE OF CONSTRUCTION AND DEMOLITION (C&D) WASTE AS A ROAD BASE MATERIAL

Narantuya Batmunkh[1], Komsun Siripun[2], Peerapong Jitsangiam[3] and Hamid Nikraz [4]

[1]PhD candidate, Department of Civil Engineering, Curtin University of Technology, P.O. Box U1987, Perth, WA, Australia 9845; PH (61) 8 9266-7609; FAX (61) 9266-2681 ;

E-mail: narantuya.batmunkh@postgrad.curtin.edu.au ;

[2]PhD candidate, Department of Civil Engineering, Curtin University of Technology, P.O. Box U1987, Perth, WA, Australia 9845; PH (61) 8 9266-7609; FAX (61) 9266-2681 ;

E-mail: komsun.siripun@postgrad.curtin.edu.au ;

[3]Lecturer, Department of Civil Engineering, Curtin University of Technology, P.O. Box U1987, Perth, WA, Australia 9845; PH (61) 8 9266-4527; FAX (61) 9266-2681;

E-mail: p.jitsangiam@curtin.edu.au ;

[4]Professor, Department of Civil Engineering, Curtin University of Technology, P.O. Box U1987, Perth, WA, Australia 9845; PH (61) 8 9266-7609; FAX (61) 9266-2681;

E-mail: h.nikraz@curtin.edu.au;

Corresponding author: Mr. Komsun Siripun: komsun.siripun@postgrad.curtin.edu.au

**ABSTRACT**

Crushed concrete waste is a by-product from building demolition and constitutes a principal component of municipal solid waste consisting of concrete, sand, brick, rock, metals and timber. Over 50% of this waste is commonly sent to land-filled sites, resulting in the impact on the limited capacity of land-filled sites. Nowadays, the sources of virgin natural aggregates are depleted by increasing in demand of using a virgin material in building and infrastructure construction and maintenance facilities. This depletion leads to the utilisation of crushed concrete waste to replace natural aggregates in road and highway construction. Of key significance of this study is to present alternative materials for road and highway construction on the production of the proper guideline for road base by using crushed concrete waste. Sophisticated tests were conducted to investigate the mechanical responses of compacted crushed concrete subjected to applied loads simulated from traffic loads. Unconfined compressive strength, shear strength parameters, resilient modulus and permanent deformation of such material were determined. Our findings showed that crushed concrete waste is able to utilise as a road base material. The results of this study will enhance increased use of crushed concrete waste in road and highway construction and will, therefore, alternatively reduce consumption and costs in manufacturing virgin aggregates.

## Introduction

In recent times there has been an increasing shift towards finding more environmentally sustainable practices in an effort to tackle modern challenges related to climate change, population growth and pollution. This study presents the latest research in the performance of recycled construction and demolition material as a road building material in Western Australia and recommends new technology for this growth industry. Local Governments and industry are encouraged to be sustainable development leaders with more sustainable road building practices and materials. The objective of this research is to investigate crushed concrete waste material for a pavement material with sufficient background information to enable interested parties constructing roads to have a good understanding of the background as to how utilisation limits were derived. The study investigates the properties and performance characteristics to enable practitioners to confidently make use of recycled construction and demolition materials in the construction of road pavements.

The existing road construction material specifications for recycled products were generally derived from specifications for virgin rock aggregates, and modified slightly to allow for the different properties of the recycled products (Austroads 2004). The materials used as source material for the manufacture of recycled roadbase stem from the construction and demolition industry. The term commonly used to describe this material is construction and demolition (C&D) waste. A literature review was undertaken of Australian and international test methods for analysing the performance of pavement materials. There seemed to be gaps in the previous research undertaken into the performance and characterisation of recycled materials. A significant amount of laboratory characterisation of recycled materials and virgin quarried aggregates was undertaken by Curtin University. This testing provided a greater understanding about the characteristics of recycled roadbase sourced from construction and demolition materials and how these materials relate to newly quarried materials. These test results were compared to those, and they indicated that the materials produced in Western Australia are comparable in performance to those in other parts of Australia and the world.

It seems that where recycled products are used, they are generally considered a second class material suitable for subbase generally or base material for low volume roads. There are some interesting new test methods which could be applied not only to recycled materials but also virgin aggregates used for road construction. From field trials using recycled concrete products an increase in stiffness was observed after one to two years of service. There is a potential for recycled concrete products to rehydrate and re-cement with time. This process of rehydration has not been fully investigated and may result in a material that is too stiff and brittle. If rehydration continues and creates a bound material the pavement then may be subject to fatigue failure of the basecourse. Further research is needed to study the effects of blending other materials such as clay bricks and tiles, reclaimed asphalt pavement (RAP) and small amounts of clay material with the crushed concrete to minimise the effects of rehydration.

This study provides pavement material characterisations for use by Local Governments and Industry for the supply of recycled concrete based roadbase, subbase and fill materials. The limit use has been developed based on the results of laboratory testing and field performance of pavements constructed using recycled materials. Laboratory characterisations of three sources of recycled materials (All Earth, C&D Recycling and Capital Demolition) and two sources of virgin quarried aggregates (Boral and Cemex) were undertaken by Curtin University. These results were used to give a greater understanding about the characteristics of recycled materials sourced from construction and demolition materials and how these materials compared to local virgin materials. These tests results were also compared to those in selected engineering papers and indicate that the materials produced in Western Australia are comparable in performance to those in other parts of Australia and the world. The laboratory test results show that the constituents of the source material are significant to the long-term performance of the pavement materials. Where pavement material contain only structural concrete crushed from the demolition of reinforced concrete structures, rehydration of the cement within the material may produce an excessively stiff material likely to fatigue with time. The blending of small amounts of crushed clay bricks and tiles appears to help minimise the risks of excessive rehydration of recycled structural concrete. The possible effect of cement reacting with bitumen, where cement fines may be pumped by traffic action into the seal coat has been noted and will require further research.

## Laboratory Results and Discussion

### *Maximum dry density and optimum moisture content*

The maximum dry density (MDD) and optimum moisture content (OMC) tests were undertaken and compaction curves are shown for C&D Recycling in Figure 1, All Earth in Figure 2 and Capital Demolition in Figure 3 (Main Roads Western Australia 2007).

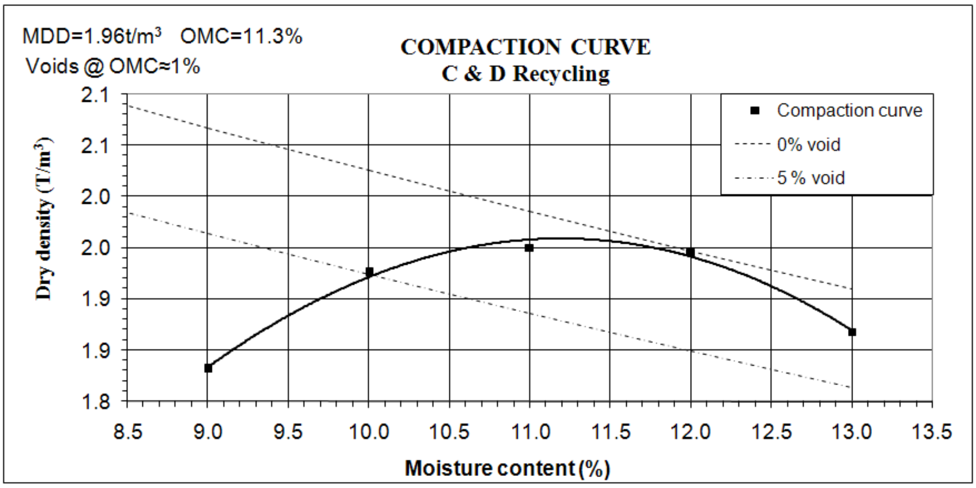



Figure 1: *C & D Recycling compaction curve.*

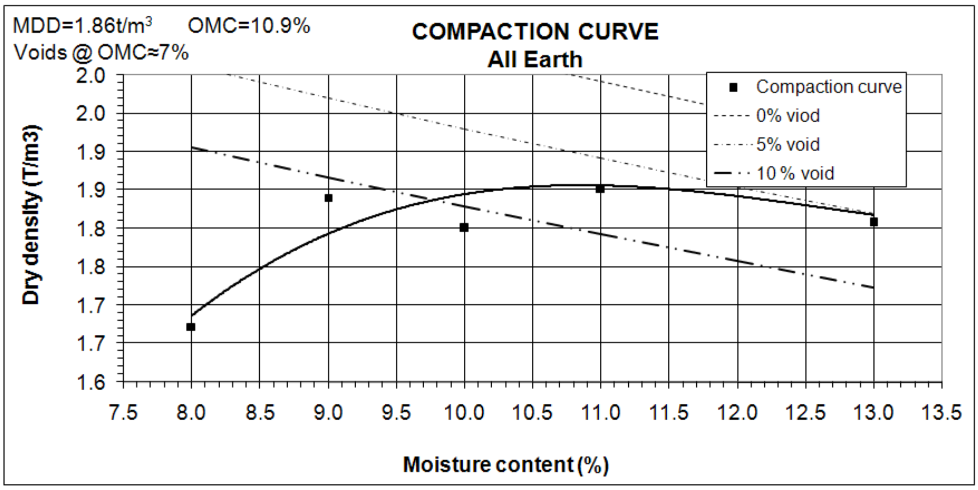



Figure 2: *All Earth compaction curve.*

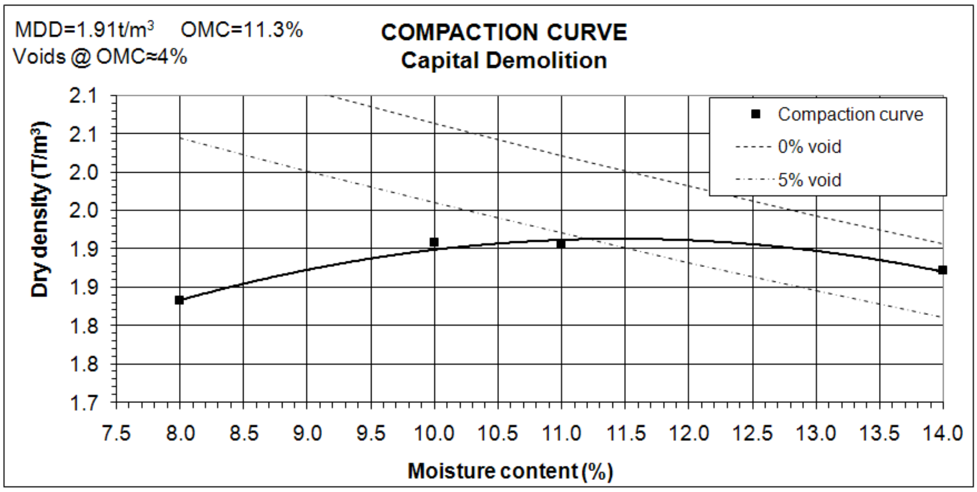



Figure 3: *Capital Demolition compaction curve.*

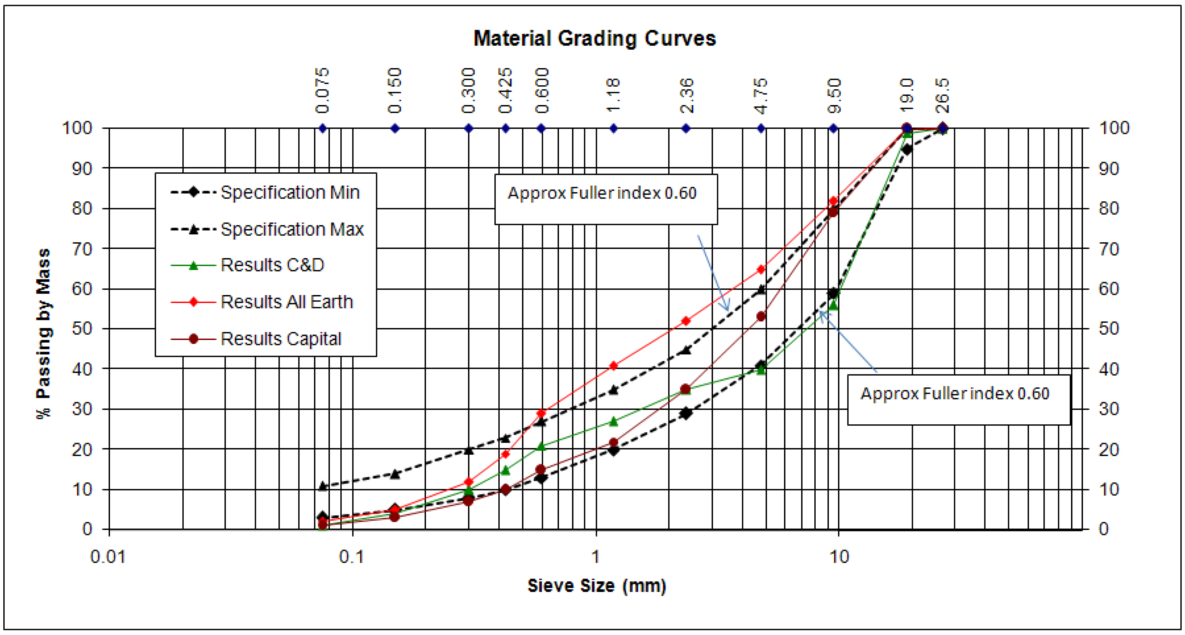



Figure 4: *Particle size distribution for recycled materials.*

***Particle size distribution***

The particle size distribution is shown in Figure 4 (Main Roads Western Australia 2007). The grading shows that all materials are low in fines, and that for these samples at least, the grading of the All earth material is well out of specification on the fine side, and the grading for C&D Recycling is slightly out of specification on the coarse side. The coefficient of curvature and Coefficient of Uniformity for each material are shown in Table 1. Whilst the grading is out of specification for All Earth, the coefficient of uniformity and coefficient of curvature for all materials are within a tolerable range.

Table 1: *Grading properties of recycled materials.*

| **Property** | **Results** | | |
|---|---|---|---|
| | **C&D** | **All Earth** | **Capital** |
| $D_{60}$ | 10 | 3.5 | 5.6 |
| $D_{30}$ | 1.5 | 0.65 | 1.8 |
| $D_{10}$ | 0.3 | 0.23 | 0.425 |
| Coefficient of Curvature | 33.3 | 15.2 | 13.2 |
| Coefficient of Uniformity | 0.8 | 0.5 | 1.4 |

***Unconfined compressive strength (UCS)***

Tests were undertaken on samples at 1 day and after 28 days curing as shown in Table 2. These results show that there is some degree of cementing action for recycled materials, and that there is considerable variability between suppliers. A limit is applied to UCS to prevent excessive brittleness of a granular pavement material. This testing confirms that there is a gradual increase of strength with time, and should this strength gain be related to rehydration of cement bonds as suspected, the strength gain will be very dependent on the curing regime. As pavements are typically dried back before sealing, ideal curing conditions will not be established as is the case in the test conditions. In practice, the increase in strength with time should not be as marked as that in the laboratory.

Capital Demolition sources its concrete from structural concrete, and the samples collected were only concrete. All Earth and C&D Recycling receive supplies of material from a wide range of sources, and much of the concrete is non-structural house pads, paving and kerbing from roadworks. There is also a blend of other products mainly brick, tile, sand, aggregate and asphalt.

Whilst there is a significant difference between C&D Recycling and All Earth, this should not be viewed as a difference in process, rather an indication of the potential variation in materials. MRWA specification 501 requires a 7 day UCS value of 0.6 MPa to 1.0 MPa. Based on the results in Table 2, it is likely that 7 day value for UCS will be potentially in the range of 0.37 MPa to 1.2 MPa (Main Roads Western Australia 2006). Further testing is required, and extraction of cores from existing road pavements will be undertaken by Curtin as part of ongoing research.

It is recommended that further research is undertaken to investigate the potential for recycled pavement materials to rehydrate and under what conditions rehydration occurs. There is little research into the rate of rehydration of crushed concrete and the assumption that 90% of the potential rehydration is achieved in 28 days is not supported from the testing of field trials. Further UCS testing of specimens cured in a condition that represents the as-constructed pavement for 60, 90, 180 and 360 days should be undertaken and if necessary the specification reviewed.

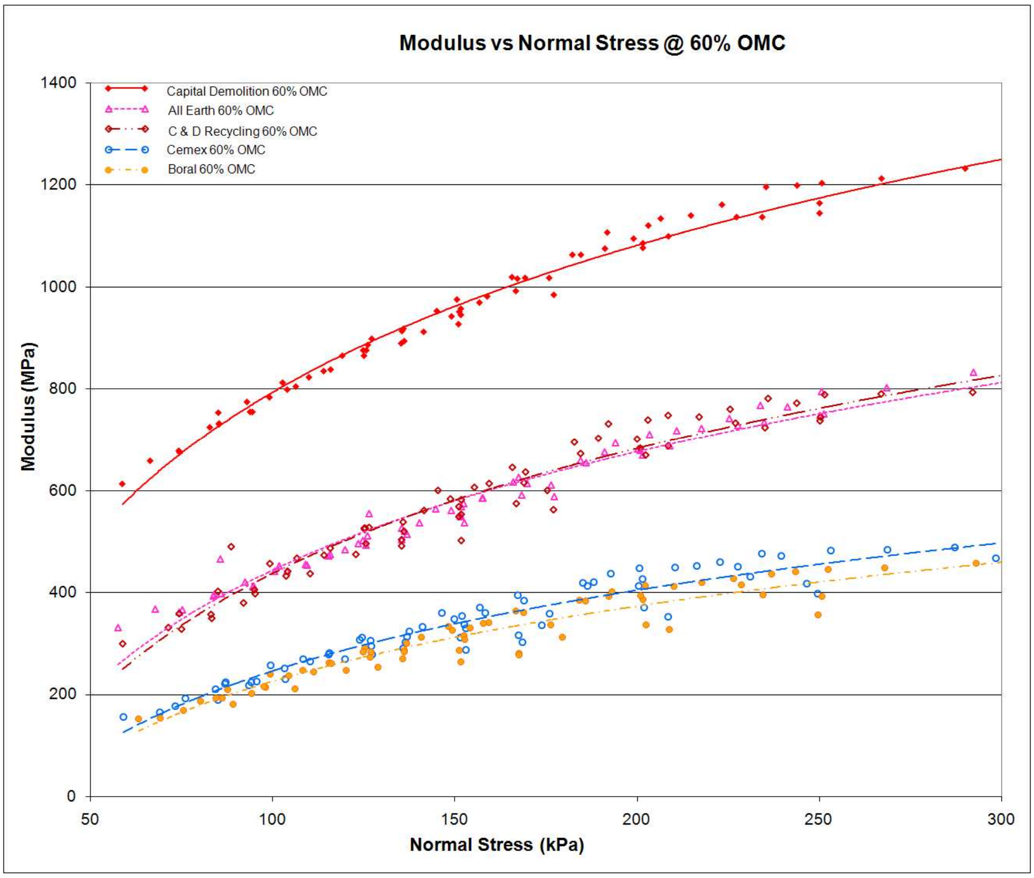
Table 2: *Results of UCS testing.*

| **Supplier** | **UCS 1 day cure (kPa)** | **UCS 28 day cure (kPa)** |
|---|---|---|
| Capital Demolition | 668 | 1625 |
| C&D Recycling | 220 | 474 |
| All Earth | 541 | 1323 |

***Repeat load triaxial testing (RLT)***

RLT tests were undertaken on triplicate samples of recycled demolition roadbase sourced from the three suppliers from All Earth, Capital Demolition and C&D Recycling, and two samples of new quarried roadbase from Cemex and Boral (Voung and Brimble 2000). The relationships between modulus and normal stress for the various materials tested at 60% and 80% OMC are shown in Figure 5. In this figure, AE represents All Earth, CAP represents Capital Demolition and CD represents C&D Recycling, all of which are the recycled materials. CEM represents Cemex and BOR represents Boral, both of which are high quality quarried and crushed granite roadbase materials (CGRB). The highest modulus values were obtained by Capital Demolition, with All Earth and C&D Recycling showing similar values. Both Boral and Cemex roadbase also showed very similar values. All of the recycled materials showed significantly higher modulus values than that obtained from the CGRB materials.

The Capital Demolition roadbase, which is predominantly structural grade concrete with no brick and tile clearly showed the highest modulus values. The All Earth and C&D Recycling roadbase which contained predominantly non structural concrete with some brick and tile, estimated to be between 10% and 15% by weight, showed good modulus values significantly higher than the values shown by the new CRB of Boral and Cemex.



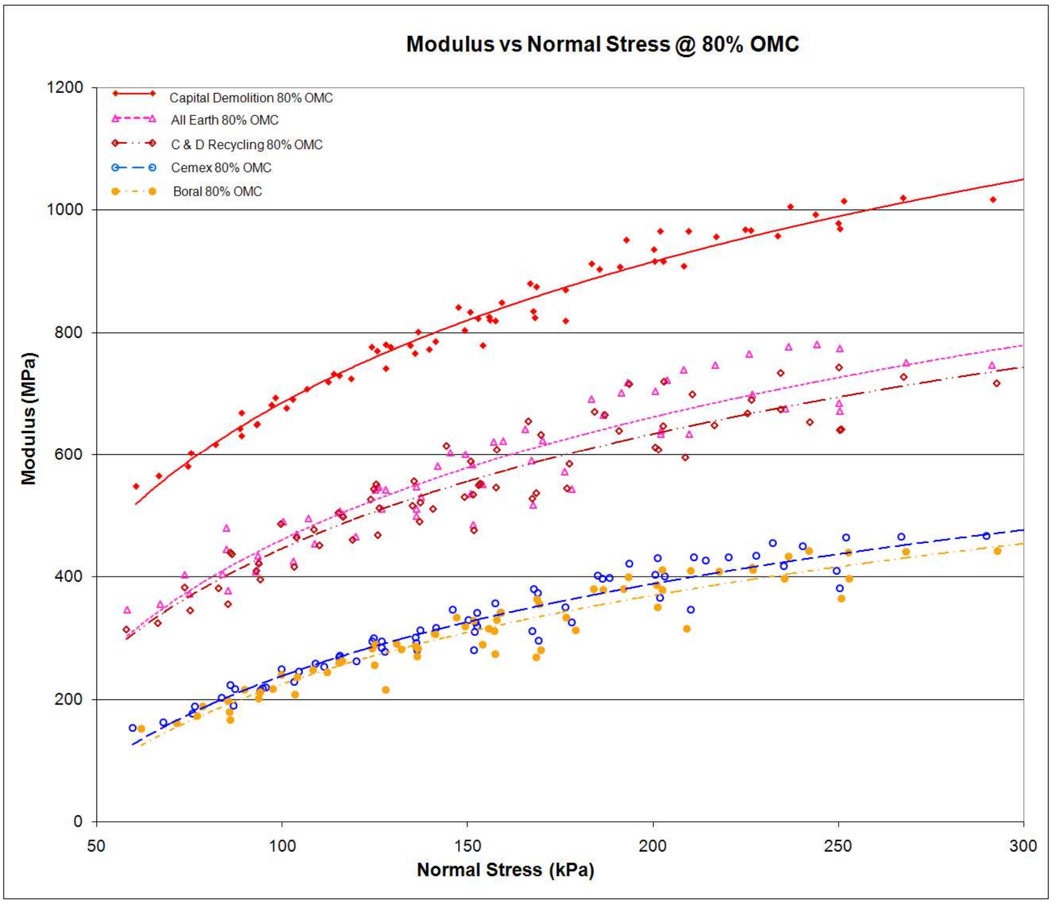




Figure 5: *Modulus vs. normal stress for recycled and quarried roadbase materials.*

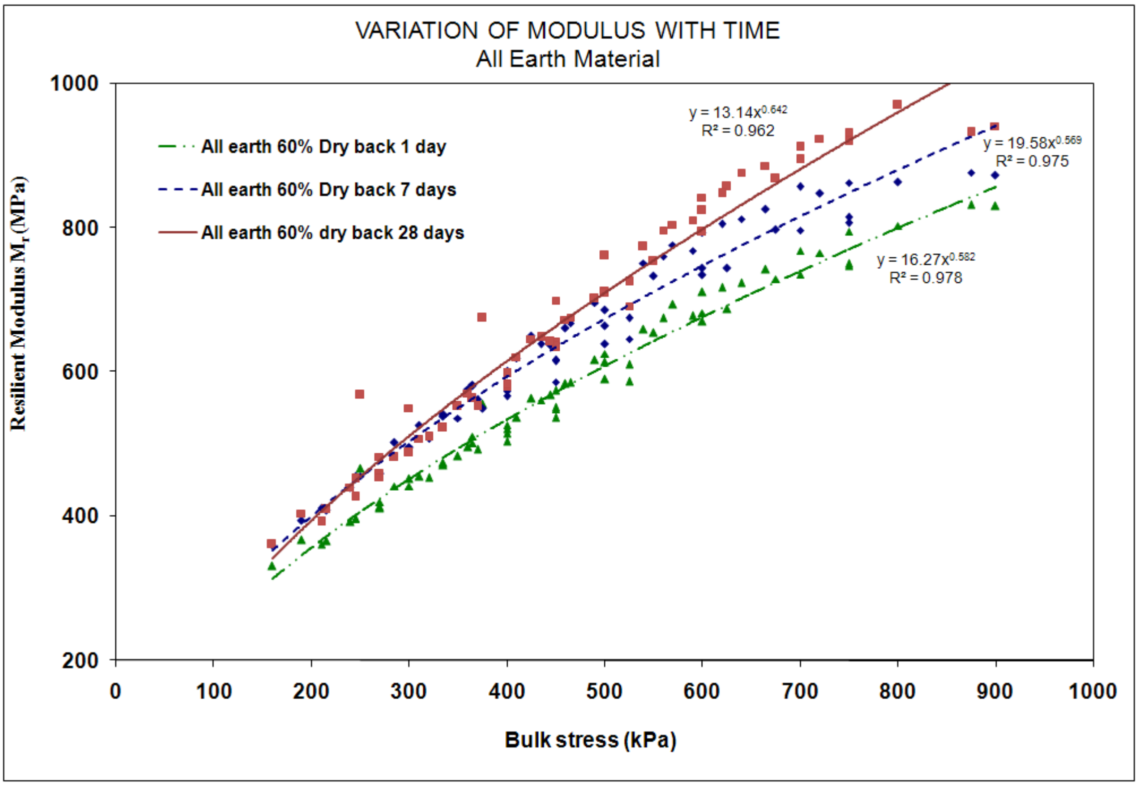



Figure 6: *All Earth resilient modulus at 1, 7 and 28 days.*

The above tests were undertaken without curing, and in order to assess the potential changes of modulus with time, one product was selected to undertake resilient modulus testing at 60% OMC over a period of 1 day, 7 day and 28 day curing. All Earth material was chosen, as it represents the middle range of the uncured performance testing. The results of this testing are shown in Figure 6, and indicate a clear increase in stiffness with time. In practical terms, a higher modulus value will result in lower curvature values and hence a longer fatigue life of an asphalt surface.

## Conclusions

Test results have confirmed that the optimum moisture content for recycled crushed demolition roadbase (CDRB) materials is higher than for crushed granite roadbase (CGRB). It has also been determined by repeat load triaxial testing (RLTT) that the resilient modulus of the CDRB is superior to that of CGRB by a factor of two or more. The RLTT also showed that the source of material strongly influences the modulus of the recycled material.

In order to differentiate between products, new definitions should be included to differentiate materials into sources. It is proposed that the following definitions be used, and will be used in the discussion from here on:

| | |
|---|---|
| CGRB | – conventional new roadbase manufactured from virgin granite rock |
| CCRB(S) | – crushed concrete roadbase containing only structural grade concrete |
| CCRB(N) | – crushed concrete roadbase containing only non-structural grade concrete |
| CDRB(x) | – crushed demolition roadbase containing a mix of predominantly concrete with clay brick, tile and sand containing approximately x% concrete. |

The reason for this is that CCRB(S) may be too stiff in its raw state for a basecourse and could lead to premature shrinkage and or fatigue type cracking. CCRB(N) may be suitable for roadbase, as is CDRB(x) where x is yet to be determined. There may be a value of x suitable for base application, and a value for subbase application. The Capital Demolition material would fall into CCRB(S) class, and the RLTT shows a very stiff material. This may be suitable under thick layers of asphalt, or as a subbase for heavy trafficked pavements. However, it may not be suitable for a heavy trafficked pavement with thin asphalt due to possible fatigue cracking. The Capital Demolition material showed a modulus value of approximately 3 times that of the CGRB.

CCRB(S) can be blended with clay brick or tile to form CDRB(90), that is 90% concrete with 10% foreign material, which would make a good general-purpose base for heavy trafficked pavements.
The CDRB(x) which comprises the All Earth material and C&D Recycling material, where x is thought to be around 90%, showed very similar modulus values of about twice that of CRB. Particle size distribution (PSD) of the materials to not completely fall within the current MRWA specification, but this does not seem to affect the performance of the material in the RLTT. Unconfined compressive strengths were undertaken on all recycled materials, and here significant variations were observed. Tests were undertaken at 1 days and 28 days curing. The results are interesting in that in this test the high modulus Capital Demolition material and the lower modulus All Earth material showed similar results, where the C&D Recycling material which has a similar modulus to All Earth showed a much lower value. The All Earth material, considered to be the best compromise between modulus and UCS was subjected to more study using the RLTT and static triaxial testing at 1 day, 7 days and 28 days curing to determine if there was significant rehydration occurring in the material.

This investigation has considered Australian and international trends in the use, performance and specification for roadbase material sourced from construction and demolition materials. However, no specifications currently give limits for modulus, compressive strength. Therefore as a starting point, limits for these values have been estimated based on the results obtained from Curtin University testing associated with this investigation. These values will need revision as more data becomes available, but by including them now in the specification and testing for these properties, a data base will be established which will allow a more critical review in the future.

## References

Austroads (2004). Pavement Design-A Guide to the Structural Design of Road Pavements, Austroad Inc.2004.

Main Roads Western Australia. (2006). "A guide to the selection and use of naturally occurring materials as base and subbase in roads in Western Australia " Retrieved September, 2008, from http://standards.mainroads.wa.gov.au/NR/mrwa/frames/standards/standards.asp?G={E582C897-FF5E-4C02-8B46-51E88C1E5DD8}.

Main Roads Western Australia. (2007). "Dry density/moisture content relationship: modified compaction fine and medium grained soils." Retrieved September, 2008, from http://standards.mainroads.wa.gov.au/NR/mrwa/frames/standards/standards.asp?G={E582C897-FF5E-4C02-8B46-51E88C1E5DD8}.

Main Roads Western Australia. (2007). "Test Method (Aggregate)." Retrieved September, 2008, from http://standards.mainroads.wa.gov.au/NR/mrwa/frames/standards/standards.asp?G={E582C897-FF5E-4C02-8B46-51E88C1E5DD8}.

Voung, B. T. and R. Brimble (2000). Austroads Repeated Load Triaxial Test Method-Determination of Permanent Deformation and Resilient Modulus Characteristics of Unbound Granular Materials Under Drained Conditions. APRG DOCUMENT APRG 00/33(MA), Austroads.

# USE OF THE PARALLEL GRADATION TECHNIQUE TO ASSESS THE SHEAR STRENGTH OF RECLAIMED ASPHALT PAVEMENT IN DIRECT SHEAR

P. S. K. Ooi[1], P. Selvarajah[2]

[1]Associate Professor, Department of Civil and Environmental Engineering, University of Hawaii at Manoa, Holmes Hall 383, 2540 Dole Street, Honolulu, HI 96822, USA.
[1]E-mail: ooi@eng.hawaii.edu

[2]Graduate Student, Partheeban Selvarajah, Department of Civil and Environmental Engineering, University of Hawaii at Manoa, Holmes Hall 383, 2540 Dole Street, Honolulu, HI 96822, USA.
[2]E-mail: ps2@hawaii.edu

**Abstract:** The demand for pavement construction aggregate is growing significantly. With a rise in aggregate consumption, the use of recycled materials has become an attractive solution in pavement construction from both economical and environmental perspectives. Use of recycled materials in the base course demands understanding of their properties including its shear strength and recognition of potential problems that may arise. The parallel gradation technique involves evaluating the parameters of a coarse material using a gradation that is finer but parallel to the prototype. It has been successfully used for estimating the shear strength parameters under certain conditions. The main objectives of this study are to: assess the applicability of the parallel gradation technique to estimate the shear strength of Reclaimed Asphalt Pavement (RAP) and determine the shear strength parameters of RAP in direct shear at two relative compactions.

**Keywords:** shear strength, parallel gradation, prototype, direct shear, relative compactions

## 1 Introduction

The demand for construction aggregate is growing significantly. In the United States, 2 billion tons of aggregate are produced each year and it is expected to increase to more than 2.5 billion tons annually by the year 2020 [1]. This growing need has raised concerns in the highway-related industries and there has been a push for finding alternative materials to replace quarried virgin aggregate.

The U.S. highway system includes nearly 4 million miles of public roads in the country [2]. Many of these roads are maintained by some form of reclamation be it partial or full depth. Removal of these pavements produces waste material. Rehabilitation for these pavements requires large quantities of aggregate. So if the waste material can be recycled for reuse as aggregate, it would reduce the need for virgin aggregate thereby preserving the environment in two ways: (1) reduce the need to quarry as much and (2) eliminate the need to dispose of the waste materials in landfills. Recycling of pavement materials date back as early as 1915 but its interest grew significantly in response to inflated construction costs [3]. However, there is a need for the recycled materials to perform well over the pavement life cycle. Therefore, there is a need to understand their engineering properties.

Some encouraging studies have shown that pavements constructed using recycled materials are as durable as those constructed with 100 percent virgin aggregate [3]. In this study, the shear strength of Reclaimed Asphalt Pavement or RAP having a gradation that is close to a base course gradation in Hawaii is estimated by conducting direct shear tests in conventional size equipment.

## 2 Literature review

### *2.1 Applications of recycled materials*

The engineering properties of RAP are of particular interest to pavement and geotechnical engineers. They include gradation, shear strength, stiffness, etc. One way of assessing the quality of an aggregate for use as a road base or sub-base is by looking at its shear strength [4 & 5]. Two of the more common laboratory tests to measure a granular material's shear strength are the triaxial and direct shear tests

### *2.2 Reclaimed Asphalt Pavement (RAP)*

RAP is obtained by milling or from full-depth removal of asphalt concrete. In the latter, a milling machine removes the asphalt concrete in a single pass. In full-depth removal, a rhino horn on a bulldozer and/or pneumatic pavement breakers are used to rip and break the pavement.

The properties of RAP are mainly dependent on the properties of the constituent materials used in the old pavement. The quality of RAP varies since it can be obtained from any number of pavement sources. RAP can be blended with virgin aggregate to produce a higher quality unbound material.

#### *2.2.1 Shear strength of RAP*

Several researchers have measured the shear strength of RAP, a summary of which is provided in Table 1. In the literature reviewed, the shear strength of RAP was measured either via consolidated drained (CD) or unconsolidated undrained (UU) triaxial tests. CD test friction angles ($\phi$)ranged from $37^0$ to $57.5^0$ with cohesion (c) ranging from 0 to 55.2 kPa.

### *2.3 Parallel gradation technique*

The parallel gradation technique has been previously used for estimating the shear strength parameters of very coarse granular materials by testing the same material having a gradation that is parallel to the original. The main advantage is that without the oversized particles, conventional-size testing equipment can be used [6, 7, 8, 9, and 10] while at the same time maintaining the soil gradation relative to the original material. Several researchers have found that a finer parallel gradation gave similar friction angles to the same geological material with a coarser gradation provided the mineralogy, hardness of grains, particle shape, and particle roughness do not vary with particle size [10, 11, 12, 13, 14, and 15]. It can be an attractive alternative especially if the particle shape, particle surface roughness, hardness of grains and mineralogy are similar for all particle sizes [10].

## 3 Material characterization

### *3.1 Index properties and test gradation*

The index properties of RAP are presented in Table 2. Of note are the very low absorption, very low optimum moisture content and very high maximum dry density.

To test the samples in a small shear box, the material was first dried in an oven at $60^0$C. This temperature was selected to avoid softening the asphalt. If the RAP did soften, it can act as an adhesive and glue the particles together. The RAP was then sieved to various particle sizes for rebatching. Then, a gradation that is close to the State of Hawaii Department of Transportation's (HDOT) specification for 0.75-inch maximum nominal base course was selected as the target test gradation (Figure 1). This gradation contains little to no fines and is slightly more uniform than HDOT's base course. As such, it will provide conservative shear strength parameters.

Also shown in Figure 1 are two gradations that are parallel to but finer than the target test gradation. These two finer gradations were batched and tested in direct shear. The coarser of the two gradations (scalped on the 0.265-inch sieve) was tested in a 100-mm-square shear box while the finer (scalped on the 4.75 mm sieve) was tested in a 61.4-mm-diameter shear box.

Table 1: *Summary of RAP characteristics from the literature*

| State | Aggregate type | Confining Stress/ kPa | Failure deviator stress/ kPa | USCS and AASHTO Symbol | $G_s$ | $w_{opt}$ (%) | $\gamma_d$ (kN/m$^3$) | $\gamma_{d\ max}$ (kN/m$^3$) | Relative Compaction (%) | Test Type | Peak | | Reference Note |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | | $\phi$ ($^0$) | c (kPa) | |
| Texas | NK | 34.5, 68.9, 137.9, 206.8, 344.7 | NK | GW A-1-a | 2.33 | 3 | NK | 18.4 | NK | CD[1] Triaxial | 37.0 | 55.2 | Viyanant, (2006), Rathje et al. (2006[a]), and Rathje et al. (2006[b]) |
| California | NK | 0, 35, 70,105 | NK | GW A-1-a | | 5.5 | NK | 22.9 | 95 | CD[1] Triaxial | 51.5 | 0 | Bejarano, (2001) |
| | | | | | | | | | 100 | | 57.5 | 0 | |
| Texas | NK | 82.7, 117.2, 158.6, 255.1, 310.3 | 515, 642.6, 739.8, 1149.4, 1254.2 | GW A-1-a | 2.28 | 6.7 | [a]18.3 | NK | NK | CD[1] Triaxial | 39.0 | 55.2 | Carley (2001) |
| | | | | | | 4.9 | [a] 18.6 | | | | | | |
| | | | | | | 3.9 | [a] 18.9 | | | | | | |
| | | | | | | 3.9 | [a] 18.8 | | | | | | |
| | | | | | | 4.1 | [a] 18.7 | | | | | | |
| Florida-Hammermill | Lime rock | 35,70, 105 | NK | SW A-1-a | NK | 5.5 | [b] 18.5 | NK | NK | UU[2] Triaxial | 44.0 | 15.9 | Cosentino and Kalajian, (2001) |
| Florida-Tubgrinder | | | NK | SP A-1-a | NK | 6.6 | [b] 19.0 | NK | NK | | 35.0 | 46.2 | |
| Illinois | NK | 34.5, 68.9, 103.4, 137.8 | NK | GW A-1-a | NK | 7.2 | NK | 19.7 | NK | UU[2] Triaxial | 45.0 | 130.9 | Garg and Thompson (1996) |
| Mississippi – OGDL[3] | Limestone | 0, 34.5, 103 | 185, 283, 450 | GP A-1-a | NK | 6.3 | NK | 19.5 | 95 | UU[2] Triaxial | 38.5 | 28.8 | Saeed (2008) |
| Louisiana – OGDL[3] | | | 163, 261, 425 | GP A-1-a | NK | 5.4 | NK | 19.4 | 95 | | 39.0 | 19.2 | |
| Denver – finer DGBL[4] | Granite | | 77.2, 222, 441 | GW A-1-a | NK | 10.3 | NK | 19.8 | 95 | | 41.0 | 14.3 | |

Note: (1) CD = consolidated drained; (2) UU= unconsolidated undrained; (3) OGDL= Open Graded Drainage Layer; (4) DGBL =Dense Graded Base Layer.
NK= not known; [a] Condition not given; [b] Modified proctor test

Table 2 *Summary of index properties*

| Properties | RAP[1] | |
|---|---|---|
| LA Abrasion (%)[1]: Grading C | 33 | |
| LA Abrasion (%)[1]: Grading D | 27 | |
| | Fine[2] | Coarse[3] |
| Bulk Specific Gravity | 2.17 | 2.54 |
| Bulk Specific Gravity (SSD)[4] | 2.31 | 2.61 |
| Apparent Specific Gravity | 2.52 | 2.72 |
| Absorption (%) | 6.3 | 2.5 |
| Void Content (%)[5] | 50.0 | |
| Optimum Water Content (%)[6] | 5.05 | |
| Maximum Dry Density (kgm$^{-3}$)[6] | 1808 | |
| Asphalt Content (%)[7] | 5.8 | |

Notes
1) Per AASHTO T96.
2) Per AASHTO T84.
3) Per AASHTO T85.
4) SSD = Specific surface dry.
5) Per AASHTO TP56.
6) Per ASTM D1557 Method C (modified Proctor).
7) Per AASHTO T308.

Based on the gradations shown in Figure 1, the RAP can be classified as GW using the Unified Soil Classification System (USCS), and as A-1-a using the American Association of State Highway and Transportation Officials (AASHTO) classification system. Based on the parallel gradations with a maximum particle size of 4.75 mm, the RAP is classified as SP using the USCS, and as A-1-a using AASHTO.

### *3.2 Parallel gradation test results*

A comparison of the Mohr-Coulomb failure envelopes for the small and large shear boxes are shown in Figure 2. The RAP samples were compacted to the same dry densities and moisture contents in both the large and small shear boxes. It can be seen that the Mohr Coulomb envelopes for the two shear box sizes are very similar.

One limitation of the parallel gradation technique is that if the original coarse material contains a significant amount of fines, a parallel gradation would contain even more fines and if excessive, the overall behavior of the material will be governed by the fines [8]. Since the sample contained little fines, the parallel gradation technique appears applicable for the RAP gradations tested as seen by the results in Figure 2.

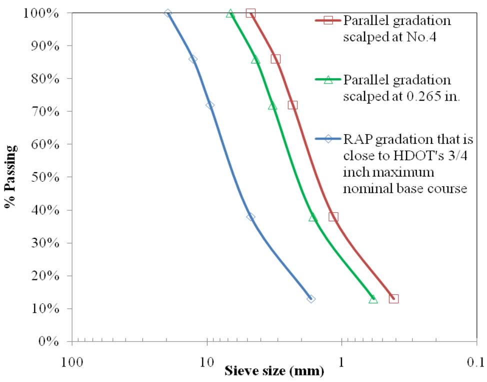



Figure 1: *Target and parallel gradations of RAP*

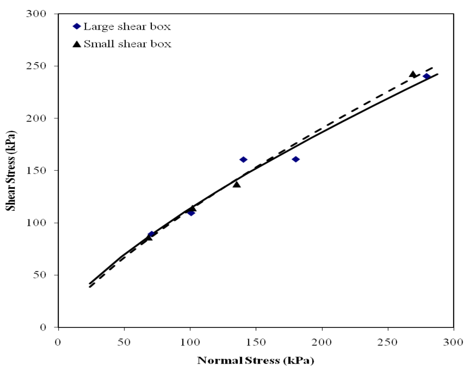



Figure 2: *Mohr-Coulomb failure envelop based on direct shear testing for large and small shear boxes on RAP*

### *3.3 Direct Shear Tests Results*

For the direct shear testing program, “loose” and “dense” RAP samples were tested in the shear box at four different normal stresses. Details of the target sample densities and water contents are summarized in Table 3. All samples were 28.8 mm high.

Table 3: *Details of samples tested in direct shear*

| Physical State | Dry Density (kg m$^{-3}$) | Water Content (%) | Relative Compaction (%) |
|---|---|---|---|
| "Loose" | 1722 | 4.76 | 95 |
| "Dense" | 1809 | 4.76 | 100 |

A summary of the direct shear results are provided in Figures 3 and 4 for "dense" and "loose" RAP, respectively. The test results are plotted in terms of shear stress ($\tau$ = horizontal shear force/initial area) normalized with the normal stress ($\sigma$ = vertical load/initial area) versus shear strain ($\gamma$ = horizontal displacement normalized by the original sample height) even though it is known that the shear strains are not uniform along the failure plane. Also, the volumetric strain ($\varepsilon_v$), equal to the change in height divided by the original sample height, is plotted versus $\gamma$. $\varepsilon_v$ is positive for compression and negative for dilation.

Figures 3a and 4a provide an instant indication of the linearity (or nonlinearity) of the Mohr-Coulomb envelops at the peaks and at large strains. In figures 3b and 4b, the volumetric strain is plotted versus shear strain. Failure was defined when the shear stress peaked or at 20% shear strain if there was no peak in shear stress. Table 4 summarizes the shear strains at failure and at critical state.

From figures 3 and 4, the following observations are offered:

- The highest $\tau/\sigma$ occurs at the lowest normal stress and this ratio decreases with increasing $\sigma$.This is an indication that increasing $\sigma$ reduces the brittleness. Also, increasing $\sigma$ increases the strain to failure, and decreases the tendency to dilate.
- The strains at failure for "loose" RAP (16.5% to 20.0%) are higher than those for "dense" RAP (7.7% to 20.0%). For "loose" RAP, a $\tau/\sigma$ peak at shear strains less than 20% was obtained only at the lowest normal stress. On the other hand, "dense" specimens have peak $\tau/\sigma$ at shear strains less than 20% except for the highest normal stress. Therefore, brittleness is affected by both normal stress and relative density.
- It was interpreted that critical state occurred at a shear strain of 34.4%. Critical state is said to be reached when shearing occurs at constant volume. In this study, the shear strain at which critical state occurred was taken to be the strain at which the coefficient of variation of $\tau/\sigma$ is a minimum. Using this definition, values of $\tau/\sigma$ at critical state varied from 0.876 to 0.975.
- All the RAP samples were dilative at failure except for "loose" RAP at the two highest normal stresses.

### *3.4 Mohr-Coulomb envelop*

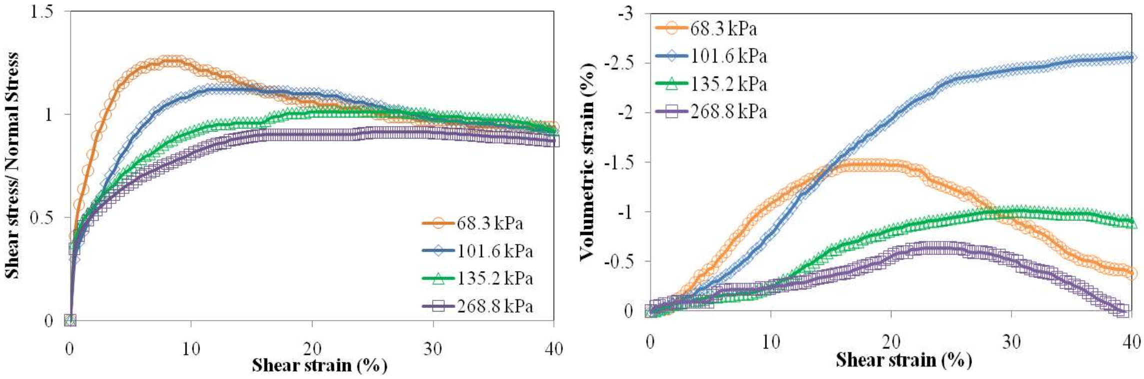
Figure 5 presents a plot of the Mohr-Coulomb failure envelop for "dense" and "loose" RAP. The peak failure envelope was nonlinear with secant friction angles varying from $41.2^0$ to $48.3^0$ for "loose" RAP and $42.1^0$ to $51.6^0$ for "dense" RAP at the highest and lowest normal stresses, respectively. The friction angles for "dense" RAP are only slightly higher than those in the "loose" state. This is expected for poorly graded materials since the advantage of a higher relative compaction cannot be realized if there are insufficient fines available to fill in the voids. Critical state friction angles for "loose" RAP ranged from $41.2^0$ to $43.3^0$ and for "dense" RAP from $41.8^0$ to $44.3^0$.

For a cohesionless material, the Mohr-Coulomb failure envelope can be expressed as follows:

$$\tau = \sigma \tan\phi \qquad (1)$$

where $\tau$ = shear stress, $\sigma$ = normal stress and $\phi$ = friction angle. The curvature of the envelop and the secant value of $\phi$ decreases as the normal stress increases due to increased particle breakage according to Duncan and Wright [16]. Secant values of $\phi$ with curved envelopes can be modeled as follows [16]:

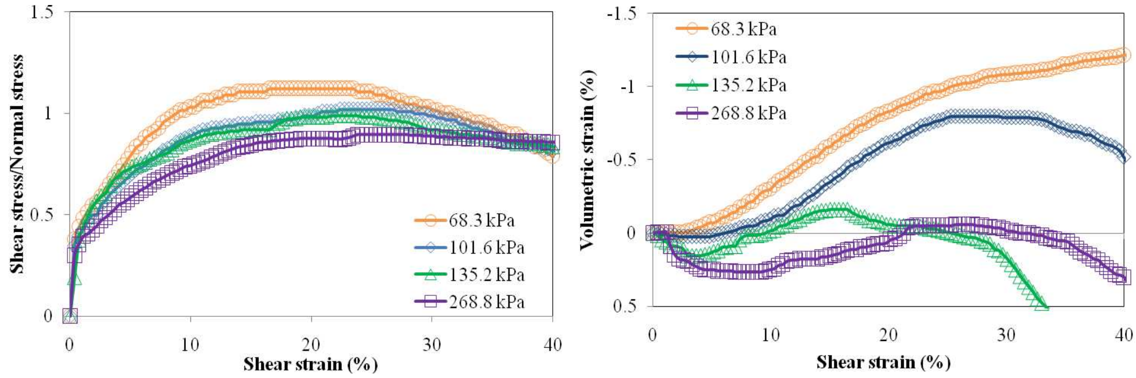
$$\phi = \phi_0 - \Delta\phi \tan\frac{\sigma}{p_a} \qquad (2)$$




(a) (b)

Figure 3: *Direct shear test results for "dense" RAP scalped on the No. 4 sieve: (a) normalized shear stress and (b) volumetric strain versus shear strain. [Normal stresses shown in legend. Volume change sign convention: (+) for compression and (–) for dilation.]*

where $\phi_0$ = friction angle at a normal stress = 1 atmosphere, $p_a$ = atmospheric pressure, and $\Delta\phi$ = slope of $\phi$ versus $\log(\sigma/p_a)$ plot. Values of $\phi_0 = 45.7^0$ and $\Delta\phi = 11.2^0$ fitted the peak failure envelop well ($R^2$ =0.9384) for "loose" RAP and $\phi_0 = 48.3^0$ and $\Delta\phi = 15.9^0$ fitted well ($R^2$ =0.9707) for "dense" RAP.




(a) (b)

Figure 4: *Direct shear test results for "loose" RAP: (a) normalized shear stress and (b) volumetric strain versus shear strain. [Normal stresses shown in legend. Volume change sign convention: (+) for compression and (–) for dilation.]*

*Table 4: Test results for "loose" and "dense" RAP*

| Relative Compaction[1] (%) | Normal Stress (kPa) | Failure[2] | | Critical State | |
|---|---|---|---|---|---|
| | | Friction Angle (degrees) | Shear Strain (%) | Friction Angle (degrees) | Shear Strain (%) |
| 95 | 68.3 | 48.3 | 16.5 | 43.3 | 34.4 |
| | 101.6 | 44.7 | 20.0 | 42.6 | 34.4 |
| | 135.2 | 44.5 | 20.0 | 41.4 | 34.4 |
| | 268.8 | 41.2 | 20.0 | 41.2 | 34.4 |
| 100 | 68.3 | 51.6 | 7.7 | 43.3 | 34.4 |
| | 101.6 | 48.3 | 11.4 | 43.8 | 34.4 |
| | 135.2 | 45.4 | 19.8 | 44.3 | 34.4 |
| | 268.8 | 42.1 | 20.0 | 41.8 | 34.4 |

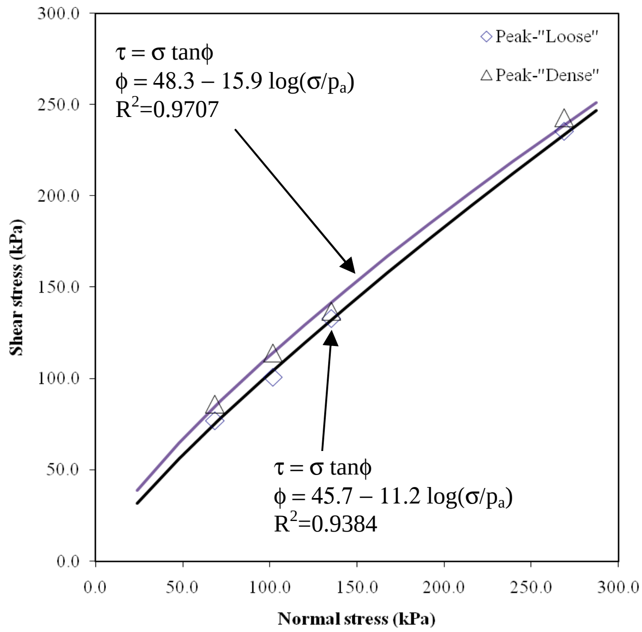



Figure 5: *Peak strength Mohr-Coulomb envelop based on direct shear testing for "Dense" and "Loose" RAP*

### *3.5 Validity of Measured Friction Angle*

It is well known that the direct shear test suffers from many shortcomings. As a consequence, different friction angles and failure envelops can be obtained if the shear strength were to be measured using other types of tests. There is an interesting question as to how different the friction angles will be. The following equation [17] relates the friction angle from direct shear ($\phi_{ds}$) with the friction angle from triaxial compression ($\phi_{tc}$).

$$\phi_{tc} = \frac{\tan^{-1}\left[\frac{\tan\phi_{ds}}{\cos\phi_{crit}}\right]}{1.12} \qquad (3)$$

where $\phi_{crit}$ = friction angle at critical state. Based on an average $\phi_{crit} = 42.7^0$ for RAP, Equation 3 suggests $\phi_{ds} \sim \phi_{tc}$ at $59.3^0$. Above $59.3^0$, $\phi_{ds} > \phi_{tc}$ and below $59.3^0$, $\phi_{ds} < \phi_{tc}$. However, the difference between $\phi_{ds}$ and $\phi_{tc}$ is at most $3.3^0$ for the range of measured $\phi_{ds}$ values at failure.

Many design problems are treated as plane strain in practice. It can be shown using Rowe's [18] equation below that the friction angle in plane strain testing ($\phi_{ps}$) is at most $8.8^0$ higher than in direct shear for the range of $\phi_{ds}$ measured and assuming $\phi_{crit} = 42.7^0$.

$$\tan\phi_{ds} = \tan\phi_{ps}\cos\phi_{crit} \qquad (4)$$

Thus for the RAP tested, $\phi_{ds}$ is at most $3.3^0$ lower than $\phi_{tc}$ and the direct shear test provides a conservative value of $\phi$ compared with plane strain testing, which is not common in practice. Thus, it can be concluded that the results presented herein are appropriate for practical purposes.

## 4 Summary and conclusions

In this study, the shear strength of RAP having a gradation that is close to a base course gradation was estimated by conducting direct shear tests on a gradation that is parallel to it. This parallel gradation is finer and allows the use of conventional size test equipment (61.4-mm-diameter shear box). The following conclusions are offered:

- The peak Mohr-Coulomb failure envelope was nonlinear with secant friction angles varying from $41.2^0$ to $48.3^0$ for "loose" and $42.1^0$ to $51.6^0$ for "dense" RAP at the highest and lowest normal stresses, respectively. The friction angles for "dense" RAP are only slightly higher than those in the "loose" state indicating the effects of relative compaction are small for the range of values (95% to 100%) adopted and for this test gradation. Of note is that the values are not insignificant.
- The highest $\tau/\sigma$ occurred at the lowest normal stress and this ratio decreased with increasing $\sigma$. Also, increasing $\sigma$ increased the strain to failure (ductility), and decreased the tendency to dilate.
- The strains at failure for "loose" RAP (16.5% to 20.0%) are higher than those for "dense" RAP (7.7% to 20.0%). Therefore, brittleness was affected by both normal stress and relative density.
- All RAP samples were dilative at failure except for the "loose" RAP tested at the two highest normal stresses.
- Critical state friction angles for "loose" RAP ($41.2^0$ to $43.3^0$) are close to those for "dense" RAP ($41.8^0$ to $44.3^0$). This is reasonable as the critical state friction angle is independent of the molding density.
- Based on theoretical equations relating the friction angle from direct shear ($\phi_{ds}$) to the friction angle from triaxial compression ($\phi_{tc}$) and to those from plane strain testing ($\phi_{ps}$), it was found that the direct shear test provides conservative values of $\phi$ that are appropriate for practical purposes.

## 5 List of references

1. Gonzalez, G.P., and Moo-Young, H.K. (2004), “Transportation applications of recycled concrete aggregate,” FHWA state of the Practice National Review, September, 2004, Washington, DC.
2. U.S. House of Representatives Committee on Transportation and Infrastructure (2008), “Maintaining our nation’s highway and transit infrastructure.” Washington, DC 20515.
3. Rebecca, D (2001). “RAP variability in Hot-Mix-Asphalt.” Year of the Recycled Roadway Materials, Texas.
4. Earland, M.G. and Pike, D.C. (1985). “Stability of gravel sub-base.” RR64 Transport and Road Research Laboratory, U.K.
5. Margaret , M.M. (1996). “An analysis of the shear strength of recycled aggregates.” Materials and Structures, Vol. 30, December 1997. pp. 599- 606.
6. Lowe, J. (1964). “Shear strength of coarse embankment dam material.” Proceedings, 8th Congress on Large Dams, pp. 745-761.
7. Marachi, D., Chan, C. and Seed, H. (1972). “Evaluation of properties of rock fill materials.” ASCE, Journal of the Soil Mechanics and Foundations Division, 98(1), pp. 95-114.
8. Verdugo, R., Gesche, R. and De La Hoz, K. (2003). “Metodologia de evaluacion de parametros de Resistencia al corte de suelos granulares gruesos.” 12th Pan American Conference on Soil Mechanics & Geotechnical Engineering, Cambridge, MA, Vol. 1, pp. 691-696.
9. Varadajan, A., Sharma, K., Venkatachalam, K. and Gupta, K. (2003). “Testing and modeling two rockfill materials.” ASCE, Journal of Geotechnical and Geoenvironmental Engineering, 129(3), pp. 206-218.
10. Verdugo, R. and Karem de la Hoz, (2006). “Strength and stiffness of coarse granular soils.” Geotechnical Symposium in Rome, March 16 & 17.
11. Al-Hussaini, M. (1983). “Effect of particle size and strain conditions on the strength of crushed basalt.” Canadian Geotechnical Journal, 20(4), pp. 706-717.
12. Cho, G., Dodds, J. and Santamarina, J. (2005). “Particle shape effects on parking density, stiffness and strength: Natural and crushed sands.” Internal report- Georgia Institute of Technology.
13. Lee, K. and Seed, H. (1967). “Drained strength characteristics of sands.” ASCE, Journal of the Soil Mechanics and Foundations Division, 93(6), pp. 117-141.
14. Santamarina, J. and Cho, G. (2004). “Soil behavior: The role of particle shape.” Proceedings Skempton Conference. London.
15. Santamarina, J. and Diaz-Rodriguez, J. (2003). “Friction in soils: Micro and Macroscale Observations.” 12th Pan American Conference on Soil Mechanics & Geotechnical Engineering, Vol. 1, MA, pp. 633-638.
16. Duncan, M. J., and Wright, G. S. (2005). Soil strength and slope stability. John Wiley & Sons, Inc., Hoboken, New Jersey.
17. Kulhawy, F. H., and P. W. Mayne (1990). “Manual on estimating soil properties for foundation design.” EPRI Report EL-6800, Projet 1493-6. Electric Power Research Institute, Palo Alto, Calif.
18. Rowe, P. W (1962). “The stress dilatancy relation for static equilibrium of an assembly of particles in contact.” Proceedings of the Royal Society of London, Series A, Vol. 269, pp. 500-527.

# RECYCLED AGGREGATE CONCRETE: A SUSTAINABLE BUILT ENVIRONMENT

M. Chakradhara Rao[1], S. K. Bhattacharyya[2], S. V. Barai[3 *]

[1] Research Scholar, Department of Civil Engg., Indian Institute of Technology, Kharagpur.
E-mail: chakradhar@civil.iitkgp.ernet.in, Phone: +91 9933589603, Fax: 03222-282254

[2] Director, Central Building Research Institute, CSIR, Roorkee 247 667, India
E-mail: director@cbri.in, Phone: 01332 272391, Fax: 01332 272272
(On deputation from IIT Kharagpur)

[3] Professor, Department of Civil Engg., Indian Institute of Technology, Kharagpur 721 302, India
E-mail: skbarai@civil.iitkgp.ernet.in, Phone: 03222-283408, Fax: 03222-282254

**Abstract**
In the present study the influence of recycled coarse aggregate (RCA) obtained from three different sources having different ages of structures are on the properties of recycled aggregate concrete (RAC) are investigated. RAC mixes are prepared with each source of RCA separately. In order to assess the performance of RAC in comparison with normal concrete, two normal concrete mixes prepared with 100% natural coarse aggregate. Locally available natural sand is used in all mixes. The compressive strength and the characteristics of interfacial transition zone (ITZ) viz. porosity and microhardness of RAC are studied. The results reveal that the ITZ of RAC is relatively loose and porous than the ITZ in normal concrete.

**Keywords:** *Recycled aggregate concrete (RAC), compressive strength, interfacial transition zone (ITZ), porosity, microhardness.*

## 1 Introduction

Recycling of construction and demolition waste has been considered from two main environmental aspects point of view: solving the increasing waste disposal crisis and save the depletion of natural resources. In the recent time due to significant increase in prices of natural raw materials of construction, and rise in the cost of waste storage in many regions, it has also become a burning issue (Ajdukiewwicz and Kliszczewicz, 2007). There is a scarcity of conventional building materials due to rapid construction activity and growing demand of houses in urban areas. Rapid industrialization has lead to the generation of huge quantities of construction and demolition wastes, which arises major problems of disposal. The disposal and utilization of construction and demolition waste is one of the major problems in India. Factors such as sustainability, economy, shortage of land for the disposal and shortage of good quality of raw materials for construction make it imperative that the construction and demolition waste should be properly recycled (Asnani, 1996). The recycling technology, not only solves the problem of waste disposal, but reduces the cost and preserves environment also. This also gives the way for the sustainable built environment in the construction industry.

## 2 Review of Literature

In the recent past, several researchers have studied the influence of recycled aggregate obtained from both aspects of laboratory crushed materials as well as field demolished structural materials on mechanical properties of recycled aggregate concrete (Hansen, 1985; Bairagi et al., 1993., Sagoe-Crentsil et al., 2001., Rao et al., 2007., Padmini et al., 2009). Nevertheless, the information available on the influence of recycled aggregate on the characteristics of ITZ is scarce. In the present study an objective is to investigate the influence of RCA obtained from different demolished structures on strength and on the characteristics of ITZ viz. porosity and microhardness of recycled aggregate concrete.

[*] Corresponding Author

## 3 Experimental Details

### *3.1 Materials*

Ordinary Portland Cement (OPC) of 43 grade conforming to Bureau of India Standard Specifications of IS: 8112 (1989) with specific gravity 3.14 is used in this study. The locally available natural sand and 20 mm maximum size natural coarse aggregate available from the local quarries conforming to the grading requirements of IS: 383 (1970) are used. The recycled coarse aggregates are obtained from three different demolished structures: two different demolished RCC culverts of different locations and a RCC slab of an old residential building. The recycled coarse aggregate obtained from the three sources are designated as RCA-S1, RCA-S2 and RCA-S3 respectively. The important properties of both natural and recycled coarse aggregates are presented in Table 1.

Table 1: *Properties of coarse aggregate*

| Property | Coarse aggregate | | | |
|---|---|---|---|---|
| | Natural | RCA-S1 | RCA-S2 | RCA-S3 |
| Specific gravity (SSD) | 2.75 | 2.51 | 2.47 | 2.417 |
| Water absorption (%) | 1.129 | 3.92 | 3.009 | 3.934 |
| Density (kg/l) | 1.581 | 1.413 | 1.34 | 1.35 |

### *3.2 Concrete Mixes*

Three recycled aggregate concrete mixes were prepared with 100% RCA obtained from each source. These mixes are designated as MM-RAC, MK-RAC and MV-RAC respectively. In all mixes, the letter M stands for the mix, the second letter represents the source of recycled coarse aggregate and RAC indicate the recycled aggregate concrete. In order to assess the performance of recycled aggregate concrete in comparison with normal concrete, two normal concrete mixes were prepared with 100% natural coarse aggregate and these are designated as M1-NAC and M2-NAC respectively. One corresponding to the RAC made with RCA obtained from the sources 1 and 2, and the other corresponding to RAC with source 3 RCA. The cement used in the test is OPC 43 grade manufactured by Ultratech Cement Co., India. While testing, some variation in the chemical composition is observed. Cement 1 is used for the RAC made with sources 1 and 2 RCA and corresponding normal concrete and cement 2 is used in RAC made with source 3 RCA and in corresponding normal concrete. In all mixes the locally available natural sand used as fine aggregate. All mixes were designed for M25 grade concrete in accordance with BIS (IS: 10262-1982). In all mixes, the free water-cement ratio was kept constant at 0.43 and slump was maintained in the range of 50-60 mm by adding Sika Viscocrete R-550 (l) superplasticiser. As the density of natural and recycled coarse aggregates is different, there is a little difference in the quantity of coarse aggregates. The details of all mixes are presented in Table 2.

Table 2: *Details of mix proportions (kg/m$^3$ of concrete)*

| Mix Designation | Cement (kg) | Natural FA (kg) | Natural CA (kg) | RCA (kg) | Free w/c ratio | Super - plasticizer* | Slump (mm) |
|---|---|---|---|---|---|---|---|
| M1-NAC | 401 | 574 | 1261 | 0 | 0.43 | 0.05 | 57.5 |
| MM-RAC | 401 | 574 | 0 | 1119 | 0.43 | 0.225 | 50 |
| MK-RAC | 401 | 574 | 0 | 1114 | 0.43 | 0.225 | 49 |
| M2-NAC | 401 | 574 | 1261 | 0 | 0.43 | 0.05 | 56 |
| MV-RAC | 401 | 574 | 0 | 1078 | 0.43 | 0.225 | 51 |

*percentage by weight of cement

### *3.3 Testing Procedure*

#### *3.3.1 Compressive strength*

The compressive strength test was conducted on 100 mm cubes after 7 and 28 curing. Three cube specimens were tested for 7 days curing and six cube specimens were tested for 28 days curing. A total of 9 cube specimens were tested for compressive strength in each mix.

#### *3.3.2 Sample preparation for measuring porosity and microhardness*

Specimen preparation is very important for identifying the features in scanning electron microscopy (SEM). Also a flat polished smooth surface is required for measuring the microhardness. In the present study after 28 days of curing 10 – 12 mm thick slices were cut from a 100 mm diameter × 200 mm height cylinder at different heights using a precision diamond saw and kerosene as lubricant. From each slice again approximately 10 × 10 mm rectangular sections were cut. The specimens were then dried in desiccators for more than 3 days. The dried specimens were then vacuum impregnated with a low viscosity epoxy coded as Epoxil-43 and hardener as Epoxil-MH43 in a 3:1 ratio and allowed to harden at room temperature for 1- 6 hours. The impregnated specimens are then carefully ground and polished with different sizes of grit paper and finer grades of diamond paste. The polished specimens are then cleaned in an ultrasonic bath and dried in vacuum to remove any remaining lubricant from the surface. The specimens are then coated with a thin layer of carbon to prevent charging during backscatter scanning electron (BSE) imaging.

#### *3.3.3 Scanning Electron Microscope*

The JEOL-JSM-6490 is a high performance scanning electron microscope with a high resolution of 3.0 nm was used in the present study. It is coupled with an energy dispersive spectrometer (EDS), which facilitates the qualitative analysis of the major elements on the surface.

#### *3.3.4 Microhardness*

A UHL VMHT microhardness tester (VMH-001) was used in the present study to measure the microhardness of ITZ of both normal concrete and recycled aggregate concrete made with all the three sources of RCA. The usefulness of this method is its ability to determine the response to load of a volume element that is considerably smaller than the ITZ (Igarashi, 1996). The Vickers microhardness test was conducted on the same samples on which the BSE images were acquired using scanning electron microscopy for measuring the porosity. The configuration of the Vicker's microhardness test is shown in Fig. 1. The Vickers microhardness was measured at 14 - 16 points within the distance of 235 μm from the aggregate surface. The measurements were taken randomly at a constant load of 10 gf with 10 s time. The test was conducted on three samples for each mix and the average results are reported. Here the microhardness symbol HV 0.01 means the test load is of 10 gf.

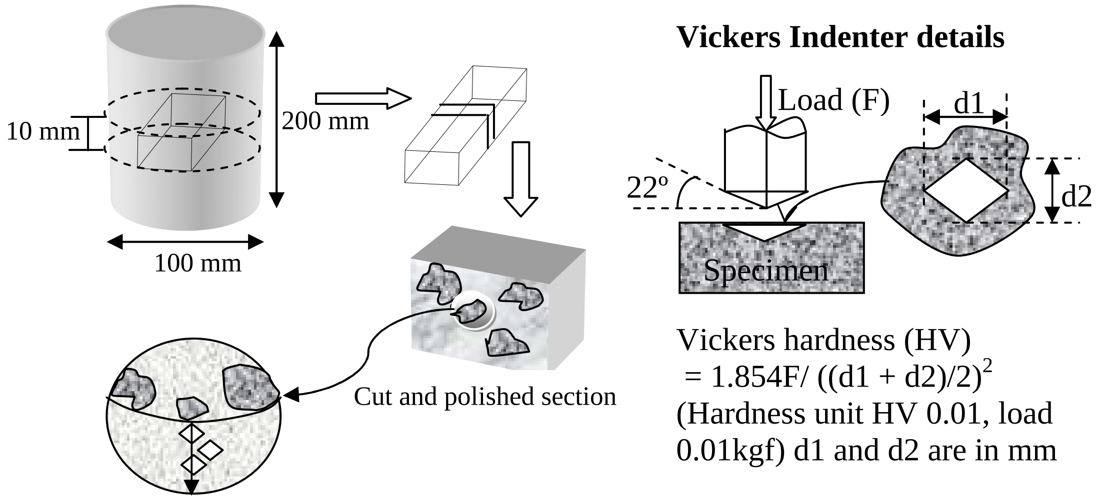



Fig. 1 *Configuration of the Vicker's microhardness*

## 4 Results and Discussion

### *4.1 Porosity*

Porosity is the volume which has not been filled by the cement grains or by the hydration products. The resolution in backscatter SEM limits the measurement of pore sizes. In the present study approximately 20 BSE images were analysed using image processing techniques from three samples of each mix (M1-NAC, MM-RAC, MK-RAC, M2-NAC, MV-RAC). All images were captured at 512 × 512 pixels; each pixel is approximately 0.3 µm in each direction which covers an area of 0.09 $\mu m^2$. Therefore the minimum pore size that can be measured is 0.3 µm and these are generally called capillary pores. The mean area percentages of porosity of both normal concrete and recycled aggregate concrete made with recycled aggregate obtained from all the three sources are presented in Table 3.

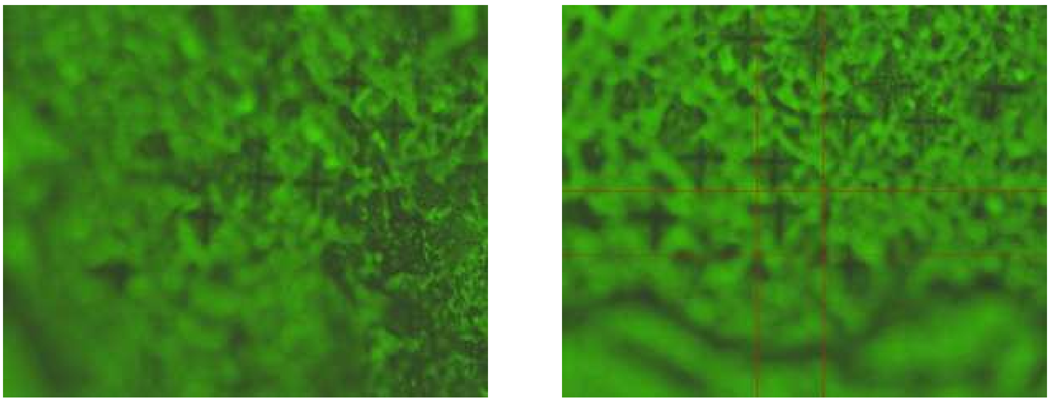
Table3: *Mean area percentages of porosity at ITZ*

| Source of RCA | Mix designation | Porosity (%) |
|---|---|---|
| RCC culvert near Midinapur | M1-NAC0 | 15.22 |
| | MM-RAC | 20.28 |
| RCC culvert near Kharagpur | M1-NAC | 15.22 |
| | MK-RAC | 21.02 |
| RCC slab of a old residential building, Vizianagaram | M2-NAC | 16.79 |
| | MV-RAC | 21.00 |

It reveals that the porosity in recycled aggregate concrete made with RCA obtained from all the three sources are more than those of corresponding normal concretes. The area of porosity in normal concrete is in the range of 15.22% - 16.7%. Whereas, the porosity in RAC made with RCA obtained from sources 1, 2 and 3 are 20.28%, 21.02% and 21% respectively. Albeit there are some chemical reactions expected between the remnant cement particles in recycled aggregates and new cement mortar would create some interfacial bonding effects, the results indicates that the ITZ in recycled aggregate concrete is loose and porous than the ITZ in normal concrete. This may be due to the presence of old mortar in recycled aggregates, which absorbs more water during the initial stages of mixing leads to the higher porosity.

### *4.2 Microhardness*

Vickers microhardness test is used to measure the microhardness along and across the width of the old ITZ. Typical microhardness indentations on various samples are shown in Fig. 2. The distribution of Vickers microhardness across the ITZ for both normal concrete and recycled aggregate concrete made with all the three sources of RCA are presented in Fig. 3.



(a) (b)

Fig. 2 *Microhardness indentations (a) normal concrete (M1-NAC) and (b) recycled aggregate concrete (MM-RAC)*

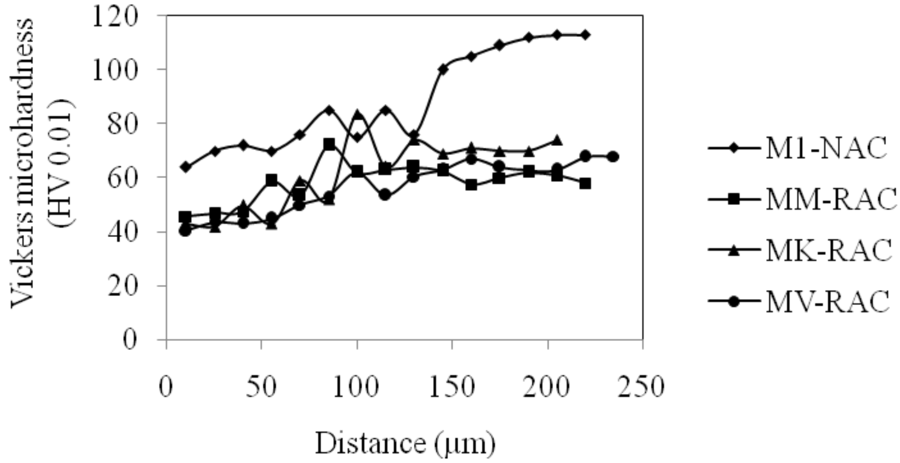



Fig. 3 *Gradients of Vickers microhardness of both normal and recycled aggregate concretes*

From the Figure it is ascertained that the Vickers microhardness increased with the distance increased from the aggregate surface in both normal concrete and recycled aggregate concretes made with all the three sources of RCA. In addition, it is observed that the Vickers microhardness upto 40 – 55 µm distance from the aggregate surface is not varying much in both normal concrete and recycled aggregate concrete. However, beyond these distances the Vickers microhardness is progressively increasing upto around 150 µm distance and thereafter almost it is constant. Lower value of Vickers microhardness indicates the presence of more microcracks and micropores which is an indication of higher porosity. The variation of Vickers microhardness across the ITZ defines the width of ITZ: the width of ITZ in both normal concrete and recycled aggregate concrete is within the range of 40 – 55 µm. Similar results are obtained in SEM examinations, the porosity near the aggregate surface (10-50 µm) is much higher than that of bulk paste in all the concretes. The variation in Vickers microhardness of RAC is almost same irrespective of the source of RCA. However, the Vickers microhardness of normal concrete is higher than that of recycled aggregate concrete made with all the sources of RCA across the ITZ. This may be due to the presence of old mortar adhered to RCA in RAC, which absorbs more water at the initial stages of mixing leads to the lesser hydration compounds and more porosity.

### *4.3 Compressive Strength*

The compressive strength of both normal and recycled aggregate concretes for different testing ages is presented in Fig. 4.

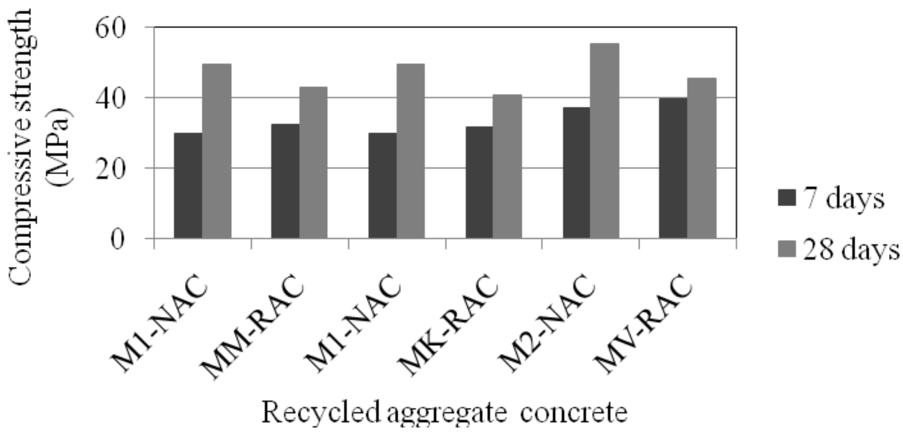



Fig. 4 *Development of compressive strength with age*

It is observed that in all mixes the target strength is achieved. It reveals that the strength gaining rate is relatively slow in case of all the recycled aggregates concrete compared to normal concrete between 7 days to 28 days curing period. There is an increase of approximately 13% - 25% of the compressive strength in recycled aggregate concretes when normal concrete strength increased by approximately

33% - 40% in the last 21 days of 28 days curing. This indicates that the recycled aggregate concretes attained more early strength than normal concretes. It was reported in the literature that the increase in compressive strength of RAC at early age is mainly due to high absorption capacity of old mortar adhered to the recycled aggregates and the rough texture of recycled aggregates that provide improved bonding and interlocking characteristic between the mortar and recycled aggregate themselves (Etxeberria et al., 2007). The compressive strength of RAC after 28 days of curing is less than that of normal concrete. The reduction in compressive strength after 28 days of curing in recycled aggregate concrete is in the order of 13% - 17% compared to the corresponding normal concrete.

## 5 Closing Remarks

The influence of recycled coarse aggregate obtained from three different demolished structures on strength and microstructural characteristics of ITZ are investigated. Based on the experimental results the following conclusions may be drawn.

- The target strength can be achieved in recycled aggregate concrete irrespective of the type of RCA, However, the compressive strength of recycled aggregate concrete was found to be lower than that of corresponding normal concrete after 28 days of curing.
- A relatively loose and porous interfacial transition zone is present in recycled aggregate concrete made with all sources of RCA compared to ITZ in normal concrete. The porous interfacial transition zone in recycled aggregate concrete may be attributed to the higher absorption capacity of recycled aggregates.
- The microhardness of ITZ in recycled aggregate concrete was found to be lower than that of normal concrete ITZ. The lower values of microhardness indicate the higher percentage of porosity.

**Acknowledgements**

The work presented in this paper is a part of Doctoral Thesis carried out by first author, who received the financial support from University Grants Commission (UGC), Government of India. The authors also wish to thank the supplier of Sika Viscocrete Superplasticizer, SIKA India Pvt. Ltd., Kolkata.

**References**

Ajdukiewicz, A.B., Kliszczewicz, A.T. (2007), “Comparative tests of beams and columns made of recycled aggregate concrete and natural aggregate concrete.” *Journal of Advanced Concrete Technology*, 5(2): 259 – 273.

Asnani, P.U. (1996), “Municipal solid waste management in India.” *Proceedings of the waste management workshop*, 24-28 June 1996, Nicosia, Cyprus.

Bairagi, N.K., Kishore, R., Pareek, V.K. (1993), “Behaviour of concrete with different proportions of natural and recycled aggregates.” *Resources Conservation and Recycling*, 9(1-2): 109 – 126.

Hansen, T.C. (1985), “Recycled aggregates and recycled aggregate concrete second state of art report developments 1945 – 9185.” *RILEM Technical Committee - 37 - DRC.*

Sagoe-Crentsil, K.K., Brown, T., Taylor, A.H. (2001), “Performance of concrete made with commercially produced coarse recycled aggregates.” *Cement and Concrete Research*, 31(5): 707 – 712.

Rao, A., Jha, K.N., Misra, S. (2006), “Use of aggregates from recycled construction and demolition waste in concrete.” *Resources, Conservation and Recycling*, 50(1): 71 – 81.

Padmini, A.K., Ramamurthy, K., Mathews, M.S. (2009), “Influence of parent concrete on the properties of recycled aggregate concrete.” *Construction and Building Materials*, 23(2): 829 – 836.

Igarashi, S., Bentur, A., Mindess, S. (1996), “Microhardness testing of cementitious materials.” *Advanced Cement Based Materials*, 4(2): 48 – 57.

Etxeberria, M., Vazquez, E., Mari, A., Barra, M. (2007), “Influence of amount of recycled coarse aggregates and production process on properties of recycled aggregate concrete.” *Cement and Concrete Research*, 37(5): 735 – 742.

# EFFECT OF DIMENSIONAL VARIATION OF CSE BLOCKS

C. Jayasinghe[1], T.S. Pathirage[2], K.R.T.I. Perera[3], C. Kariyapperuma[4]

[1]Professor, Department of Civil Engineering, University of Moratuwa, Moratuwa, Sri Lanka.
[1]E-mail: chintha@civil.mrt.ac.lk
[1]Telephone: +94-11-2650567-8 (Ext.2111) ; Fax: + 94-11-2651216

[2]Department of Civil Engineering, University of Moratuwa, Moratuwa, Sri Lanka.
[2]E-mail: pathiragets@gmail.com
[2]Telephone: +94-91-2292374

[3]Department of Civil Engineering, University of Moratuwa, Moratuwa, Sri Lanka.
[3]E-mail: thilina_uom@yahoo.com
[3]Telephone: +94-11-2298378

[4]Department of Civil Engineering, University of Moratuwa, Moratuwa, Sri Lanka.
[4]E-mail: chamath85@gamil.com
[4]Telephone: +94-34-2247242

**Abstract:** Compressed stabilized earth (CSE) blocks are one of the alternative building materials that are becoming popular due to insufficiency of conventional building materials and its sustainability. CSE blocks are manufactured with different unit dimensions. This research is focused on determination of effect of unit dimensions of CSE blocks on masonry construction. Unit dimensions basically affect on the compressive strength and cost of construction of masonry. Ratio of height to least horizontal dimension (H/W) is a governing factor of compressive strength. In this research, plain CSE blocks having H/W ratio of less than 0.6 have been considered for which there is no provision for characteristics strength ($f_k$) of masonry in BS 5628: part1:1992 [6]. Four different sizes of blocks with H/W ratio of less than 0.6 were used for the test. Relationships between Characteristic compressive strength of masonry, unit strength, and H/W ratio and load deformation characteristics were developed. Studies were done to determine the cost of construction of each panel. Results show that strengths of all panels are adequate for load bearing construction. Wall strengths increases with the H/W ratio. Panels provide a sufficient warning before ultimate failure. When the H/W ratio is close to 0.6 panel strength is comparable with the values provided in Table 2.0 of BS 5628: part1:1992.

**Keywords:** Compressed stabilized earth (CSE) blocks, Height to least horizontal dimension ratio (H/W), Compressive strength, Embodied energy, cost of construction

## 1. Introduction

Masonry has been used for many years as a popular walling material. Masonry wall construction has a number of advantages including relatively low cost, fire protection, thermal and sound insulation, weather protection, wider availability and attractive appearance [2, 3]. Masonry wall construction has undergone a considerable change in last few decades with the introduction of new materials and new type of units [2].

Use of alternative walling material has become increasingly popular due to the scarcity of conventional building materials, such as burnt clay bricks, river sand etc. Compressed stabilized earth (CSE) blocks are one such material that is becoming popular in the recent time. Use of earth as a walling material for houses is gradually regaining the popularity in many parts of the world due to recent development in stabilization techniques [1].

Compressive strength of masonry is an important parameter in designing masonry structures. It is

greatly influenced by unit characteristics such as strength, type and geometry [3, 8, 9]. Lack of quality controlling in masonry units manufacturing process in Sri Lanka has resulted various sizes of bricks and blocks coming into the market. Compressed earth blocks are produced in a greater variety than many other masonry blocks [11]. This has resulted a need of a study of effect of unit dimensions of CSE blocks on masonry construction.
It has been shown by Jayasinghe (2007) [1] that the characteristic compressive strength ( fk ) of CSE masonry with the ratio of height to least horizontal dimension (H/W) of 0.6 can be determined when the unit strengths are known by using the wall strength values specified in Table 2.0 of BS 5628: part1:1992 with some modification factors. But not enough studies were done on the walls constructed with CSE blocks having a ratio of height to least horizontal dimension of less than 0.6. This research paper covers a comprehensive study on effect of dimensional variation of CSE blocks with H/W ratio of less than 0.6 on masonry construction.

## 2. Objectives

This research was carried out to determine the effect of dimensional variation of CSE blocks with a ratio of height to least horizontal dimension of less than 0.6 on compressive strength of masonry.

## 3. Methodology

In order to achieve above objectives, following methodology was used:

I. Wall panels were made with plain CSE blocks with four different H/W ratios of less than 0.6 but same horizontal dimensions.
II. Two identical panels were made from each block type.
III. In order to determine the cost of construction, materials required and time taken for construction were measured for each panel.
IV. Wall panels were tested for the compressive strength 28 days after construction.
V. Failure patterns and load deformation characteristics were also observed.
VI. Individual blocks were tested to determine the unit compressive strength.

Results of above tests were used to find the correlations between unit strength, panel strength, load-deformation characteristics and H/W ratio.

## 4. CSE as a sustainable material

Due to limited resources in the world for construction activities sustainability would be a great important concept. A main reason for CSE blocks to gain its popularity is the keenness of developers to attempt the use of alternative building materials to improve the sustainability of building construction industry. In this context compressed stabilized earth bricks and blocks can be considered as viable alternatives [1]. Major advantages of CSE blocks [10, 12] can be listed as follows:

a) Energy efficient; consuming less than half of the energy required for conventional building methods leading to energy conservation
b) Economical; 20–40% savings in cost when compared to brick masonry
c) Plastering can be eliminated
d) Better block finish and aesthetically pleasing appearance
e) Techniques are simple and employ maximum local resources and skills
f) Decentralized production systems and small-scale operations that generate local employment
g) Reduce cost and energy involved in transportation of building products

### *4.1. Embodied Energy*

Embodied energy is the energy needed in preparing and extracting the raw materials, energy for transportation of the same and the external energy applied to raw materials in producing or assembling the final product [10]. When comparing the embodied energy in different materials, what is important is the energy flow of each and every unit including formation, transformation, transportation and installation. Some data available in literature for basic materials have been used in this study. This data can be used to determine and compare the embodied energy in CSE blocks with conventional burnt clay bricks.

CSE blocks contain 5%-6% of cement used for the manufacturing. Block can be either electrically operated machine compacted or manually compacted. Drying is done by solar radiation and no extra energy is consumed. Ordinary bricks have higher energy usage when burning. As CSE blocks do not require burning, it saves about 70% of the energy when compared to burnt clay bricks [12]. In tropical climatic conditions laterite soils are commonly found as laterite hills. Since it is readily available in most of the locations, energy requirement in transportation is comparatively less [10]. As it uses simple techniques, employ maximum local resources and skills, and can finish without a plaster (hence minimum use of cement) embodied energy in final product is much less than in conventional burnt clay bricks. A study done by Reddy (2004) [12] shows that the energy consumed by the load bearing conventional two-storied brickwork building is 2.92 GJ/m2. Two-storied building using alternative building materials like CSE walls is highly energy efficient. The energy consumed by this building is 1.61 GJ/m2, which is about 55% of that consumed by conventional brick wall building respectively.

### *4.2. Life Cycle energy*

Any comprehensive assessment of architectural energy consumption must in fact consider the entire life cycle of the building, which can be divided into three phases: pre-use phase (embodied energy), use phase (operational energy) and post-use phase (demolishing or possible recycling and reuse) [5]. Extensive testing carried out by many researchers has indicated that CSE block masonry is of adequate strength. Since cement based products tend to gain strength with age, the durability of CSE masonry will be comparable with conventional materials. Thus the life cycle energy will primarily depend on the embodied energy and operational energy [10]. Embodied energy of CSE blocks has been already discussed in the previous section.

When considering the life cycle energy, conventional masonry such as cement blocks and other ordinary bricks and blocks have a higher operational and maintenance energy. Replacement of one unit or maintenance in a usage level consumes much high energy. Comparing heat and thermal comfort, CSE blocks perform far better than conventional units.

### *4.3. Environmental concerns*

CSE blocks cause less environmental problem compared to other conventional bricks and blocks. Extensive use of burnt clay bricks and cement sand blocks has given rise to many environmental problems. Extensive clay mining has created deep pits that led to lowering the ground water table. Stagnation of water has become breeding grounds for mosquitoes. Cement sand blocks need high amount of sand for the manufacturing. Both conventional bricks and cement blocks require high amount of sand and cement for the plaster. Excessive sand mining in rivers has caused many problems including lowering water table and salt water intrusion. High usage of cement and burning in the case of burnt clay bricks increases the $CO_2$ emission to the atmosphere. As CSE block walls can be finished without a plaster, use of sand and cement is less.

At the end of the life cycle, decaying of the materials would cause hazards for the environment in direct and indirect way. Cementing materials cause direct problems in underground water paths and spill ways. And toxic elements which added to the soil and water when at the end also cause problems. As cement blocks contain much higher amount of cement, at the end of the life cycle decaying percentage of the cement is high. Ordinary bricks with plaster are having high percentage of cement. But CSE without a plaster and contains low cement percentage compared to others and less environmental problems are caused.

## 5. Experimental programme and results

In order to determine the effect of H/W ratio of CSE blocks on masonry construction four sizes of blocks were selected for testing. All the blocks are having the same horizontal dimensions but different heights so that the H/W ratios are different. H/W ratio was kept less than 0.6 as the scope of this research is limited to that. Selected block sizes are shown in the Table 1. Figure 1 shows the blocks used for the experiment.

Table 1- *Selected block sizes*

| Block dimensions (mm) | H/W ratio |
|---|---|
| 225x220x100 | 0.45 |
| 225x220x110 | 0.50 |
| 225x220x120 | 0.55 |
| 225x220x128 | 0.58 |

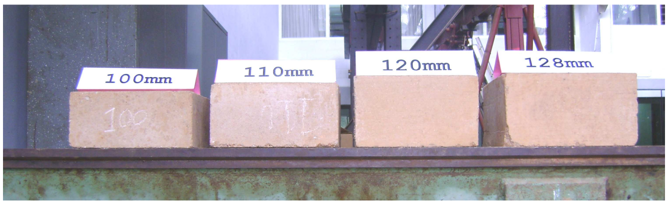


Figure 1- *Blocks used for the experiment*

### *5.1. Construction of wall panels*

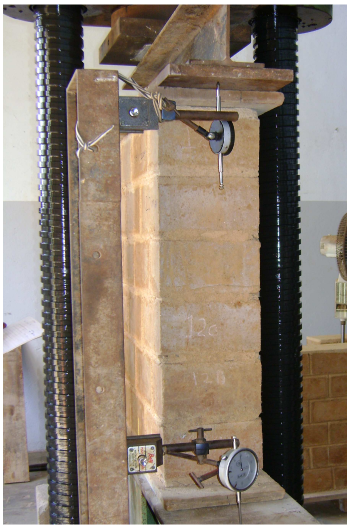
Appendix A of BS 5628: part1:1992, specifies the sizes of wall panels that should be used to test the compressive strength of masonry [6]. Size of a test panel was limited to a length of 3 blocks and height of 6 courses to avoid slenderness effect and for easy handling. Heights of wall panels were not the same due to the variation of block height. But the number of courses was kept equal. Bond pattern used was stretcher bond.
Panels were made using 1:2:6 cement: soil: sand mortar. Soil used for the mortar was laterite soil sieved with 2.36mm sieve. Soil was kept 24 hrs in water and saturated soil was used. Top of the panel was capped with the same mortar to have a level surface. Figure 2 shows the test panels constructed for testing.


Figure 2 - *Test panels constructed for testing*

Materials used to prepare the mortar were measured using a gauge box. Amount of mortar used for each panel and time taken to construct was recorded to determine the effect on cost of construction of masonry.

### *5.2. Unit strength and compressive strength of masonry*

Compressive strength has become a basic and universally accepted unit of measurement to specify the quality of masonry units. The relative easiness of undertaking laboratory compressive strength testing has also contributed to its universality as an expression of material quality [11]. Dry strength of CSE block units were tested according to the standard test method. Three blocks from each size were tested and the average value was taken as the compressive strength unit.
Compressive strength of masonry can be determined from the ultimate strength of block panels tested in accordance with the test procedure given in BS 5628: part1:1992 [6]. Test was carried out on two nominally identical wall panels. Deformation of the wall with the load was observed using two dial gauges fixed to the top and bottom of the test panel. Figure 3 shows a test panel prepared for testing.



### *5.3. Results*

***Unit strength of CSE block***

Average unit strengths of CSE blocks obtained from test are shown in Table2.

Table 2 - *Average unit strengths of CSE blocks*

| Block dimensions (mm) | H/W ratio | Average strength (N/mm$^2$) |
|---|---|---|
| 225x220x100 | 0.45 | 5.879 |
| 225x220x110 | 0.50 | 4.336 |
| 225x220x120 | 0.55 | 4.851 |
| 225x220x128 | 0.58 | 6.387 |

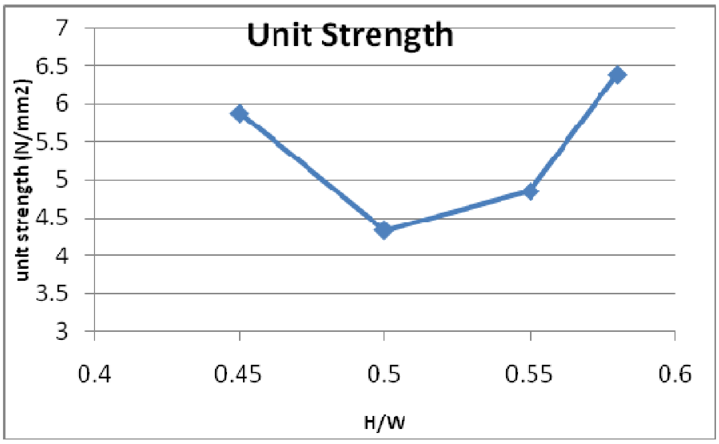



Figure 4- *Unit strength variation*

***Compressive strength of masonry***

Load at the first crack is one of the most important indications of suitability of brickwork for construction. It is of paramount importance to ensure that the wall is free from cracks under working load stresses [15]. Ultimate strength is important to determine the characteristic compressive strength. Compressive strengths of wall panels were tested according to standard method to determine the load at the first crack and failure load. Figure 5 shows two test panels after ultimate failure. Results are shown in the Table 3.

Figure 5 - *Test panels after ultimate failure*

Table 3 – *Average panel strengths*

| Block dimensions (mm) | H/W ratio | Stress at first crack (N/mm$^2$) | Average panel strength (N/mm$^2$) |
|---|---|---|---|
| 225x220x**100** | 0.45 | 1.038 | 1.965 |
| 225x220x**110** | 0.50 | 1.301 | 2.032 |
| 225x220x**120** | 0.55 | 1.752 | 2.275 |
| 225x220x**128** | 0.58 | 1.956 | 2.384 |

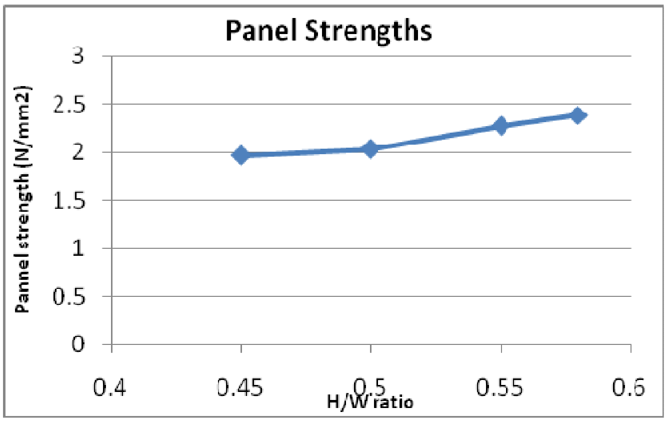



Figure 6 – *Panel strength variation*

***Load deformation characteristics***

Determination of load deformation characteristics for CSE block masonry is important because CSE blocks use for load bearing wall construction, it should give sufficient warnings prior to failure [1, 14].
Load deformation relationships for different blocks were developed by using two dial gauge readings. Those curves are shown below.

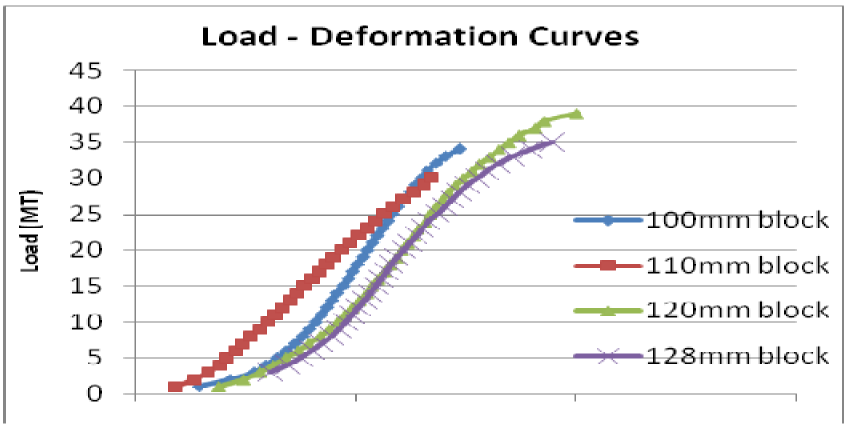



Figure 7 – *Load-Deformation curves*

## 6. Analysis of results

Generally the compressive strength of blocks decreases with the increasing height [8, 9]. But the test results of blocks used for this research did not show such a variation. That may be due to some defects of blocks. Further studies should be done to verify the variation.

A number of investigations done on effect of unit height of masonry units on compressive strength of masonry, show that the compressive strength of walls increases with the unit height [8, 9]. Test results of this research are also complying with that. Compressive strength of masonry increases from 1.965N/mm$^2$ to 2.384 N/mm$^2$ when H/W ratio increases from 0.45 to 0.58. Stress at the first crack also increases. It is shown that for two storey houses with normal room sizes wall strength of 1.5 kN/mm$^2$ is sufficient [4]. Hence compressive strengths of all panels are adequate for load bearing wall construction.

Load deformation characteristics obtained by plotting test results shows that wall panels undergo sufficient deformation before they fail. Hence it provides sufficient warning before ultimate failure. Panels made with blocks having H/W ratio of 0.55 and 0.58 show more ductile behavior than those made with blocks having H/W ratio of 0.45 and 0.5.

Australian earth building handbook recommends a design E value of about 0.2 kN/mm$^2$ [13]. Panels tested for this research are having E values in the range of 0.27-0.37 kN/mm$^2$.

Cost of construction is an important parameter when considering the viability of using alternative building materials. Determination of effect of unit dimensions on cost of construction is one objective of this research. Cost of construction for different panels is shown in Table 4.

Table 4 – *Cost of construction*

| | Sizes of block thickness | | | |
|---|---|---|---|---|
| | 128mm | 120mm | 110mm | 100mm |
| Cost of panel (Rs/-) | 2079.31 | 2045.20 | 2214.83 | 2437.37 |
| Cost for unit area(Rs/m2) | 3572.60 | 3725.31 | 4359.90 | 5208.05 |

## 7. Comparison of results with burnt clay bricks

The mortar used for the construction of wall panels is 1:2:6 Cement: Soil: Sand mortar. This can be considered as equivalent to mortar designation iv. When tested for the compressive strength of panels, the panel made of 128 mm high (H/W ratio of 0.58) blocks gave strength of 2.384N/mm$^2$. This can be compared with the wall strength values given for the masonry, constructed with blocks having H/W ratio of 0.6, in table 2 (b) of BS 5628: part1:1992 [6] as 0.58 is close enough to 0.6.

A study done by Jayasinghe and Mallawarachchi (2009) [3] has shown that cement stabilized earth bricks and blocks walls would be capable of performing in a manner comparable to good quality burnt clay bricks of 5 N/mm$^2$ compressive strength. Thus the CSE stabilized with 5% cement has the potential to provide an alternative that can be manufactured to perform very similar to burnt clay bricks of 5N/mm$^2$ compressive strength [3]. Thus for a unit strength of 5N/mm$^2$, Table 2 (b) of BS 5628: part1:1992 [6] gives a characteristics compressive strength of 2.2 N/mm$^2$ which is very close to the value we got for blocks of H/W ratio of 0.58.

## 8. Conclusion and Recommendations

CSE blocks used in masonry construction are becoming popular in order to meet sustainable construction concepts. These blocks are manufactured in different scale using manual, semi automated and fully automated machines. Different quality controlling procedures can give rise to dimensional variations for the CSE blocks. It was found in the experimental program covered in this paper, maintaining H/W ratio around 0.6 would be beneficial in terms of characteristics wall strength. This can also lead to use BS 5628: part1:1992 for design of masonry constructed with CSE blocks. It is also recommended to maintain good quality controlling at the manufacturing stage.

## Reference


1. C. Jayasinghe, Influence of bond pattern on cement stabilized earth (CSE) brick and block walls, masonry international, 20 (2007).
2. A.W Hendry, Masonry walls: materials and construction, Construction and Building materials 15 (2001) 323-330.
3. C. Jayasinghe, R.S. Mallawarachchi, Flexural strength of compressed stabilized earth masonry materials, Materials and Design 30 (2009) 3859-3868.
4. Jayasinghe M.T.R, Load bearing construction with local bricks, Engineers journal of institution of engineers Sri Lanka, xxvii (1), pp 49-57, 1998.
5. N. Huberman, D. Pearlmutter. Life-cycle energy analysis of building materials in the Negev desert, Energy and Buildings 40 (2008) 837-848.
6. BS 5628: Part 1: 1992, Code of practice for Use of masonry, British standard institute, United Kingdom.
7. B.V. Venkatarama Reddy, P. Prasanna Kumar, Embodied energy in cement stabilized rammed earth walls, Energy and Buildings 42 (2010) 380-385.
8. A.W. Hendry, B.P.Sinha, S.R. Davies, Design of Masonry Structures, E & FN Spon, London. 1997.
9. A.W. Hendry, Structural Masonry, Macmillan education ltd, London, 1990.
10. C. Jayasinghe. Embodied energy of alternative building materials and their impact on life cycle cost parameters.
11. Jean-Claude Morel, Abalo Pkla, Peter Walker, Compressive strength testing of compressed earth blocks, Construction and Building Materials 21 (2007) 303–309
12. B. V. Venkatarama Reddy, Sustainable building technologies, current science, vol. 87, no. 7, 10 October 2004
13. The Australian Earth building handbook, standards Australia, Sydney, Australia.2002.
14. C. Jayasinghe, Structural Design of Earth Buildings, united printers, 2009.
15. M.T.R Jayasinghe, Load bearing brickwork construction for Sri Lanka, Middleway Ltd, Sri Lanka, 1997.



## Acknowledgements

The technical staff of the Department of Civil Engineering, Messrs S.P.Madanayake, S.L Kapuruge assisted this research at the experimental programme.


## About the Authors


**C. Jayasinghe,** B.Sc. Hons. Moratuwa., M.Eng.(Highway & Traffic Engineering), Ph.D., C.Eng., MIE(SL). is a Professor at the Department of Civil Engineering, University of Moratuwa.

**T.S. Pathirage,** B.Sc. Hons. Moratuwa.

**K.R.T.I. Perera,** B.Sc. Hons. Moratuwa.

**C. Kariyapperuma**, B.Sc. Hons. Moratuwa.

# EFFICIENCY FACTORS FOR BOTTLE-SHAPED STRUTS IN DEEP BEAMS MADE OF RECYCLED COARSE AGGREGATE

Sahoo Dipak Kumar[1], Singh Bhupinder[2], Bhargava Pradeep[3]

[1] Ph.D., Reader, School of Engineering, Cochin University of Science & Technology, Kochi-682022, Kerala, India.(Corresponding author. E-mail: dipaksahoo2000@yahoo.com)

[2] Ph.D., Assistant Professor, Department of Civil Engineering, Indian Institute of Technology Roorkee, Roorkee-247667, India.

[3] Ph.D., Professor, Department of Civil Engineering, Indian Institute of Technology Roorkee, Roorkee-247667, India.

**Abstract**
Based on reanalysis of the shear test results of reinforced concrete beam specimens made of recycled coarse aggregates reported in the literature, this study points out that the strut-and-tie modeling (STM) provisions developed for natural coarse aggregate concrete can be applied to recycled coarse aggregate concrete with no reduction in the efficiency factors of bottle-shaped struts. The experimentally obtained strut efficiency factors in beams made of recycled coarse aggregate concrete were comparable to those in beams made of natural coarse aggregate concrete. The study also highlights lack of conservatism in the STM provisions of current design codes irrespective of the type of coarse aggregates used.

***Keywords:*** *Recycled coarse aggregate; strut-and-tie model; bottle-shaped strut; efficiency factor; shear.*

## 1. Introduction

The idea of recycling demolished old concrete to manufacture recycled coarse aggregates (RCA) for new structural concrete is driven by an ever increasing global concern for the environment. Amidst controversies over its strength and durability, RCA concrete is gradually receiving due acceptance as an efficient structural material with properties well comparable with conventional natural aggregate concrete. For structural engineers, the foremost concern with RCA concrete is whether the design provisions in the current concrete codes which are developed for natural coarse aggregate concrete can be applied without alteration to RCA concrete. Choi et al. (2010) observed that the direct application of current design methods is acceptable for RCA concrete with RCA replacement ratio of up to 50 % beyond which the shear strength may be reduced by as much as 30 %. On the contrary, Fathifazl et al. (2008 and 2009(2)) have argued that the apparent reduction in the shear strength of reinforced RCA concrete (RRC) beams reported by other researchers are attributable to conventional method of mix proportioning. They have demonstrated that if RCA concrete is proportioned by their equivalent mortar volume (EMV) method (Fathifazl et al. 2009 and 2010), then RCA concrete may even outperform conventional natural coarse aggregate concrete beams in terms of shear strength.

In the present study, the shear strength tests on 12 RRC beams performed and reported by Han et al. (2001) have been reanalyzed using strut-and-tie model to ascertain whether the efficiency factors recommended in the current design codes and the literature can be used for RRC beams.

## 2. Material and methods

Han et al. (2001) tested 12 reinforced concrete beams using natural, washed recycled and non-washed recycled coarse aggregates. All the beams were of 170 mm width and 300 mm overall depth (effective depth = 270 mm) and were tested under two point symmetrical loading with shear span-to-effective depth ( $a_v / d$ ) ratios varying from 1.5 to 4.0, **Fig. 1**. The main reinforcement provided in one layer at the beam bottom (1.11 % for specimens at S. Nos. 1 to 6 and 2.21 % for those at S. Nos. 7 to 12) was adequate to ensure that the beams did not fail in flexure. The cover to the tension reinforcement was 30 mm whereby the width of tie was considered to be 60 mm. The specimen details are given **Table 1**, with reference to **Fig. 1**. The two specimens, C-2.0-N and C-2.0-W2, wherein natural coarse aggregates were used served as control specimens. For the two specimens, NR-2.0-N and NR-2.0-W2, recycled coarse aggregate was not washed of the dust etc. For the rest of the specimens, recycled coarse aggregate was washed clean to make them free from surface dirt. The properties of the natural and recycled (washed and non-washed) coarse aggregates and the mixture proportions of different types of concrete used in the specimens can be found in the original paper by Han et al. (2001). The beams were tested under two-point symmetrical loading, interspaced by 540 mm (twice the effective depth), **Fig. 1**.

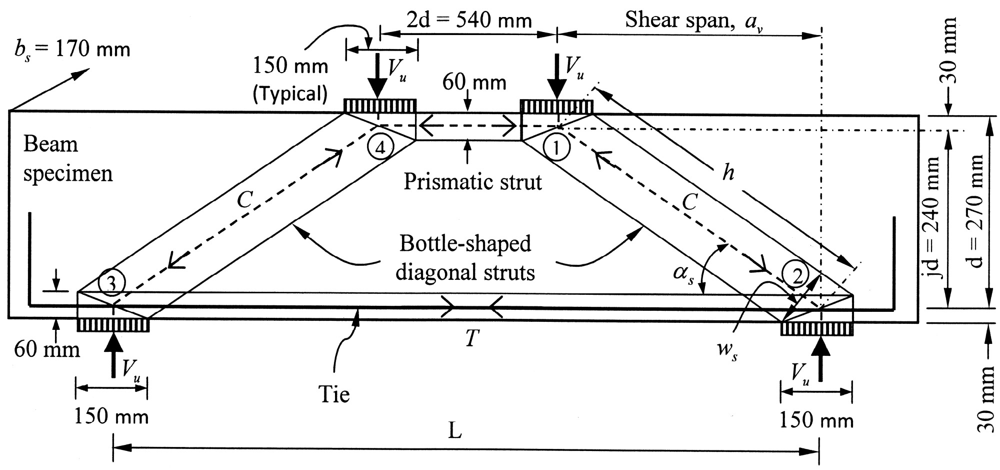



**Fig. 1**: *Dimensions of a typical beam specimen with strut-and-tie model*

**Table 1:** *Details of the beam specimens*

| S. No. | Beam ID | Coarse aggregate type | $f_c'$ (MPa) | $a_v/d$ | $\alpha_s$ (degrees) | Web steel ratio, $\rho_V$ | $\rho_T$ | $h$ | $w_s$ |
|---|---|---|---|---|---|---|---|---|---|
| I | II | III | IV | V | VI | VII | VIII | IX | X |
| 1 | R-1.5-N | Washed Recycled | 39.62 | 1.5 | 31.0 | 0 | 0 | 466 | 129 |
| 2 | R-2.0-N | Washed Recycled | 30.57 | 2.0 | 24.0 | 0 | 0 | 590 | 116 |
| 3 | R-3.0-N | Washed Recycled | 31.23 | 3.0 | 16.5 | 0 | 0 | 845 | 100 |
| 4 | R-4.0-N | Washed Recycled | 31.89 | 4.0 | 12.5 | 0 | 0 | 1108 | 91 |
| 5 | NR-2.0-N | Non-washed Recycled | 32.56 | 2.0 | 24.0 | 0 | 0 | 590 | 116 |
| 6 | C-2.0-N | Natural | 37.43 | 2.0 | 24.0 | 0 | 0 | 590 | 116 |
| 7 | R-2.0-W1 | Washed Recycled | 41.86 | 2.0 | 24.0 | 0.00089 | 0.0007 | 590 | 116 |
| 8 | R-2.0-W2 | Washed Recycled | 41.11 | 2.0 | 24.0 | 0.00244 | 0.0020 | 590 | 116 |
| 9 | R-2.0-W5 | Washed Recycled | 31.58 | 2.0 | 24.0 | 0.00507 | 0.0042 | 590 | 116 |
| 10 | R-2.0-W8 | Washed Recycled | 41.11 | 2.0 | 24.0 | 0.00823 | 0.0069 | 590 | 116 |
| 11 | NR-2.0-W2 | Non-washed Recycled | 37.43 | 2.0 | 24.0 | 0.00244 | 0.0020 | 590 | 116 |
| 12 | C-2.0-W2 | Natural | 49.83 | 2.0 | 24.0 | 0.00244 | 0.0020 | 590 | 116 |

*Note: Nomenclature: Coarse aggregate type–$a_v/d$ – web reinforcement detailing. R: washed recycled coarse aggregats. NR: non-washed recycled coarse aggregate. C: natural coarse aggregate. N: No web reinforcement. W1 through W5: $\rho_V$ varies from 0.00089 to 0.00823 as shown in Col. (VI).*

## 3. Theory/calculation

The beams were analyzed by strut-and-tie models, **Fig. 1.** The transfer of loads to the adjacent supports was assumed to take place through arch (direct strut) action as six of the twelve beams at S. Nos. 1 through 6, had no shear reinforcement. For ease of comparison, the same type of load transfer mechanism was assumed for the rest of the beams which had varying amounts of shear reinforcements in the form of vertical stirrups. Since the length of the load and support bearing plates were not reported by Han et al. (2001), the same was scaled from their figures and assumed to be 150 mm, **Fig. 1**. For simplicity, the depth of the top nodes was assumed to be 60 mm and hence, the width of the prismatic strut 1-4 was taken as 60 mm. Accordingly, the lever arm, $jd$, was calculated as 240 mm, **Fig. 1**. From the given $a_v/d$ ratios, $a_v$ values were calculated knowing $d$ = 270 mm. The angle of inclination of the diagonal struts, 1-2 or 4-3, was computed using the relationship $\cot\alpha_s = a_v / jd$. The length of the strut was calculated as $h = 240/\sin\alpha_s$ and the width of the strut, $w_s$, was calculated as $w_s = 150\sin\alpha_s + 60\cos\alpha_s$. The effective transverse reinforcement ratio, $\rho_T$, was calculated using the corrected transformation suggested by Sahoo et al. (2009) given below and presented in **Table 1**.

$$\rho_T = \sum \frac{A_{si}}{b_s s_i} \sin^2 \alpha_i \tag{1}$$

where $A_{si}$ is the area of web reinforcement in each layer in the $i^{th}$ orientation crossing the strut, $b_s$ is the thickness of the strut (170 mm), $s_i$ is the spacing of the web reinforcement in the $i^{th}$ orientation, and $\alpha_i$ is the angle between the axis of the strut and the bars in the $i^{th}$ orientation. In the present beams, since the web reinforcement consists of only vertical stirrups, $A_{si}/b_s s_i = \rho_V$ and $\alpha_i = \alpha_s$. Therefore, $\rho_T$ can be expressed as $\rho_V \sin^2\alpha_s$, **Table 1**.

## 4. Results

The efficiency factor suggested by these authors (Sahoo 2009, Sahoo et al. 2010) for natural coarse aggregate concrete is given below.

$$\beta_s = \left(0.6 + \frac{0.05}{r_c} + 55\rho_T\right)\frac{\alpha_s}{90} \tag{2}$$

The load concentration ratio (ratio of the load bearing length at the node-strut interface and the width of the imaginary rectangle enclosing the bottle-shaped strut), $r_c$, was taken as $r_c = w_s/(h/2) = 2w_s/h$, **Table 2**.

The ultimate shear force, $V_u$, resisted by the beams, **Table 2**, was obtained from the $v_u$ values reported by Han et al. (2001).

$$V_u = v_u b_s d = v_u (170\times 270)/1000 \text{ kN} = 45.9 v_u \text{ (kN)} \tag{3}$$

The beams were reanalyzed using strut-and-tie models and the compression resisted by the diagonal struts 1-2 or 4-3 was calculated from statics as $C = V_u/\sin\alpha_s$ and the tension resisted by the tie 2-3 was calculated as $T = V_u \cot\alpha_s$. From statics, the compressive force resisted by the prismatic strut 1-4 will be equal in magnitude to $T$. The axial forces in the struts and the ties have been compiled in **Table 2**. From the $C$ values, the experimental strut efficiency factor, $\beta_{se}$, in the ACI 318-08 format, was calculated as below.

$$\beta_{se} = \frac{C\times 10^3}{0.85 w_s b_s f_c^{'}} \tag{4}$$

**Table 2:** *Test results*

| S. No. | Beam ID | $f_c'$ (MPa) | $\alpha_s$ (deg.) | $r_c$ | $\rho_T$ | $V_u$ (kN) | $C$ (kN) | $T$ (kN) | $\beta_{se}$ | $\beta_s$ (ACI) | $\beta_s$ (EC2) | $\beta_s$ (AASHTO) | $\beta_s$ [Eq.(2)] |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| I | II | III | IV | V | VI | VII | VIII | IX | X | XI | XII | XIII | XIV |
| 1 | R-1.5-N | 39.62 | 31.0 | 0.55 | 0 | 144 | 280 | 240 | 0.38 | 0.60 | 0.51 | 0.39 | 0.24 |
| 2 | R-2.0-N | 30.57 | 24.0 | 0.39 | 0 | 118 | 290 | 265 | 0.57 | 0.60 | 0.53 | 0.26 | 0.19 |
| 3 | R-3.0-N | 31.23 | 16.5 | 0.24 | 0 | 55 | 194 | 186 | 0.43 | 0.60 | 0.53 | 0.13 | 0.15 |
| 4 | R-4.0-N | 31.89 | 12.5 | 0.16 | 0 | 51 | 236 | 230 | 0.56 | 0.60 | 0.52 | 0.08 | 0.13 |
| 5 | NR-2.0-N | 32.56 | 24.0 | 0.39 | 0 | 113 | 278 | 254 | 0.51 | 0.60 | 0.52 | 0.26 | 0.19 |
| 6 | C-2.0-N | 37.43 | 24.0 | 0.39 | 0 | 118 | 290 | 265 | 0.46 | 0.60 | 0.51 | 0.26 | 0.19 |
| 7 | R-2.0-W1 | 41.86 | 24.0 | 0.39 | 0.0007 | 150 | 369 | 337 | 0.53 | 0.60 | 0.50 | 0.26 | 0.20 |
| 8 | R-2.0-W2 | 41.11 | 24.0 | 0.39 | 0.0020 | 153 | 376 | 344 | 0.55 | 0.60 | 0.50 | 0.26 | 0.22 |
| 9 | R-2.0-W5 | 31.58 | 24.0 | 0.39 | 0.0042 | 174 | 428 | 391 | 0.81 | 0.75 | 0.52 | 0.26 | 0.26 |
| 10 | R-2.0-W8 | 41.11 | 24.0 | 0.39 | 0.0069 | 174 | 428 | 391 | 0.61 | 0.75 | 0.50 | 0.26 | 0.30 |
| 11 | NR-2.0-W2 | 37.43 | 24.0 | 0.39 | 0.0020 | 142 | 349 | 319 | 0.56 | 0.60 | 0.51 | 0.26 | 0.22 |
| 12 | C-2.0-W2 | 49.83 | 24.0 | 0.39 | 0.0020 | 154 | 379 | 346 | 0.45 | 0.60 | 0.48 | 0.26 | 0.22 |

The predicted efficiency factors for the beam specimens have been calculated from the authors' model, **Eq. (2)**, and presented in **Table 2**. The EC2 efficiency factors in the ACI format have been calculated from the expression $\beta_s = 0.6(1 - f_{ck}/250)$ wherein the characteristic strength of concrete, $f_{ck}$, is taken as the specified cylinder compressive strength, $f_c'$. The AASHTO efficiency factors in the ACI format have been obtained from the expression $\beta_s = 1/0.85(0.8 + 170\varepsilon_1)$ where the principal tensile strain in concrete in the bottle-shaped strut, $\varepsilon_1$, is obtained from $\varepsilon_1 = \varepsilon_s + (\varepsilon_s + 0.002)\cot^2\alpha_s$ assuming conservatively the strain in the tie reinforcement, $\varepsilon_s$, to be the yield strain of steel used in the tie (0.002).

## 5. Discussion

The experimentally obtained values of efficiency factor of the diagonal bottle-shaped struts in the 12 beams have been compared in four different groups.

Comparison of $\beta_{se}$ of bottle-shaped struts in the three beams made of washed recycled, non-washed recycled and natural coarse aggregates at S. Nos. 2, 5 and 6 respectively, with identical sizes, shear span-to-effective depth ratios and having no web reinforcement, clearly shows that the use of recycled aggregate has resulted in no reduction in the strength of the diagonal struts. The experimental efficiency factors of the bottle-shaped diagonal struts in these beams are 0.57, 0.51 and 0.46 respectively, which indicate that recycled (washed/non-washed) coarse aggregate concrete can even outperform the natural coarse aggregate concrete in shear strength expressed in terms of strut efficiency, **Fig. 2**.

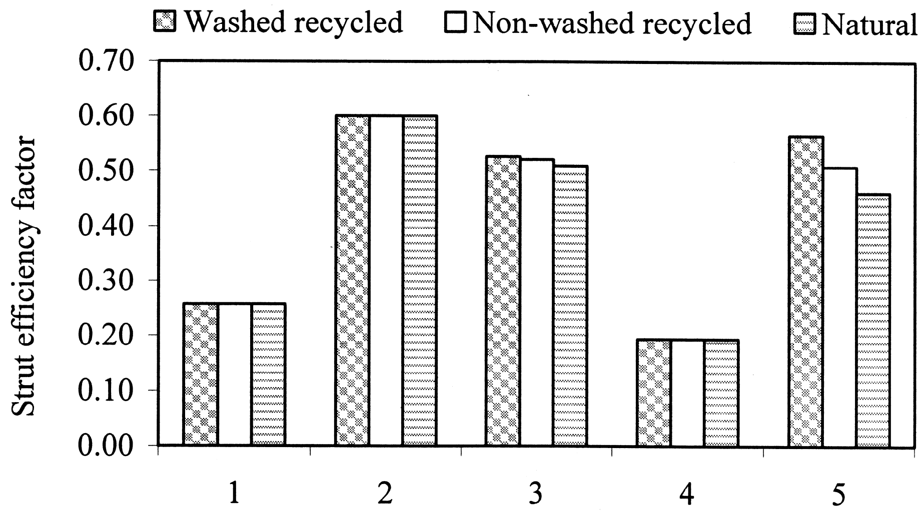



**Fig. 2** *Influence of type of aggregates on the efficiency factor of bottle-shaped struts without transverse reinforcement*
*[1. AASHTO, 2. ACI, 3. EC2, 4. Authors, 5. Experimental]*

The $\beta_{se}$ values of the three web-reinforced beams made of washed recycled, non-washed recycled and natural coarse aggregate, at S. Nos. 8, 11 and 12 respectively, with identical sizes, identical $a_v / d$ ratios and having identical effective transverse reinforcement, $\rho_T$ , of 0.002, are 0.55, 0.56 and 0.45 in that order. Thus, for RRC beams with web reinforcement also, efficiency factors of bottle-shaped struts are higher than that of natural aggregate concrete beam, **Fig. 3**. Washing of recycled aggregates does not seem to have any significant effect on the strength of the bottle-shaped struts, **Figs. 2 and 3**.

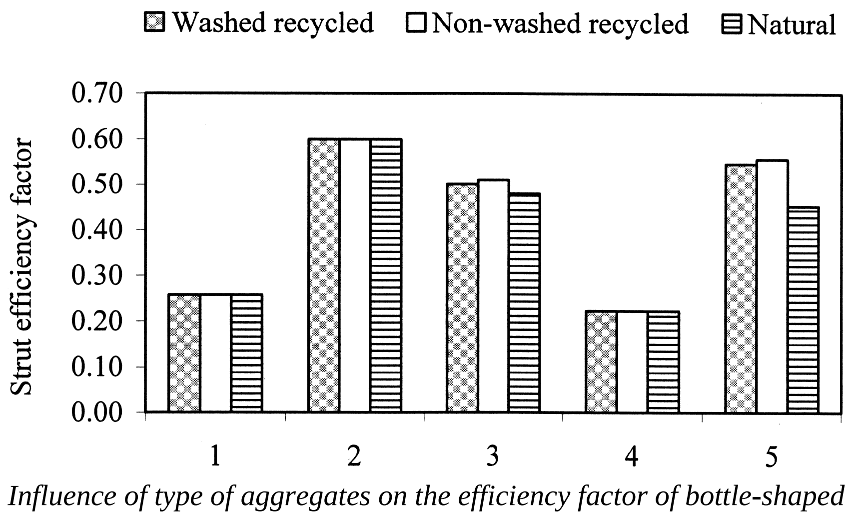



**Fig. 3** *Influence of type of aggregates on the efficiency factor of bottle-shaped struts with 0.2% transverse reinforcement*
*[1. AASHTO, 2. ACI, 3. EC2, 4. Authors, 5. Experimental]*

The $\beta_{se}$ values of the five specimens at S. Nos. 2 and 7 through 10, made of washed recycled coarse aggregates, are plotted against the effective transverse reinforcement ratio, $\rho_T$ , in **Fig. 4**. The linear trend line (broken line) of experimentally obtained efficiency factor values shows the dependence of $\beta_{se}$ on $\rho_T$ although the correlation between the experimental values and their linear trend line is weak. The trend of $\beta_s$ obtained from the authors' model (Sahoo 2009, Sahoo et al. 2010), **Eq. (2)**, is similar to the linear trend in the experimental values. The authors' model is most conservative of all and is in close agreement with the predictions of the AASHTO (2005) model.

Although the EC2 (British Standards Institution, 2004) predictions are close to the experimental values, the margin of conservatism is low. The ACI recommended $\beta_s$ values of 0.60 (for $\rho_T$ < 0.003) and 0.75 (for $\rho_T \geq$ 0.003) are found to be unconservative in all cases except R-2.0-W5. It may be noted that the low experimental efficiency factor values for bottle-shaped struts compared to the ACI recommended values are not attributable to the use of recycled aggregates, rather the ACI efficiency factor values are unconservative irrespective of the types of aggregate primarily because the ACI efficiency factors do not account for the inclination of bottle-shaped struts with adjoining tie(s) which has a strong influence on the strength of bottle-shaped diagonal struts in beams (Sahoo 2009, Sahoo et al. 2010).

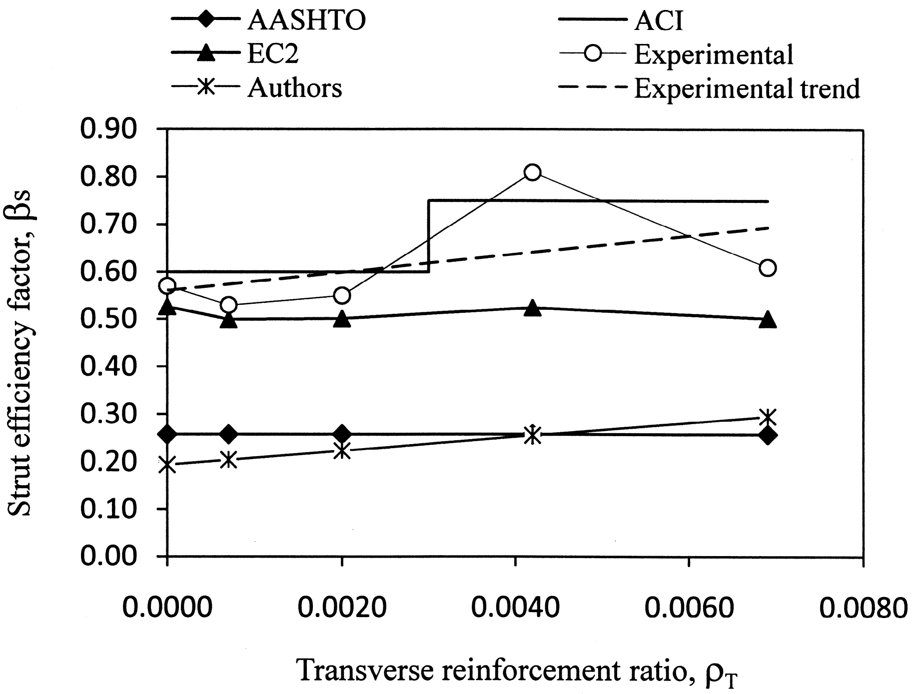



**Fig. 4** *Trend in experimental and predicted strut efficiency factors with varying transverse reinforcement ratio (washed recycled aggregate concrete)*

The four beams at S. Nos. 1 through 4 with no web reinforcement for the diagonal bottle-shaped struts had $a_v / d$ ratios of 1.5, 2, 3 and 4 respectively. In the absence of vertical shear reinforcement, it is rational to assume that the load transfer from the loading points to the adjacent supports will take place through direct strut mechanism between the loads and adjacent supports. In **Fig. 5** the experimentally obtained efficiency factors for the diagonal bottle-shaped struts have been plotted against the corresponding strut angles, $\alpha_s$. The number of data points being few and the strut angles being mostly less than 25°, the experimental efficiency factors show large scatter and no clear trend is discernible. However, the experimentally observed efficiency factor values for all the four beams made of recycled aggregates when compared with those predicted by the AASHTO and the authors' models for natural coarse aggregate concrete indicate that RCA concrete can be treated at par with natural aggregate concrete in terms of strut efficiency factors. However, the experimental values are less than the ACI recommended values in all four specimens and less than the EC2 recommendations in two of the four specimens. However, as mentioned earlier, the apparent low experimental results vis-à-vis the ACI or the EC2 recommended efficiency factors do not indicate inferior strength of RCA concrete; rather it is indicative of the inherent lack of conservatism in the ACI and the EC2 efficiency factor models which do not account for the inclination of struts with adjoining tie(s).

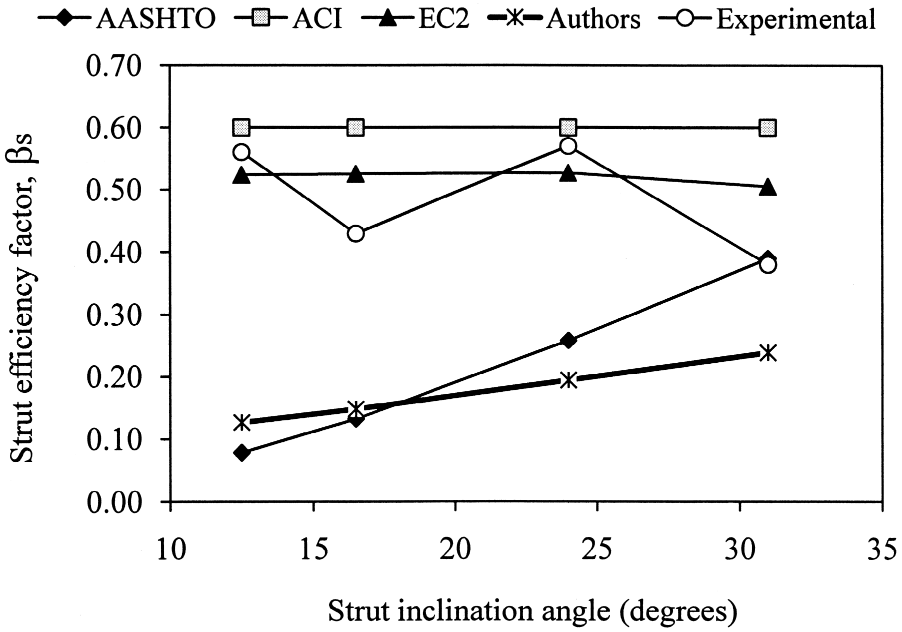



**Fig. 5** *Trends in experimental and predicted strut efficiency factors with varying strut inclination (washed recycled aggregate concrete)*

What is notable from the above discussions is that the experimentally observed efficiency factors of bottle-shaped struts in the recycled coarse aggregate concrete beams and the trends in the efficiency factor values with varying transverse reinforcement contents and strut inclinations do not suggest any loss of strut efficiency attributable to the substitution of natural aggregate in concrete with recycled coarse aggregate. Therefore, the use of recycled coarse aggregate in concrete *per se* does not call for any reduction in the values of strut efficiency factors prescribed for conventional natural coarse aggregate concrete.

## 6. Conclusions

On the basis of strut-and-tie modeling, the results of beam tests reported in the literature were reanalyzed to arrive at the following conclusions.

a) Shear strengths of reinforced concrete beams in terms of strut efficiency factors were found to be no inferior when recycled concrete coarse aggregate is used. The experimentally observed efficiency factors of bottle-shaped struts in the recycled coarse aggregate concrete beams and the trends in the efficiency factor values with varying transverse reinforcement contents and strut inclinations do not suggest any loss of strut efficiency attributable to the substitution of natural coarse aggregate in concrete with recycled coarse aggregate. Therefore, the use of recycled coarse aggregate in concrete *per se* does not call for any reduction in the values of strut efficiency factors prescribed for conventional natural coarse aggregate concrete.
b) The efficiency factors of bottle-shaped struts predicted by the authors' efficiency factor model (2009) were most conservative yet close to the AASHTO recommended values. The Eurocode 2 predicted efficiency factors were most accurate for the test specimens but the extent of conservatism was marginal. The ACI recommended strut efficiency factors were found to be unconservative when compared with the observed values.
c) Washing of recycled aggregates does not seem to have any significant effect on the strength of bottle-shaped struts. Therefore, coarse recycled concrete aggregate can be straightaway used for making fresh concrete without washing.

## References

ACI Committee 318 (2008), *Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (318R-08)*, American Concrete Institute, Farmington Hills, Michigan: 465 pp.

American Association of State Highway and Transportation Officials (2005), *AASHTO LRFD Bridge Design Specifications, Third Edition*, Washington DC: 1436 pp.

British Standards Institution (2004), *The European Standard EN 1992-1-1:2004. Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings*, BSI, London.

Choi, H.B., Yi, C.K., Cho, H.H., Kang, K.I. (2010), "Experimental study on the shear strength of recycled aggregate concrete beams", *Magazine of Concrete Research*, 62**(**2): 103-114.

Fathifazl, G., Abbas, A., Razaqpur, A.G., Isgor, O.B., Fournier, B., Foo, S. (2008), "The key to the design and production of high quality structural-grade recycled aggregate concrete", *Proceedings of the 2008 Concrete Technology Forum: Focus on Sustainable Development*, Denver, CO, USA.

Fathifazl, G., Razaqpur, A.G., Isgor, O.B., Abbas, A., Fournier, B., Foo, S. (2009), "Shear strength of reinforced recycled concrete beams without stirrups", *Magazine of Concrete Research*, 61(7): 477-490.

Fathifazl, G., Razaqpur, A.G., Isgor, O.B., Abbas, A., Fournier, B., Foo, S. (2009), "New mixture proportioning method for concrete made with coarse recycled concrete aggregate", *Journal of Materials in civil Engineering, ASCE*, 21(10): 601-611.

Fathifazl, G., Razaqpur, A.G., Isgor, O.B., Abbas, A., Fournier, B., Foo, S. (2009), "Flexural performance of steel-reinforced recycled concrete beams", *ACI Structural Journal*, 106(6): 858-867.

Fathifazl, G., Razaqpur, A.G., Isgor, O.B., Abbas, A., Fournier, B., Foo, S. (2010), "Proportioning concrete mixtures with recycled concrete aggregates", *Concrete International*: 37-43.

Han, B.C., Yun, H.D., Chung, S.Y. (2001), "Shear capacity of reinforced concrete beams made with recycled-aggregate", *American Concrete Institute SP 200-31*: 503-515.

Sahoo, D. K., Singh, B., Bhargava, P. (2009), "An appraisal of the ACI strut efficiency factors", *Magazine of Concrete Research*, 61(6): 445-456.

Sahoo, D.K. (2009), *An investigation of the strength of bottle-shaped struts*, Ph.D. thesis, Indian Institute of Technology Roorkee, Roorkee: 324 pp.

Sahoo, D.K., Singh, B., Bhargava, P. (2010), "Strut efficiency factors for structural concrete made from coarse recycled concrete aggregates", *Journal of Structural Engineering*, 37(1): 49-54.

# SUSTAINABLE ROAD CONSTRUCTION WITH COMPRESSED STABILIZED EARTH

K. P. Arandara[1], C. Jayasinghe[2], M. T. R. Jayasinghe[3]

[1] MOS Executive (Former Project Engineer), MAS Intimates Thurulie (Pvt) Ltd, Thulhiriya, Sri Lanka.
[1]E-mail: kokilaar@masholdings.com
[1]Telephone: +94-114-768600 (Ext: 8647); Fax: +94-114-768666

[2] Professor, Department of Civil Engineering, University of Moratuwa, Moratuwa, Sri Lanka.
[2]E-mail: chintha@civil.mrt.ac.lk
[2]Telephone: +94-11-2650567-8 (Ext.2111); Fax: + 94-11-2651216

[3] Professor, Department of Civil Engineering, University of Moratuwa, Moratuwa, Sri Lanka.
[2]E-mail: thishan@civil.mrt.ac.lk
[2]Telephone: +94-11-2650567-8 (Ext.2006); Fax: + 94-11-2651216

**Abstract:** Heat island effect has been given significant recognition in LEED certification process. The idea is to reduce the heat island effect to minimize impacts on micro climates and human and wild life habitats. In this context, it is useful to have roads with Solar Reflectance Index of more than 29% and also less than 50% impervious. Cement stabilized earth has the potential to satisfy this requirement as an alternative to asphalt paved roads. This paper describes some field experience gained while constructing a cement stabilized earth road with adequate strength and durability.

## 1. Introduction

The use of dark non – reflective surfaces for parking areas, walkways and access roads contributes to the heat island effect by absorbing sun's warmth, which then radiates into the surroundings. Because of heat island effect, ambient temperatures in urban areas could be artificially elevated by about 1.5$^{o}$C to 5$^{o}$C compared to suburban and undeveloped areas. In this context, under sustainable sites credit 7.1of LEED, emphasis is placed on reducing the heat island effect that can be caused by hard cape materials. It is recommended to have such strategies for at least 50% of the site hardscape [1]. Therefore, development of an alternative material that can be successfully used for car parking areas, access roads and walkways will be extremely useful.

## 2. Soil stabilization with cement

Laterite soil available in tropical climatic condition is an ideal material for stabilization with cement. Generally, 7 -14 % cement by volume is recommended [2]. The thickness of the base recommended is 100 – 150 mm. The soils with less that 35% clay and silt content could give good results.
The important characteristic of stabilization with cement is that the relatively high strengths which may be obtained with dry compacted soils are retained by the mixtures when they become wet. When the unconfined compressive strength of cement stabilized soil is more than 1.5 N/mm$^2$, the corresponding CBR value will be 100% or more [3]. A compressive strength of 1.7 N/mm$^2$ is recommended in Kadyali and Lal [4]. Since the strength characteristics and performance has been found to be adequate with many previous trials, it was decided to have a detailed study addressing various construction related issues. Such experience would allow the wider use of this technique for projects intended to achieve a greater degree of sustainability.

## 3. The objective and methodology

The main objective of this field trial was to achieve a road and also with acceptable performance with very good durability characteristics. In order to achieve this objective, the field trials were carried out with different cement percentage and addition of chips. The mixes that gave good constructability, strength and durability have been recommended for future applications.

## 4. The thickness of road base material

Since the road was intended for heavy traffic loads such as containers, a suitable base thickness had to be selected. According to the Road Note 29 [5], it is possible to allow 1 x $10^6$ cumulative axles when the thickness of the base is more than 150 mm. Hence, a decision was taken to maintain a minimum thickness of 150 mm when the base and the wearing course thickness were combined.

## 5. The soil cement mixes

In a cement stabilized road, there are two requirements to be fulfilled. One is the strength. The other is the durability. In order to address these two issues in a cost effective manner, a decision was taken to complete the road in two main layers. The lower layer laid on well compacted soil was intended to provide sufficient strength. The upper layer was intended to provide a good wearing surface while ensuring a light colour surface and also contribute to the strength of the base.

### *5.1. The soil composition*

It was found that the clay content of the soil that was available at the site due to earthwork associated with landscaping and road work has clay and silt content of about 40%. Therefore, a soil with lower clay and silt content had to be brought to adjust the composition. The resulting mix had a clay and silt content in between 30 - 35 %. In order to facilitate proper mixing, soil was sieved using a 25 mm mesh and then kept covered as shown in Figure 1 to control the moisture content.

Figure 1: Selected soil is stored with a cover to ensure proper moisture content

### *5.2. The cement content*

For the selection of cement content, reliance was placed on the results obtained with laterite soils stabilized with cement for rammed earth concentration. A compressive strength of about 2.5 N/mm$^2$ could be obtained with 10% cement with a wet strength of about 1.5 N/mm$^2$ [6]. The cement and soil was mixed with a drum type concrete mixer as shown in Figure 2.

Figure 2: Cement and soil mixed with desired ratio

### *5.3. The compaction*

In cement stabilized soil, the level of compaction is very important for the strength characteristics. A compaction ratio in excess of 1.7 would be ideal. A minimum compaction ratio of 1.65 has been specified by Jayasinghe [7]. It was found that a 5 tonne roller shown in Figure 3 could provide a compaction ratio of excess of 1.7 after four passes. The thickness of the base was maintained between 75 – 100 mm after compaction.

Figure 3: Five Ton roller was used for compaction

### *5.4. The wearing course*

For this road, the durability was very important. For this, the wearing course was completed with a cement, soil and aggregate mix. The aggregate was 6 – 8 mm ships. The addition of chips was useful in improving the resistance to driving rain while ensuring adequate abrasion resistance to moving vehicles. After few trials, it was found that a mix of 1:4:4 cement, soil and aggregate could ensure a surface that would be free of shrinkage cracks. A thickness of 100 -150 mm was maintained for this wearing course while ensuring the required camber. The finished wearing surface is shown in Figure 4. Due to formation of a light colour dust, it could maintain a light colour thus fulfilling the requirement as Solar Reflective Index.

Figure 4: The light colour road surface without significant cracks

### *5.5. The drainage*

In a road of this nature, it is important to ensure good drainage. This was achieved by using precast concrete half round drains laid on part of the road construction as shown in Figure 5.

Figure 5: Arrangement used for drains

## 6. The long term performance

The road has retained its light colour during its usage. The reason is that the chips would become exposed as the road is used and hence the colour of the road would be light thus facilitating a high solar Reflectance Index as the road is used. Since a small degree of wearing would occur with time, the road was able to maintain its light color without much maintenance. Figures 6 to 8 indicate the present situation of the road and parking areas after nearly three years of operation.

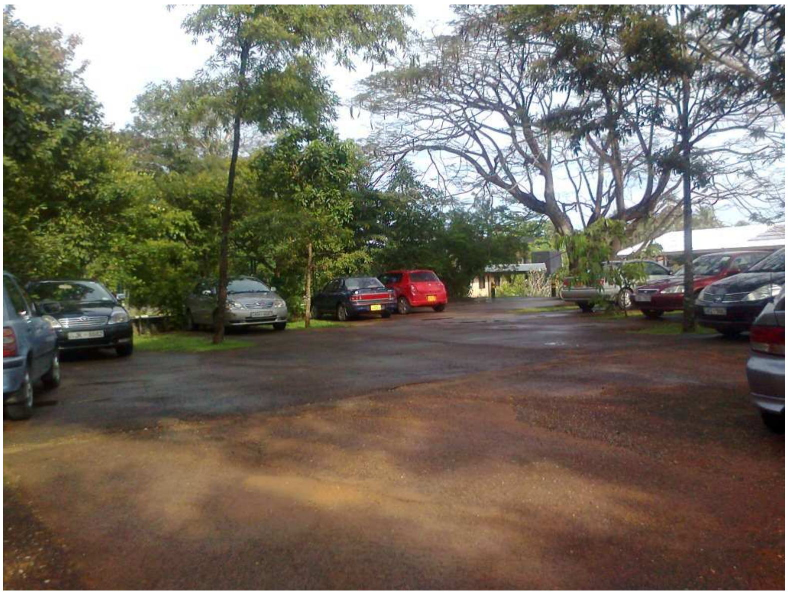


Figure 6: The parking area provided with many trees

Figure 7: One of the main access road


Figure 8: Gradual erosion that has exposed aggregates mixed in wearing course

## 7. Conclusion

The minimization of the effect of heat island is an important concept when planning built environments. The solution proposed in white concrete or gray concrete hard tops. The alternative proposed is cement stabilized soil bases finished with light colour aggregates. When combined with strategically planned vegetation, such roads can provide a good solution for minimization of heat island effect in a cost effective and environmentally friendly manner. Some of the key advantages would be the use of soil that is available at the site thus promoting the minimization of waste material generation while reducing the need for new material

## Reference

[1] LEED reference guide for “Green Building Design and Construction” U.S. Green Building Council, 2009, Edition

[2] Wright, P.H., Dixon, K.K., “Highway Engineering”, John Wiley and Sons, USA, 667 p

[3] Odier, L., Millard, R.S., Santos, P.D., Mehra, S.R., Low cost roads, Butterworth, London, 153 p

[4] Kadyaki, L. R., Lal, N. B., Principles and Practices of Highway Engineering, Khanna Publications, India, 845 p

[5] Road Research Laboratory, A guide to the structural design of flexible and rigid parameters for new roads, Development of Scientific and Industrial Research, Road Note No 29, H.M. Stationary office, London, 1965.

[6] Jayasinhe C., Kamaladasa N., Compressive strength characteristics of cement stabilized rammed earth walls, Construction and Building Materials, Vol. 21, Issue 11, Nov. 2007, pp 1971 -1976.

[7]Jayasinghe, C., Alternative building material and methods for Sri Lanka, Ph.D. Thesis, Department of Civil Engineering, University of Moratuwa, Sri Lanka.

## Acknowledgements

The authors wish to express their sincere thanks to MAS Intimates Thurulie (Pvt) Ltd for all the support given for this study as the client of the project. Also we wish to thank to MAGA Engineering (Pte) Ltd and BVN Construction (Pvt) Ltd assisted this detailed experimental programme with much enthusiasm. And also the support given to success the research by all the officers of each company is highly appreciated.

## About the Authors

**K. P. ARANDARA,** B.Sc. Hons. Moratuwa., M.Sc. Moratuwa, AMIE(SL) was the client's Project Engineer for the Thurulie Green Factory project and is currently working as MOS Executive in MAS Intimates Thurulie (Pvt) Ltd. in Thulhiriya.

**C. JAYASINGHE,** B.Sc.(Eng), M.Eng.(Highway & Traffic Engineering), Ph.D., C.Eng., MIE(SL) is a Senior Lecturer at the Department of Civil Engineering, University of Moratuwa.

**M. T. R. JAYASINGHE,** B.Sc. Hons. Moratuwa., Ph.D. Cambridge, MIE (SL), C.Eng. is a Professor at the Department of Civil Engineering, University of Moratuwa. His research interests are in the areas of tall buildings, energy efficiency, sustainable development and disaster resistant structures.

# USING SHREDDED PLASTIC SHOPPING BAGS WASTES IN SOIL IMPROVEMENT

Kalumba, D.

Snape Building, Department of Civil Engineering, University of Cape Town, 7701 Rondebosch, South Africa.
Telephone +27 21 650 2590, Fax +27 21 689 7471, Email denis.kalumba@uct.ac.za

**Abstract**
Colossal tonnages of waste are produced each year worldwide, with a considerable amount being in the form of plastic (polyethylene) grocery bags. Most of this is non-degradable and destined for landfill. This study investigated the potential of utilising this type of waste to reinforce soils paving way for its use in civil engineering projects such as in road bases, embankments and structure foundations. A comprehensive test programme was undertaken including direct shear tests on two selected sandy soils. Plastic strips were used as reinforcement inclusions at concentrations of up to 0.3% by weight. The effect of the dimensions of the strip was investigated by varying the length of the strips from 15 to 45 mm and the width from 6 to 18 mm. Shear strength parameters were obtained for each composite material from which analyses were done to identify the extent of the soil improvement. The laboratory experiments favourably suggest that inclusion of these strips in sandy soils would be an effective soil reinforcement method.

***Keywords***
*Environment, Engineering, Geotechnical, Plastic, Soil Reinforcement, Waste.*

## 1. Introduction

Plastic (polyethylene) shopping bags have been used extensively since their first introduction over fifty years ago. These bags are reliable and easily adaptable, very light in weight but strong, highly convenient, inexpensive and readily available everywhere. For these reasons, consumers and businesses alike are depending on them to deliver goods between places. However, their major weakness is that they severely pollute the environment: littering parks and roadsides, clogging sewers and filling landfills (Kalumba and Petersen, 2010). With landfills rapidly filling up, finding a solution to this non-degradable polyethylene waste is key to a sustainable environment. This research, therefore, was undertaken to investigate the potential increase in soil shearing strength when it is reinforced with plastic strips. It was anticipated that positive laboratory results could trigger field applications, success of which would permit reduction of the plastic grocery bag wastes destined to landfills bringing along environmental and economic benefits.

## 2. Material and Methods

### *2.1 Plastic Shopping Bag*

The materials used in the study were plastic grocery shopping bags sourced from the local Woolworths Supermarket (Woolworths House, 93 Longmarket Street, Cape Town, South Africa). The bags were medium sized and manufactured, from high-density polyethylene by Transpaco, Sixth Street, Wynberg, Johannesburg, South Africa. The material density was measured to average 798 kg/m$^3$, with the tensile strength ranging between 14 and 20 MPa.

### *2.2 Soil Material*

The two soils used were Cape Flats and Klipheuwel sands which were obtained locally. Both are clean, consistent and easily controllable, making it possible to form identical samples if prepared the same way. Cape Flats sand is uniformly graded medium dense, light grey, clean quartz sand, whereas Klipheuwel sand is uniformly graded medium dense, reddish brown sand. Figure 1 shows the soils grading while Table 1 summarises their physical properties determined according to BS 1377: 1990.

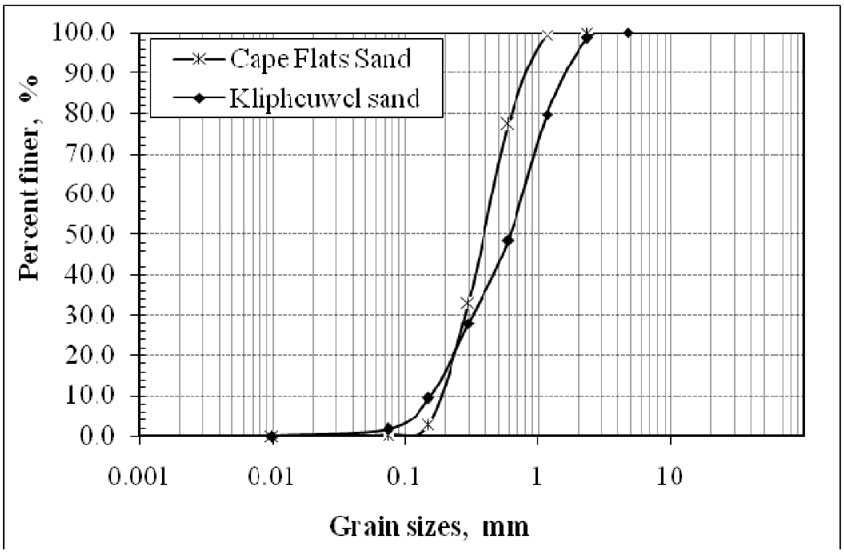



**Figure 1.** *Soils Grading Graph*

**Table 1.** *Soils physical properties*

| Characteristics | Cape Flats Sand | Klipheuwel sands |
|---|---|---|
| Specific gravity | 2.66 | 2.63 |
| Optimum moisture content (%) | 15 | 6.5 |
| Maximum dry density (Mg/m$^3$) | 1.710 | 1.985 |
| Average densest dry density (Mg/m$^3$) | 1.720 | 1.824 |
| Average loose dry density (Mg/m$^3$) | 1.538 | 1.587 |
| Angle of internal friction (°) | 33. 9 | 39.0 |
| Residual shear strength (°) | 28.0 | 35.9 |
| Apparent cohesion (kN/m$^2$) | 9.4 | 8.2 |

## 3. Procedure

The plastic bag material were cut into strips of 5 distinct rectangular dimensions using a guillotine allow for an investigation of the effect of reinforcement length, width and concentration to the soil strength characteristics of the composite material. The 5 reinforcement strips dimensions used were; 6 x 15 mm, 6 x 30 mm, 6 x 45 mm, 12 x 15 mm and 18 x 15 mm. The elements dimensions chosen were in the range of 0.06 – 0.45 of the shear box dimensions so as to control entanglement between the reinforcing strips. Strips entanglement would limit soil particles forming surface attachments with the reinforcement resulting into lower shearing strengths of the composite material (Kalumba and Petersen, 2010). For each testing regime, a predetermined weight of plastic strips of known dimensions was added and mixed randomly with a known mass of dry soil to form a composite material with the required reinforcement concentration. Dry soil was used in all experiments order to eliminate any effect of moisture fluctuations. Three reinforcement concentrations of 0.1, 0.2 and 0.3% by mass were adopted. The low concentration values were based on the fact that although the strips were light in weight, they occupied large volumes. Besides, at those respective concentrations, it was easier to ensure consistency and even distribution of reinforcing elements within the soil sample without entanglement between strips.

With the test samples thoroughly mixed, the composite specimens were poured into the 100mm square direct shear box in three layers compacting each to the required density. Three normal stresses of 25, 50 and 100 kPa and a shear speed of 1.2 mm/min until a residual state were applied. The peak shear stress from each composite sample was then recorded for the respective applied normal stresses. These values were plotted against normal stress to determine the friction angles for particular composite material tested.

## 4. Test Results and Discussion

In Figures 2 to 4, the relationship between the friction angle and the studied reinforcement parameters are plotted. It is clear from Figure 2 that addition of high-density polyethylene strips of any length enhances the peak friction angle for both Cape Flats and Klipheuwel sands. Studying the effect of lengthening the reinforcement shows a non-linear relationship with each sandy soil exhibiting a unique characteristic response (Figure 2a). In the Cape Flats, the soil shear strength improved with increased strip length over specified lengths of 15 and 45 mm, dropping when strips 30 mm long were used. It is likely that this point could have been an anomaly in that test. The results also displayed that when the fibre length was increased in the Klipheuwel sand, the soil friction angle also became better peaking with the 15 mm long strip elements (shortest strips tested). Therefore, it is likely that there are limiting plastic strip lengths in the soil composites where the reinforcements intersect potential failure surfaces most effectively in the soil mass. Beyond this limit any reinforcement lengthening results in decreases in the shear strength.

In Figure 2b, the effect of reinforcement width on composite peak friction angle is shown. It is demonstrated that the inclusion of plastic strips significantly raises the peak shear strength. Further testing revealed that beyond a specific reinforcement width of 6 mm (narrowest strip tested), the strength decreased. It is possible that more testing could have revealed that the greatest strength gain occurs for strips narrower than 6mm. These results suggest that the gains in strength decrease as the reinforcement strips widen. The plastic material used in this study being smooth, it is likely that when longer and wider strips are used, they overlap each other more during shearing thereby reducing the soil/reinforcement interaction. As expected, there would be less friction generated between strips than between soil and embedded strips. It was again observed that Klipheuwel composites generally had higher peak friction angles.

The results of effect of the quantity of strips in the soil/reinforcement system are shown in Figure 3. It is observed that there was an increment from the initial friction angle of 33.9° in the Cape Flats sand, to 41.7° for the 0.1% concentration composite, after which there was an almost linear increase in the friction angle with concentration. The pattern in Klipheuwel composites was however different. In Klipheuwel, the reinforcement concentration considerably increased the peak friction angle initially. However; further testing revealed that beyond the reinforcement concentration of 0.15 the strength decreased. It can be concluded that for various soils, with different grading, independent tests would need to be conducted to determine the individual soil strength enhancement performances. The laboratory experiments also favourably suggest that inclusion of polyethylene strips in sandy soils would be an effective soil reinforcement method.

Analysis of the results, in Figure 4 shows that there is a general increase in friction angle at lower aspect ratios in both types of sands. However, the friction angles peak at the aspect ratio of 0.4, beyond which the gains in shear strength (tan phi) start to reduce.

Assessment of the laboratory test results shows that inclusion of polyethylene strips in both sands, results in a definite increase in shear strength (tan phi). Strains in the soil mass generated strains in the strips, which in turn, generated tensile loads in the strips. These tensile loads acted to restrict soil movements and thus impart additional shear strength. This resulted in the composite soil/strip system having significantly greater strength than the soil mass alone. Polyethylene as a material has low frictional properties and therefore interacts with the soil particles in a unique way. Instead of particles adhering to the polyethylene surface, the particles ‘punched’ and moulded around the ‘soft’ strips. As the vertical load was applied and increased this ‘punching’ became amplified, and due to the fact that this material has good elongation characteristics it could withstand the high strains.

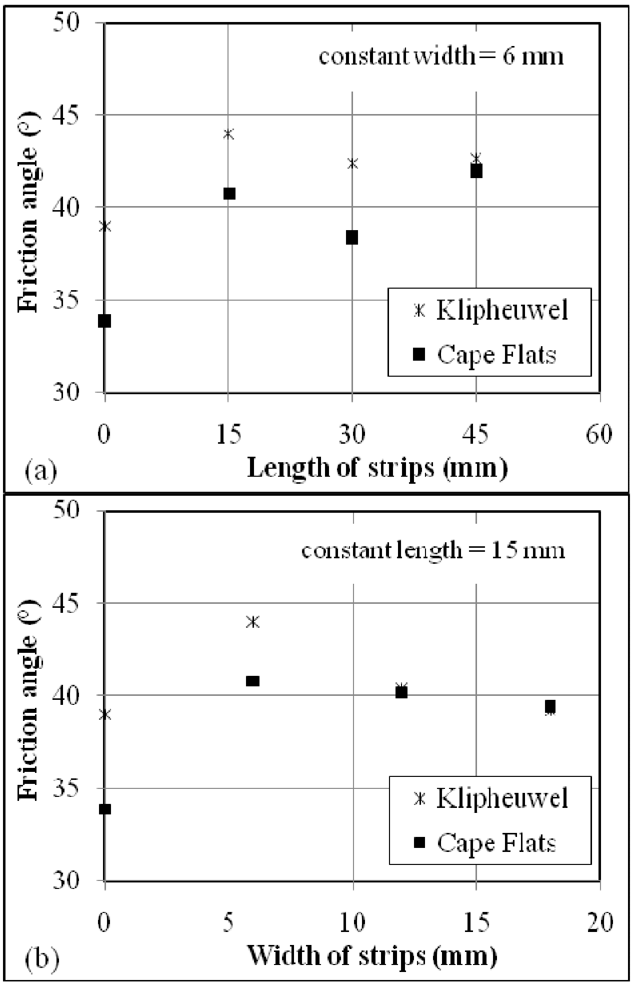



**Figure 2.** *The friction angle versus reinforcement (a) width, and (b) width*

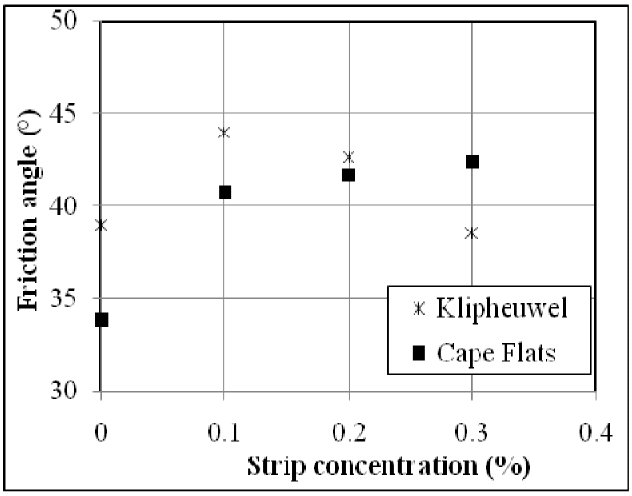



**Figure 3.** *The friction angle versus reinforcement concentration*

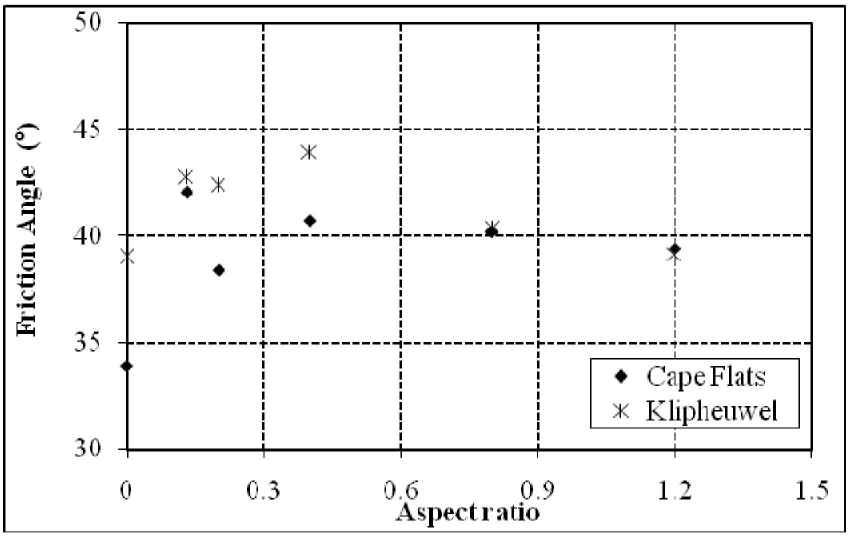



**Figure 4.** *The friction angle versus aspect ratio (after Petersen, 2009)*

## 5. Conclusions

It can be concluded from this work that:

Inclusion of plastics sourced from shopping grocery bags can result to an increase of more than 20% in angle of internal friction. Consequently, this can result in significant enhancement in shear strength and soil bearing capacity.

The addition of the strips improved the angle of internal friction but lengthening and widening the strips reduced the improvement. The optimum reinforcement aspect ratio was 0.4.

The results are, however, specific to the particular type of plastic shopping bag used and the soil with which the reinforcement was mixed. In order to properly document behaviour, testing in a range of soil types with inclusion of plastics from different sources, thickness and roughness is recommended.

## References

BS1377 – 1990, British Standard Methods of Test for Soils for Civil Engineering Purposes

Petersen, M.A. (2009), “Utilizing consumer polyethylene bags to reinforce soil”, BSc Thesis, University of Cape Town, South Africa

Kalumba, D, Petersen, M.A. (2010), “Using shredded plastic shopping bags in soil improvement for civil engineering projects”, Proc of 20th WasteCon Conference, 4-8 October, Gauteng, South Africa

# A QUANTITATIVE APPROACH FOR MEASURING SUSTAINABILITY IN GEOTECHNICAL ENGINEERING

Aditi Misra and Dipanjan Basu*

Department of Civil and Environmental Engineering
University of Connecticut
261 Glenbrook Road, Unit 2037
Storrs, CT, USA.
*E-mail: dbasu@engr.uconn.edu
*Telephone: 1 860 486 5023

**Abstract:** Civil Engineering is the major instrument of anthropocentric development over centuries through ever expanding infrastructure, cities and facilities. Over the last two decades, a growing awareness is noted towards making such growth sustainable as well. Efforts in setting up standards in construction management are mostly directed towards high level construction and material management but geotechnical engineering that can produce the most permanent change of the land use pattern, lacks proportional attention. Literatures available in this field are found to stress more on qualitative aspects of construction management than on developing quantitative efficiency parameters. This paper studies the energy efficiency of two types of pile foundation, drilled shaft and driven reinforced concrete pile, based on available energy-centric methods like exergy and emergy and provides an aid to the practitioner in making a sustainable choice.

***Keywords***: *sustainability, emergy, exergy, drilled shaft, driven pile*

## 1. Introduction

Civil engineering processes (e.g., planning, design and construction of a road network) are both resource and fuel intensive**.** The building industry alone, during the construction stage, uses about 30-40% of the total resources used in the industrialized countries (Pulselli et al. 2003). But this intensive consumption of energy goes unnoticed mainly because of the indirect nature of the energy used in the form of materials and natural resources (e.g., water, wood and land use). Resource efficiency as a decision making metric is slowly gaining momentum in the civil engineering industry, particularly in the construction sector (Jeffris 2008). In fact, sustainable development, which is closely related to efficient resource management, is the current focus of the civil engineering industry and academia. Sustainable development is defined by the Brundtland Commission of the United Nations as 'the development that meets the needs of the present generation without compromising the ability of the future generations to meet their own needs' (Brundtland 1987). Most of the efforts in incorporating sustainability in civil engineering practices are directed towards construction management and material re-engineering (Jeffris 2008).

Geotechnical engineering is most material intensive and produces the most permanent change in the land-use pattern. Consequently, sustainability metrics must be an inherent part of geotechnical planning, design and construction processes. However, a major problem in introducing sustainability in geotechnical engineering is inadequate knowledge of the effect of the processes on the ecological balance of the area (Abreu et al. 2008). There is also an absence of a reference framework which can help in determining the best engineering solution balancing both economy and ecology. These drawbacks are compounded by the scarcity of the geo-sustainability literature in and by the fact that most of the sustainability indicators for geotechnical practices are qualitative in nature (Abreu et al. 2008). Foundation engineering is also plagued by a general reluctance in accepting any other efficiency criterion beside the traditional considerations of cost and technical efficiency (Jefferis 2008).

There is only one set of guidelines available, developed by Jefferson et al. (2007), which couple sustainability with geotechnical practices. It essentially evaluates the effect of a geotechnical construction process on four sectors of efficiency: economic, environmental, social and technical. These broad sectors are then subdivided into subsectors that are of relevance to the project.

The entire system is represented on a circle and a project is marked closer or further from the centre of the circle depending on its achievement level in that subsector. This provides a qualitative guideline at the construction stage of geotechnical projects and is modeled after the general green building codes like BREEAM or LEED. Although these guidelines serve well at the construction stage, there is little or no help available in the decision-making process during the planning and design stages of geotechnical engineering. Considerable energy efficiency can be ensured at the design stage itself with the help of a quantitative framework that considers the energy equivalence of the material and natural resources used in the process.

In this paper, a quantitative sustainability framework is proposed for use with geotechnical engineering, particularly with foundation design. The quantitative framework is based on thermodynamic principles, and two different approaches based on emergy and exergy are used (Odum 1996, Scuibba&Wall 2010). Emergy accounting is an ecocentric method that considers all the work done by nature and man together to make a product. Exergy is the entropy-free energy of a material that can do useful work. The procedure follows a “cradle to grave” approach (McDonough & Braungart,2002) in which the reuse of the materials after decommissioning of the project is not considered. This framework is applied to pile foundations, particularly drilled shafts and driven piles, in order to determine the most environmentally-friendly solution for a few particular sites.

## 2. Thermodynamic Calculations for Drilled Shafts and Driven Piles

The laws of thermodynamics have been used to develop different sustainability parameters for applications in different processes, e.g., ecological and chemical processes. Both the concepts of emergy and exergy take into account the important fact that, although energy is conserved in any process, its quality is not (Odum 1996, Dincer 2007). This is a particular consequence of the second law of thermodynamics according to which it is impossible to have 100% efficiency for any cyclic process, that is, the generation of a product is always accompanied by an irrecoverable loss of useful energy to its environment. In the following sections a brief overview and calculation methods are discussed for both emergy and exergy and the discussions are followed by applying the particular method for the case study. Since all processes are interlinked it is important to decide a system window consisting of the parts of the process that are of importance. It is necessary for both the methods to have a well defined system boundary across which mass & energy flows. In our case, that boundary is decided to be the physical limit of the construction area. This automatically excludes any environmental effect of transportation to or from the site which is probably not a justified assumption given that distance of construction site from manufacturing unit and landfill site are important considerations in calculating fuel use and emissions from construction related work.

### *2.1 Emergy Based Calculations*

Emergy, spelled with an ‘m’, measures both the work of nature and that of human beings in generating services and products. While energy is a measure of the amount of work that can be obtained from a product, emergy is the available energy already used up to make that product (Odum1996). Products for economic use are made from both renewable and non renewable natural resources and services. The resources can be local to the production process or brought in from outside. Emergy of all the inputs, resources and services are added up to arrive at the emergy of the product. However, the quality of energy content of one resource is not the same as that of another and they have different work capacities. Hence, for the purpose of comparison, it is necessary to have a common basis to which all other forms can be converted. Commonly, solar energy is used for the purpose. The available solar energy used up directly or indirectly to make a service or product is defined as solar emergy and its unit is solar emjoules (sej). Different energy forms are converted to equivalent solar emergy by a transformation coefficient, also known as transformity, which is defined as the solar emergy required directly or indirectly to produce 1J of a product or service. The solar emergy $U$ of a product coming from a process is given by:

$$U = \sum_i \left[ (Tr)_i \, E_i \right] \qquad i = 1,\, 2,\, \ldots,\, n \tag{1}$$

where $E_i$ is the available energy content of the $i^{th}$ independent input material/energy flow to the process and $(Tr)_i$ is the solar transformity of the $i^{th}$ input material/energy flow and $n$ is the total number of material/energy flows.

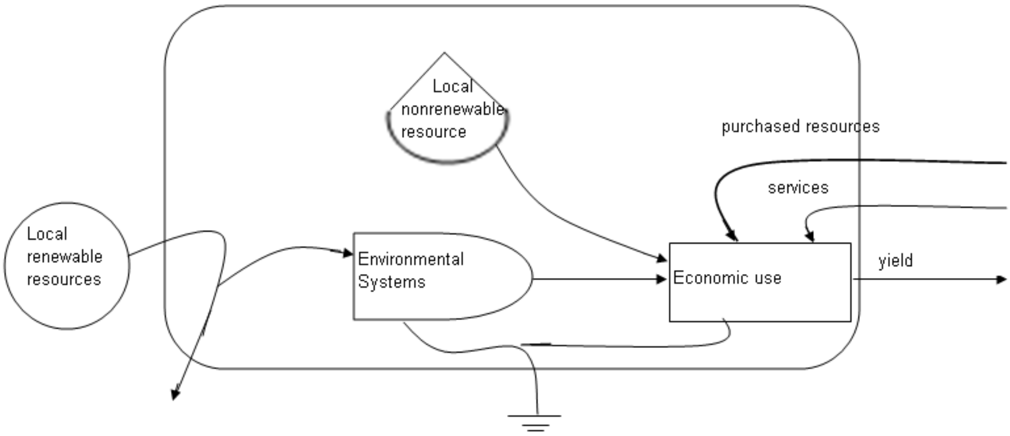



Figure 1. *Diagram illustrating the production process by emergy flow diagram (from Ulgiati and Brown 1997).*

### *2.2 Emergy calculation for driven and drilled piles*:

A comparative analysis of the requirement of the natural resources and materials for two common type of pile foundations, drilled shafts and, reinforced concrete driven piles are made in this paper. This is a hypothetical problem of a single pile required to carry a structural load of 10000KN. The pile is embedded in a homogeneous sand deposit with a relative density of 60%. The water table is assumed to be at the ground surface and the pile length is 12m The piles are embedded in a fully submerged sandy soil with relative density of 60%. From the above data, the diameter of the driven pile is calculated to be 1.5m and that of the drilled shaft to be 2.5m. The corresponding volumes of land use, concrete and steel are also calculated for each case. The following tables detail the emergy calculation for this case study.

TABLE 1: *EMERGY Calculation for Drilled Shaft Foundation*

| **Item** | **Specification** | **Volume m3** | **Density Kg/m3** | **Raw Data** | **Unit** | **Transformity sej/unit** | **Reference:** | **Emergy Sej** |
|---|---|---|---|---|---|---|---|---|
| Solar Irradiation | NOTE 1 | | | | | | | |
| Land Use NOTE 2 | Soil Erosion (Soil organic matter =3%) | 9.62 | $20.31\times10^4$ | $19.53\times10^5 \times0.03 = 0.59\times10^5$ | Kg | $5.4 \times 4186 \times1.24\times10^5 = 2.8\times10^9$ | Odum (2000) | $1.65\times10^{14}$ |
| | Soil Excavation (Soil Organic matter =1%) | 105.83 | $20.31\times10^4$ | $214.9\times10^5 \times 0.01 =2.14\times10^5$ | Kg | $5.4\times4186\times1.24\times10^5 = 2.8\times10^9$ | | $5.99\times10^{14}$ |
| Concrete NOTE 3 | Pile | 58.9 | 2500 | $1.47\times10^5$ | Kg | $1.54\times10^{12}$ | Brown & Buranakaran (2003) | $2.26\times10^{17}$ |
| Steel | As pile reinforcement | 35.3 | 7850 | $2.77\times10^5$ | Kg | $4.13\times10^{12}$ | Brown & Buranakaran (2003) | $11.44\times10^{17}$ |
| | As in Construction machinery | NOTE 4 | | | | | | |
| Fuel | For Electricity generator | NOTE 5 | | | | | | |
| | For machinery operation | | | | | | | |
| **Total Emergy driving the process of drilled shaft construction is $13.7\times10^{17}$ sej (based only on soil, concrete and steel used)** | | | | | | | | |

TABLE 2: *EMERGY Calculation for Driven Pile*

| Item | Specification | Volume m3 | Density Kg/m3 | Raw Data | Unit | Transformity sej/unit | Reference: | Emergy Sej |
|---|---|---|---|---|---|---|---|---|
| Solar Irradiation | NOTE 1 | | | | | | | |
| Land Use NOTE 2 | Soil Erosion (Soil organic matter =3%) | 4.9 | $20.31\times10^4$ | $9.97\times10^5 \times0.03 = 0.3\times10^5$ | Kg | $5.4 \times 4186 \times 1.24\times10^5 = 2.8\times10^9$ | Odum(1996) | $0.814\times10^{14}$ |
| | Soil Excavation (Soil Organic matter =1%) | 0 | $20.31\times10^4$ | 0 | Kg | $5.4 \times 4186 \times 1.24\times10^5 = 2.8\times10^9$ | | 0 |
| Concrete NOTE 3 | Pile | 21.2 | 2500 | $1.47\times10^5$ | Kg | $1.54\times10^{12}$ | Brown & Buranakaran (2003) | $0.816\times10^{17}$ |
| Steel | As pile reinforcement | 12.72 | 7850 | $0.9985\times10^5$ | Kg | $4.13\times10^{12}$ | Brown & Buranakaran (2003) | $4.12\times10^{17}$ |
| | As in Construction machinery | NOTE 4 | | | | | | |
| Fuel | For Electricity generator | NOTE 5 | | | | | | |
| | For machinery operation | | | | | | | |
| Water | Water expelled during compaction | 21.2 | 1000 | $0.21\times10^5$ | Kg | $1.95\times10^9$ | Pulselli et al. (2007) | $0.14\times10^{14}$ |
| **Total Emergy driving the process of driven pile construction is $4.95\times10^{17}$ sej (based only on soil, concrete and steel used)** | | | | | | | | |

***Notes to the Tables***:

NOTE 1: Solar irradiance is calculated as the solar energy received by the construction area during the construction period (Pulselli et al., 2007) In this hypothetical case study the difference is negligible as we consider single pile areas but in reality the number of piles required in a foundation depends on pile capacity. As can be deduced from the calculation, driven piles have a higher capacity than drilled piles and hence the foundation area required might substantially differ in large scale construction projects.

NOTE 2: It is assumed that top 1m soil suffers erosion due to any construction activity and its organic matter content is about 3% decreasing to 1% at depths greater than that (Pulselli et al., 2007). For driven pile, only the top 1m is affected by construction while for drilled shaft the entire volume of soil mass needs to be excavated to put the shaft in place. Removal of soil mass removes with it soil nutrients that are essential for thriving of bacterial colonies. Not much study is available correlating these two factors but commonly it can be concluded that the lesser a system is forced to deviate from its original state, the more sustainable it is.

NOTE 3: Cement industry accounts for 30-40% of $CO_2$ emissions to the environment. As construction debris also, cement is mainly responsible for clogging drainage systems in the locality. Cement particles suspended in air is a predominant health hazard. It is only evident that a foundation option that uses lesser quantity of cement is more acceptable than one that uses more of it.

NOTE 4: Steel used in machinery is calculated as percentage present by weight in the machinery. Multiplied by transformity it gives the emergy in that account (Pulselli et al., 2007).The inclusion of steel in machinery as an input is not always obvious particularly when system boundary does not include the machine manufacturing unit. This is typical of emergy analysis which provides a holistic approach including every form of energy that is required for the process to take place. Since machinery is an integral part of construction process, it is assumed that energy that went into making the machinery also goes to the making of the pile.

NOTE 5: Fuel efficiency is important both for environmental and economic resource. Fuel use varies widely depending on machine type and process. Spaulding et al.(2008) has shown that the $CO_2$ emission increases 1450 times only on the basis of fuel usage if a traditional ground improvement technique is used instead of dynamic compaction .
For this particular case study, literature from DELMAG about equipments show that the fuel usage of excavators is 25-30l/hr while that for diesel hammer for pile driving(of length 11-60m) is only about 7.5l/hr.

***2.3 Remarks***:

The calculations made so far leads us to conclude that driven pile is a more sustainable choice than drilled shafts. However, use of drilled pile is limited by the site condition- dense or rocky strata may be uneconomical and even technically unfeasible for driven pile. In such cases, alternative ways to make the construction process sustainable can be thought of like using bio diesel instead of fossil fuel, replacing non renewable materials by bio-engineered materials and such other newer approaches to make the built environment eco-friendly.

Since foundation is an almost permanent structure, it is considered as storage of the emergy inflow in the process of its construction. Emergy in the output flow should be calculated at the stage of dismantling of the structure. This study gives the partial picture and can be used as a decision making tool when the energy input is the major concern without any consideration for reuse of the materials. Thus, for a cradle to cradle approach, the emergy of reusable/recyclable materials obtained after decommissioning should also be considered to arrive at the net emergy used in the process.

## 3.0 EXERGY and EXERGY Analysis Applied to Drilled Shaft and Driven Pile

### *3.1 EXERGY*

Exergy is defined as the amount of work that a system can perform when it is brought into thermodynamic equilibrium with its environment. In terms of energy, exergy is the available or entropy free energy of a system. However, unlike energy, exergy is not conserved and depends on the state of the reference environment.

Exergy of a homogeneous system at a defined state 1 is given by:
ex1=ex1,t+ex1,c+ex1,k+ex1,p+ex1,n+ ….

Where ex1,t ex1,c ex1,k ex1,p and ex1,n are the thermodynamic, chemical, kinetic, potential and nuclear exergy components of the total exergy . (Sciubba &Wall 2010)

Parameters have been developed over years to quantify efficiency of processes on the basis of exergy Dewulf et al.(2000) defined 'Renewability Parameter' as (exergy consumption of renewable resources)/(total exergy consumption) and 'Efficiency parameter' as (exergy value of the useful products)/(exergy consumed in the process + exergy required for the abatement of the harmful emissions). Lems et al. (2003) defined exergy efficiency as the useful exergy flow out/ exergy flow into the process (Hau et al. 2004)

Mathematically, exergy is commonly represented per unit mass as

$$\mathbf{B = \Delta[H - T_0S + \Sigma x_i\mu_i + v^2/2 + gz]}$$

where H = enthalpy
$T_0$ = temperature of the reference environment
S=entropy
$x_i$=mole fraction of component i
$\mu_i$ =chemical exergy of the component i
v=velocity
z=height

And Δ is the difference w.r.t temperature, pressure and the composition between current and reference state

As is evident from the definition, exergy analysis requires the definition of a reference state that should remain constant throughout the calculation. The environmental reference state commonly used is 1atm and 25C and composition of the air, oceans, and a selected thickness of the earth's crust. Standard chemical exergy values are available from literature (Szargut et al.1988).

### *3.2 Exergy Analysis of Drilled and Driven Pile*:

Cumulative exergy consumption (CExC) developed by Szargut (1988) calculates the total exergy consumed in the making of a product. For our particular case study, we will use the CExC method to determine efficiency of the two types of foundation on the basis of consumption. As seen in emergy analysis, in this case also, to arrive at the net exegy consumption, we need to know the state of materials at the dismantling stage and their exergies. Then, net exergy consumption = cumulative exergy consumption – exergy of the residual materials.

For the inflow only, exergy being additive, we can have:

***Cumulative exergy flowing into the process of construction (CExC) = Exergy of Cement + Exergy of Steel + Exergy of Fuel.***

Berthiume & Buchard(1999) has calculated exergy of cement concrete for dry process to be 5.35 MJ/Kg and for wet process to be 10.2 MJ/Kg. The exergy of steel is 41MJ/Kg with the assumption that steel is fully oxidized at the end of its useful life (Szargut 1988). Exergy of diesel fuel is 42.7 MJ/kg (Dincer & Rosen 2007)

Since this is a hypothetical case study, the exergy due to actual fuel use cannot be determined but the high exergy content of diesel fuel indicates that use of heavy machinery that consumes large amount of fuel will end up with higher CExC.

From calculations shown in Table 1 &2, both the mass of cement concrete and steel used in construction are higher for drilled shaft than driven pile. Hence, the cumulative exergy consumption will also be higher for drilled shaft than driven pile.

### *3.3 Remarks*

Exergy analysis includes the raw materials used in process but it fails to account for the energy contribution of the natural resources that are in their natural (standard) state (Berthiume & Buchard ,1999). For example, in our case, exergy of soil excavated for the purpose of construction does not make any contribution to the process exergy until it undergoes a chemical property change when disposed in landfill as soil is considered to be in its standard state in the lithosphere. Similarly, the emission of $CO_2$ in cement manufacturing process does not affect the exergy of cement until a significant change in noticed in the standard atmospheric conditions.

## 4. Conclusion

Major anthropogenic changes of the environment are due to indiscriminate use of natural resources for technical advancement. We as engineers are the main sculptors of this technology oriented society and it should be a primary concern for us to rethink and re-evaluate existing systems so that the future generation does not have to compromise on their requirement for our contribution to this system. Towards this goal, civil engineers have a greater responsibility as they provide the basic infrastructure of social development. Geotechnology as the foundation of any civil engineering construction and also as an interface between nature as soil and the built environment has an immense potential to economize the use of the resources and energy if properly managed.

Foundation construction is a large and complex process engineering that involves exploitation of natural capital in the form of land and water use, human labor and material use. It is clearly evident that indiscriminate use of any of these is going to affect the ecosystem adversely in both short and long term. However, this industry is still focused on technological and economic efficiency and absence of proper study into a possible contradiction between technical efficiency and energy efficiency has led to a lack of general consciousness. Both drilled shaft and driven pile are two most commonly and traditionally used pile foundation whose usage till date has been singularly dictated by market economics and technical considerations. Being engineers, we admit that technical feasibility is of paramount importance in all projects but energy considerations can bring a third dimension in decision making when alternative choice is technically not limiting.

As methods, emergy analysis seems to better represent the energy consumption in geotechnical processes. Emergy provides an ecocentric economic valuation of ecosystem goods and services and is considered by many as a more holistic approach to environmentally conscious decision making. The methods of LCA or exergy analysis are more focused on emissions and their impacts and fail to capture the critical nature of contribution of ecosystems to human well being (Hau & Bakshi 2004). This paper provides a quantitative reference framework based on both the methods of energy utilization to help the future practitioners in this field take a more informed decision that will promote sustainable growth.

## Reference:


1. Abreu D.G, Jefferson I, Braithwaite P.A, Chapman D N- Why is Sustainability Important in Geotechnical Engineering-Geo Congress 2008-Geosustainabiltity and Geohazard Mitigation
2. Berthiaume R, Bouchard C- Exergy Analysis of the environmental impact of paving material manufacture -Transactions of the Canadian Society for Mechanical Engineering. 1999
3. Brown M T & Buranakaran V-Emergy indices and ratios for sustainable material cycle and recycle options-Resources, Conservation and Recycling 2003
4. Brown M T & Ulgiati S- Emergy-based Indices and Ratios to Evaluate Sustainability:monitoring economies and technology towards environmentally sound innovation-Ecological Engineering 1997
5. Brundtland G H "Our Common Future: Report of the World Commission on Environment and Development-Oxford University Press-1987
6. Dincer I, Rosen M-Exergy: Energy, Environment & Sustainable Development-Elsevier,2007
7. Edgerton H. Robert-Available Energy and Environmental Economics-Lexington Books 1982
8. Hau J L& Bakshi B R-Promise and problems of Emergy Analysis-Ecological Modelling 2004
9. Hau J L, Ukidwe N U, Bakshi B R-Hierarchical Thermodynamic metrics for evaluating the environmental sustainability of industrial processes-Environmental Progress and Sustainable Energy,2004
10. Holt D G A, Jefferson I, Braithwaite P A, Chapman D N- Sustainable Geotechnical Design-Geoflorida 2010
11. Jefferis A Stephen- Moving towards Sustainability in Geotechnical Engineering-GeoCongress 2008-Geosustainability and Geohazard Mitigation
12. Jefferson I, Hunt D V L, Birchall C A, Rogers C D F –Sustainability Indicators for Environmental Geotechnics- Proceedings of the Institute oF Civil Engineers , Engineering Sustainability,2007
13. Kibert J. Charles- Environmental Cost Internalization for Sustainable Construction-Evaluation of the built environment for Sustainability1997
14. McDonough William & Braungart Michael-Cradle to Cradle-North Point Press-2002
15. Odum Howard T- Environmental Accounting- Wiley Publishing Company- 1996
16. Pulselli RM, Simoncini E, Pulselli FM, Bastianoni S- Emergy Analysis of Building manufacturing, maintenance and use: Em-building indices to evaluate housing sustainability-Energy and Buildings,2007
17. Sciubba E, Wall G – A brief commented history of Exergy from the beginnings till 2004-International Journal of Thermodynamics,2010
18. Spaulding Charles, Masse Frederic, LaBrozzi Justin- Ground Improvement Technologies for a Sustainable World-GeoCongress 2008-Geosustainability and Geohazard Mitigation
19. Szargut J, Morris R. David, Stewart R. Frank –Exergy Analysis of Thermal, Chemical and Metallurgical Processes-Hemishere Publishing Company,1988

# SOIL CONSTITUTIVE MODEL FOR SUSTAINABLE GEOTECHNICAL DESIGN

William Higgins[1], Harry Martindale[1], Tanusree Chakraborty[2*] and Dipanjan Basu[1]

[1]Department of Civil and Environmental Engineering, University of Connecticut, Storrs, CT 06269, USA.
[*]Telephone 860 486 5023, E-mail: dbasu@engr.uconn.edu

[2]Department of Civil Engineering, Indian Institute of Technology Delhi, India.
*Telephone: +1 765 337 6019, *E-mail: tanusree.research@gmail.com

**ABSTRACT:** Design of sustainable civil infrastructure requires that the built environment is resilient against natural and man-made hazards which can cause catastrophic failures. As a result, high rates of strain ($10^2$-$10^4$/sec) are generated in the soil which plays a significant effect on the strength and stiffness of soil. In this paper, we investigate the high strain-rate behavior of sand by developing a rate-dependent, multi-axial, viscoplastic two-surface constitutive model based on the concepts of critical-state soil mechanics. Perzyna's overstress theory and non-associated flow rule are used in this model. The rate-dependent model parameters are determined from experimental data of split Hopkinson pressure bar test under high rate loading. Model performance is demonstrated for various sands.

## 1. INTRODUCTION

An important requirement of sustainable infrastructure design is that the built environment is resilient against natural and man-made disasters. Natural hazards like landslide, mudflow, debris flow, earthquake and tsunami and man-made hazards like terror attack and collision cause catastrophic failures in civil infrastructure. Hazardous flows (landslide, mudflow and debris flow) can move rapidly along down slope with a flow speed as high as 0.03 km/sec. Earthquake induced P and S wave speed can be up to 6 km/sec (Kumar et al. 1987, Tseng and Chen 2006). A bomb blast can create strain rates in materials up to $10^4$/sec (DeSilva 2005, Barsoum and Philip 2007, Ishihara 1996). Often, large geo-structures like earth embankments, slopes and tunnels involving large masses of soil are affected by these hazards. As a result, high rates of strain, of the order of $10^2$-$10^4$/sec, are generated in the soil. Soil is the weakest of all civil engineering materials and often collapse of a civil engineering structure is initiated from within the soil. In order to safeguard civil engineering facilities against different catastrophic hazards, it is essential that soils subjected to high strain rates are properly characterized and modeled. The rate of induced strain (or stress) plays a significant effect on the strength and stiffness of soil.

Casagrande and Shannon (1948) were the first to study the effect of strain rate on the strength of soil. They performed drained triaxial compression tests on dense Manchester sand with the strain rates varying from $1\times10^{-5}$/sec to 1/sec and observed that the compressive strength of sand increased by about 10% from the corresponding rate-independent (static) value. Since then, many researchers have performed drained and undrained triaxial tests on sand under different loading rates (Whitman and Healy 1962, Yamamuro and Lade 1998, Yamamuro and Abrantes 2003). Jackson et al. (1980) conducted uniaxial strain tests on sand at 200/sec strain rate. From these triaxial and uniaxial tests it was observed that the shear strength of sand increased by about 10% with each log-cycle increase in the strain rate and that an increase in the applied strain rate resulted in increased dilatancy and earlier peak generation. It was further observed that the dynamic shear modulus of sand was 5-40% higher than the static shear modulus. The split Hopkinson pressure bar (SHPB) tests have been performed on sand by several researchers in order to investigate sand behavior at strain rates as high as $10^4$/sec (Felice 1985, Veyera and Ross 1995, Semblat et al. 1999, Song et al. 2009). The results showed that the compressive response of the dry sand was significantly dependent on the initial density, compaction and lateral confinement level. The stress-strain response of highly saturated sand (saturation > 80%) followed nearly the same slope as of the uniaxial stress-strain response of water under SHPB test.

Very few researchers (Laine and Sandvik 2001, Wang et al. 2004, Grujicic et al. 2006, Tong and Tuan 2007, Deshpande et al. 2009, Chakraborty et al. 2010) have attempted to develop soil constitutive models for high strain rates. Although some of the existing constitutive models can capture strain rates as high as 200/sec and have been applied to simulate blast loading in soil, they are mostly not capable of capturing the path dependent, multi-axial soil behavior with all the important features like the peak and critical states and phase transformation under both rate-independent and rate-dependent loading.

In this paper, a rate-dependent, viscoplastic constitutive model for sand is developed that can simulate all the important features — e.g., dilatancy, critical state and phase transformation — of the multi-axial, stress-path dependent behavior of soil under both drained and undrained loading, and can capture extremely high strain rates. The model is developed by extending the modified Manzari-Dafalias two-surface plasticity model for sands (Manzari and Dafalias 1997, Papadimitriou and Bouckovalas 2001, Dafalias and Manzari 2004, Loukidis and Salgado 2009). Viscoplasticity is incorporated in the model using Perzyna's overstress theory (Perzyna 1963 and 1966). The strain-rate dependence of the initial shear modulus is incorporated explicitly in the model. The model performance is demonstrated by comparing with test results obtained from high-speed SHPB tests for up to 2000/sec strain rate. The research presented here is at the initial stages of an ongoing project on systematic quantification of soil behavior under high strain rates.

## 2. BASIC PLASTICITY MODEL

The rate-independent, two-surface sand plasticity model adopted in the study was proposed by Manzari and Dafalias (1997) and later modified by Loukidis and Salgado (2009). Figure 1(a) shows the model in the normalized deviatoric stress space. The model contains four conical shear surfaces, the yield, bounding, dilatancy and critical-state (CS) surfaces, with straight surface meridians and apex at the origin. The projection and interpolation rules are exclusively contained in the deviatoric plane. The yield surface of the model is given by

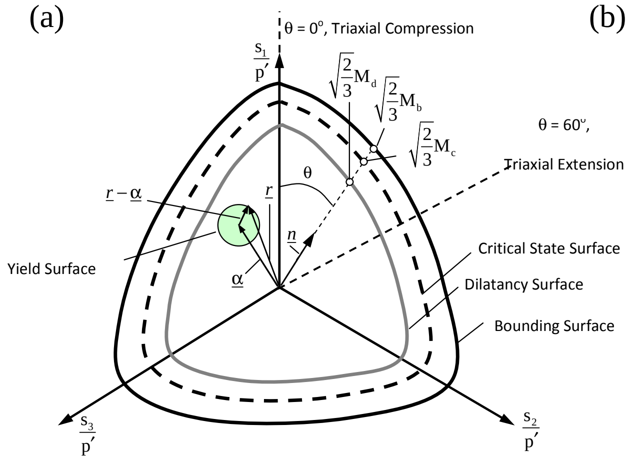
$$f = \sqrt{\rho_{ij}\rho_{ij}} - \sqrt{2/3}\,m = 0 \tag{1}$$

where m is the radius of the yield surface and $\rho_{ij}$ is the stress ratio given by

$$\rho_{ij} = r_{ij} - \alpha_{ij} \tag{2}$$

in which $r_{ij}$ is the normalized deviatoric stress tensor ($r_{ij} = s_{ij}/p'$; $s_{ij}$ is the deviatoric stress tensor and p' is the effective mean stress) and $\alpha_{ij}$ is a kinematic hardening tensor.

(a)




(b)

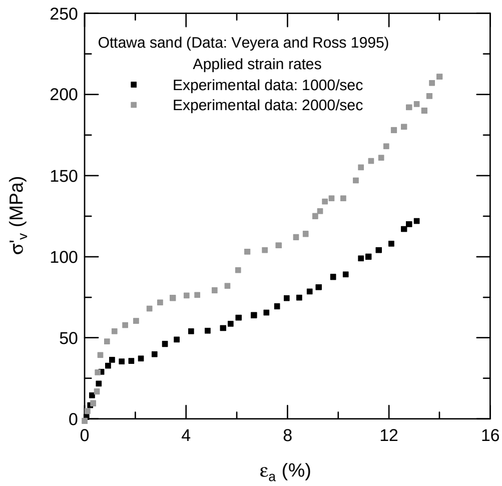



Figure 1. (a) Modified Manzari-Dafalias two-surface plasticity model for sand (from Loukidis and Salgado 2009) and (b) a typical vertical stress-axial strain plot for Ottawa sand in SHPB test (from Veyera and Ross 1995)

The yield surface can harden only kinematically through the use of the kinematic hardening tensor $\alpha_{ij}$. The bounding and the dilatancy surfaces can harden or soften isotropically through the dependence of the corresponding stress ratios $M_b$ and $M_d$ on the state parameter ψ (ψ = e – $e_c$; where e and $e_c$ are the current and the critical-state void ratios at the same mean stress) (Been and Jefferies 1985)

$$M_b = g(\theta)M_{cc}e^{-k_b\psi} \quad (3)$$

$$M_d = g(\theta)M_{cc}e^{k_d\psi} \quad (4)$$

where $M_{cc}$ is the critical-state stress ratio in triaxial compression [$M_{cc} = 3(\sigma'_{1,CS} - \sigma'_3)/(\sigma'_{1,CS} + 2\sigma'_3)$]. In the current model formulation, $M_{cc}$ is a model parameter, $k_b$ and $k_d$ are fitting parameters and $g(\theta)$ is a function of the Lode's angle $\theta$ that determines the shapes of the critical-state, bounding and dilatancy surfaces on the deviatoric plane (Loukidis and Salgado 2009).

## 3. DEVELOPMENT OF THE HIGH STRAIN-RATE CONSTITUTIVE MODEL

Figure 1(b) illustrates a typical vertical stress vs. axial strain response of Ottawa sand under SHPB test under maximum strain rates of 1000/sec and 2000/sec at 0% saturation (data from Veyera and Ross 1995). Three important features of sand stress–strain behavior in impact loading are observed in this figure which the constitutive model needs to be able to capture: (1) an inertial response early in the event when the soil sample at rest is suddenly accelerated after initial contact with the striker bar; inertial response becomes more prominent at higher impact velocities (e.g., higher strain rates), (2) gradual transition from stiff initial inertial response to a viscous flow behavior and (3) a strain hardening behavior at large strains where the stress-strain response looks like an exponential curve. In the following sections we will discuss how the model captures the first two features. The third feature is captured through the evolution of $\alpha_{ij}$ (the kinematic hardening tensor).

### *3.1 Initial shear modulus*

In the current model, the stress state is assumed nonlinear elastic inside the yield surface. The SHPB tests on sand (Veyera and Ross 1995, Semblat et al. 1999) showed that the initial shear modulus up to 1% of axial strain was between 300 to 6000 MPa, which is almost 5-40% higher than the shear modulus of sand in static loading. This increase in shear modulus is due to the inertial resonse of sand under suddenly applied impact load (as observed by Dupaix and Boyce 2007 for polymers). However, systematic quantification of the increase in shear modulus for sands is not yet done in the literature. Hence, in the current model, we use an curve-fitting approach through the experimental data for the very initial portion of the stress-strain response (i.e. when the axial strain is less than 1%). The initial shear modulus $G_0$ is determined from the slope of vertical stress-axial strain plot. After 1% of axial strain, when the viscous flow behavior governs material response, the initial shear modulus $G_0$ is calculated from the initial void ratio and mean stress (Hardin and Richart 1963). Since the initial stiffness increases with increasing strain rate (Matesic and Vucetic 2003), $G_0$ in the proposed model is given by

$$G_0 = C_g\left[(2.17 - e_0)^2 \big/ (1+e_0)\right]\sqrt{p'p_a}\left(1 + b_{rate}\ln\left(1+\dot{\varepsilon}_{eq}\right)\right) \quad (5)$$

where $C_g$ is a model parameter, $e_0$ is the initial void ratio, $p_a$ is a reference mean stress (= 100 kPa) and $b_{rate}$ is a parameter that determines the dependence of $G_0$ on the applied deviatoric strain rate $\dot{\varepsilon}_{eq}$. The shear modulus follows the Ramberg-Osgood type degradation given by Loukidis and Salgado (2009):

$$G = G_0 \Big/ 1 + 2\left(1/\alpha_1 - 1\right)\left(\sqrt{3/2}\sqrt{r_{ij} - \alpha_{ini,ij}} \Big/ 2\alpha_1 LI\left(G_0/p'\right)\gamma_1\right) \quad (6)$$

where G is the degraded shear modulus, $\alpha_1$ and $\gamma_1$ are model parameters and $\alpha_{ini,ij}$ is the initial value of the kinematic hardening tensor $\alpha_{ij.}$ The parameter LI represents the loading index: LI = 1 for loading, = 2 for subsequent unloading and reloading. The degradation of the shear modulus occurs both inside and outside of the yield surface and G is not allowed to degrade below $G_0/2(1/\alpha_1 - 1)$.

### *3.2 Incorporation of viscoplastic rate-dependence*

The viscoplastic process begins as the stress-state reaches the yield surface. In this paper, Perzyna's overstress theory (Figure 2a) is used to incorporate the viscoplastic behavior of sand. The overstress theory is based on the viscoplastic overstress function $\phi$ defined as

$$\langle \phi(F) \rangle = \begin{cases} F \text{ if } F > 0 \\ 0 \text{ if } F \leq 0 \end{cases} \qquad (7)$$

where the parameter quantifies the amount of overstress and is given by $F = f_d - f_s$ in which $f_d$ and $f_s$ are the dynamic and static yield surfaces, respectively.

Unlike the conventional, single yield-surface plasticity models, there is no static yield surface $f_s$ in our model. In order to use the overstress theory, we assume that, at any given instance of time n, the yield surface f, given by equation (1), represents the static yield surface $f_s$ and the "current" stress state, represented by $r_n$ in Figure 2b, is on $f_s$. For the next strain increment at time n+1, if the stress state lies outside this static yield surface, then the stress state is viscoplastic. According to Liingaard et al. (2004), the "overstress" is the amount of stress by which a stress state exceeds the yield surface. Therefore, the stress state $r_{n+1}^{visco}$ in Figure 2b, representing the stress state at time n+1, is on a dynamic yield surface $f_d$ and the difference $\left| r_{n+1}^{visco} - r_n \right|$ represents the overstress. The dynamic yield surface is assumed to have the same form as equation (1). Thus, $f_d$ is given by

$$f_d = \sqrt{\rho_{ij}^d \rho_{ij}^d} - \sqrt{2/3}m = 0 \qquad (8)$$

where $\rho_{ij}^d$ is the viscoplastic stress ratio, given by

$$\rho_{ij}^d = r_{ij}^d - \alpha_{ij} \qquad (9)$$

In which $r_{ij}^d$ is the measure of the current normalized deviatoric stress. Note that $\rho_{ij}^d$ is the amount of "extra" stress from the centre $\alpha_{ij}$ of the yield surface ($\rho_{ij}^d$ represents the distance of $r_{n+1}^{visco}$ from the center of the yield surface in Figure 2b). Therefore, the measure of the overstress $\left| r_{n+1}^{visco} - r_n \right|$ can be obtained by appropriately subtracting the radius m of the yield surface from $\rho_{ij}^d$. The right hand side of equation (9) represents this "distance" $\left| r_{n+1}^{visco} - r_n \right|$ and hence $f_d$ is the overstress in our model. Thus, we choose $F = f_d$ in our model.

Following Perzyna (1966), the total strain rate $\dot{\varepsilon}_{ij}$ is split into elastic and viscoplastic components $\dot{\varepsilon}_{ij}^e$ and $\dot{\varepsilon}_{ij}^{vp}$ as

$$\dot{\varepsilon}_{ij} = \dot{\varepsilon}_{ij}^e + \dot{\varepsilon}_{ij}^{vp} \qquad (10)$$

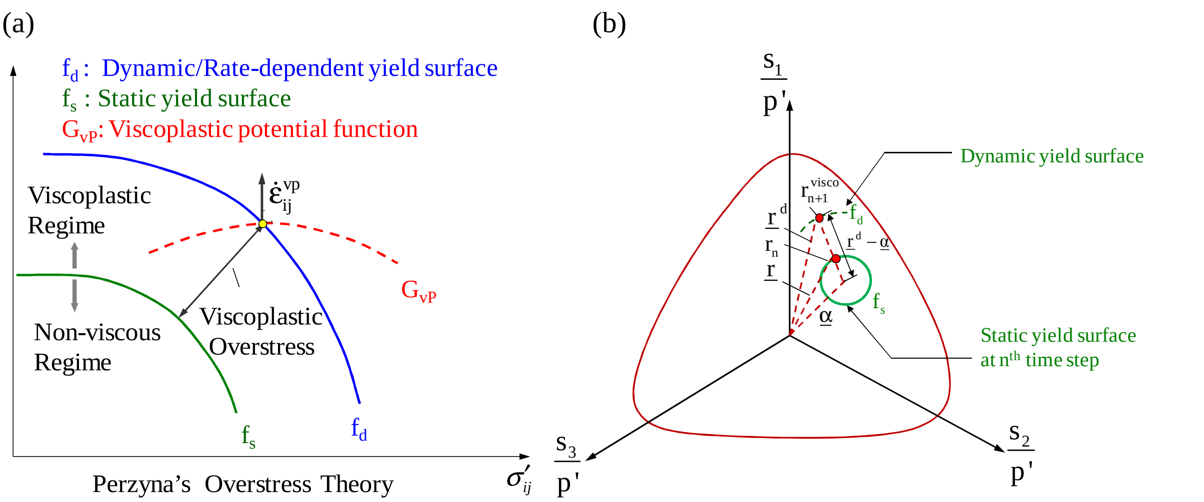



Figure 2. (a) The concept of overstress viscoplastic model (from Liingaard et al. 2004) and (b) initial ('static') and dynamic yield surfaces in the current model

The viscoplastic strain-rate $(\dot{\varepsilon}_{ij}^{vp})$ is given by a non-associated flow rule

$$\dot{\varepsilon}_{ij}^{vp} = \dot{\lambda}_{vp}\left(\partial G_{vP}/\partial\sigma_{ij}\right) \quad (11)$$

where $G_{vP}$ is the viscoplastic potential function and $\dot{\lambda}_{vp}$ is the viscoplastic multiplier given by

$$\dot{\lambda}_{vp} = \langle\phi(F)\rangle/\eta \quad (12)$$

In which the parameter $\eta$ is the viscoplastic coefficient. During the stress-strain integration, the viscoplastic multiplier is determined incrementally (Martindale et al. 2010). The gradient $\left(\partial G_{vP}/\partial\sigma_{ij}\right)$ of the viscoplastic potential in stress space is divided into a deviatoric component $R'_{ij}$ and a mean component that relates to the dilatancy D (Loukidis and Salgado 2009):

$$\partial G_{vP}/\partial\sigma_{ij} = \left(R'_{ij} + 1/3 D\delta_{ij}\right) \quad (13)$$

$R'_{ij}$ gives the direction of the deviatoric viscoplastic strain rate $\dot{e}_{ij}^{vp}$. The dilatancy D controls the shear-induced viscoplastic volumetric strain rate $\dot{\varepsilon}_{kk}^{vp}$. D depends on the distance between the current stress state and the projected stress state on the dilatancy surface (Manzari and Dafalias 1997):

$$D = D_0/M_{cc}\left(\sqrt{2/3}\left(M_d - m\right) - \alpha_{ij}n_{ij}\right) \quad (14)$$

where $D_0$ is an input parameter controlling the inclination of the stress ratio-dilatancy curve.

## 4. MODEL PARAMETERS

We demonstrate the performance of the constitutive model by comparing the stress-strain responses obtained from our model with those obtained from SHPB tests performed by Felice (1985) on New Mexico clayey sand, Semblat et al. (1999) on Fontainebleau sand, and Veyera and Ross (1995) on Ottawa sand. Details of these sands are presented in Table 1.

The rate-indepedent parameters for Ottawa sand are available from Loukidis and Salgado (2009). Determination of the rate-independent model parameters for New Mexico clayey sand and Fontainebleau sand for modified Manzari-Dafalias model is underway. The current viscoplastic model formulation has two rate-dependent parameters $\eta$ and $b_{rate}$. The coefficient of viscosity $\eta$ of sand is assumed to be equal to 0.005 MPa-sec following Towhata (2008). The parameter $b_{rate}$ is assumed to be equal to 0.002 for Ottawa sand and New Mexico clayey sand considering the fact that a 10% increase in the initial shear modulus value was observed for each log-cycle increase in the strain rate (Matesic and Vucetic 2003). For Fontainebleau sand, Semblat et al. (1999) observed a 0.2% decrease in the initial shear modulus value with each log-cycle increase in strain rate. Therefore, $b_{rate}$ is assumed to be equal to –0.0001 for this sand. The rate-dependent model parameters for all the three sands are presented in Table 1.

Table 1: Description of sands used in model parameter determination

| Sand | Type | Density (kg/m$^3$) | Critical state friction angle ($^o$) | Rate-dependent model parameters (from calibration) | | References |
|---|---|---|---|---|---|---|
| | | | | $\eta$ ( MPa-sec) | $b_{rate}$ | |
| Ottawa sand | Silica sand | 1715.00 | 29 | 0.005 | 0.002 | Veyera and Ross (1995) |
| New Mexico clayey sand | Quartz sand | 1870.00 | ≈ 33 (considered to be the same as a Quartz sand) | 0.005 | 0.002 | Felice (1985) Lancelot (2006) |
| Fontainebleau sand | Quartz sand | 1667.00 | 29 | 0.005 | -0.0001 | Semblat et al. (1999), Gaudin et al. (2005) |

## 5. MODEL VALIDATIONS

### *5.1 Split Hopkinson Pressure Bar test*

The developed constitutive model was incorporated in the finite element (FE) software Abaqus through a user material subroutine UMAT. SHPB tests were simulated at different strain rates for the New Mexico clayey sand, Fontainebleau sand and Ottawa sand using Abaqus. Table 2 presents the initial conditions of the SHPB simulations — sample dimension, density, initial void ratio and amplitude of loading — as used by Felice (1985) for New Mexico clayey sand, Veyera and Ross (1995) for Ottawa sand and Semblat et al. (1999) for Fontainebleau sand. The sand samples were assumed to be dry for the simulation. The tests were simulated using an axisymmetic 8-noded full integration element. Zero vertical-displacement and zero radial-displacement conditions were applied at the bottom and the side boundaries of the element, respectively, to simulate the uniaxial loading condition of the actual tests. Pressure loading (for New Mexico clayey sand and for Ottawa sand) or velocity boundary condition (for Fontainebleau sand) was applied on the top boundary of the specimen with exactly similar amplitudes as used in the actual experiments to simulate the uniaxial loading condition of the actual tests. Figure 4a illustrates the geometry of the sample for the New Mexico clayey sand. The analysis was performed in two steps: (1) geostatic equilibrium and (2) dynamic loading. Although there was no initial confining pressure applied in the actual tests, we applied a minimal initial confining stress of 20 kPa in the geostatic equilibrium stage to avoid numerical singularity. The dynamic loading step is simulated using the implicit dynamic procedure in Abaqus. Damping is applied in the dynamic loading step through material viscoplasticity.

Table 2: Description of initial test conditions and loading

| Sand | Sample Dimension | | Initial void ratio | Applied strain rates | Loading | Reference |
|---|---|---|---|---|---|---|
| | Height (cm) | Diameter (cm) | | | | |
| Ottawa sand | 0.635 | 5.08 | 0.545 | 1000/sec, 2000/sec | Applied pressure pulse, peak stress rise time 50 μsec, 257μsec pulse width | Veyera and Ross (1995) |
| New Mexico clayey sand | 0.65 | 6.12 | 0.46 | 1051/sec | Applied pressure pulse, peak stress rise time 100 μsec, 140μsec pulse width | Felice (1985) |
| Fontainebleau sand | 1.00 | 4.00 | 0.54 (same as $e_{min}$) | 393/sec, 771/sec, 1245/sec | Applied impact velocity, 3.4m/sec, 5.8m/sec, 9.9m/sec | Semblat et al. (1999), Vincens et al. (2003) |

Figure 4b shows the vertical stress-time response of New Mexico clayey sand at 1051/sec strain rate. Figures 5a and 5b show the axial stress-strain response obtained from simulations of Fontainebleau sand and Ottawa sand respectively at different strain rates. The peak strengths of sands at high strain rate are predicted reasonably well. The model captures the initial high stiffness of the stress-strain curves for Ottawa sand and Fontainebleau sand through the initial increase in shear modulus. According to Veyera and Ross (1995), the initial steep slope of the stress-strain curves is caused by particle reorientation under high impact loading. The constitutive model in its present form does not capture sand behavior at the particle level. Further investigation is in progress to capture the particular behavior of sand at high loading rate and the gradual transition from the initial inertial response to the final exponential response of the curve.

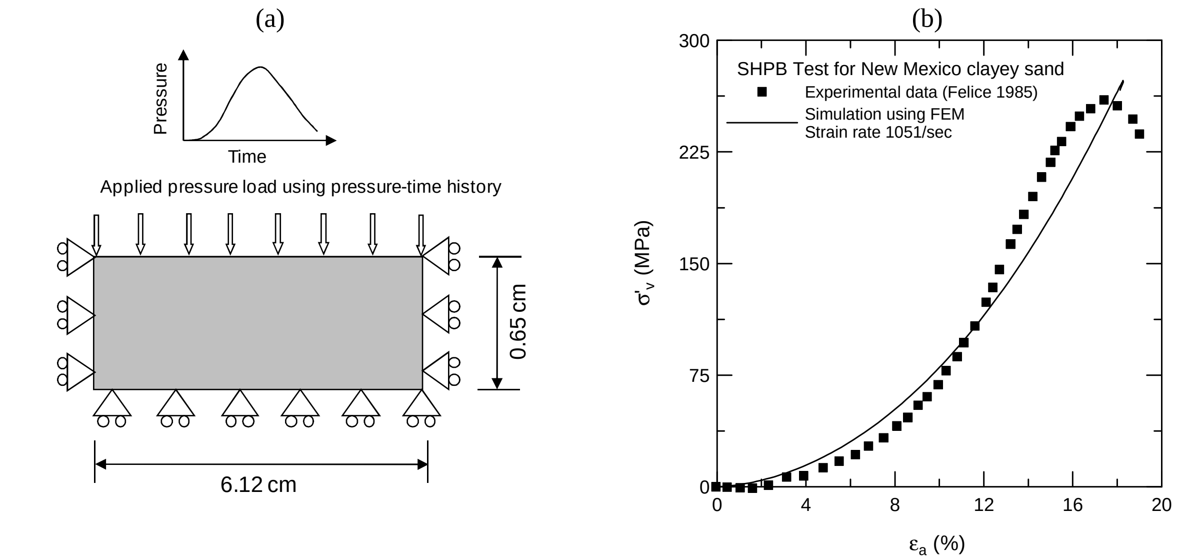



Figure 4. (a) Geometry of the SHPB test sample and (b) vertical stress-axial strain response of New Mexico clayey sand in SHPB test.

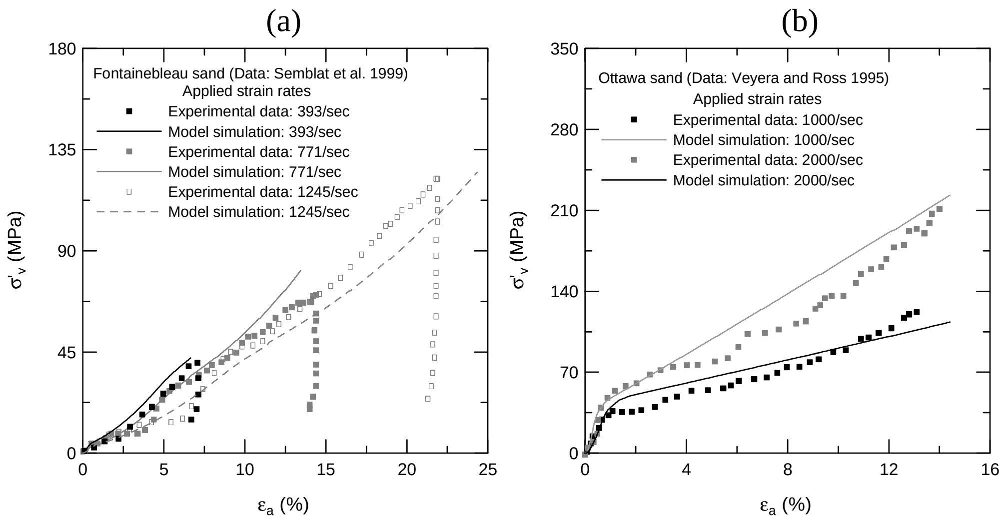



Figure 5. Vertical stress-axial strain response of (a) Fontainebleau sand and (b) Ottawa sand in SHPB test.

## 6. CONCLUSIONS

The paper presents a viscoplastic constitutive model for sand based on the concepts of critical-state soil mechanics for the design of sustainable civil infrastructure. The model is developed from an existing rate-independent sand constitutive model with open, “cone”-shaped yield and bounding surfaces. We added Perzyna’s overstress function and the strain-rate dependence of the initial shear modulus to the existing rate-independent model in order to capture the viscoplastic, rate-dependent behavior of sand. The model is currently capable of simulating sand behavior up to a strain rate of 3000/sec. The peak strength of sand at high loading rates is captured reasonably well. Further investigation is in progress to capture the particular behavior of sand and the gradual transition from the initial inertial response to the final exponential response of the curve.

The incorporation of the rate-dependence was achieved by using two additional parameters that can be directly determined either through inspection of the experimental data or by fitting simple equations to laboratory test data. The model performance under high loading rate was demonstrated for Ottawa sand, New Mexico clayey sand and Fontainebleau sands. The paper outlined a part of an ongoing research on a systematic study of the mechanical response of soil subjected to extremely high strain rates.

## References


Barsoum, R. G. S. and Philip, D. 2007. Armor including a strain rate hardening elastomer, *US Patent 7300893*.

Been, K. and Jefferies, M. G. 1985. A state parameter for sands. *Géotechnique*, 35 (2): 99-112.

Casagrande, A., and Shannon, W. L. 1948. Strength of soils under dynamic Loads. *Proceedings of the American Society of Civil Engineers*, 74(4): 591–608.

Chakraborty, T., Salgado, R. and Loukidis, D. 2010. A two-surface plasticity model for clay. *Submitted to Computers and Geotechnics*.

Dafalias, Y. F. and Manzari, M. T. 2004. Simple plasticity sand model accounting for fabric change effects. *Journal of Engineering Mechanics*, *ASCE*, 130: 622- 634.

DeSilva, C. W. 2005. Vibration and shock handbook. *Taylor and Francis*, *CRC Press*, 1872p.

Dupaix, R. B., and Boyce, M. C. 2007. Constitutive modeling of the finite strain behavior of amorphous polymers in and above the glass transition. *Mechanics of Materials*, 39:39–52.

Felice, C. W. 1985. The response of soil to impulse loads using the split-Hopkinson pressure bar technique. *PhD thesis*, The University of Utah, Utah, USA.

Gaudin, C., Schnaid, F. and Garnier, J. 2005. Sand characterization by combined centrifuge and laboratory tests. *International Journal of Physical Modelling in Geotechnics*, 1: 42-56.

Grujicic, M., Pandurangan, B, Cheeseman, B. A. 2006. A computational analysis of detonation of buried mines. *Multidiscipline Modeling in Materials and Structures,* 2: 363–387.

Hardin, B. O. and Richart, F. E., Jr. 1963. The nature of stress-strain behavior of soils. *Journal of Soil Mechanics and Foundations Division*, ASCE, 89 (1): 33-65.

Ishihara K. 1996. Soil behaviour in earthquake geotechnics. *Oxford Clarendon Press*, 259-260.

Jackson, J. G., Rohani, B., and Ehrgott, J. Q. 1980. Loading rate effects on compressibility of sand. *Journal of Geotechnical Engineering Division, ASCE*, 106(8): 839–852.

Kelln, C., Sharma, J. and Hughes, D. 2008. A finite element solution scheme for an elastic–viscoplastic soil model. *Computers and Geotechnics*. 35: 524–536.

Kumar, S., Chander, R. and Khattri, K.N. 1987. Compressional wave speed in the second crustal layer in Garhwal Himalaya. *Journal of Association of Exploration Geophysicists*, 8(4): 219-25.

Laine, P. and Sandvik. A. 2001. Derivation of mechanical properties for sand., *Proceedings of the 4$^{th}$ Asia–Pacific conference on Shock and Impact Loads on Structures.* Singapore 361–368.

Lancelot, L., Shahrour, I. and Mahmoud, M. A. 2006. Failure and dilatancy properties of sand at relatively low stresses. *Journal of Engineering Mechanics*, 132(12): 1396-1399.

Li, X. S. and Dafalias, Y. F. 2000. Dilatancy of cohesionless soils. *Géotechnique,* 50: 449-460.

Liingaard, M. Augustesen, A. and Lade, P. V. 2004. Characterization of models for time-dependent behavior of soils. *International Journal of Geomechanics*, 4(3): 157-177.

Loukidis, D. and Salgado, R. 2009. Modeling sand response using two-surface plasticity. *Computers and Geotechnics*, 36: 166–186.

Manzari, M. T. and Dafalias, Y. F. 1997. A critical state two-surface plasticity model for sands. *Géotechnique*, 47(2): 255-272.

Martindale, H., Higgins, W., Chakraborty, T., Basu, D. 2010. Two-surface viscoplastic sand model for high rate loading. Accepted in *International Conference on Geotechnical Engineering, ICGE Tunisia.*

Matešić, L. and Vucetic, M. 2003. Strain-rate effect on soil secant modulus at small cyclic strains. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, 129 (6): 536–549.

Ortiz, M. and Simo, J. C. 1986. An analysis of a new class of integration algorithms for elastoplastic constitutive relations. *International Journal for Numerical Methods in Engineering,* 23: 353-366.

Perzyna, P. 1963. The constitutive equations for rate sensitive plastic materials. *Quarterly of Applied Mathematics,* 20: 321-332.

Perzyna, P. 1966. Fundamental problems in viscoplasticity. *Advances in Applied Mechanics,* 9: 244–377.

Semblat, J. F., Phong, L. M. and Gary, G. 1999. 3D-Hopkinson bar: new experiments for dynamic testing on soils. *Soils and foundation*, 39(1): 1-10.

Simo, J. C. and Hughes, T. J. R. 1998. Computational Inelasticity, *Springer*, 392p.

Sloan, S.W. 1987. Substepping schemes for the numerical integration of elastoplastic stress-strain relations. *International Journal for Numerical Methods in Engineering,* 24(5): 893-911

Song, B., Chen, W. and Luk, V. 2009. Impact compressive response of dry sand, *Mechanics of Materials*, 41: 777–785.

Tong, X. and Tuan, C. 2007. Viscoplastic cap model for soils under high strain rate loading. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE,* 133(2): 206-214.

Towhata, I. 2008. Geotechnical Earthquake Engineering. *Springer.*

Tseng, T. and Chen, W. 2004. Contrasts in seismic wave speeds and density across the 660-km discontinuity beneath the Philippine and the Japan Seas. *Journal of Geophysical Research*, 109: B4, 12 pp.

Veyera, G. E. and Ross, C.A. 1995. High strain rate testing of unsaturated sands using a split-Hopkinson pressure bar, *Proceedings of 3rd International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*, St-Louis, USA, 31-34.

Vincens, E. Dubujet, Ph. and Cambou, B. 2003. Cyclic behavior of soils, a prediction of permanent deformations. *Deformation Characteristics of Geomaterials*, eds. Di Bendetto et al., Swets & Zeitlinger, Lisse: 1027-1034.

Wang, Z., Hao, H. and Lu, Y. 2004. A three-phase soil model for simulating stress wave propagation due to blast loading, *International Journal for Numerical and Analytical Methods in Geomechanics*, 28: 33–56.

Whitman, R. V. and Healy, K. A. 1962. Shear strength of sands during rapid loading, *Journal of the Soil Mechanics and Foundations Division*, *ASCE*, 88 SM2: 99-132.

Yamamuro, J. A. and Abrantes, A. E. 2003. Behavior of medium sand under very high strain rates. *Proceedings of 1st Japan-U. S. Workshop on Testing, Modeling, and Simulation*, Boston, MA, USA, 61-70.

Yamamuro, J. A. and Lade, P. V. 1998. Steady-state concepts and static liquefaction of silty sands. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, 124(9): 868–877.

# UTILIZATION OF COIR DUST TO PARTIALLY REPLACE SAND IN VERTICAL DRAINS FOR SOFT GROUND IMPROVEMENT

U.P. Nawagamuwa[1*] and G.H.A.J.J. Kumara[2*]

[1]Department of Civil Engineering, University of Moratuwa, Moratuwa, Sri Lanka

[2]Department of Civil Engineering, Yokohama National University, Japan

*Formerly, Department of Civil and Environmental Engineering, University of Ruhuna, Galle, Sri Lanka

**Abstract**
During recent decades, ground improvements using waste materials have been encouraged in many countries as a cost effective way. This paper discusses application of coir wastes in vertical drains as a substitution material to sand.

The permeability behaviours of the combined material of sea sand and coir waste were studied and they were sufficient for vertical drains. The inclusion of vertical drains in the soft clay was physically modeled and the drains were filled by the mixed material. This study showed that consolidation rate of soft clay increased largely with the vertical drains. Finally, an optimum coir percentage was proposed.

***Keywords:*** *coir waste, consolidation, permeability, sea sand, vertical drain*

## 1. Introduction

During recent decades, ground improvements using waste materials have been encouraged in many countries to solve mainly two problems at the same time. The usage of waste materials solve some environmental and economical problems caused by wastes itself as waste disposal needs a large budget. Ground improvements too need some materials, mostly expensive materials. Therefore, in many ways, the concept of using waste materials for ground improvements seems to be cost effective and reasonable especially for developing countries like Sri Lanka.

This paper discusses application of coir wastes in vertical drains as a substitution material to sand, the traditionally used material. The sand drains are used as a ground improvement technique to accelerate settlements of soft ground. The sand, normally taken from rivers also had been a major environmental issue. Therefore, in this research, sea sand was used. It was previously reported that Sri Lanka produces over 0.5 million tons of coir wastes annually [1]. In some part of Sri Lanka, especially along coastal areas, coir waste dumping have become a serious environmental issue as the waste are dumped as open dumps as shown in Figure 1. Hence, in this research, application of coir wastes to partially replace sand for vertical drains was studied.

As the main function of vertical drain is to flow water quickly, permeability behaviours of the combined material of sea sand and coir waste were studied. The constant head permeability tests were conducted to study permeability characteristics. The materials were mixed with certain percentage by weight. The consolidation behaviour of soft clays, collected from a section of Southern expressway was also studied using consolidation tests. The inclusion of vertical drains in the soft clay was physically modeled. The consolidation tests were done as three cases, where effects of vertical drains in soft clay and then as a separate case, the effects of sand and combined material in vertical drains were studied.

The permeability and consolidation characteristics as well as some data of cost of the materials and environmental issues were considered to decide how much coir wastes would be mixed for the combined material. Finally, a model was proposed to determine a percentage of coir waste to use for vertical drains.

## 2. Material and methods

As mentioned above, sea sand and coir wastes were mixed to make a combine material which was used as a filling material for vertical drains.

### *2.1 Coir wastes*

The coir wastes were collected at a dumping site, located in Nugaduwa, Galle and Figure 2 shows the site. The brown colour coir is harvested from fully ripened coconuts. It has been reported that the coir are thick, strong and has high abrasion resistance [2]. Further, it was found that the coir fibres contain more lignin and less cellulose which made them resilient, strong and highly durable.

The content of fibres is important as they make the vertical drains more stable in soft grounds. The sieve analysis tests were done to evaluate particle distribution and percentage of fibres on the collected coir wastes.

Figure 1: *Coir wastes Hikkaduwa*

Figure 2: *Coir wastes dumping site at Nugaduwa*

### *2.2 Sea sand*

As the research was done at University of Ruhuna, Galle, the sea sands were collected from the coast near to Galle fort area. The sieve analysis was done to evaluate particle size distribution.

### *2.3 Soft clay*

The soft clays were collected from a section of Sothern expressway (CH: 15+000 from Kurundugaha Hetekma). There were many places along the expressway where soft clays were removed or structural measures were done for ground improvements. Figure 3 shows the soft clays collected area.

Atterberg limit tests were conducted to classify the soft clay collected. The liquid limit, plastic limit and plasticity index were evaluated with Atterberg limit test.

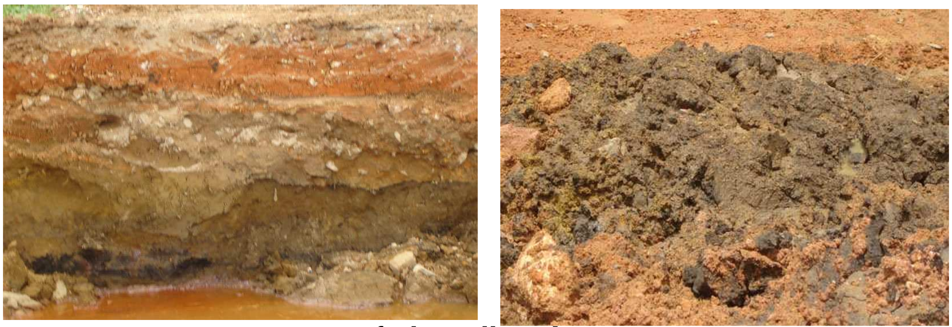


*Figure 3: Soft clay collected area*

### *2.4 Mixing of coir wastes and sea sand*

The mixing of coir wastes and sea sand were done using the mixing machine shown in Figure 4 and Figure 5. The percentage of coir wastes for the mixture was changed from 0 to 100 (on weight based) as shown in Table 1.

The mixing time (5 mins) was kept constant for each sample as to keep same conditions for every sample.

Figure 4: *Material mixing machine*

Figure 5: *After mixing coir wastes and sand*

***2.5 Permeability tests***

The permeability characteristics were examined by Constant head permeability tests. The sample preparations were done slightly different to the standard as in some cases, coir fibres tend to separate from the sand during sample preparations. Therefore, as shown in Figure 6, constant number of blows from a small rod (5 blows) was applied while applying some water into the sample. The samples were prepared in three layers. Figure 7 shows a prepared sample.

Figure 6: *Sample preparation (a) application of small compaction and (b) addition of water*

Figure 7: *The prepared sample*

***2.6 Consolidation tests***

The laboratory one-dimensional consolidation tests were conducted for three cases, one is for the soft clay specimen without a vertical drain. The other two cases were done with vertical drains, at the centre of the specimen. The vertical drains were filled by sea sand and combined material respectively. As a typical case, the combined material was selected as 50% of coir wastes with 50% of sea sand by weight.

The specimens were prepared as 50mm in diameter and 20mm in thickness. The ratio of vertical drain diameter to sample diameter was selected as 1/10. Figure 8 shows the specimen prepared for consolidation tests.

Figure 8: *(a) Application of vertical drain and (b) vertical drain filled by combined material*

## 3. Results

The results of laboratory experiments are shown in graphical forms.

### *3.1 Particle size distribution*

Figure 9 and Figure 10 show the particle size distribution curves of sea sand and coir wastes used for experiments.

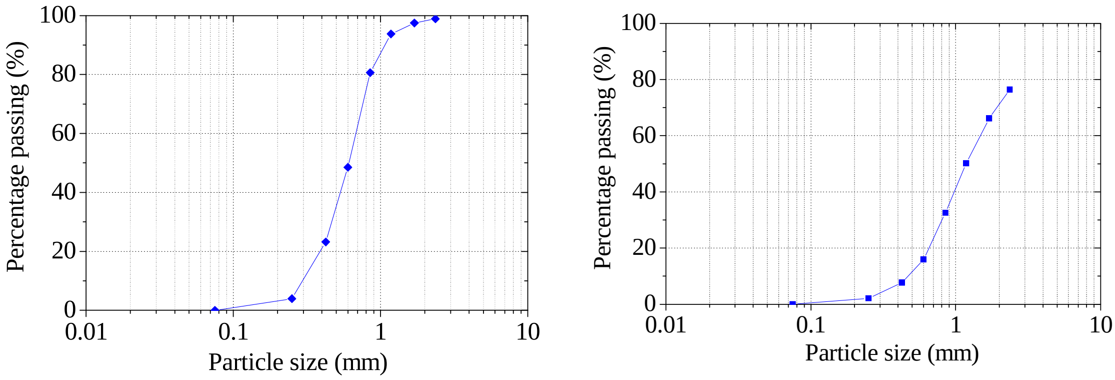



Figure 9: *Particle size distribution (a) sea sand and (b) coir wastes*

### *3.2 Permeability tests*

The Constant head permeability tests were done for 8 cases, ranging coir wastes from 0% (sea sand) to 50% on weight based. As typical cases, only data from few tests are shown in figure 10 and Figure 11.

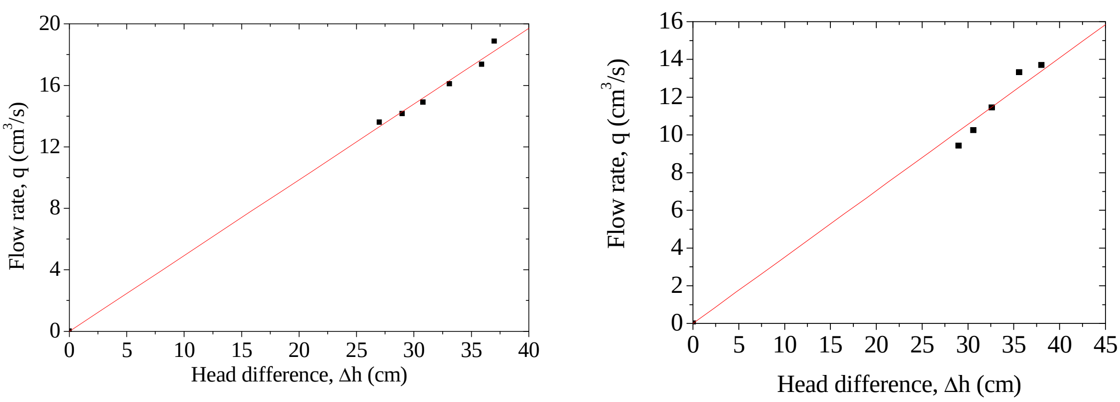



Figure 10: *The graph of q vs Δh for coir waste of (a) 5% and (b) 20%*

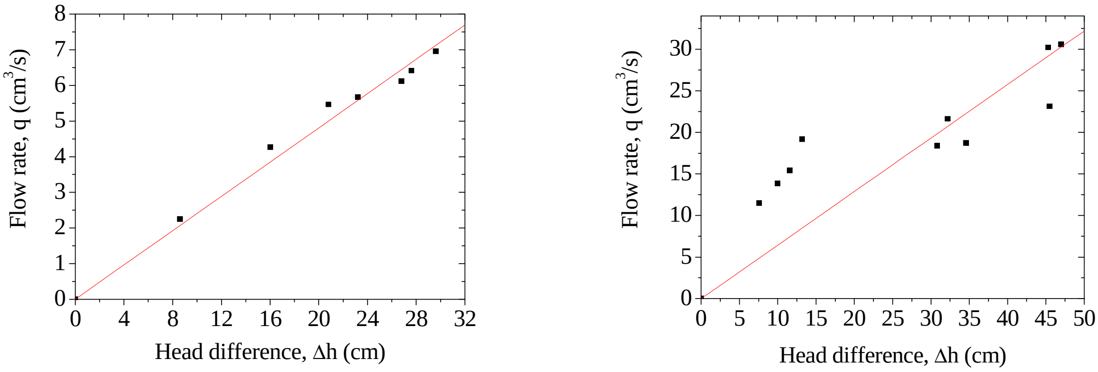



Figure 11: *The graph of q vs Δh for coir waste of (a) 50% and (b) 0% 9 sea sand sample)*

### *3.3 consolidation tests*

The results of consolidation tests for three cases are shown in Figure 12.

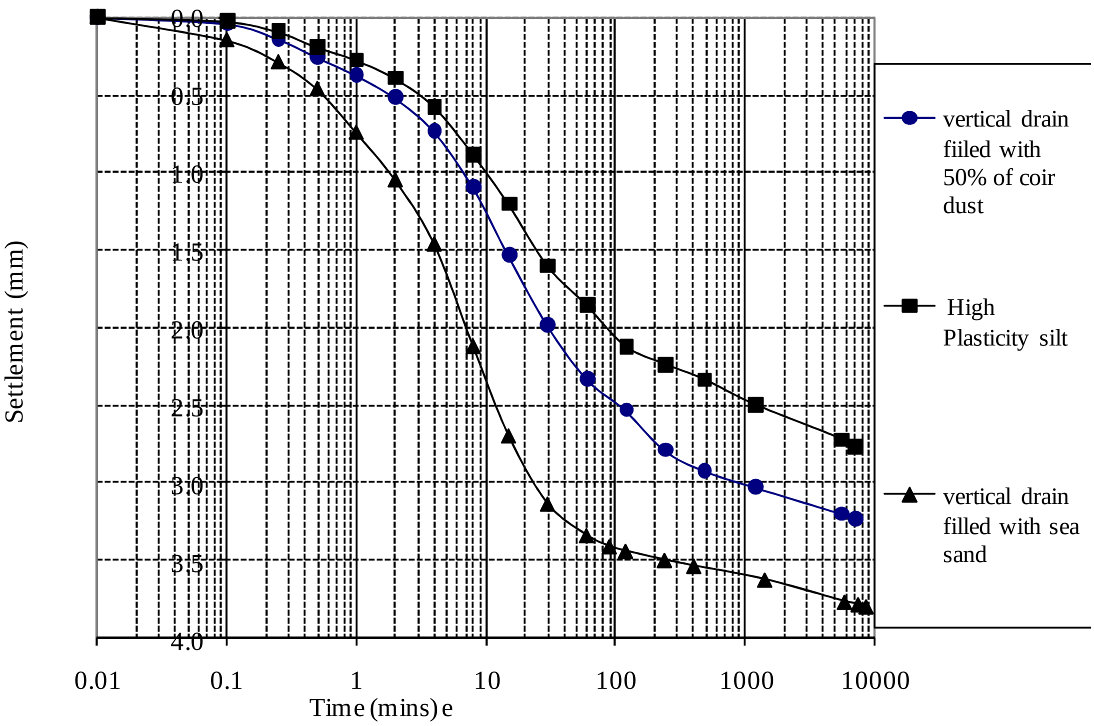



Figure 12: *The consolidation graphs for all cases*

## 4. Discussions

The permeability tests indicated that coefficient of permeability ($k$) reduces with amount of coir wastes added. The value of $k$ for sea sand was 0.121 (cm/s) while that of combined material (50% of coir wastes) was 0.048 (cm/s) as shown in Table 1 and Figure 13. Therefore, it is clear that the reduction of $k$ with 50% of coir wastes is about half of sea sand. The 50% coir wastes by weight based is equivalent to about 80% by volume. Therefore, with costs involved, the usage of coir wastes for vertical drains can be justified though it reduces permeability slightly.

As practical application will be based on volume, the coefficient of permeability was plotted against the coir percentage by volume as shown in Figure 14. A model to determine required k with coir percentage was also proposed as $k = 0.1173 – 0.00009c_v$ where $c_v$ is coir percentage by volume.

Table 1: *The details of the samples*

| Case No | Coir by weight (%) | Coir by volume (%) | $k$ (cm/s) |
|---|---|---|---|
| 1 | 0 | 0 | 0.121 |
| 2 | 5 | 18 | 0.098 |
| 3 | 10 | 32 | 0.091 |
| 4 | 15 | 43 | 0.077 |
| 5 | 20 | 52 | 0.070 |
| 6 | 30 | 65 | 0.064 |
| 7 | 40 | 74 | 0.052 |
| 8 | 50 | 81 | 0.048 |

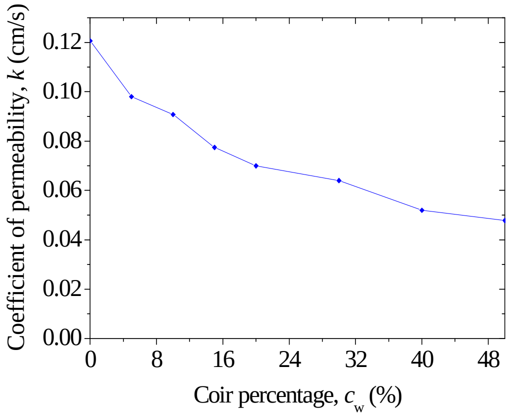



Figure 13: *The variation of k with % of coir*

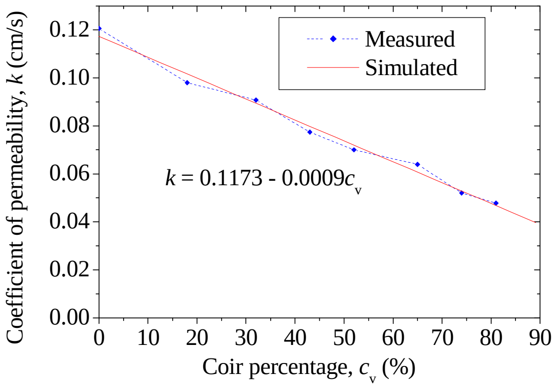



Figure 14: *The proposed model for k*

The consolidation tests showed that inclusion of vertical drains increased the coefficient of consolidations from 1.446 ($mm^2$/min) to 3.457 ($mm^2$/min) where vertical drains were filled by sea sand. However, the typical case of the combined material (50% of coir wastes) increased it only to 1.973 ($mm^2$/min).

The soft clay used was classified as high plasticity silt from the Atterberg limit tests where liquid limit, plastic limit and plasticity index read as 67, 40 and 27(%) respectively.

As shown in Figure 9 (b), the fibre (over 2mm in diameter) content is about 25%. It had been found in the literature that inclusion of fibres increase the strength of soil [2 and 3]. This high amount of fibres will increase the stability of the vertical drains.

## 5. Conclusions

Following conclusions could be made.

The permeability reduces slightly but even with 80% of coir wastes (on volume based), the combined material gives about half of permeability given by sea sand.

The coefficient of permeability gives a linear relation with percentage of coir wastes mixed. Therefore, once the value of $k$ is determined based on site conditions, the required amount of coir wastes mixed can be determined (Figure 14).

Consolidation tests showed that inclusion of vertical drains increased the consolidation rate. Further, it was observed that addition of coir wastes as a filling material to vertical drains gives a reasonable consolidation rate.

As coir wastes make some environmental problems, here it is recommended to use them as filling material to vertical drains.

Based on the experimental results and some literature data on costs of materials [1], it is proposed to use coir percentage about 50% (on weight based) for filling materials. However, depending on *k*, the percentage can de determined as shown in Figure 14.

## References

[1] Kumara, G.H.A.J.J. and Nawagamuwa, U. P., "Application of coir dusts in vertical drains for improvement of soft clay", Proceedings of 2nd International Symposium, Sabaragamuwa University of Sri Lanka, July 8th ~ 12th, 2008.

[2] Mwasha, A.P. (2009), Coir fibre: A sustainable engineering material for the Caribbean environment. The College of The Bahamas Research Journal, 15, 36-44.

[3] Prabakar, J. and Sridhar, R.S. (2002), Effects of random inclusion of sisal fibre on strength behaviour of soil, Construction and Building Materials, 16, 123-131.

# METHODOLOGY TO DEMONSTRATE PILE CAPACITY IN RELAXING GROUND

T. A. Peiris[1], B. Morris[2], R. Kelly[1], K. Chan[3]

[1]Coffey Geotechnics, Sydney, Australia
[1]E-mail: Ashok.Peiris@leicon.com.au, Ashok_Peiris@coffey.com

[2]SMEC, Sydney, Australia

[3]GHD, Sydney, Australia (Formerly Coffey Geotechnics, Sydney, Australia)

**Abstract:** Driven pre-stressed concrete piles have been used as a foundation system to support abutments and piers of a bridge constructed near Ballina in New South Wales, Australia. In order to achieve the required geotechnical capacity, the piles were required to be driven through soft clay and sand to moderately weathered rock. Pile Driving Analyser (PDA) testing together with CAPWAP analysis was performed to assess the integrity and geotechnical capacity of the driven piles. Pile damage was observed during driving. To prevent damage a rock shoe was retrofitted to the piles prior to installation. Reductions in pile capacity (or relaxation) were observed between end of drive (EOD) and later restrike testing (RST). A substantial amount of additional pile testing was performed at different times after driving to assess the changes in pile capacity over time. Most piles were re-driven to achieve higher capacity. Pile capacity could not be achieved in one pier and additional piles were installed to reduce the required pile test load

**Keywords:** Water Conservation, Rainwater, Harvesting

## 1. Introduction

Twin bridges have been constructed as part of the Ballina Bypass Project. The bridges extend over a length of approximately 133m (between centrelines of abutments). Each bridge has four spans of approximately 33m length with a carriageway width of about 10.5m. The bridge alignment is generally orientated on a north-south bearing over estuarine and alluvial soil deposits.

The adopted foundation system for the bridges comprised 550mm octagonal pre-stressed concrete piles. One row of 7 piles was adopted for the abutments and two rows with 5 piles in each row in a staggered configuration were adopted for the piers. In order to achieve the required geotechnical capacity, piles were driven into moderately weathered bedrock.

Some of the piles were damaged during driving. The cause of damage and method used to avoid damage in piles is discussed in the following sections.

In addition, reductions in pile capacity (or relaxation) were observed between PDA end of drive (EOD) and restrike testing (RST) for many of the piles. Data showing relaxation with time are presented and potential mechanisms for relaxation discussed. Methods adopted to overcome pile relaxation and testing performed to meet acceptance criteria are discussed.

## 2. Geological conditions and Model

Subsurface ground conditions have been assessed using borehole drilling and cone penetration testing. The investigation results indicate that the twin bridges are underlain by Holocene age estuarine clay deposits overlying Pleistocene age sand, stiff clay, residual soils and weathered Argillite (interbedded metasiltstone / metasandstone).

The underlying Argillite is a fine grained interbedded metasiltstone and metasandstone. The Argillite can be divided into three sub-units; decomposed clays, highly to moderately weathered Argillite and slightly weathered to fresh Argillite. A long section showing the ground conditions is presented in Figure 1.

The bridge is oriented in a north-south direction. To the west of the bridge lies a hillside that dips steeply toward the bridge location and continues to dip from west to east beneath the bridge location.

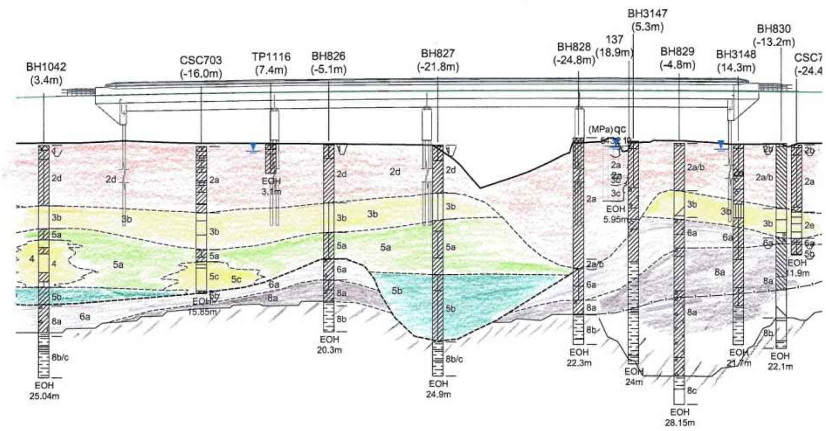



Figure 1: *Geological long section*

The geotechnical model developed for pile design for Pier 2 (northbound) is presented in Table 1 to provide an indication of the soil and rock properties. The geotechnical model shows that the majority of the pile resistance comes from end bearing.

Table 1: *Geotechnical model for pile design*

| Soil unit | Reduced Level at Top of Unit (m) | Description | Undrained Shear Strength, $c_u$ (kPa) | Ultimate Skin Friction, $f_s$ (MPa) | Ultimate Base Resistance $f_b$ (MPa) |
|---|---|---|---|---|---|
| Unit 2d | 0.4 | Very soft high plasticity silty clay. | 10 | 0.01 | - |
| Unit 3b | -5.1 | Loose silty sand. | - | 0.02 | - |
| Unit 5a | -8.1 | Firm clay | 25 | 0.03 | - |
| Unit 5b | -14.1 | Stiff to hard clay | 150 | 0.06 | - |
| Unit 8b | -18.1 | Highly to medium weathered Argillite | - | 0.5 | 15 |

## 3. Pile Type

The foundation system for each support comprised 550mm octagonal driven pre-stressed concrete piles with a precast conical driving tip. Figure 2 shows the geometry of the pile.

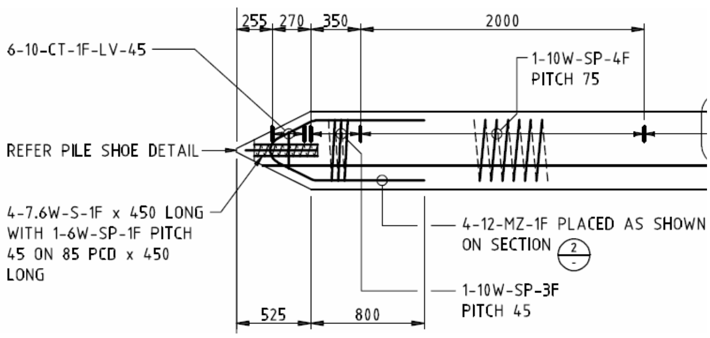



Figure 2: *Precast Octagonal Pile Details*

## 4. Hammer details

Due to the close proximity of the bridge piers to the creek edge a conventional piling rig was unable to be utilised due to potential instability within the near surface soft ground. For the purpose of positioning heavy plant away from the creek edge the piles were driven using a free hammer attached to a modified 50t crawler crane.

The hammer used for this operation was a 6t hydraulic hammer. It applied up to 55kJ of energy with a 1m drop. Sets measured manually varied from 3mm/blow to 0.5mm/blow at end of drive (EOD).

## 5. Testing methodology

Project specific QA specifications cited that 10% of piles (at least 1 test per pier or abutment) should be dynamically tested at EOD and upon re-strike (RST) with CAPWAP signal matching analysis. RST testing is performed a minimum of 12 hours post installation. These piles are termed as representative piles and are driven prior to the remainder of the pile group. The remaining production piles were then installed to the driving parameters set by the representative pile.

In addition to the representative pile testing an additional 5% of the remaining production piles were required to be dynamically tested on RST with CAPWAP signal matching analysis.

Acceptance criteria for pile test loads (or pile capacity) are also stipulated in the project specifications that "the acceptance criteria for a restrike test on a pile are that the driven parameters achieved must be equal or better than those measured at the end of driving and the distribution of resistance along the pile must be effectively unchanged". According to the specification, piles with lower capacity at RST than EOD cannot be accepted.

Australian Standard AS2159 allows the geotechnical strength reduction factor ($\phi_g$) to vary with the amount of testing performed. Based on the above quantum of testing $\phi_g$=0.8 was adopted for the heaviest loaded piles in the group and $\phi_g$=0.7 for the remainder of the piles in the group. The maximum pile test load to be achieved was 3966kN and all piles were intended to be driven to achieve this capacity.

## 6. Pile damage and integrity testing

Representative piles S2-1 (i.e. Pile No. 1 of Pier S2) and S2-10 were installed and PDA tested at EOD. The PDA equipment calculates an integrity factor called BETA. This is based on the change of impedance from one section of the pile to another and is an indication of the reduction in cross sectional area of the pile (Webster, 1996). The PDA data showed that the piles had BETA values of 57% and 78% respectively. The BETA value alone was not the sole indication of pile integrity but was rather used as a guide together with reviewing the force-velocity curve that a pile may be undergoing bending and subsequent loss of integrity at a given point during the driving process. These BETA values together with a review of PDA data indicated that the piles were broken and damaged respectively. PDA testing on these piles was applied after the pile had been driven some

distance into the founding stratum and so the stage of driving where damage initiated was not identifiable.
For the remainder of the piles PDA testing was initiated early in the drive whilst the pile was still being driven through the upper weak soil layers. This was undertaken to allow real time monitoring of the pile integrity and inferred capacity as the pile toe penetrated the founding stratum. For each pile the force-velocity curve of the dynamic impact was viewed for each blow and an assessment of the pile integrity was made as the pile was being installed.
Additional boreholes were drilled at Pier 2 to further investigate the ground conditions and to gain understanding of the bedrock conditions across Pier 2. The subsurface profile was identified to generally consist of weak upper soil layers followed by a rapid transition into high strength bedrock. Following a review of the borehole information and the PDA data on initial S2 piles it was assessed that these piles may have been damaged as a result of a tendency for the conical driving shoe to slip down the surface of the bedrock prior to penetrating the founding stratum. Bending is likely to occur in the pile between the free tip and the pile being held within a guide frame at ground surface level. It was assessed that there was a high level of risk of piles in Pier N2 and S2 groups being subjected to this mechanism and remedial action would be necessary to enable the pile to drive vertically into the founding stratum.
The original conical driving shoe was not suitable to allow the pile to vertically penetrate such a distinct bedrock surface. A steel rock driving shoe was introduced which would encase the pile toe and provide a pile base surface that would minimise the opportunity for the pile toe to deviate from its position caused by following the contours of the bedrock.
The conical toe of all piles thereafter was cut off to suit the expected founding level. The purpose of this was to remove any unnecessary length of pile that may “whip” during driving caused by the effect of the hammer impact and the low resistance in the weak upper soil layers. It was not clear that pile whip was affecting the pile capacity but the piles were cut as short as possible to minimise potential whip effects. The piles were then subsequently retrofitted with a pre-fabricated steel rock driving shoe which comprised of a steel sleeve and a stiffened steel end plate fitted to the end of the pile using epoxy resin. Figures 3 and 4 show section and end detail respectively of the modified rock driving shoe.

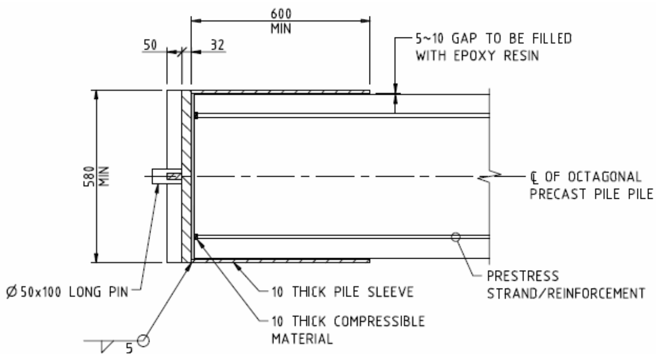



Figure 3: *Rock Driving Shoe Section*

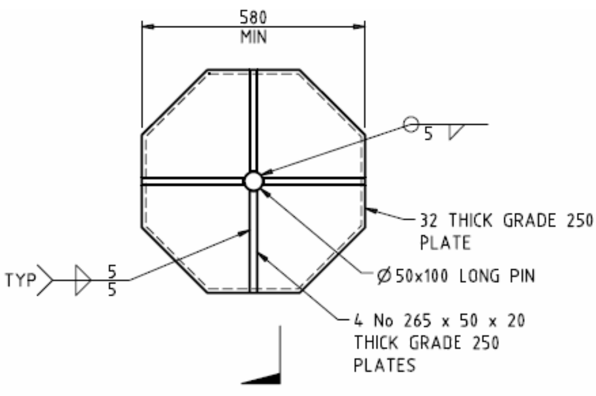



Figure 4: *Rock Driving Shoe Details*

Review of PDA test data during installation and subsequent review of CAPWAP signal matching analysis showed that the integrity of the modified piles was good with BETA values of 100% on all tests. Following adoption of the revised rock driving shoe no loss of integrity was observed in any of the piles.

## 7. Pile Relaxation

### *Observations of Relaxation*

Pier 3 piles were the first piles to be installed at the bridge and it was at this location that pile relaxation was identified. These piles did not have problems with integrity during driving and were installed using the conical tip. Pier 3 piles were originally driven to just beyond the pile test load and upon RST testing were found to have relaxed below the test load. Piles were then subjected to additional driving to significantly higher capacities in excess of the pile test load to account for the extent of relaxation.
Given the experiences of pile capacity relaxation at Pier 3 the approach for driving Pier 2 piles was adopted such that piles would be over driven to account for the extent of relaxation. In the northbound Pier, piles N2-1, N2-3 and N2-5 were selected for extensive PDA testing over time since their locations span the Pier N2 pile group and provide a reasonable representation of the relaxation behaviour. These piles were driven to final EOD capacities between 4,490kN and 4,670kN. RST testing was performed over a period of up to 63 days which showed that the piles had relaxed then stabilised in capacity. Results of the testing are shown in Fiure 5. The maximum measured relaxation was 840kN. CAPWAP analysis was performed on selected tests and the analyses demonstrated similar capacity to PDA estimates. The results of testing show that relaxation did occur however the final pile capacities were in excess of the individual pile test load. Once it was assessed that piles were relaxing, the piles were driven to achieve a PDA field estimated capacity in excess of the required pile test load. In some cases piles were re-driven up to 1m to achieve the desired PDA inferred capacity which was up to 1500kN above the individual pile test load. Samson and Authier (1986) report a case where piles were re-driven up to 0.3m at RST to regain the required pile resistance.

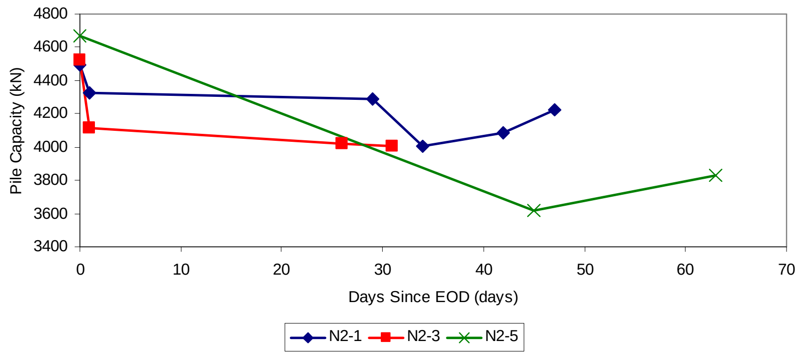



Figure 5: *N2 Relaxation Behaviour*

In the southbound pier, piles S2-5 and S2-10 were selected for extensive testing as they had the lowest PDA capacities at EOD. Piles S2-3, S2-6 and S2-8 were also selected for additional PDA testing over time due to their locations along the pier group and thus providing a reasonable representation of the relaxation behaviour. CAPWAP analysis was performed on selected tests and the analyses demonstrated similar capacity to PDA estimates. The results of testing show that relaxation occurred and that the final pile capacities were in excess of the individual pile test load. Figure 6 illustrates the pile relaxation behaviour of Pier S2 piles and demonstrates the trend for stabilisation of relaxation with time.

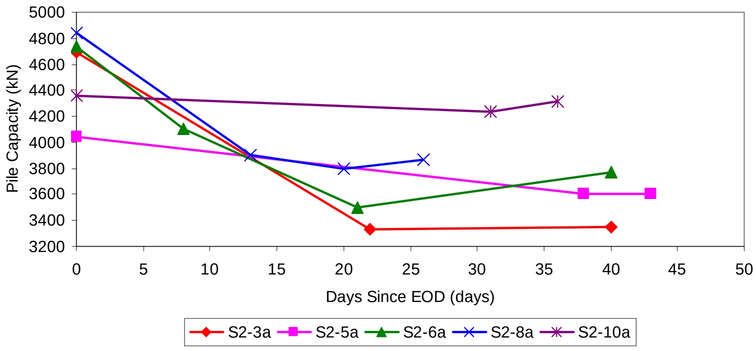



Figure 6: *S2 Relaxation Behaviour*

The available PDA data shows relaxation occurring over approximately 20-30 days before stabilising. The CAPWAP analyses for tested piles suggested that relaxation occurred on the shaft capacity and conversely the base capacity increased with time as shown in Figure 7a and Figure 7b respectively.

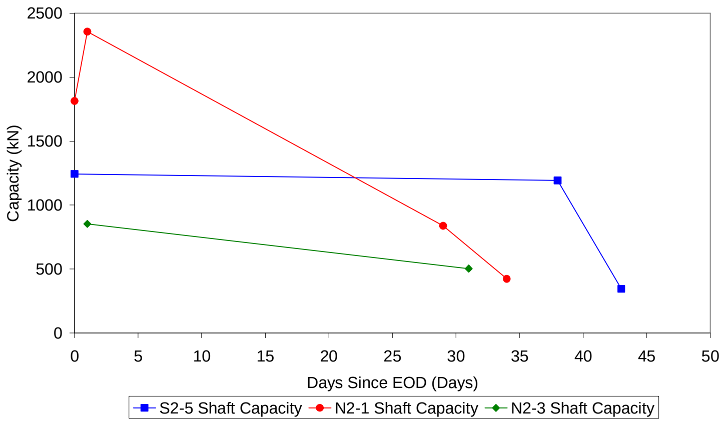



Figure 7a: *Pier 2 Shaft Capacity Change with Time*

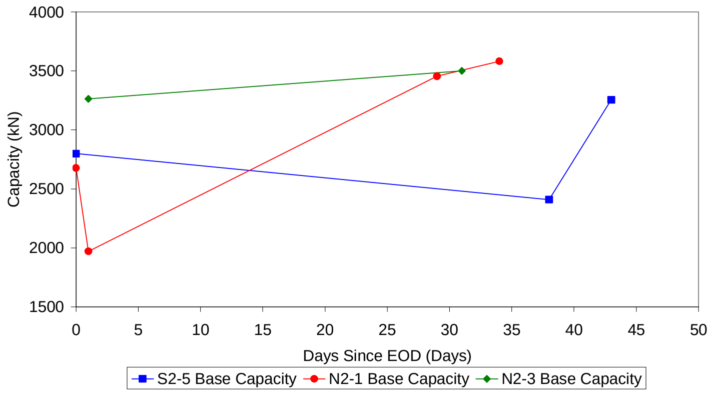



Figure 7b: *Pier 2 Base Capacity Change with Time*

### *Potential Reasons for Relaxation*

An assessment of reductions in pile capacity between PDA EOD and 1st RST testing of pier piles is presented in Table 2. Table 2 illustrates similar relaxation range (as a percentage) for all pier piles. Given that the conical shoe was used in Pier 3 and the rock shoe used in the other piers it appears that the type of driving shoe did not affect pile relaxation.

Moller and Bergdahl (1981) suggested that during driving displacement piles into very dense sand dilation causes negative pore pressures are developed giving a temporary increase in effective stresse causing a temporary increase in capacity and that the effect of relaxation occurs due to the reduction in the effective stress with time as the pore pressures dissipate (York et al. 1994). The founding bedrock at ECC Bridge consists of a fine grained metasandstone and it may be possible that as the piles were driven into the bedrock the material has fractured to some degree and under this change in rock structure development of negative pore pressures may have occurred.

Table 2: *Reductions in Pile Capacity Between PDA EOD and 1st RTS Testing*

| Pier | EOD capacity | 1st RST capacity | Relaxation (%) |
|---|---|---|---|
| 1 | 4075kN to 5142kN | 3000kN to 4500kN | 9 to 30 (average of 20) |
| 2 | 4057kN to 5157kN | 3066kN to 4920kN | 4.6 to 26 (average of 13) |
| 3 | 4125kN to 4821kN | 2959kN to 4316kN | 1 to 32 (average of 14) |

Samson and Authier (1986) presented two case studies where reduction in capacity of toe bearing piles on shale bedrock has been observed. These piles were driven through soft strata into hard rock which is a similar stratigraphy to that encountered at this bridge.
Low shaft friction and high end bearing may be ground conditions that favour pile relaxation.

## 8. Pile group design revision

The 2 representative piles on Pier S2 were disregarded from the pier group after it was confirmed that the piles were damaged beyond acceptable contribution to the group. Consequently a new pile layout was adopted by flipping the original layout plan to enable installation of 10 piles (S2-1 to S2-10) into new locations whilst not being obstructed by the 2 disregarded piles. The new configuration of Pier S2 is shown in Figure 8.

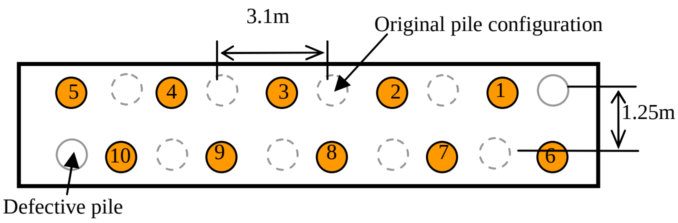



Figure 8: *Revised Pier S2 Pile Configuration*

Relaxation made achieving the original pile test load difficult. Continually redriving and retesting the piles was taking time and costing money. There was a high risk that the piles would not achieve the required pile test load and as such an additional two piles were incorporated into each pile group to reduce the individual pile loads. The final pile configurations for Pier S2 and Pier N2 are as shown in Figure 9a and Figure 9b respectively.

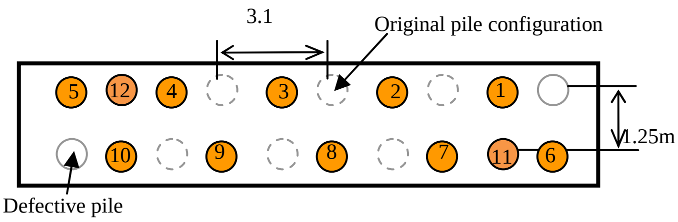



Figure 9a: *Revised Pier S2 Pile Configuration with 12 Piles*

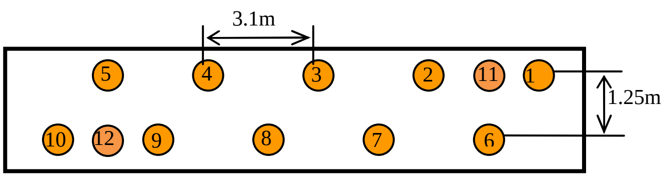



Figure 9b: *Revised Pier N2 Pile Configuration with 12 Piles*

Foundation designs for the revised pile group configurations were carried out. Table 3 shows the revised ultimate limit state loads and the associated individual pile test loads (i.e. maximum ultimate limit state load/appropriate geotechnical reduction factor).

Table 3: *Revised Pile Test Loads for Pier 2*

| Pier N2 | | | Pier S2 | | |
|---|---|---|---|---|---|
| Pile No. | ULS Load (kN) | Pile Test Load (kN) | Pile No. | ULS Load (kN) | Pile Test Load (kN) |
| N2-1 | 2800 | 3500 | S2-1 | 2850 | 3563 |
| N2-2 | 2330 | 3329 | S2-2 | 2580 | 3225 |
| N2-3 | 2230 | 3186 | S2-3 | 2320 | 2900 |
| N2-4 | 2080 | 2971 | S2-4 | 2350 | 3357 |
| N2-5 | 2420 | 3457 | S2-5 | 2740 | 3425 |
| N2-6 | 2420 | 3457 | S2-6 | 2740 | 3425 |
| N2-7 | 2080 | 2971 | S2-7 | 2350 | 3357 |
| N2-8 | 2230 | 3186 | S2-8 | 2320 | 3314 |
| N2-9 | 2330 | 3329 | S2-9 | 2580 | 3225 |
| N2-10 | 2800 | 3500 | S2-10 | 2850 | 3563 |
| N2-11 | 2420 | 3457 | S2-11 | 2850 | 3563 |
| N2-12 | 2420 | 3457 | S2-12 | 2850 | 3563 |

## 9. Acceptance of pile capacity

***Pile Capacity Acceptance Procedure***

The procedure for accepting piles was developed according to the following sequence:

i) Revised pile test loads were allocated to each individual pile based on the pile group analysis. Individual pile test loads are shown in Table 1;

ii) Selected piles were extensively RST tested over a period of time to establish the extent of relaxation, that relaxation ceases over time and to show that the pile capacity at the lower bound of relaxation is above the individual pile test load;

iii) Spot checks were performed by RST testing other selected piles to demonstrate the pile capacity is in excess of the individual pile test load;

iv) For piles without RST testing the lower bound pile capacity has been represented by nearby piles which have similar EOD capacities to those which have been extensively RST tested.

Shown in Table 4 is a summary of the developed categorization of piles according to the acceptance criteria listed above.

Table 4: *Categorization of Pier 2 Piles with respect to Acceptance Criteria*

| Category | Description | Pier N2 | Pier S2 |
|---|---|---|---|
| A | Piles subjected to RST testing which demonstrate stabilisation of relaxation above the individual pile test load | N2-1<br>N2-3<br>N2-5 | S2-3<br>S2-5<br>S2-6<br>S2-8<br>S2-10 |
| B | Piles which were subjected to RST testing a similar time after EOD to those in Category A to demonstrate pile capacity in relation to individual pile test load | N2-4<br>N2-8<br>N2-9<br>N2-12 | S2-1<br>S2-2<br>S2-9<br>S2-11 |
| C | Piles which were only subjected to EOD testing at final drive of which are represented by nearby piles in Category A | N2-2<br>N2-6<br>N2-7<br>N2-10<br>N2-11 | S2-4<br>S2-7<br>S2-12 |

***Northbound Pier (N2) Piles***

Piles N2-4, N2-8, N2-9 and N2-12 were selected for RST testing between 24 and 63 days post installation. The piles were driven to final EOD capacities of between 4,530kN and 4,860kN. Results of the RST tests show that the piles relaxed a maximum 830kN to level in excess of their individual pile test loads. The amount of relaxation on these piles was shown to be less than the maximum relaxation demonstrated by Category A piles. These piles were not subjected to ongoing RST testing but demonstrated that over time the extent of relaxation is similar to Category A piles.

The remaining piles in the group were subjected to EOD PDA testing only. These piles were driven to EOD capacities of between 4,570kN to 4,770kN and were considered to be represented by the nearby Category A piles which were driven to a similar yet lower EOD capacity. The extent of relaxation shown by piles in Category A demonstrated that with similar relaxation the final capacities of the above piles were in excess of their individual pile test load.

***Southbound pier (S2) piles***

Piles S2-1, S2-2, S2-9 and S2-11 were selected for RST testing between 8 and 46 days since installation. The piles were driven to final EOD capacities of between 4,700kN and 4,830kN. Results of the RST tests show that the piles relaxed a maximum of 1,100kN to levels in excess of their individual pile test loads. The RST tests performed on these piles showed that the amount of relaxation was less than the maximum relaxation demonstrated by Category A piles, their behaviour over time was similar to Category A piles and that their capacities were greater than the individual pile test loads.

The remaining piles in the group were subjected to EOD PDA testing only and were driven to EOD capacities of between 4,850kN and 4,910kN. These piles were represented by the nearby Category A pile S2-8 which was driven to a similar yet lower EOD capacity of 4,840kN. Findings of the extensive testing on pile S2-8 showed that the relaxation has stabilised at a level above the individual pile test load for these 3 piles.

## 12. Concluding remarks

Piles were driven through challenging ground conditions that created problems with pile relaxation and attainment of pile capacity. Piles with precast conical driving shoes are not suited to being driven through low strength soil deposits onto a high strength sloping rock surface. Under these conditions the retrofitted ribbed flat plate driving show was found to allow the pile to be installed without being damaged.

Ground conditions where limited shaft capacity and a strong fine-grained founding stratum exist appear to be associated with pile relaxation. The mechanism for pile relaxation is not clear but may be associated with dissipation of negative pore pressure around the base of the pile.

The type of driving shoe did not appear to affect the magnitude of relaxation.

To avoid retesting every pile over a period of time acceptance criteria have been developed to relate piles tested at EOD to the performance of piles tested over a longer time period. However, many piles require testing over a long period in order to develop confidence that the less tested piles will achieve the design intent.
Continual redriving and retesting of piles to demonstrate capacity in relaxing ground is a time consuming process. In these circumstances installation of additional piles to reduce the pile test load provides the fastest method to achieve design criteria. The time savings are reflected in cost savings.

## Acknowledgements

The authors would like to acknowledge the Roads and Traffic Authority of New South Wales, Australia for permission to publish data from the Ballina Bypass.

## References

43. Moller, B., and Bergdahl, V. (1981). “Dynamic pore pressure during pile driving in fine sand.” Proc., 10th Int. Conf. on Soil Mech. And Found. Engrg., A. A. Balkema, Rotterdam, The Netherlands, 2, pp 791-794.
44. Samson, L., and Authier, J. (1986). “Change in Pile Capacity with Time: Case Histories.” Can. Geotech. J. 23, pp 174-180.
45. Webster, S., Teferra, W., September, (1996). Pile Damage Assessments Using the Pile Driving Analyzer. Proceedings of the Fifth International Conference on the Application of Stress-wave Theory to Piles 1996: Orlando FL; 980 – 990.
46. York, D., Brusey, W., Clemente, F., and Law, S. (1994). “Setup and Relaxation in Glacial Sand.” Journal of Geotechnical Engineering, Vol 120, No. 9, pp 1498-1513.

## About the Authors

**T. A. PEIRIS,** B.Sc. Hons. Moratuwa., M.Sc. London, DIC, D.Eng. Tokyo, MIE (SL), C.Eng. Senior Geotechnical Enginner, Coffey Geotechnics, Sydney, Australia.

# MODEL STUDY ON "PENDULOR" TYPE WAVE ENERGY DEVICE TO UTILIZE OCEAN WAVE ENERGY IN SRI LANKA

Gunawaradane S.D.G.S.P[1], Abeysekara M.P[2], Uyanwaththa, D.M.A.R[3], Tennakoon, S.B[3] , Wijekoon[3], W.M.J.S, Ranasinghe R.A.P.C[3]

[1] Senior Lecturer, Department of Mechanical Engineering, University of Peradeniya
[1]E-mail: sdgspg@pdn.ac.lk, ; sdgspg@gmail.com,
[1]Telephone: +94-81-2393610 ; Fax: + 94-81-2393610

[2] Postgraduate Student, Department of Energy Technology, KTH University, Sweden
[2]E-mail: muditha.abeysekera@gmail.com

[3] Department of Mechanical Engineering, University of Peradeniya, Sri Lanka
[3]E-mail: darkera87@gmail.com, Telephone : +94779147180

**Abstract**
Sri Lanka being an island it is inevitable to utilize the ocean wave energy as a sustainable energy source. The device called "Pendulor", which is invented in Japan and has proven 40-50% of wave energy conversion efficiency at the sea of Muroran, Japan. The low wave energy density oceanic climate of Sri Lanka makes this device more preferred, because of its high energy conversion capability. The device consist a pendulum flap hinged inside a caisson and the caisson is faced to the ocean waves. The economical feasibility of this device depends on the construction cost. The conventional caisson is a straight chamber and this type of a device is infeasible because of the length that causes increase in construction cost. A modified caisson is proposed to shorten the caisson length towards the sea and to increase the frequency bandwidth of operation.

Model testing of this modified caisson with a solid friction damper is addressed in this endeavor. Laboratory experiments were conducted with a 1/16 scale model of the proposed caisson. A solid friction damper was used for the model testing and a mathematical model for the device was developed and the parametric modeling was done by using MATLAB. The power conversion efficiency with a modified caisson was discussed in this endeavor.

***Key wards:*** *"Pendulor", Wave energy, caisson, excitation moment*

## 1. Introduction

The "pendulor" device is invented in Japan. The device consists of a flap hanged inside a caisson which faces towards the ocean waves (Fig 1). The flap is driven by the horizontal water particle motion of the waves. The device operates on standing waves which are created by superposition of the incident waves and reflected waves from the back wall. This excitation moment ($M_o$) of the pendulum depends on the caisson configuration as well as the wave frequency (f) and wave height (H). A rotary vane pump is driven by this pendulum and it drives an oil motor which drives a generator.

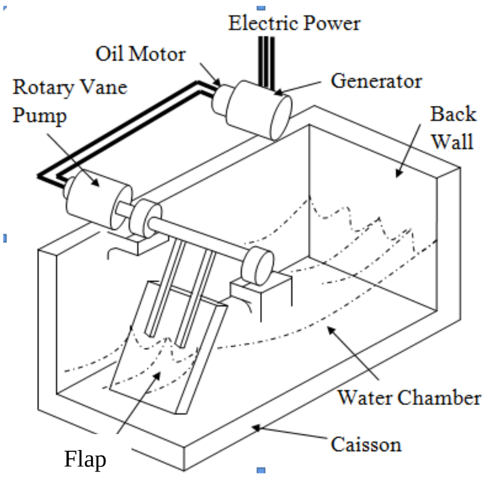



*Fig 1 Basic pendulum device*

The flap is placed at the node point of the standing waves (Fig 2), where the water particle motion is completely horizontal. The node point is at a distance d from the caisson back wall. Where d = $\lambda/4$ for a wave inside a straight caisson, ($\lambda$ = wave length).Therefore the chamber length depends on the wave length.

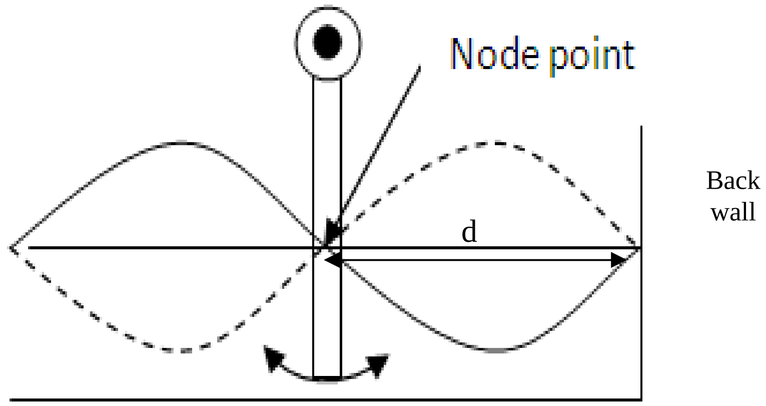
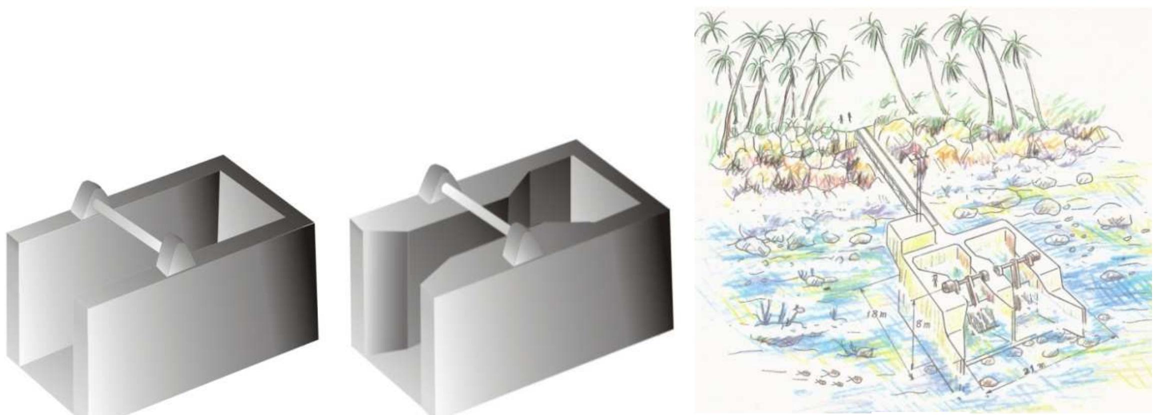



*Fig 2 Node point and pendulum placement*

**Any sustainable system needs to be ecologically viable and economically feasible and socially accepted.**

In Sri Lankan ocean climate, the swell wave length is around 100m causing the operating point of flap to be placed at a 25m distance away from the back wall and increases the associated construction cost. A new modified caisson is proposed to shorten the caisson length [3], where a converging section is introduced at the mouth of the chamber and a diverging section is introduced at the back of the caisson (Fig 3). This converging section is expected to create a horizontal particle motion inside the caisson which can drive the flap. Another expected behavior of the modified caisson is to have a broader frequency bandwidth of operation than the straight caisson.

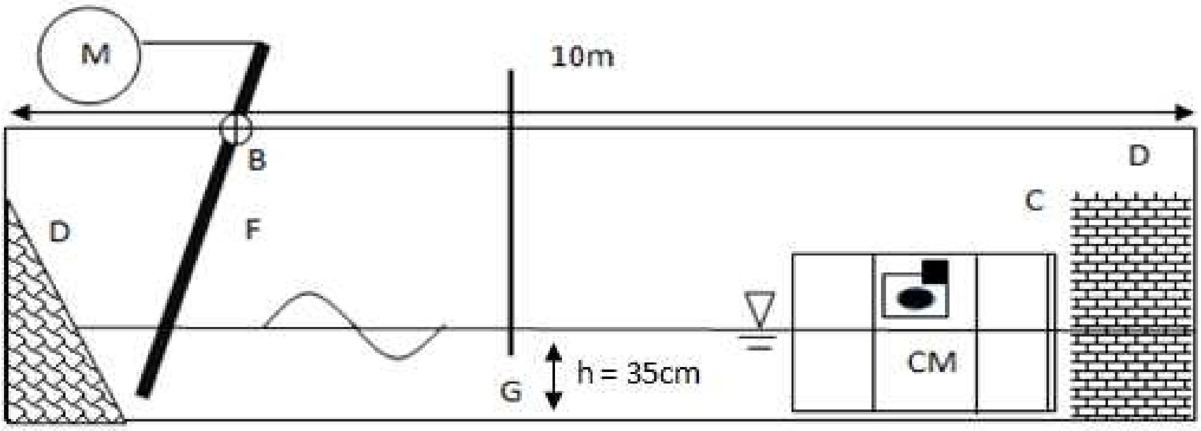


*(a)* *(b)* *(c)*

*Fig 3 Two caisson types (a) straight caisson, (b) modified caisson,*
*(c) An artistic view of proposed power plant (Watabe)*

### *1.1 Node point variation inside the caisson with the wave frequency*

The incoming wave length changes with the time due to the randomness of the ocean , therefore the node point varies and the flap placement point deviate from the node point hence affect the device efficiency. To investigate the node point variation inside the modified caisson laboratory experiments were conducted with a 1/16 scale model of the proposed modified caisson. A visual method was used to identify the node point of the standing waves created inside the caisson




*B: Bearing, C: Caisson, D: Dampers, F: Wave Generating Flap, G Wave Gauge*
*M: Motor, CM: Camera*
*Fig 4 Experimental set up*

Wave patterns inside the caisson were observed and snapshots were taken to identify the node points. The node point was identified and the distance from the back wall was measured.
A partial standing wave was created inside the modified caisson due to its geometric construction. The incident and the reflected waves are different due to the effects of reflections and diffractions at the converging and diverging sections

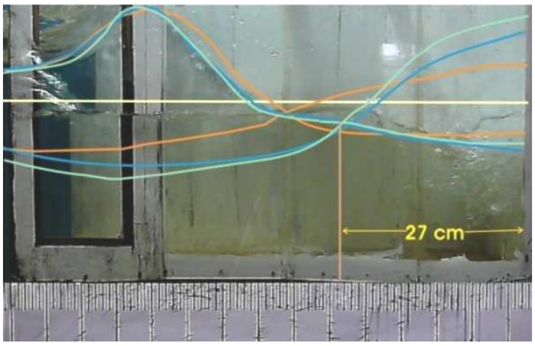



*Fig 5 Node point variation inside the modified caisson for T = 0.9s*

### *1.2 Node point variation inside straight caisson*

As shown in Fig05 it was evident that the modified caisson has a lesser nodal point distance from back wall to the nodal point. This implies the effect of the modification is beneficial on the device construction because the caisson length can be reduced. Since the pendulum has to be placed at the node point, it is implied that, lesser the node point deviation broader the operating frequency bandwidth. From the fig 5 it is visible that the gradient of the modified caisson is lesser than the straight caisson implying that the operating frequency bandwidth is broader in the modified caisson.
The length reduction compared to straight caisson is about 30-40% in the considered frequency range.
The predicted device efficiency behavior for the pendulor model is shown in (Fig 6)[2]

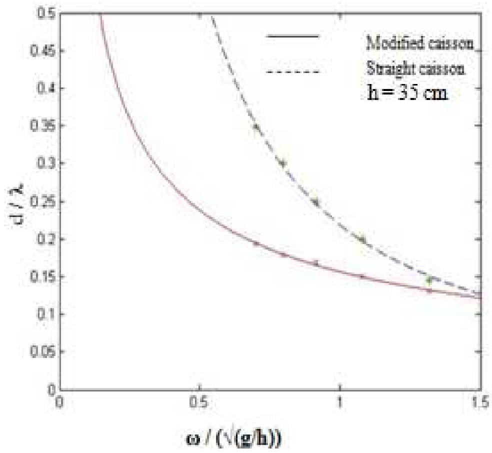



*Fig 6 Node point variation for the modified and straight caisson.*

## 2. Mathematical modeling of the pendulor device with modified caisson

The mathematical model for the device with a straight caisson under regular type waves were used in the analysis of the modified caisson [1]. The actual working model of a pendulor device consist a viscous damper (hydraulic pump), but the experiments were conducted under a constant friction damper.

Disc brakes were used as the mechanical damper of the system and the mechanical damping torques were changed by changing the applied pressure on the friction plate. It was assumed that the disc brake plate gives a constant torque at a constant applied pressure on it.

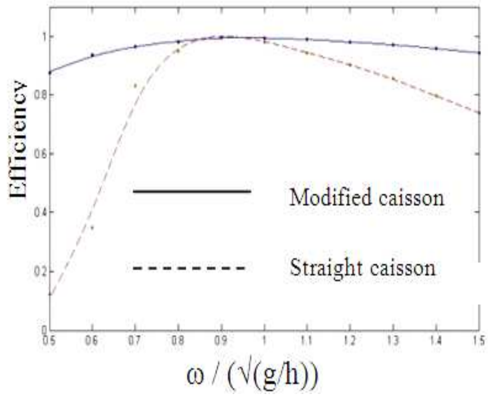



*Fig 7 Predicted device performance*

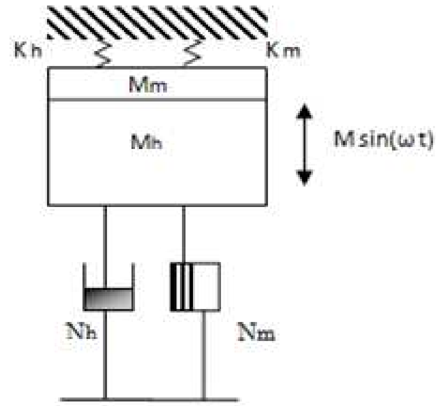
Model for the constant damping brake torque device

$$\sum I\ddot{\theta} + (N_h)\dot{\theta} + N_m + \sum K\theta = M_0 \sin \omega t$$

Where $\sum I = I_h + I_m$, $\sum K = K_h + K_m$

$I_m$: Moment of inertia of the flap, $I_h$: Moment of inertia of the added water mass,

$N_m$: Mechanical Damping factor, $N_h$: Hydrodynamic damping Factor, $K_m$: Coefficient of restoring moment by the flap gravitation, $K_h$: Coefficient of restoring moment by the water mass elevation, *Mo*: wave excitation moment.




h- Hydrodynamic, m – mechanical

*Fig 8 Experimental set up with a mechanical brake (constant damping torque)*

For a straight caisson, the relationship between the above parameters and the wave parameters were discussed in literature [1]. The change of the water chamber cross section changes the velocity potential of the wave inside the caisson. The added moment of inertia and the hydrodynamic damping and the restoring moment from the added water mass elevation depend on the caisson configuration and frequency. Therefore the dynamic system parameters vary from the straight caisson parameters and the complex shape of the modified caisson makes the theoretical investigation complex. Therefore an experimental method was used to investigate the effect of straight caisson.

The input was varied by changing the $M_0$ (changing the input wave), and the output $\theta$ was measured while the $N_m$ (Mechanical damping) was at a constant brake torque value for regular waves. Then the parameters were found by minimizing the error between the measured values and the model values.

### *a. Parametric modeling of the Pendulor device with constant brake torque (Non linear damping torque)*

The output of the system was the flap rotation (θ) and the input of the system was the moment of excitation by the waves inside the caisson ($M_o$).Least square method was used to minimize the square of error between the model values and the measured system values, then optimization techniques were used to optimize the parameters of the model equation to fit the experimentally measured output data. MATLAB programs were used to optimize and find the system parameters (Fig 9).

The MATLAB model was used to obtain Numerical Values for the following parameters

$$\frac{N_h}{\sum I}, \frac{N_m}{\sum I}, \frac{\sum K}{\sum I}, \frac{M}{\sum I}$$

Following parameters of the system were measured.

Mass of the flap (m):2.12kg, distance to the center of gravity of the flap (lg) - 0.95m

Therefore
Inertia of the flap $I_m$ – 1.91 kgm$^2$
Restoring moment by gravity of the flap $K_m = l_g mg$ = 19.75 Nm

These calculated values were used with the numerical values of the paramters to find the Parametric Model $9.9\ddot{\theta} + 0.33\dot{\theta} + N_m + 23.3\theta = 15.17\sin(3.60t)$

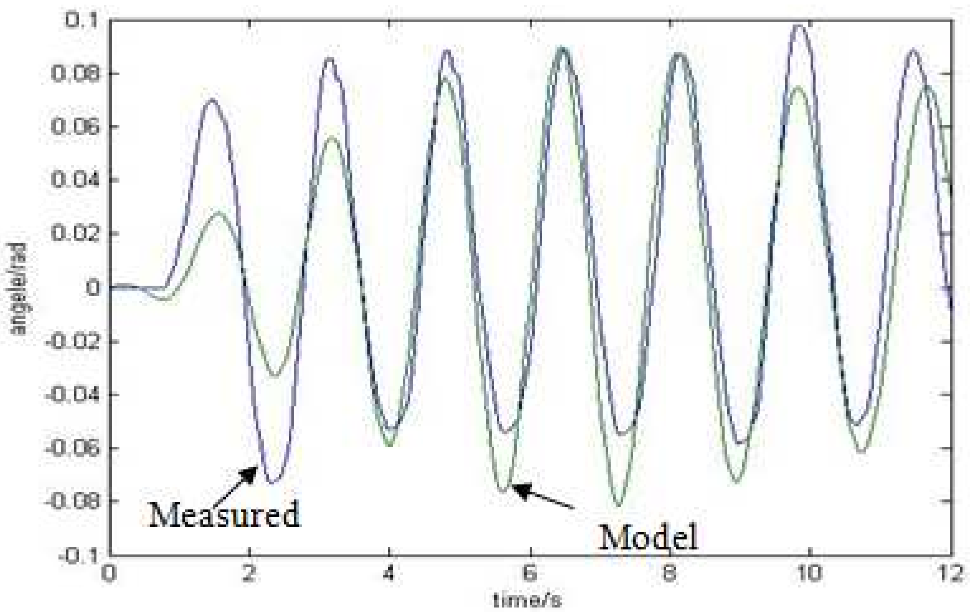



Fig 9 - model and the measured values for T = 1.75s

***b. Variation of excitation moment with the frequency***

The excitation moment is influenced by the incoming wave frequency. The excitation moment $M_o$ was found from the Mathematical Model and the excitation moment variation for the modified caisson was investigated (Fig 10).

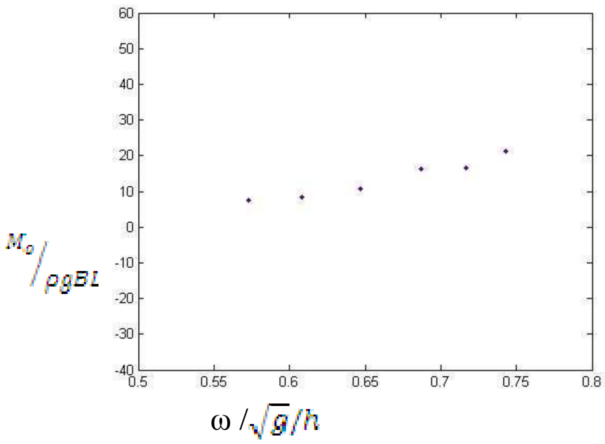



*Fig 10 – The variation of excitation moment with the wave frequency*

Where, ρ: density of sea water, *B*: width of the mouth of the caisson, *L*: wave length, *H*: wave height and *h*: water depth.

***2.1 Power conversion efficiency of the Pendulor device, with a modified caisson.***

The applied brake torque (N) was varied for different wave frequencies. (Fig11)

The work done per cycle $W = N.\theta$

$\theta$, the flap swing angle per cycle.

From Linear Wave Theory, the energy content of an incoming wave for a unit width

$$E = \frac{1}{8} \rho g H^2 L$$

Power conversion ratio for a cycle $\eta = \frac{W}{E}$

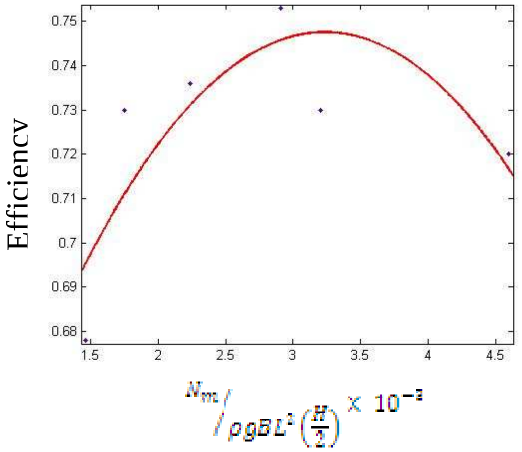



*Fig 11 Efficiency Vs damping resistance for wave period 1.35s*

The applied mechanical damping torque ($N_m$) was varied for different wave frequencies to obtain the resonance (damping matching and frequency matching conditions) for the device to perform with maximum power conversion. The device efficiency changes with the Mechanical damping torque and the incoming wave frequency.

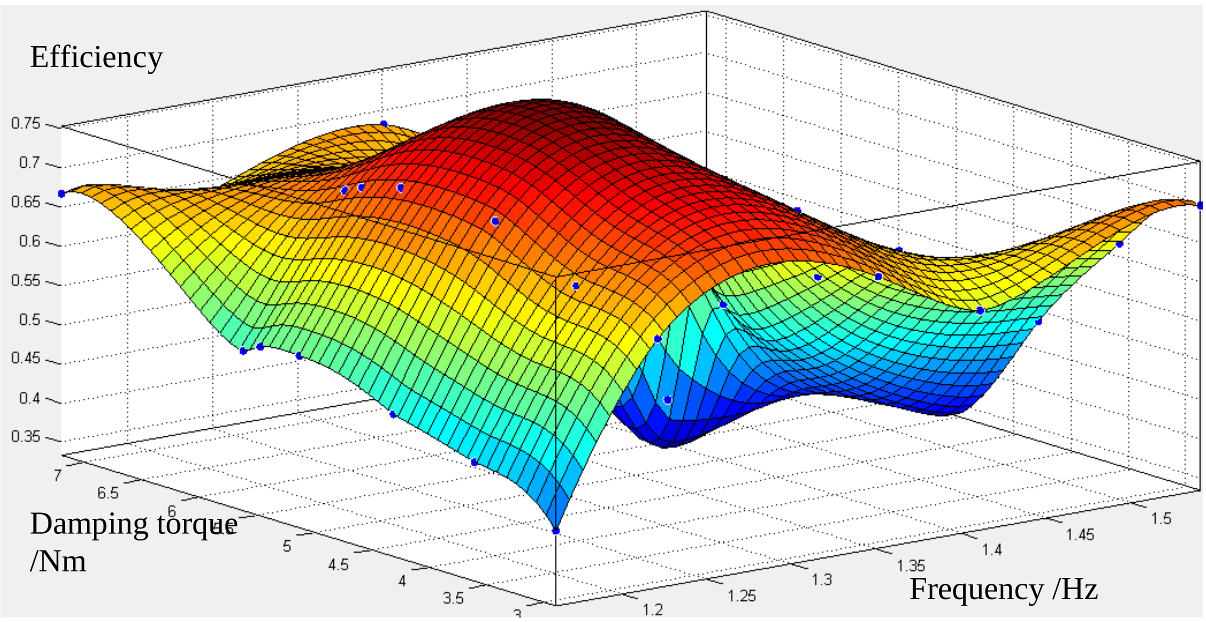



*Fig 12 Device performance and operating frequency bandwidth*

## 3. Conclusion

The Pendulor device with the modified caisson has proven caisson length shortening capability of about 40-30%. The device has proven 70% wave power conversion efficiency at the experimental model with the solid friction damper. The economical feasibility of the pendulor system depends upon the construction cost as well as the device performance. The operating frequency bandwidth of this device is expected to be broadened by the introduction of this modified caisson. The modified caisson has to be compared with the straight caisson for this frequency range and investigate the device performance with the modified caisson. The further studies on the performance with the modified caisson are currently being conducted.

**References**

1. *Utilization of the Ocean wave energy , 2007.* Tomiji WATABE
2. Ando A., Ohtani S., Takagi M., Kuroi M., Kondo H., Yano K., Watabe T., On a Flap TypeWave Energy Converter, Proceedings of the Engineering Committee on Oceanic Resources (ECOR) International conference, 1984 Oct.
3. Tomiji Watabe, Hirotaka Yokouchi, S.D.G.S.P. Gunawardane, "Heave and Pitch Buoy (HPB), A miniature Wave Energy Converter", Proceeding of the Twelfth (2002) international offshore and Polar Engineering Conference, Kitakyushu, Japan, May 26-31, 2002, pp. 649-654.

**About the Authors**

- Dr S.P Gunawardane, **PhD** : Muroran Institute of Technology, Japan,**MEng** : Muroran Institute of Technology, Japan, **BScEng:** University of Peradeniya, Research interests : renewable energy studies mainly on small wind power and ocean wave energy systems, sound and vibration problems and machine dynamics.
- Abeseykara M.P , currently reading for MSc at the department of Energy Technology , KTH , Sweden.
- Uyanwaththe. D.M.A.R, Tennakoon S.B, Wijekoon W.M.J.S, Ranasinghe R.A.P.C, graduates of Mechanical Engineering from the University of Peradeniya.

# ENERGY PLANNING MODEL FOR SUSTAINABLE DEVELOPMENT

Bimlesh Kumar[1]

[1]Assistantr Professor, Department of Civil Engineering, Indian Institute of technology Guwahati, Guwahati-781039, India.
[1]E-mail: bimk@iitg.ernet.in
[1]Telephone: +91-361-2582420; Fax: + 91-361-2582440

**Abstract:** The objective of this study is to develop a mathematical model for the effective utilization of renewable energy sources in a developing country like India. DEP is one of the options to meet the rural and small scale energy needs in a reliable, affordable and environmentally sustainable way. The main aspect of the energy planning at decentralised level would be to prepare an area-based DEP to meet energy needs and development of alternate energy sources at least-cost to the economy and environment. The geographical coverage and scale reflects the level at which the analysis takes place, which is an important factor in determining the structure of models. DEP planning involves multiple objectives and different kinds of constraints Present work present the methodology for the DEP. The kinds of objective functions and constraints which have to be included in the DEP have been presented in the present work. Present developed model has been applied to a typical Indian block unit, which comprises of several villages. Based on the analysis made in the present work, it is found that biomass-based energy systems have the potential to meet all the energy needs of Kunigal block.

**Keywords:** Decentralized Energy Planning; Energy planning; Goal Programming; Scenarios; Sustainable Energy Development Scenario

## 1. Introduction

Energy is universally recognized as one of the most significant inputs for economic growth and human development. The growth of a nation, encompassing all sectors of the economy and all sections of society, is contingent on meeting its energy requirements adequately. Efficient use of energy and utilization of renewable energy sources are the orders of the day when it comes to mitigating greenhouse-gas emissions and, thus, the risks of possible global climate change with unpredictable consequences for our current way of life [1, 2].

The current pattern of commercial energy-oriented development, particularly focused on fossil fuels and centralized electricity generation, has resulted in inequities, external debt and environmental degradation. For example, large proportions of rural population and urban poor continue to depend on low quality energy sources and inefficient devices, leading to low quality of life. The current status is largely a result of adoption of centralized energy planning, which ignores energy needs of the rural areas and poor, and has also led to environmental degradation due to fossil fuel consumption and forest degradation. As suggested by Reddy and Subramanian [3] and Ravindranath and Hall [4], Decentralised energy planning (DEP) is one of the options to meet the rural and small scale energy needs in a reliable, affordable and environmentally sustainable way. DEP is a concept of recent origin with limited applications. Literature shows that different models are being developed and used worldwide. The central theme of the energy planning at decentralised level would be to prepare an area-based DEP to meet energy needs and development of alternate energy sources at least-cost to the economy and environment. Ecologically sound development of the region is possible when energy needs are integrated with the environmental concerns at the local and global levels [5]. Taking into account these features, present work explains the methodology adopted for DEP by estimating the end-use energy requirement and energy resource in the region. Present work also tries to estimate the end-use energy requirement and energy resource in a typical block unit of India.

## 2. Models for Decentralized Energy Planning

After presenting the resources available technologies, constraints and data availability for the region, this section deals with formulating a DEP model with multiple objectives and constraints to develop an optimal energy plan at different scales is presented in this section. Energy-planning involves finding a set of sources and conversion devices so as to meet the energy requirements of all the activities in an optimal manner. This optimality depends on the objective; such as to minimize the total annual costs of energy or minimization of non-local resources or maximization of system overall efficiency. Factors such as availability of resources in the region, costs and energy requirements specific activities impose constraints on the regional energy planning exercise. Thus, the DEP turns out to be a constrained optimization problem. ‘Optimization’ refers to the generation of the best result given certain constraints and circumstances as introduced by the programmer [6, 7]. All optimization models have an objective function, a function one is trying to optimize and constraint functions, those functions that place a limit on the objective function. Optimization modeling therefore allows one to set certain constraints, and within those constraints determine an optimal function.

### *2.1 Goal programming model (GP)*

All the linear programming models developed so far had a single over-riding objective, such as maximizing the revenue of the study area or maximizing the biomass energy production or minimizing the total cost of energy sources or maximizing the generation of surplus biomass, etc. However, in reality, achieving such a single objective through mathematically feasible, the outputs have little utility. Very often optimizing an energy system could involve multiple objectives namely minimizing the cost, maximizing use of local energy sources, maximizing employment, reducing emission of pollutants, etc. Thus an approach or model to optimize multiple objectives for a given set of constraints is necessary. Goal programming (GP) is powerful and flexible modeling tool to deal with the above types of multiple criteria decision-making problems in energy planning and management for sustainable development of rural areas. Goal programming provides a way of striving towards several such objectives simultaneously. The basic approach of goal programming is to establish a specific numeric goal for each of the objectives, formulate an objective function for each objective and then seek a solution that minimizes the weighted sum of deviation of nine objective functions from their respective goals. There are three possible types of goals:

1. A lower one-sided goal which sets a lower limit that we do not want to fall under (but exceeding the limit is fine).
2. An upper one-sided goal which sets an upper limit that we do not want to exceed (but falling under the limit is fine).
3. A two-sided goal which sets a specific target that we do not want to miss on either side.

GP is the most suitable technique for solving multi-objective resource allocation problems. Thus GP has been chosen for the analysis here. DEP problems have been applied most frequently in practice relative to other multi-objective decision-making techniques.

### *2.2 Data needs for DEP*

DEP model requires the following set of data.

- Socio-economic features
- Land use: forests land, wasteland, fallow land, cropping pattern, etc.
- Energy; activities, end use devices, efficiency of devices
- Biomass production for energy; area under forests and plantations, biomass productivity, production and availability of crop residue for energy
- Energy efficiency, energy conversions, energy use
- Energy: RET (Renewable Energy Technologies) and FF (fossil fuel) technologies

- Cost of energy systems operation and maintenance cost and financial value of energy and products.

## 3. Description of the model

The quality and quantity of energy dictates how the societies will evolve [8]. Thus they are like "dissipative" structures [9]. For a self-organizing structure, a critical mass is required before it can sustain and grow. To achieve the goal of integrated and sustainable energy planning, energy models such as goal programming approach are used. Selection of the appropriate model is based on the requirement of data and suitability of the model at the Decentralized level to account for multiple energy needs. In a Goal Programming problem there are multiple objectives (with trade-offs) and the deviations from constraints are penalized. The optimization model used in the study consists of 7 objective functions subjected to 19 constraints. This model cannot be solved by using ordinary linear optimization and hence goal programming has been employed to solve the optimization problem. These 7 goals may be either *over* (1) or *under* (2) achieved. Deviation variables are introduced to represent the over or under achievement of the goals:
$d_1^+$ represents the amount of over-achievement of goal (1)
$d_1^-$ represents the amount of under-achievement of goal (1)
$d_2^+$ represents the amount of over-achievement of goal (2)
$d_2^-$ represents the amount of under-achievement of goal (2).
Note that for any goal K:
If the goal is exactly achieved: $d_1^+ = 0$; $d_1^- = 0$
If the goal is over-achieved: $d_1^+ > 0$; $d_1^- = 0$
If the goal is under-achieved: $d_1^+ = 0$; $d_1^- > 0$
In all cases all deviation variables $\geq 0$
The objective of the goal programmer is to minimize deviations from the goals given by:

$$\text{Minimize: } \sum d_i^- + d_i^+ \qquad \text{where } (j = 1, 2\ldots 7) \tag{1}$$

Adding pairs of deviation variables to the goals transforms them into a set of constraints:
Subjected to,

$$L_j + w_j d_j^- - w_j d_j^+ = b_j \tag{2}$$

Where, $d_j^-$ and $d_j^+$ represent the under-achievement and over-achievement of the goal respectively and $w_j$ represents the weighing factors and $b_j$ represents the goal values.

## 4. Scenarios considered for modeling

Energy scenarios provide a framework for exploring future energy perspectives, including various combinations of technology options and their implications. Many scenarios used in the literature illustrate how energy system development will affect economic development and environment. The historic trends and current priorities for the study area described earlier provide a starting point for the development of various scenarios, with and without the implementation of various technologies and policy measures. Seven scenarios are considered for analysis, namely, Business As Usual scenario (no specific policies to promote alternative energy technologies or to reduce emissions), Economic Objective Scenario (government subsidy plays an important role in this scenario) Renewable Energy Scenario (maximum use of locally available renewable energy resources), Biomass Intensive Scenario (biogas and biomass power along with energy plantations for meeting electricity needs) and Sustainable Development Scenario (high quality fuels, efficiency improvements, low environmental impacts, equitable allocation of energy resources, etc). Based on the outputs of scenarios, energy demand, cost and number people employed and $CO_2$, $NO_x$ and $SO_2$ emissions are estimated. Though scenario approach is one of the forecasting techniques, it is especially attractive for development planning due to uncertainties arising due to various factors such as availability of resources, changing demand scenarios, technological options and cost implications. Present case scenario is for the year 2005 and BAU, RES, EOS and SDS are for the year 2020.

## 5. Results and Discussion

This section presents the results of energy resource allocation at Kunigal Block level for the base year (2005). Different scenarios are developed for the year 2020 with an aim to identify the optimal scenario for implementation. The selection of scenario is carried out on the basis of cost incurred in energy supply, associated emissions and use of local resources. The optimization model is solved using WINQSB package. Kunigal Block has 36 Panchayats (GPs) and 314 villages. The total area of the block is 99,110 ha and its total population is over 0.2 million. The block has 47,200 households out of which 8853 households are unelectrified. Table 1 presents a summary of the DEP for results of different scenarios, which are explained in detail in the following sections.

**Table 1:** Optimized energy resource allocation for Kunigal block under different scenarios

| | **PECS (2005)** | **BAU (2020)** | | **Economic objectives scenario (2020)** | **RES (2020)** | | **SDS (2020)** |
|---|---|---|---|---|---|---|---|
| Activities | | No priority | Equal priority | Cost (1)[b], Employment (2)[b], Efficiency and Reliability (3)[b] | ERS; Emission (1)[b], Economic (2)[b] and Security (3)[b] | BIS Cost, Employment, Local and Emissions (1)[b] | Economic, Security and Emission (1)[b] |
| Cooking | Biomass (65%) LPG (22%) Kerosene (13%) | Biomass (50%) LPG (50%) | Biomass (22%) LPG (21%) Biogas (57%) | Biogas (100%) | Biogas (100%) | Biogas (100%) | Biogas (100%) |
| Home Lighting | Kerosene (50%) Grid (50%) | Grid (100%) | PV Electricity (100%) | Biomass Electricity (100%) | PV electricity (100%) | Biomass Electricity (100%) | Biomass Electricity (100%) |
| Water Pumping | Diesel electricity (10%) Grid (90%) | Grid (100%) | Biomass electricity+ diesel electricity (100%) | Grid (100%) | PV electricity (100%) | Biomass electricity (100%) | Biomass Electricity (100%) |
| Water Heating | Biomass (100%) | Biomass (100%) | Solar thermal (100%) | Biomass using improved cook stoves (100%) | Solar thermal (100%) | Biomass using improved cook stoves (100%) | Biogas (100%) |
| Rural Industries | Diesel (10%) Grid (50%) Biomass (40%) | Grid (40%) Biomass (60%) | Biomass electricity+ diesel electricity (100%) | Biomass electricity (100%) | Biomass electricity (100%) | Biomass electricity (100%) | Biomass electricity (100%) |
| Home Appliances | Grid (100%) | Grid (100%) | Biomass electricity (100%) | Biomass electricity (100%) | PV electricity (100%) | Biomass electricity (100%) | Biomass electricity (100%) |

[a] Using improved cook stoves.
[b](1) first priority, (2) second priority and (3) third priority

Energy resource allocation in Kunigal block shows that 68% of the households used solid biomass, 22% LPG and 13% kerosene as cooking fuel under the PECS scenario. BAU (EP) scenario with equal priority for all the objective functions shows that biogas produced from the available livestock dung in the block can meet 57% of cooking energy needs and the remaining through 22% biomass and 21% from LPG. Biogas is produced from livestock dung and leaf litter collected from energy plantations raised on degraded lands. Liquefaction of biogas is not possible, so transportation of the gas is not an option. Thus, the optimized option for cooking is community biogas systems in the villages of Kunigal block under EOS, RES and SDS scenarios where 100% energy needs can be met. Under PECS and BAU (NP), traditional stoves with low efficiency are the options for water heating using solid biomass. Solar water heater is the option for heating water under BAU (EP) and RES (ERS) as over 300 days of bright sun shine is available in the region. Improved stove with efficiency higher than 25% is the option to heat water under EOS and BIS scenarios. Surplus biogas available from energy plantation appears to be the option for SDS scenario. Thus, the optimized options for water heating are surplus biogas and improved cookstoves under SDS and BIS scenarios. Water pumping energy needs under PECS are met by grid electricity (upto 90%) which is subsidized (at Rs 1 kWh) and 10% by diesel electricity which is used when grid is unavailable (shown in the Figure 1). So, PECS is largely based on grid. Kunigal is projected to have 10112 irrigation pump sets by 2020 compared to 6212 in 2005. Biomass power is the optimized solution for meeting electricity demand for agricultural pumping activity under BIS and SDS scenarios.

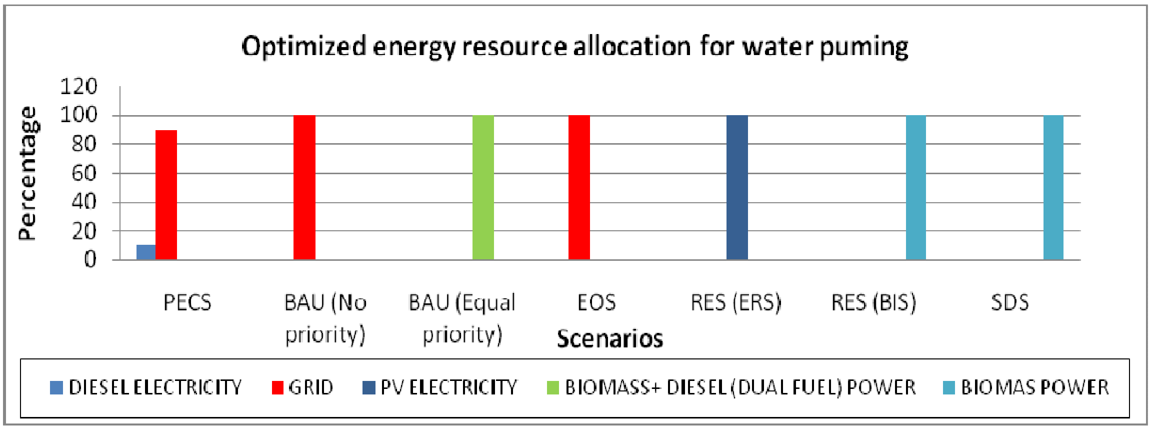



**Figure 1:** Optimized energy resource allocation and technology mix for meeting water pumping energy needs

Rural industries like Jaggery manufacturing and pottery depend on solid biomass while others like milk processing unit, rice mill and flour mill need electricity. Detailed Diesel (10%), biomass (40%) and grid electricity (50%) are used as source of energy for industries (shown in the Figure 2). BAU (No priority): Results show that grid electricity and biomass meet 40% and 60% of energy needs of rural industry in Kunigal block, respectively under BAU (NP). Biomass dual-fuel under BAU (EP), biomass power under EOS, RES and SDS scenarios are the optimized options for meeting the electricity needs of rural industry.

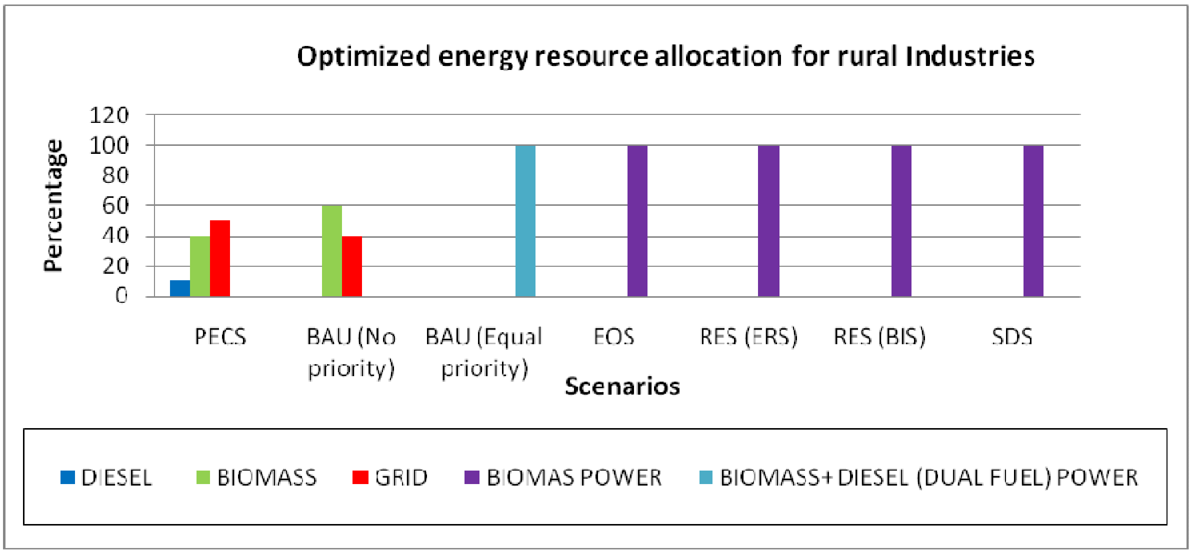



**Figure 2:** Optimized energy resource allocation and technology mix for rural industries in Kunigal block under different scenarios

## 6. Conclusions

Present work formulates the different objectives functions which have to be considered for the DEP. Present work states the constraints while formulating such problem. Although different approaches are there in the literature, Goal programming has been shown appropriate for DEP. Methodology for making objectives functions and constraints for DEP has been discussed in the manuscript. This study covers an assessment of the current energy demand and supply situation, the development of several forecasting scenarios (business-as-usual energy scenario, biomass energy intensive scenario, renewable energy scenario and sustainable development energy scenario), supply-demand balancing, the identification of intervention options, and an assessment of impacts of future trends and interventions in economic and environmental terms. Application of the present model has been shown in a typical block (Kunigal block) from India. A block constitutes a cluster of villages with distinct geographic boundary consisting of settlement, agricultural land, water bodies and any other land category, in most parts of India. Each individual village forms the distinct rural identity, each of which is generally separated by agricultural or forest land. Present model suggests that biomass-based energy systems have the potential to meet all the rural energy needs. At the block level large extent of wasteland is available (34% of geographic area). If these wastelands are used for raising energy plantation, all the electricity needs of the block could be met. High employment generation and carbon mitigation can also be achieved by adopting RES and SDS scenarios. Further, all cooking needs can be met from biogas option.

### References

47. Bruckner TH, Groscurth HM, Kümmel R, “Competition and synergy between energy technologies in municipal energy systems”, *Energy*, 1997; 22:1005-1014.
48. Bakken B, Holen A, *Energy Service Systems: Integrated Planning Case Studies*, Proc. IEEE PES General Meeting 2004, Denver, CO, 2004.
49. Reddy AKN, Subramanian DK, *The design of rural energy centers*, Indian Academy of Sciences, Bangalore, 1980, 109-130.
50. Ravindranath NH, Hall DO, *Biomass, energy, and environment: A developing country perspective from India*, Oxford University Press, Oxford, United Kingdom, 1995.
51. Hiremath RB, Shikha P, Ravindranath NH, “Decentralized energy planning; modeling and application-a review”, *Renewable and Sustainable Energy Reviews*, 2007; 11: 729 - 752.
52. Juan J, Tarjuelo JM, Valiente M, Garcia P, “Model for optimal cropping patterns within the farm based on crop water production functions and irrigation uniformity I: Development of a decision model”, *Agricultural Water Management*, 1996; **31:** 115-143.
53. Kanniappan P, Ramachandran T, “Optimization model for energy generation from agricultural residue”, *International Journal of Energy Research*, 1998; **22:** 1121-1132.
54. Prigogine I, *From-Being to Becoming*, W. H. Freeman and Co., San Francisco, 1980.
55. Goldemberg J, Johansson TB, Reddy AKN, Williams RH, *Energy for a Sustainable World*, Wiley Eastern Ltd, New Delhi, 1988.

# ENHANCING SUSTAINABILITY OF LOCAL RICE MILLS BY CLEANER PRODUCTION AND INDUSTRIAL ECOLOGICAL PRINCIPLES

H.M.A.P Rathnayake[1*], A.K. Kulatunga[2], T.M.R. Dissanayake[1]

[1*]Acting Senior Mechanical Engineer, Institute of Post harvest Technology,
Jayanthi Mawatha, Anuradhapura, Sri Lanka.
[1*]E-mail: aprathnayake@hotmail.com
[1*]Telephone: +94-25-2225765-6 (Ext.48); Fax: + 94-25-2223983

[1*]Mechanical Engineer, Institute of Post harvest Technology, Jayanthi Mawatha, Anuradhapura, Sri Lanka.
[1]E-mail: mrdissanayake@yahoo.com
[1]Telephone: +94-25-2225765-6 (Ext.32); Mobile +94 714192964; Fax: + 94-25-2223983

[2]Lecture. Department of Production Engineering, Faculty of Engineering,
University of Peradeniya, Peradeniya, Sri Lanka
[2]E-mail: aselakk@pdn.ac.lk; Telephone: +94-81-2393657

**Abstract-** Rice processing industry is the largest agro-based industry in the country turning more value of product than any other industry. Mainly two types of rice called raw and parboiled; famous among the local community. The rice milled from pre-treated paddy is known as parboiled rice. Parboiling is a hydro-thermal treatment followed by drying before milling for the production of parboiled grains. Nearly 70 percent of the paddy produced in Sri Lanka at present is parboiled. Two dominant wastages are released to environment in parboiled rice milling namely, soak water and paddy husks. For one metric ton of paddy, approximately 1.3 $m^3$ of soaked water and 0.2 tons of paddy husk are released to environment and soaked water is discharged to the environment without being treated. Due to this, bad odour is prevailed in the vicinities of the mills. In addition, paddy husk and ash dumps are washed away with rain to the waterways. To minimize bad smell, millers are advised to change socking every 10-12 hours. Even though socking water is changed every 12 hours time COD and $BOD_5$ levels of releasing water are higher than the values set by CEA. Therefore, sustainability of this industry relies addressing the waste streams in a productive manner. Therefore in this research cleaner production principle is adapted to minimize the waste generated by attacking the point of generation and using potion of treated wastewater for soaking process. Industrial Ecology concept is adapted by using by product (waste) of the process, paddy husk as a fuel initially to operate the boiler and later ash of the husk as filtering media of wastewater. In addition, steam generated by burning paddy husk will be partly used to rotate biological rotating disk which facilitate aerobic reaction. Treatment process consists of two stages and in the first stage, BOD value is reduced by aerobic digestion with the help of rotating biological contractors (RBC) and in the second stage pre-treated wastewater sent through a carbon filter in order to reduce COD. Result shows that treatment process gives promising results of the COD and $BOD_5$ concentrations of the treated water therefore treated water can be re-used for soaking purposes again.

***Keywords:*** *Waste water, Cleaner Production, Industrial Ecology, Waste management, Sustainability*

## 1.0 Introduction

The Sri Lankan economy has traditionally been dominated by agriculture. The production of paddy has occupied an extremely important place in the agricultural sector. Rice processing industry is the largest agro-based industry in the country turning more value of product than any other industry. Sri Lanka currently produces average of 3.24 million tons of paddy annually. Large number of direct and indirect employment opportunities available in this industry. Unless this industry operates in a sustainable manner, those who are involved in this industry will get affected. Mainly two types of rice called raw and parboiled famous among the local community. The rice milled from pre-treated paddy is known as parboiled rice whereas the rice milled from untreated paddy is known as raw rice or white rice. Parboiling is a hydro-thermal treatment followed by drying before milling for the production of parboiled grains [1]. Nearly 70 percent of the paddy produced in Sri Lanka at present is parboiled. The parboiling process extensively used in Eastern, North Central, Uva, North-Western provinces. There are three major steps in parboiling process: soaking, steaming and drying. All these steps have a great influence of the final characteristics and quality of parboiled rice.

Two dominant wastages are released to environment in parboiled rice milling namely, soak water and paddy husks. For one metric ton of paddy, approximately 1.3 $m^3$ of soaked water and 0.2 tons of paddy husk are released to environment and mostly soaked water is discharged to the environment without treating. This leads to bad odour in the vicinities of the mills. In addition paddy husk also sometimes goes away with rain water to the waterways. As a rule of thumb, soaking time for short and long grains are twenty four (24h) and forty eight (48h) hours respectively [2]. Due to these extended hours of soaking effluent water contains bad odour. This problem aggravates sometimes millers re-use soaked water. In order to reduce this bad odour, currently Institute of Post Harvest Technology (IPHT) has been advocating millers to change soaking water at every twelve hours during soaking period. This leads to increase water consumption per tonnage of paddy and wastewater generated at two or three times. Even though socking water is changed every 12 hours time Chemical oxygen demand (COD) and five days biological oxygen demand ($BOD_5$) levels of releasing water are 1100 mg/l and 310 mg/l approximately and these values far higher than the values set by CEA (Central Environment Authority). According to CEA guidelines effluent has to be treated to reduce COD and BOD values and bad odour. Even there is no proper mechanism to manage ash generated out of husk burning.

There are few previous cases to reduce COD value in wastewater generated from paddy parboiling process. Ash generated from paddy husk was used as filtering media and tests were carried out in laboratory scale model to test the feasibility. However, this research managed to drop COD limits to 225mg/l [3]. Furthermore, that work did not attempted to reduce BOD and nothing was referred to reduce the bad odour. In addition, IPHT also tested similar kind of mechanism as above and that did not progress due to inherent operational difficulties as well as that method could not reduce the odour and BOD in substantial manner within considerable time frame [4].

Department of Chemical and Process engineering, University of Moratuwa and Environment and Management Lanka (Private) Ltd, have conducted several tests on that waste water treatment in parboiling. They understood that Anaerobic process (digestion) conducted could not be use totally satisfactory manner as digestion did not readily occur and cannot be applied to all rice mills. Physical chemical process was used them by adding alum and lime to reduce BOD and COD. The test results indicated that physical chemical treatment along is not sufficient to reduce BOD and COD. Also, they test Trickling filter and understand this method can applied when BOD level is remaining low. In next experiment they construct Oxidation Ditch and get down the $BOD_5$ from 1350 to 150mg/l. within the 16 hours [5].

There are number of drawbacks in the previously tested methods. Many tried to control one aspect only with the studies (such as COD reduction). None of them tried to eliminate bad odour generated from effluent water. Furthermore, some tried to treat chemically by adding chemicals. But, none of them succeed. Therefore, in this research a combined effluent treatment process to be developed. Though the main objective of this research is to develop a wastewater treatment method, there are number of additional benefits also available. By-product of the process: paddy husk is used as a fuel first for the boiler and then ash of it is used for wastewater filtering media. Furthermore, steam generated from boiler is partially used to fulfil power requirement for rotating of the drum of the treatment plant. By adapting above concepts total water consumption for the soaking and washing processes to be minimized.

## 2.0 Methodology

This research is adapted to minimize the waste generated by attacking the point of generation, what cleaner production (CP) principle mainly attempts. This is achieved by initially by developing the walk through assessment of the rice mill and later quantifying the material flow and energy requirements for the process. However, for this research main focus was to reduce water consumption for the soaking process and to treat and reuse wastewater generated from soaking process.

The process flow diagram of the rice milling process is given in Figure 01. The water consumption for soaking process has been reduced by reuse of soaked and treated water mixing with fresh water. In addition, the by product of the rice milling process, paddy husk has been used as fuel for steam generation to boil paddy under controlled burning and later used partly burned ash as filtering media in the wastewater treatment. Since this research adapted aerobic digestion to reduce high BOD strength of wastewater within shorter time line, a rotating biological contractor (RBC) is used. The advantage of RBC are simplysity of maintenance and operation, low power consumption, no pliers or objectionable odour, ability to withstand socks or toxic loads and desirable sludge setting properties [6]. In order to rotate this reactor, steam generated from the boiler is to be used for 75 % of the time it is being used. Thereby electrical energy used to rotate the disk can be minimized. The proposed methodology is shown in a schematic diagram in Figure 02.

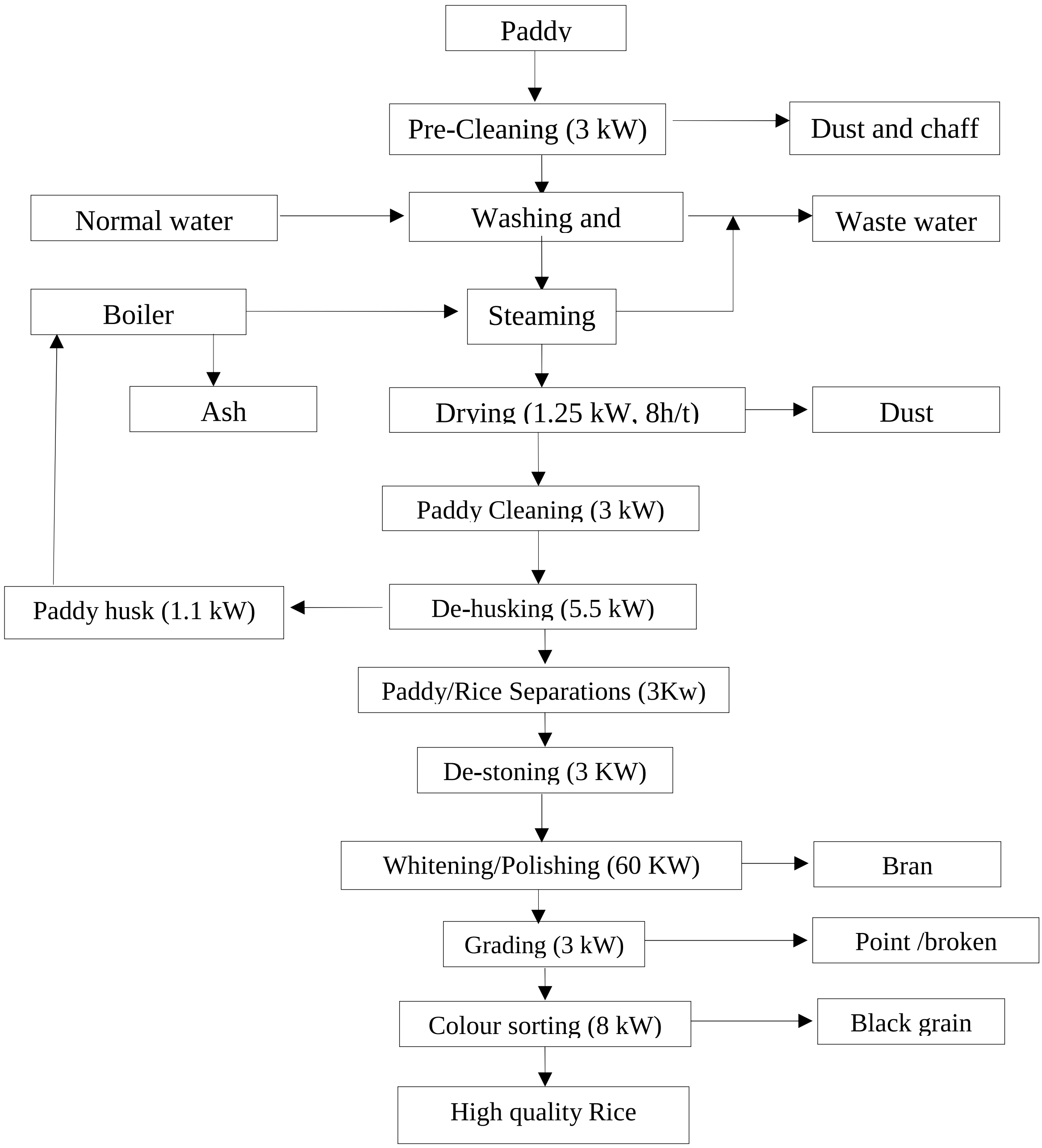



Figure 1: *Process Flow Diagram of rice production (Production Capacity: 1 t/h)*

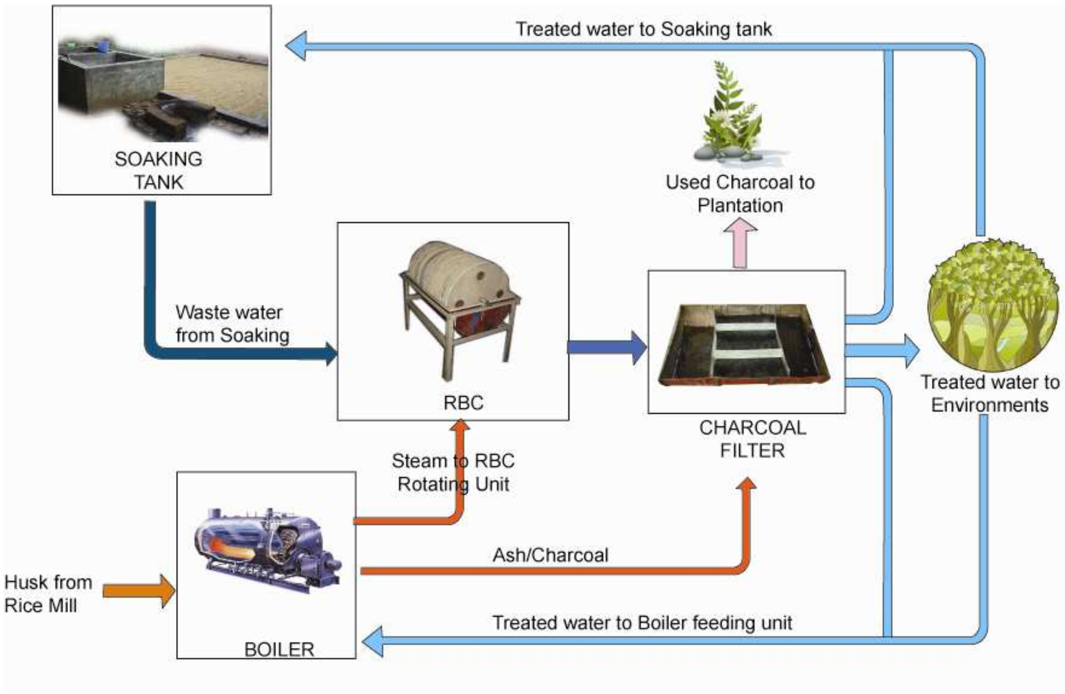



Figure 2: *Schematic diagram of the cycle proposed*

## 2.1 Experiments

In order to validate the model proposed in the paper, a laboratory scale treatment plant was design and developed. This plant mainly consisted of two sections: the rotating biological reactor and paddy husk ash based horizontal filtration bed. The photos of the lab scale model are given in Figure 3. The Rotating Biological Contactor (RBC) was fabricated in work shop of Institute of Post Harvest Technology. It consisted of a hemi cylindrical tank made of barrel by applying suitable paint coating to prevent from corrosion or rust. The rotating disk also corrosive resistant and made by BI sheet coated with suitable paint to prevent from rust and oxidization.

Two stages were used for the treatment. First stage used for RBC system and second stage contained with Paddy Husk Ash Filter (RHA). The first stage tank was 88cm long and 60 cm in radius with a volume of 100 litres. The system was developed for batch type treatment to match with soaking process of the rice mills. A central steel shaft ran through the whole length of the tank and was used to support the rotating disk. There were 39 discs. Each disk having a diameter of 56cm and attached together in each stage to allow maximum surface area for a given volume. The total surface area for 39 disks was 19.21m$^2$ providing a volume to surface area ratio of 5.2 litres/m$^2$. The disks were 40% submerged and rotated at a constant speed of 1.6 rpm [7]. The wastewater was subjected to ten hours treatment as a batch in the RBC system and then it was send to the RHA filter which has same dimension and capacity.

The tank which has same diameter as above, consist 40 inch long RHA filter in the middle part. The treated water came from RBC system was fed through the RHA filter in horizontal direction. At the end pipe was facilitate to drain out purified water.


Figure 3: *Lab scale model of the proposed batch type wastewater treatment plant*

## 3.0 Results & Discussion

Data were collected from plant period of two weeks. Five days biological oxygen demand ($BOD_5$), Chemical oxygen demand (COD) and PH were measured during this time period. The results of the six day readings are given in Table 01. The $BOD_5$ readings before treatment varied from 50 mg/l to 98 mg/l and that of the COD readings varied from 905mg/l to 1676mg/l. The $BOD_5$ values of the waste water samples were shown low when compared to values in literature. This was due to the water changing at correct time. Furthermore, the adding of dust, husk and bran to soaking water may have caused to increase pollutants of soaking water. Hence, by collecting above by products into fully closed rooms would results low $BOD_5$ and COD readings in this experiment.

The degree of pollution (i:e the quantity of substances suspend in it and their state of fermentation) depend on the processing systems and conditions specially , length of time the paddy is in water, the impurities contained in the rice and their characteristic, water temperature etc. as well as processing condition [8]

Table 1: *$BOD_5$, COD and PH concentrations before and after the treatment process*

| No | Before treatment | | | After treatment | | |
|---|---|---|---|---|---|---|
| | $BOD_5$/(mg/l) | COD/(mg/l) | pH | $BOD_5$/(mg/l) | COD/(mg/l) | pH |
| 1 | 50 | 997 | 6.98 | 25 | 245 | 7.58 |
| 2 | 53 | 1442 | 7.04 | 26 | 391 | 7.59 |
| 3 | 85 | 1029 | 5.09 | 20 | 112 | 8.2 |
| 4 | 65 | 905 | 4.29 | 55 | 214 | 7.36 |
| 5 | 98 | 1676 | 5.78 | 24 | 241 | 8.01 |
| 6 | 95 | 1230 | 5.94 | 16 | 42 | 7.75 |

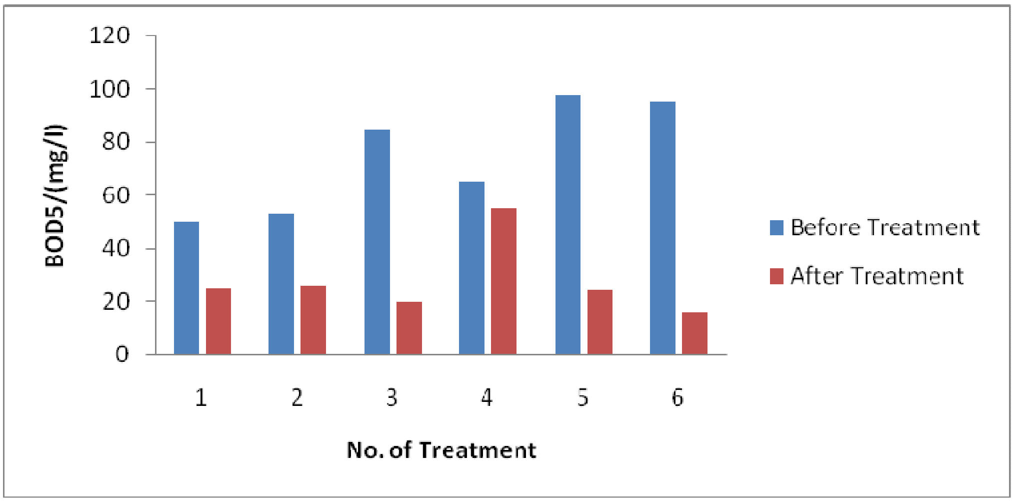



Figure 4: *The variation of $BOD_5$ change before and after the treatment process*

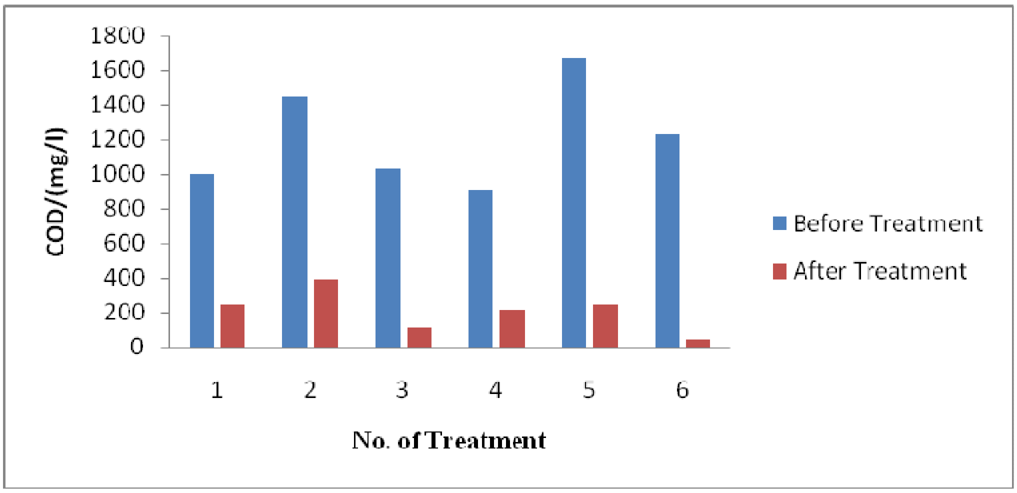



Figure 5: *The COD variation before and after the treatment process*

Based on the experiment, The $BOD_5$ values after treatment varied from 16 mg/l to 55 mg/l. Out of six trials, only one trial exceeds CEA standards in $BOD_5$ (CEA standard for $BOD_5$ is max. 30mg/l). This is at trial number 04. The COD values after treatment were from 42mg/l to 349 mg/l. In this treatments also, out of six trials, one trial exceeds CEA standards in COD (CEA standard for COD is max. 250 mg/l) at trial 02. This may be due to the collection errors of the treated wastewater sample or saturation of absorption characteristics of charcoal bed. However, when compared to percentage reduction of $BOD_5$ and COD; it is harder to see any pattern. In all these experiments the bad odour of the wastewater was eliminated after the respective batch of water was treated at two stages.

If the BOD and COD level of treated waste water is above the CEA standard, it is possible to reduce it by increasing the area in disk of existing RBC system or adding another RBC system. Further more, treated water could be diluted by adding little quantity of fresh water and could be used for boiler and soaking purpose.

## 4.0 Conclusion

This paper presented a sustainable mechanism to manage waste generated in local par-boiling rice milling industry. Main focus of the paper was to minimize the wastewater generated from the soaking process and to treat and reuse it in a sustainable manner. CP and IE based approach is presented in this research where, same industries waste and energy generated out of other processes are used for treatment process. Treatment process consists of two stages and in the first stage, BOD value is reduced by Anaerobic digestion with the help of rotating biological contractors (RBC) and in the second stage pre-treated wastewater sent through a carbon filter in order to reduce COD. Seventy five percent of energy used for RBC was supplied by stream generated for par-boiling purposes by burning paddy husk and carbon media for filtering purposes were used from the ash comes out from paddy husk burning. Result shows that treatment process gives promising results of the COD and BOD concentrations of the treated water therefore treated water can be re-used for soaking purposes again. The filtration media change frequency has not checked in this experiment. Currently this design is experimented with medium size par-boiling rice mil owners of North Central Province as a pilot project.

## References


[1] Chakraverty, A, "Principal of Parboiling of Paddy", Post Harvest Technology of Cereals, Pulses and Oil Seeds, 2006; 7:125-135.

[2] Adikarinayake, T.B, Swarnasiri, D.P.C., "An Improved Home level parboiling Technique", *Tropical Post harvest Technology*, 1988; 1(1): 19-22.

[3] Ariyarathna, S.M.W.T.P.K, Kalpage, C.S., Shanthini, R., "Design of a Paddy Husk Charcoal Adsorption Unit for the Treatment of Parboiled Rice Processing Wastewater", *Proceedings of the Peradeniya University Research Sessions*, 2007; 12: 199-201.

[4] Senadeera, W. "Disposal of Parboiling Water in Rice Mills", *RPRDC Technical Note*, 1990.

[5] Thiruchelvam, A.T., Sivakanthan, N., Dharmawardhana, M.S., Alwis, A.A.P.De., "Treatability Study of Parboiling Effluent from Paddy Processing" *Unpublished Technical Note.* University of Moratuwa.

[6] Chan, R.T., Stenstrom, M.K., "Use of the rotating biological contractor for appropriate technology waste water treatment", water resource programme school of engineering and applied science, University of California, Los Angeles, 1979; 3-5.

[7] Metcalf & Eddy, "Rotating Biological contactor -Attached Growth and Combined Biological Treatment Process", Wastewater Engineering Treatment and Reuse, 2003; 930-939

[8] F. Gariboldi, 1984, FAO Agricultural services bullting, Food and agriculture organization of the United Nations Rome, 1984)

# DEVELOPMENT OF A 3D MODEL TO STUDY THE $CO_2$ SEQUESTRATION PROCESS IN DEEP UNMINEABLE COAL SEAMS.

M.S.A. Perera[1], *P.G. Ranjith[1], S.K. Choi[2], D. Airey[3], A. Bouazza [1] and J. Kodikara[1]

[1]Department of Civil Engineering, Monash University, Building 60, Melbourne, Victoria, 3800, Australia.

[2]Australian Commonwealth Scientific and Research Organization (CSIRO), Locked Bag 10, Clayton South VIC 3169, Australia.

[3] School of Civil Engineering, University of Sydney, Sydney NSW 2006, Australia.

Corresponding author: Dr Ranjith Pathegama Gamage*
Department of Civil Engineering, Monash University, Building 60,
Melbourne, Victoria, 3800, Australia.
[1*]E-mail: ranjith.pg@monash.edu, [1*]Phone/Fax: 61-3-9905 4982

**Abstract**
This paper presents a numerical model to study the carbon dioxide ($CO_2$) sequestration process in deep coal seams and to investigate the factors that affect this process. A coal seam lying 1000 m below the ground surface was considered for the simulation. One injecting well was first inserted at the middle of the area under consideration and $CO_2$ was injected for a 10 year period. With one injection well, the storage capacity was calculated as $13\times10^7$ $m^3$. The number of injecting wells was then increased to 4. It was found that the maximum storage capacity was observed at two well conditions (an increment of 130% of the single well condition). However, further increasing the number of wells (up to 4) reduced the storage capacity to $12.5\times10^7$ $m^3$. According to the model results, it is clear that $CO_2$ storage capacity in deep unmineable coal seams is dependent on the number of injecting wells and their location and porosity, the permeability of the coal seams, coal bed moisture content and temperature.

***Keywords:*** *Storage capacity, $CO_2$ sequestration, wells, moisture content, temperature, numerical modeling*

## 1. Introduction

The world is currently facing the problem of global warming, the main cause of which has been identified as the release of green house gases such as carbon dioxide ($CO_2$) into the atmosphere. $CO_2$ sequestration in deep coal seams has been recognised as a potential method of atmospheric $CO_2$ mitigation. In addition, it could produce large amounts of value-added energy products such as methane ($CH_4$) as an outcome. According to Stevens et al.(2000), the coal mass can store a substantial amount of gases due to its large surface area and highly porous structure. For instance, it has been estimated that the combined Bowen and Sydney basins in eastern Australia can store 11.2 Gt of $CO_2$ (White et al. 2005).

Coal mass can be defined as a naturally-fractured reservoir for gas movement. The movement of gases through this highly complex coal mass structure depends on the permeability of the coal mass itself, which may be governed by Darcian Law/or non-linear laminar flow and the intrinsic permeability of the coal matrix, which is governed by Fickain FICKIAN? diffusion. Therefore, the amount of $CO_2$ that can be stored in the coal mass is highly dependent on coal's physical and chemical properties, the arrangement of injecting wells and their number. However, the process of $CO_2$ sequestration in deep coal seams remains in the experimental stage as many aspects need to be studied before it can be put into practice. Following a detailed review of the available studies related to $CO_2$ sequestration, White et al. (2005, p.?) explain that "*there is a fundamental lack of understanding concerning the physical, chemical and thermodynamic phenomena that occur when $CO_2$ is injected into a coal seam.*"

Normally coal mass has "dual" porosities. It has inter-aggregate fractures (secondary porosity system) and intra-aggregate pores (primary porosity system). The interaction of these porosities can be complex and renders simple models inaccurate (Coll et al. 1994). Experimental and numerical modelling studies can help to provide a better understanding of the flow phenomenon in coal. To date, many field-scale models have been developed for flow in porous rock masses using different computer codes, such as TOUGH 2 (Carneiro, 2009), COMSOL (Liu and Smirnov, 2009; Perera et al., 2010(a)) FEMLAB (Holzbecher, 2005) and COMET 3 (Pekot and Reeves, 2002; Perera et al., 2010(b)) which can be used to simulate gas and water flow in coal.

The main objective of this study is to develop a 3-D numerical model using COMET3 to simulate the $CO_2$ sequestration process in a deep unmineable coal seam. COMET 3 is a conventional and coal bed methane reservoir simulator, which can simulate single or two phase flow through single, dual or triple porosity reservoirs, such as coal or shale as well as conventional reservoirs (Pekot and Reeves, 2002).

### *1.1 Governing Equations Used in the Model*

In COMET 3, fluid flow in the rock mass is modelled by using the mass conservation equations for water and gas as given in Eq.[1] and[2], respectively (Sawyer et al. 1990).

$$\nabla.\left[b_g M_g\left(\nabla p_g+\gamma_g \nabla Z\right)+R_{sw} b_w M_w\left(\nabla_{pw}+\gamma_w \nabla Z\right)\right]_f+q_m+q_g=\left(\frac{d}{dt}\right)\left(\phi b_g S_g+R_{sw}\phi b_w S_w\right)_f \quad [1]$$

$$\nabla.\left[b_w M_w\left(\nabla p_w+\gamma_w \nabla Z\right)\right]_f+q_w=\left(\frac{d}{dt}\right)\left(\phi b_w S_w\right)_f \quad [2]$$

where $b_n$ (n=g or w) is the gas or water bulking factor, $\gamma_n$ (n=g or w) is the gas or water gradient, $R_{sw}$ is the gas solubility in water, $\phi$ is the fracture porosity, Z is the elevation $q_g$ is the gas flow rate, $q_w$ is the water flow rate, $q_m$ is the matrix gas flow rate, $M_n$ (n=g (gas) or w(water)) $= kk_m/\mu_n$, is the phase mobility, ($k$-permeability, $k_m$-matrix permeability, $\mu_n$ -phase viscosity), $S_n$ (n=g or w) is the gas or water saturation, and $P_n$ (n=g or w) is the gas or water pressure. Gas adsorption is calculated using the extended Langmuir model.

$$C_i\left(P_i\right)=\frac{V_{Li}\,P_i}{P_{Li}\left[1+\sum_{j=1}^{3}\left(\frac{P}{P_L}\right)_j\right]} \quad , i=1,2 \quad [3]$$

where, $V_{Li}$ is the Langmuir volume, $P_{Li}$ is the Langmuir Pressure, $P_i$ is the partial pressure of the gas component, $C_i(P_i)$ is the adsorbed gas concentration at $P_i$, and $P$ is the total pressure.
Gas flow through the matrix is modelled using Fick's law of diffusion.

$$q_{mi}=\left(V_m / \tau_i\right)\left[C_i-C_i\left(P_i\right)\right] \quad , i=1,2 \quad [4]$$

where, $q_{mi}$ is the gas component flow, $V_m$ is the bulk volume of the matrix element, $\tau_i$ is the sorption time, and $C_i$ is the average matrix gas concentration of gas component $i$ . Permeability is determined by the ARI (Advanced Resources International) model.

$$\phi=\phi_i\left[1+c_p\left(P-P_i\right)\right]-c_m\left(1-\phi_i\right)\left(\frac{\Delta P_i}{\Delta C_i}\right)\left(C-C_i\right) \quad \mathbf{[5]}$$

$$\frac{k}{k_i} = \left(\frac{\phi}{\phi_i}\right)^n \qquad [6]$$

where, $c_p$ is the pore volume compressibility, $c_m$ is the matrix shrinkage compressibility, $\phi$ is the coal mass porosity, $\phi_i$ is the initial coal mass porosity , $P$ is the reservoir pressure, $P_i$ is the initial reservoir pressure, $C$ is the reservoir concentration, $C_i$ is the initial reservoir concentration , $P$ is the reservoir permeability, and $P_i$ is the initial reservoir permeability.

## 2. Model Development

For the purpose of modeling, a 540m×500m×20 m size coal seam, lying 1000 m below the ground surface, was considered. The location of the coal layer is shown in Fig. 1. $CO_2$ was injected at 14 MPa for 10 years from the bottom of the well as shown in the figure.

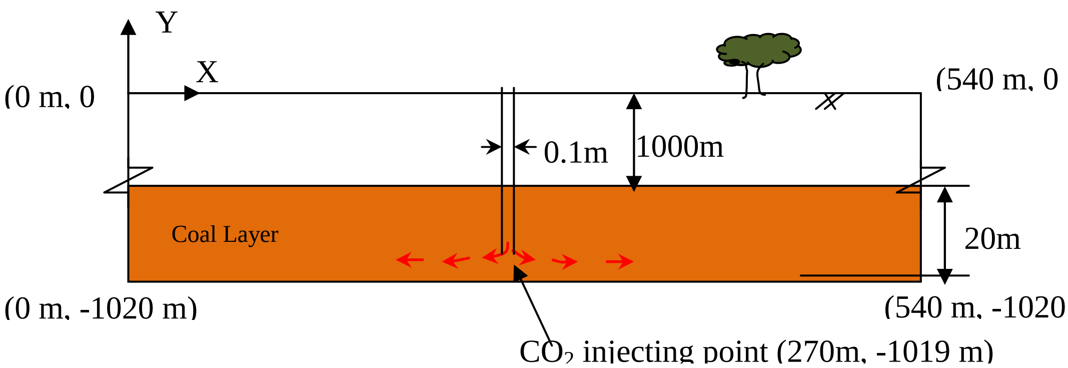



Figure 1. *Location of the coal layer.*

The model parameters used are shown in Table.1(Balan and Gumrah 2009).

Table.1. *Model parameters*

| **Model Parameter** | **Value** |
|---|---|
| Coal seam moisture content | 0.5 ($cm^3/cm^3$) |
| Coal seam initial permeability | 20 md |
| Coal seam porosity | 0.1 |
| Pore volume compressibility | $6.9e^{-5}$ (1/kPa) |
| Matrix shrinkage compressibility | $6.9e^{-7}$ (1/kPa) |
| Exponent of pressure dependent | 3 |
| Relative permeability variation | Cooray formula (Akin 2001), residual water and gas contents are 0.05 and 0.01 ($cm^3/cm^3$) |
| Temperature | 30 $^0$C |
| Langmuir volume for $CO_2$ adsorption | 16 ($m^3/m^3$) |
| Langmuir pressure for $CO_2$ adsorption | 1.56 MPa |

After developing the model, the effect of mesh size on storage capacity was examined by changing the width of the smallest grid block from 2 m to 14 m at 2 m intervals. The obtained CO2 storage capacity for 10 years and for 14 MPa gas injecting pressure is shown in Fig.2. According to the figure, when the grid width reduces from 14 m to 10 m, corresponding gas CO2 storage capacity increases from 1□108 m3 to 1.32□108 m3 and thereafter reduction of grid size does not change the storage. This is due to the fact that, when analysing the gas flow rate through a coal layer and it is a function of pressure gradient calculated using the mesh size, resulting a mesh size dependant outcome. This problem can be minimised by selecting smaller grid size, which gives more consistent results. However, a reduction of grid size causes an increase in the time required for the calculation (Nam et al. 2008). Therefore, the selection of optimum mesh size is important in any kind of finite element analysis. Considering all these factors, the size of the smallest grid block was taken as 10 m for the model. The final grid system selected for the coal layer is shown in Fig.2 (b).

(a) (b)

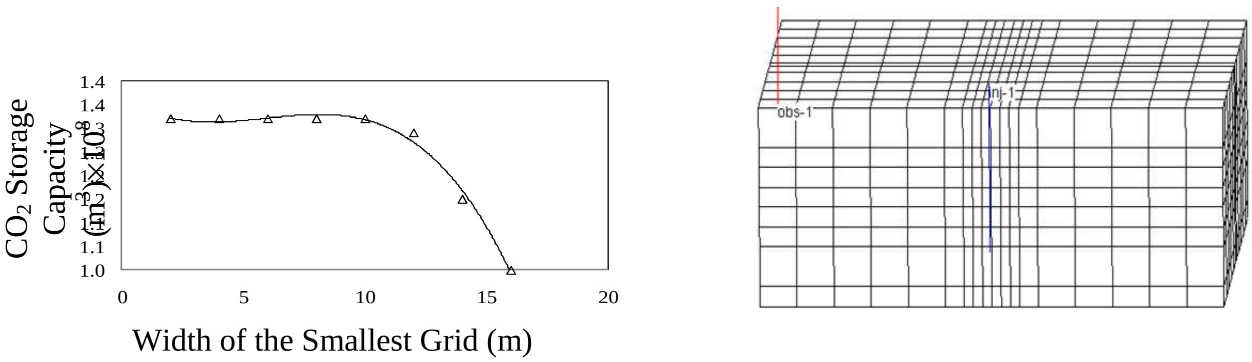



Figure 2. *(a) Effects of grid size on storage, and (b) selected model grid blocks for the coal seam.*

## 3. Model Simulation

After developing the model, the $CO_2$ migration rate along the horizontal distance was investigated for the bottom coal layer (Fig.3).

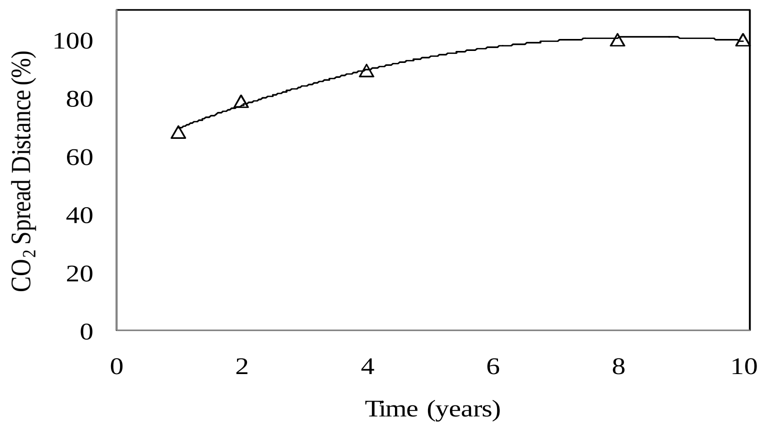



Figure 3. *Variation of $CO_2$ migration percentage with time.*

According to the above figure, after the first year $CO_2$ has spread through only around 65% of the coal layer and by the eighth year $CO_2$ has spread through the whole coal layer. It can also be seen that at the beginning the $CO_2$ spread at a fast rate. This is because at the beginning there are more pores available and the pore pressure in the reservoir is lower. Therefore, the advective flux rate is higher. However, as the pore pressure increases over time, the pressure difference between the injecting $CO_2$ and pores reduces, resulting in lower gas flow rate.

The effect of the injecting well operation on $CO_2$ storage capacity was investigated. The number of injecting wells was changed from 1 to 4, which changed the distance among the injecting wells. The variation of $CO_2$ storage capacity for 10 years of injection time with number of injecting wells is shown in Fig.4.

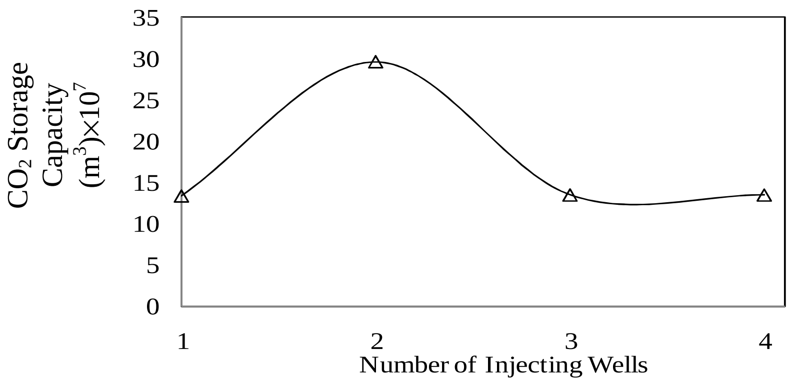



Figure 4. *Variation $CO_2$ storage capacity with number of injecting wells.*

According to Fig.4, the maximum storage capacity is obtained by having two injection wells and the addition of further injecting wells into the coal seam does not increase the $CO_2$ storage capacity by a significant amount. This may be due to the fact that further increasing the number of injecting wells after the two injecting well condition causes pressure contours to coincide, resulting in increased pore pressure and consequently reduced storage capacity. This arises because, when the number of injecting wells is increased from 1 to 4, the distance among the injecting wells reduces, such that for 2, 3 and 4 injecting well conditions the distances among the wells are 680m, 528m and 500 m respectively. In order to check this, the spread of $CO_2$ concentration contours was checked after 10 years of $CO_2$ injection for two and three injecting well conditions. The results are shown in Fig.5.

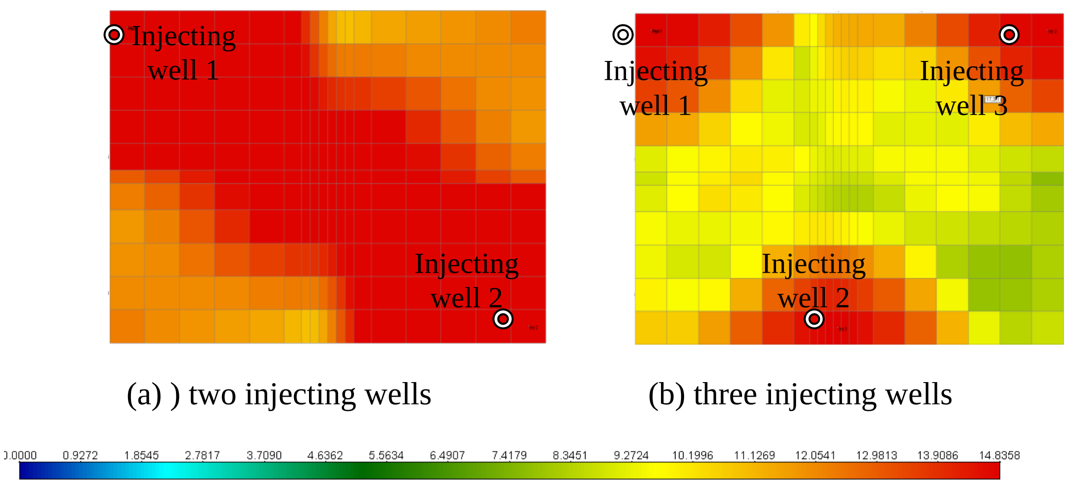



Figure 5. *$CO_2$ concentration contour cutting patterns*

According to Fig.5 (a), under the two injecting well condition, concentration has spread throughout the coal layer. However, as shown in Fig.5 (b), for more than two wells, $CO_2$ concentration is mainly limited to the surrounding areas of the injecting wells. This is because, with increased numbers of injecting wells, the distance between the injecting points is reduced, resulting in the pressure contours produced by each $CO_2$ injecting well meeting each other within a shorter time. This causes the pore pressure to increase and consequently the injecting capacity to reduce.

After checking the injecting well effect, the effect of coal mass physical properties on $CO_2$ storage capacity was investigated. The effect of coal bed moisture content on $CO_2$ storage capacity was investigated by changing the coal-bed moisture content from 0.1 to 0.5 ($cm^3/cm^3$). The variation of $CO_2$ storage capacity with the bed moisture content is shown in Fig.6 (a). Here, the gas injecting pressure and the coal mass temperature were maintained at 14 MPa and 30 $^oC$, respectively. Next, the coal bed temperature on storage was changed from 20 $^oC$ to 60 $^oC$ to investigate the temperature effect on total amount of $CO_2$, that can be injected into the coal mass (Fig.6(b)). According to Fig.6(a), up to around 0.45($cm^3/cm^3$) moisture content the storage capacity significantly reduces with moisture content and hereafter moisture content does not affect the $CO_2$ storage capacity. The reduction of $CO_2$ storage is 99% when moisture content changes from 0.1 to 0.45. The amount of $CO_2$ that can be stored in the coal mass is highly dependent on the available pore space, and the presence of water causes the coal mass pore space available for the $CO_2$ movement to largely reduce (Skawinski et al. 1991). However, according to Anderson et al. (1956), before reaching the critical moisture content (around 0.45($cm^3/cm^3$) in this study), the water molecules adsorb into the coal pore surface and obstruct the gas molecules adsorbtion into the surface. After the saturation point, the excess water (more than 0.45($cm^3/cm^3$) in this study) in the coal mass moves into the mobile phase and therefore does not affect the gas sorption. If the effect of coal bed temperature on $CO_2$ storage capacity is considered, according to Fig.6 (b), it can be seen that the increase of temperature causes the amount of $CO_{2,}$ that can be injected into the coal mass to significantly reduce. This may be due to the fact that, when the coal mass temperature increases, the gas molecules start to be released from the coal mass surface by the breakage of the bond between the molecules and the coal surfaces. As the temperature increases, this causes the kinetic energy of the gas molecules to increase and accordingly the rate of diffusion also to increase, resulting in reduction of the adsorption capacity (Levy et al. 1997). This may reduce the $CO_2$ storage capacity as the amount of $CO_2$ that can be captured inside the coal mass is totally dependent on its adsorption capacity.

(a) (b)

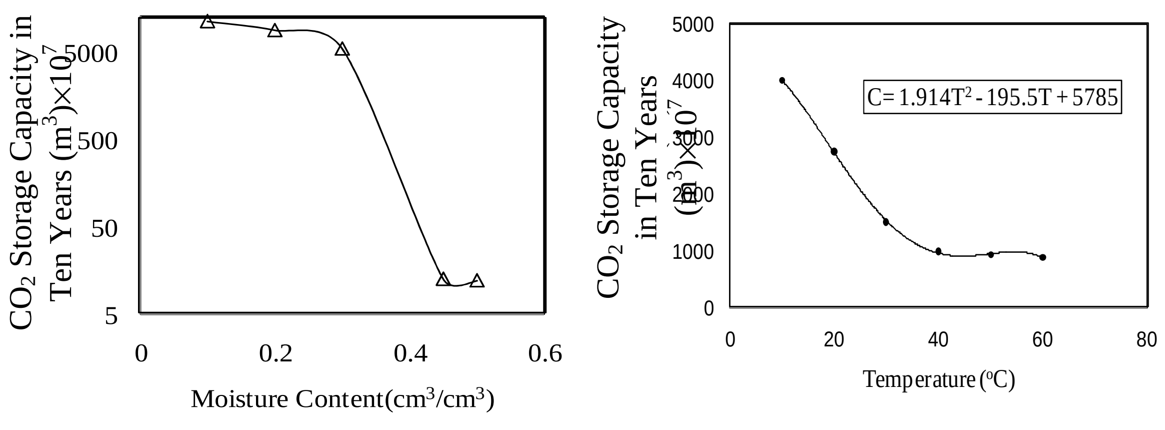



Figure 6. *Variation of $CO_2$ storage capacity of coal mass for 10 years of injection with the (a) coal bed moisture content and (b) coal bed temperature.*

## 4. Conclusions

The carbon dioxide ($CO_2$) sequestration process in deep unmineable coal seams can be successfully modelled using the COMET 3 numerical simulator. According to **the** results of this study, the number of injecting wells is a critical parameter, which should be investigated using an appropriate model before any field investigation. The reason is that according to the model results, $CO_2$ storage capacity in the coal seam cannot be increased by simply increasing the number of injecting wells. In fact, for a 500×540×20m coal seam, the two injecting well operating condition provides the optimum storage capacity and any further increase in injecting wells significantly reduces the storage capacity. When more than one injecting well is present in the coal seam, the $CO_2$ storage capacity is controlled by the pressure contours induced by all the available injecting wells. Coal mass temperature and the moisture content significantly control coal's $CO_2$ storage capacity. According to the developed model, the amount of $CO_2$ that can be injected into the coal mass reduces with both coal bed temperature and the moisture content, whereas the reduction of the storage capacity with moisture content occurs only up to the critical moisture content of the coal mass.

## References

Akin, S. (2001). "Estimation of fracture relative permeabilities from unsteady state corefloods." *Journal of Petroleum Science and Engineering*, 30(1), 1-14.

Anderson, R. B., Hall, W. K., Lecky, J. A., and Stein, K. C. (1956). "Sorption Studies on American Coals." *The Journal of Physical Chemistry*, 60(11), 1548-1558.

Balan, H. O., and Gumrah, F. (2009). "Assessment of shrinkage-swelling influences in coal seams using rank-dependent physical coal properties." *International Journal of Coal Geology*, 77(1-2), 203-213.

Carneiro, J. F. (2009). "Numerical simulations on the influence of matrix diffusion to carbon sequestration in double porosity fissured aquifers." *International Journal of Greenhouse Gas Control*, 3(4), 431-443.

Coll, M. C., Cortiula, B., Toledo, A., and S.A, M. (1994). "Identification of Productivity Problems in Clastic Sandstone Reservoirs of the Eocene Lower C Group, Block III, Lake Maracaibo, Western Venezuela using Well Log Data." SPE Latin America/Caribbean Petroleum Engineering Conference, Society of Petroleum Engineers Inc., Buenos Aires, Argentina.

Holzbecher, E. "*FEMLAB performance on 2D porous media variable density benchmarks* " *FEMLAB Konferenz 2005. Proceedings.*, Göttingen, 203--208.

Levy, J. H., Day, S. J., and Killingley, J. S. (1997). "Methane capacities of Bowen Basin coals related to coal properties." *Fuel*, 76(9), 813-819.

Liu, G., and Smirnov, A. V. (2009). "Carbon sequestration in coal-beds with structural deformation effects." *Energy Conversion and Management*, 50(6), 1586-1594.

Nam, J., Kim, J.-H., Kim, S., Yi, N., and Byun, K. (2008). "A study on mesh size dependency of finite element blast structural analysis induced by non-uniform pressure distribution from high explosive blast wave." *KSCE Journal of Civil Engineering*, 12(4), 259-265.

Pekot, L. J., and Reeves, S. R. (2002). "Modeling coal shrinkage and differential swelling with $CO_2$ injection for enhanced coal bed methane recovery and carbon sequestration application."

Perera, M. S. A., Ranjith, P.G. Choi, S.K., Bouazza , A. Kodikara, J. (Under Review). "A Parametric Study of Coal Mass and Cap Rock Behaviour and Carbon Dioxide Flow during and after Carbon Dioxide Injection " *International Journal of Petrolium Science*.

Perera, M. S. A., Ranjith, P.G. Choi, S.K., Airey, D. (Accepted). "Numerical Simulation of Gas Flow through Porous Sandstone and Its Experimental Validation
" *International Journal of Fuel*.

Sawyer, W. K., Paul, G. W., and Schraufnagel, R. A. "Development and application of a 3D coalbed simulator." *Paper CIM/SPE, Calgary* 90-119.

Skawinski, R., Zolcinska, J., and Dyrga, L. (1991). "Experimental investigations of the permeability in gas flow in coal with various water content." *Archives of Mining Sciences*, 36(3), 227-238.

Stevens, S. H., Kuuskraa, V. A., Gale, J. J., and Beecy, D. (2000). "CO (sub 2) injection and sequestration in depleted oil and gas fields and deep coal seams; worldwide potential and costs." *AAPG Bulletin*, 84(9), 1497-1498.

White, C. M., Smith, D. H., Jones, K. L., Goodman, A. L., Jikich, S. A., LaCount, R. B., DuBose, S. B., Ozdemir, E., Morsi, B. I., and Schroeder, K. T. (2005). "Sequestration of Carbon Dioxide in Coal with Enhanced Coalbed Methane Recovery A Review." *Energy & Fuels*, 19(3), 659-724.

# A STUDY OF METHODOLOGIES AND CRITICAL PARAMETERS ASSOCIATED WITH $CO_2$ STORAGE ESTIMATION IN DEEP SALINE AQUIFERS

P.N.K.De Silva[a], * P.G.Ranjith[b], S.K.Choi[c] and A.Haque[d]

[a,b,d]Department of Civil Engineering, Monash University, Building 60, Melbourne, Victoria, 3800, Australia.
[a]E-mail: Navinda.DeSilva@eng.monash.edu.au ; [a]Phone: +61 3 9905 8901; [a]Fax: +61 3 9905 4944
[*b]E-mail: Ranjith.pg@eng.monash.edu.au [*b]Phone/Fax: 61-3-9905 4982

[c]Australian Commonwealth Scientific and Research Organization (CSIRO) Locked Bag 10,
Clayton South VIC 3169, Australia.

**Abstract**
Deep saline aquifers have greater potential for carbon dioxide ($CO_2$) storage (around 12,000 Gt global capacity) than alternative storage media and could be adopted anywhere in the world. It is important to understand methodologies for the estimation of $CO_2$ storage capacities in relation to different trapping mechanisms and the extent to which critical parameters such as aquifer thickness, porosity, salinity and permeability are taken into account. Storage security will improve over time, especially as a result of mineral trapping. This paper reviews methods of estimating $CO_2$ storage potential from earlier studies and numerically estimates the storage potential in saline aquifers considering critical parameters such as saline aquifer and porosity.

***Keywords:*** *Saline aquifers, Trapping mechanisms, Mineral trapping*

## 1.0 Introduction
Carbon dioxide ($CO_2$) is a major greenhouse gas that needs to be controlled promptly to reduce its impact on global warming. Atmospheric emissions of $CO_2$ are projected to increase by 2.2% globally and 3.3% in the developed countries from 2000 to 2020 due to ever- increasing human activities (Soares et al., 2006). Underground storage of greenhouse gases in deep saline aquifers has been suggested as an effective means of significantly reducing atmospheric greenhouse gases to dampen the effects of global warming. Estimates of $CO_2$ storage capacity in saline aquifers fall into two categories with respect to the state of storage: One assumes the $CO_2$ remains as a separate fluid phase and the other assumes all $CO_2$ dissolves in the brine. However, Bruant et al., (2002) have shown that only a small fraction of an aquifer will be filled with separate phase $CO_2$ due to hydrodynamic and buoyancy effects. Estimations of $CO_2$ storage in deep saline aquifers must account for different trapping mechanisms such as physical, solubility, residual gas, mineral and hydrodynamic.

## 2.0 $CO_2$ storage capacity
Residual gas and solubility trapping of $CO_2$ in saline water have been viewed as the dominant $CO_2$ storage mechanisms (Zhang et al., 2009). Reservoir simulation and practical experience show that the injected $CO_2$ will rapidly gravitate to the top of the reservoir, limiting its contact with the reservoir and thus also limiting the storage capacity of the aquifer. Figure 1 illustrates the variation of storage with time.

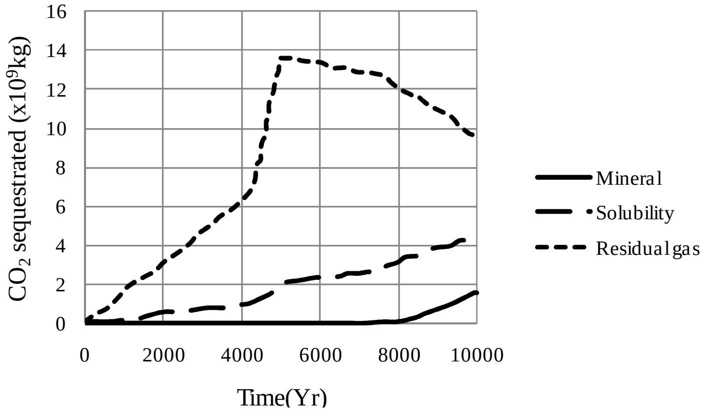



**Figure 1:** *Comparison of time evolution of the injected carbon dioxide in different trapping mechanisms (Zhang et al., 2009).*

### *2.1 Structural and stratigraphic trapping*

Storage of $CO_2$ in structural and stratigraphic traps is similar to $CO_2$ storage in oil and gas reservoirs, the only difference being that the trap is initially saturated with water in place of hydrocarbons (Bachu et al., 2007). Theoretical storage capacity is found using the following equation;

$$\mathrm{M}_{\mathrm{CO2t}} = \mathrm{Ah}\phi(1-\mathrm{S}_{\mathrm{wirr}})\rho(\mathrm{P},\mathrm{T}) \qquad \text{Equation (1)}$$

Where $\mathrm{M}_{\mathrm{CO2t}}$ is the theoretical $CO_2$ storage capacity, A is the trap area, h is the average thickness, $\rho(\mathrm{P},\mathrm{T})$ is the in situ density under local pressure (P), temperature (T), $\phi$ is the porosity and $\mathrm{S}_{\mathrm{wirr}}$ is the irreducible water saturation. Effective storage capacity is calculated using the following equation (Bachu et al., 2007);

$$\mathrm{M}_{\mathrm{CO2e}} = C_c \mathrm{M}_{\mathrm{CO2t}} \qquad \text{Equation (2)}$$

Where $\mathrm{M}_{\mathrm{CO2e}}$ is the effective $CO_2$ storage capacity and $C_c$ is the capacity coefficient that incorporates the cumulative effects of trap heterogeneity, $CO_2$ buoyancy and storage efficiency. Currently, there are no values in the literature for this capacity coefficient, which is site-specific and needs to be determined through numerical simulations and laboratory studies followed by field work (Bachu et al., 2007).

### *2.2 Residual gas trapping*

Due to the hysteretic properties of relative permeability, $CO_2$ is trapped at the end of injection of $CO_2$ as the flow is reversed (Bachu et al., 2007). This capacity can be found with the following equation;

$$\mathrm{M}_{\mathrm{CO2e}} = \Delta V_{trap} \phi S_{CO2t} \rho(\mathrm{P},\mathrm{T}) \qquad \text{Equation (3)}$$

Where $\Delta V_{trap}$ is the rock volume previously saturated with $CO_2$ that is invaded by water, $S_{CO2t}$ is the trapped $CO_2$ saturation after flow reversal. While the porosity and relative permeability characteristics can be determined through laboratory measurements on core scale rock samples, $S_{CO2t}$ and $\Delta V_{trap}$ can be determined only through numerical simulations (Bachu et al., 2007).

### *2.3 Solubility trapping*

Solubility trapping is dependent on time and continues over long periods of time in the order of centuries (Bachu et al., 2007). $CO_2$ may mix with, and then dissolve in, formation water through diffusion, dispersion and convection. Theoretical storage capacity can be estimated using the following equation;

$$\mathrm{M}_{\mathrm{CO2t}} = \mathrm{Ah}\phi(\rho_s X_s^{CO2} - \rho_o X_o^{CO2}) \qquad \text{Equation (4)}$$

Where A is the aquifer trap area, h is the average thickness, $\rho$ is the density of formation water, $X^{CO2}$ is the carbon dioxide mass fraction in formation water and the subscripts "*o*" and "*s*" stand for initial carbon dioxide content and carbon dioxide content at saturation, respectively. The effective solubility content can be estimated using the following equation;

$$\mathrm{M}_{\mathrm{CO2e}} = C * \mathrm{M}_{\mathrm{CO2t}} \qquad \text{Equation (5)}$$

Where C is a coefficient that includes the effect of all factors that affect the spread and dissolution of $CO_2$ in the whole aquifer volume under consideration.

### *2.4 Mineral trapping*

Mineral trapping is dependent on the chemical composition of formation waters and of the rock matrix, and on temperature and pressure (Bachu et al., 2007). In addition, it depends on the contact surface (interface) between the mineral grains and the formation water containing dissolved $CO_2$, and on the flow rate of fluids past the interface (Bachu et al., 2007). Only local and site level numerical simulations, supported where possible with laboratory experiments and field observations, may provide an estimate of the amount of stored $CO_2$ and the time-frame for $CO_2$ storage through mineral trapping. Previous work reveals that the $CO_2$ storage capacity through mineral trapping per unit of rock volume can be comparable to the storage capacity through solubility trapping, although it can take several thousand years for geochemical reactions to have a significant impact (Xu et al., 2004). Similar to residual gas and solubility trapping, as mineral trapping is a time-dependent process operating on the scale of centuries to millennia, the $CO_2$ storage capacity needs to be estimated for a particular point in time.

### *2.5 Hydrodynamic trapping*

Hydrodynamic trapping differs from other trapping mechanisms as it is not based on a single, specific physical or chemical trapping mechanism, but is a combination of the mechanisms reviewed earlier, which operate simultaneously but at different rates while a plume of injected $CO_2$ is expanding and migrating (Bachu et al., 1994). Because hydrodynamic trapping is based on several $CO_2$ trapping mechanisms acting at times simultaneously and sometimes being mutually exclusive, the $CO_2$ storage capacity has to be evaluated at a specific point in time as the sum of the storage capacities achieved by its component trapping mechanisms (Bachu et al., 2007). Given the combination and complexity of the processes involved and of their different time scales, it is not possible to evaluate the $CO_2$ storage capacity at basin and regional scales except in the broadest terms by extrapolating from local-scale simulations in the relevant aquifer. Numerical simulations can provide answers for specific cases of $CO_2$ injection in deep saline aquifers (Bachu et al., 2007).

### *2.6 Combined trapping method*

Nakanashi et al.(2009) propose a site-specific parameter *Sf* ("storage factor"), a ratio of immiscible $CO_2$ plume volume to total pore volume, to account for the combined effects of factors including trap heterogeneity, $CO_2$ buoyancy and sweeping efficiency. The researchers assume that the entire aquifer exists below a depth of 800 meters where $CO_2$ can be maintained at supercritical conditions, and no distinction is made between $CO_2$ stored by the various mechanisms. Further, it is assumed that injected $CO_2$ may be trapped for extended periods of time by a combination of trapping mechanisms (Nakanishi et al., 2009).

$$M_{co_{2t}} = Sf * A * h * \phi * Sg * \rho_{st} / Bg\,CO_2 \qquad \text{Equation (6)}$$

Where A is the aquifer area, h is the effective aquifer thickness, so that ($A$ x $h$ x $\phi$) represents the total pore volume within the aquifer volume under consideration. Sg is the supercritical $CO_2$ gas phase volume fraction in the injected $CO_2$ plume, assumed as 0.5 for assessment purposes. $\rho_{st}$ is $CO_2$ density at standard conditions (= 1.976 kg/m3), and $BgCO_2$ is the $CO_2$ volume factor which depends on local pressure and aquifer temperature. Further it is assumed that the storage factor, S*f*=0.5, for aquifer systems with limited areal extent due to predominance of $CO_2$ buoyancy effects and 0.25 for aquifer systems with larger areal extent(>24km$^2$).

### *2.7 Method proposed by US Department of Energy*

The US Department of Energy has proposed the following equation for $CO_2$ capacity estimation in saline aquifers (DOE 2007). Similar to earlier methods, details of the storage trapping mechanisms within a saline formation are not specified in this method. However, displacement of saline aquifer in the pore volume by immiscible CO2 is the fundamental mechanism implicit in the calculations.

$$M_{CO_2} = A h_g \phi_{tot} \rho E \qquad \text{Equation (7)}$$

Where $M_{CO_2}$ is the mass $CO_2$ storage capacity, A is the area, $h_g$ is the gross aquifer thickness, $\phi_{tot}$ is the average porosity and E is the storage efficiency factor. Monte Carlo simulations estimated a range of E between 1 and 4 percent of the bulk volume of saline formations for a 15 to 85% confidence range (DOE 2007).

### *2.8 Storage estimation of a typical saline aquifer*

Mount Simon type sandstone is used in this numerical model. The basic data for this model are listed in Table 1. The following numerical simulation using COMET3 software illustrates the saline aquifer storage estimation process. Mineral trapping and hydrodynamic trapping storage cannot be estimated using this numerical simulation procedure.

**Table 1:** *Reservoir model set-up properties and data (Kuuskra et al.,2004)*

| *Reservoir properties /data* | |
|---|---|
| Thickness of aquifer layer | 24 m |
| Area | 2.6 $km^2$ |
| Depth | 1524 m |
| Pressure | 1900 psia |
| Fracture water saturation | 100% |
| Permeability | 20 md |
| Fracture porosity | 0.02 |
| Water viscosity | 0.73 |
| Salinity | 30000ppm |
| Total compressibility | 9 E-06 |
| Gas injection rate | 25000 tons / year |
| Injection duration | 25 years |
| Shut in period | 75 years |
| Wellbore radius | 0.1m |
| Temperature | 39 deg cel. |
| No of wells | 2 |

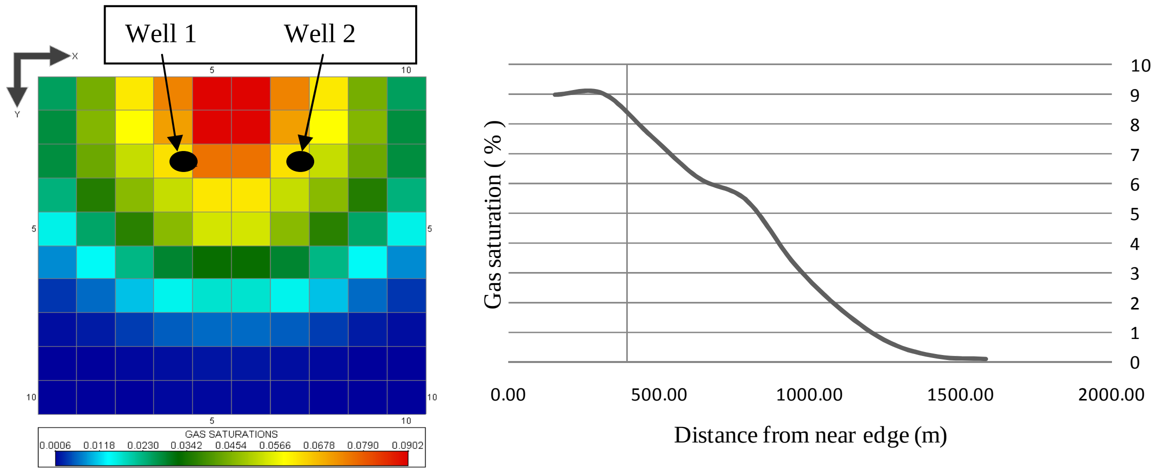



**Figure 2:** *$CO_2$ gas saturation profile at 25 years and gas saturation around a well*

The gas saturation levels and $CO_2$ contact level have been estimated under various reservoir conditions as illustrated in Figure 2 in order to estimate the storage capacity using various trapping mechanisms. Figure 3 illustrates the gas saturation level corresponding to various sand layer thicknesses from COMET3 modelling. There is a decline of gas saturation with increasing layer thickness, mainly due to the gas saturation being lowered with the reduced aquifer contact level for thicker saline aquifers.

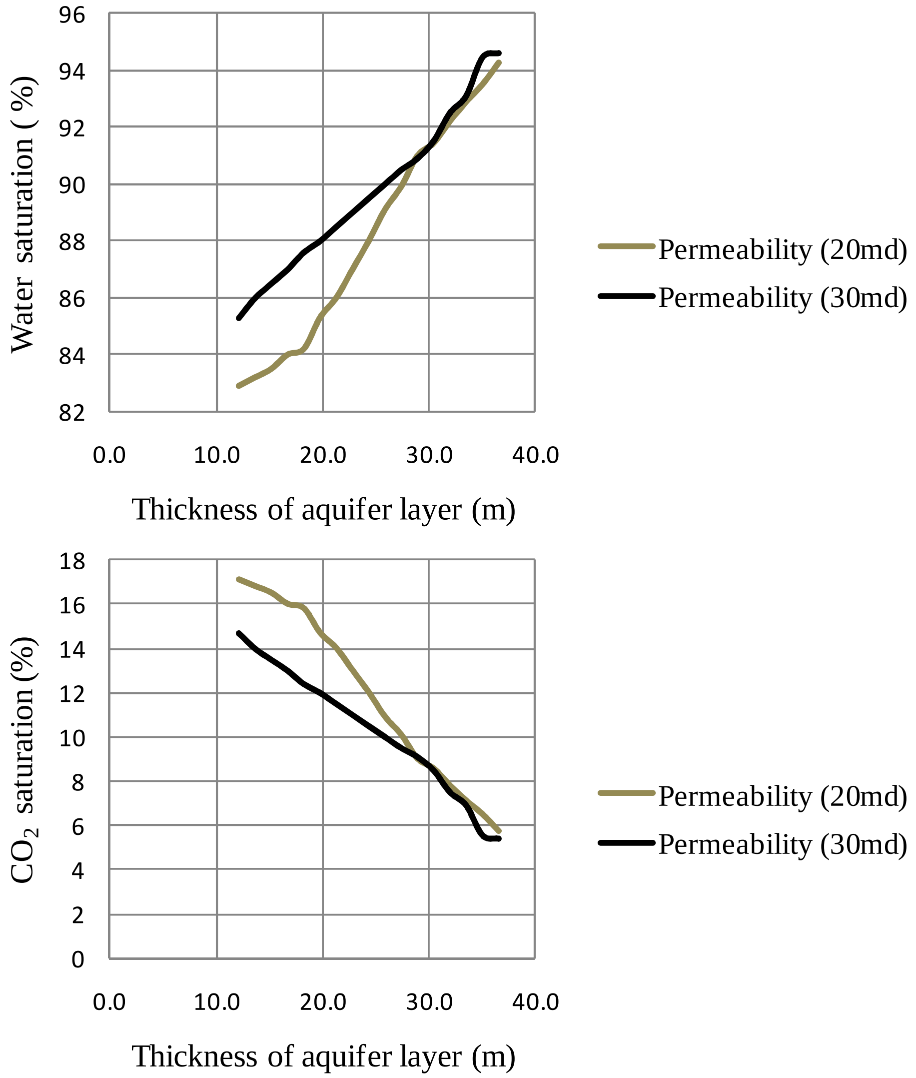



**Figure 3:** *$CO_2$ saturation variation against aquifer thickness*

Porosity is also an important parameter to be considered in terms of storage potential in a typical saline aquifer. Figure 4 illustrates the variation of $CO_2$ storage saturation and water saturation against porosity. Gas saturation is estimated for porosities ranging from 0.20 to 0.55. At a porosity of 0.55, gas saturation is estimated to be zero for 20md and 30md permeability levels. Porosity and $CO_2$ saturation show a negative relationship with the dissolution of water in saline aquifers where higher porosities have higher water saturation, and hence large dissolution of $CO_2$ in water levels; this is due to the interaction of parameters such as salinity level and permeability leading to higher dissolution in saline water with increasing porosity values. The storage variation is correlated to the varying gas saturation levels for structural, residual and solubility trapping mechanisms.

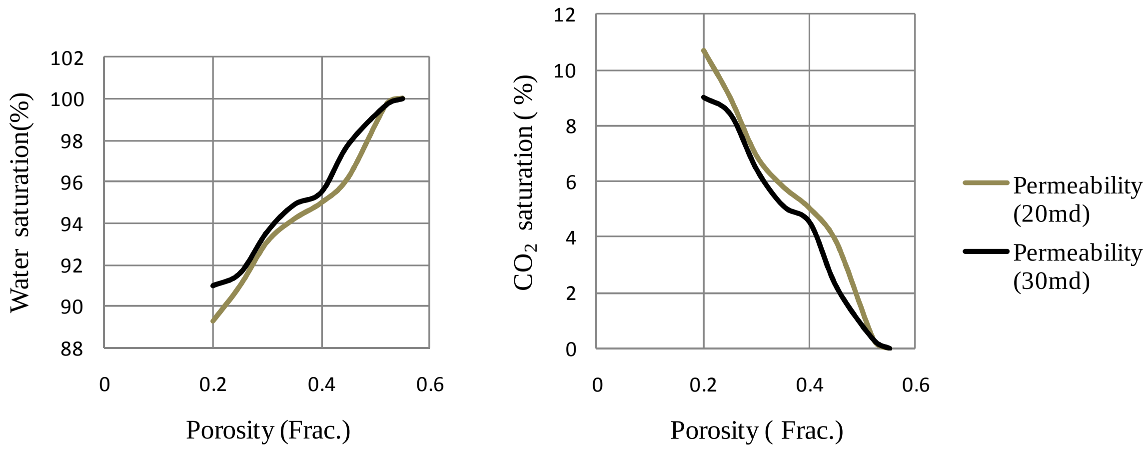



**Figure 4:** *$CO_2$ saturation, water saturation against porosity*

## 3.0 Discussion

Storage efficiency found in the existing research is still very conservative and generally yields values between 2 – 17% (Bradshaw et al., 2004). This may be due to inadequate consideration of the respective trapping mechanisms. Studies of the storage efficiency of an aquifer serve to identify the potential aquifers for carbon dioxide sequestration and are usually conducted in relation to site selection. However, storage efficiency is usually discussed on a site-specific scale and that there is a lack of a generic formula that can be used for a quick assessment of the storage efficiency at any random site. For the hydrodynamic trapping mechanism, there is a lack of mathematical formulae in the existing literature to estimate the storage capacity of carbon dioxide. The volume of carbon dioxide that can be stored by all the other trapping mechanisms except mineralisation can be calculated by a factor multiplied by the volume of the trap and its porosity. Since hydrodynamic trapping is a combination of the different trapping mechanisms, the storage capacity using hydrodynamic trapping might be calculated by the representative overall volume of traps and porosities, multiplied by a time-dependent factor, sine hydrodynamic trapping is also time-dependent. There are studies on the effect of temperature on pressure and solubility of the aquifers individually and the subsequent effects on the storage capacity of the aquifer.

## 4.0 Conclusions

This paper presents a review of current methodologies adopted for the estimation of saline aquifer storage. Gas saturation level is estimated for various porosities and thickness levels of saline aquifers in order to estimate the potential $CO_2$ capacity. The numerical estimations found using COMET3 software has been used to estimate the $CO_2$ gas saturation for a specific field scenario. This methodology does not include the provision of storage by mineral trapping and hydrodynamic trapping. In order to improve the storage efficiency levels of trapping mechanisms, one needs to review the extent of storage potential especially in mineral trapping and hydrodynamic trapping.

## 5.0 References

[1]. Bachu, S., Bonijoly, D., Bradshaw, J., "$CO_2$ storage capacity estimation: Methodology and gaps." International Journal of Greenhouse Gas Control,2007; **1**(4): 430-443.

[2]. Bachu, S.,Adams, J. J.,"Sequestration of $CO_2$ in geological media in response to climate change: capacity of deep saline aquifers to sequester $CO_2$ in solution." Energy Conversion and Management,2003;**44**(20): 3151-3175.

[3]. Bachu, S., Gunter, W. D., Perkins, E. H.,"Aquifer disposal of $CO_2$: Hydrodynamic and mineral trapping." Energy Conversion and Management,1994; **35**(4): 269-279.

[4]. Bradshaw, J., Allinson, G.,Bradshaw, B. E., "Australia's $CO_2$ geological storage potential and matching of emission sources to potential sinks,2004;." Energy **29** (9-10): 1623-1631.

[5]. Bruant, R. G., Jr, Celia, Jr. M. A., Guswa, A. J. , Peters, C. A., "Peer Reviewed: Safe Storage of $CO_2$ in Deep Saline Aquifiers." Environmental Science & Technology,2002;**36**(11):240A-245A.

[6]. DOE (2007). "Carbon sequestration atlas of the united stated and canada." 1-90.

[7]. Ghanbari, S., Al-Zaabi, Y., Pickup, G. E.,"Simulation of $CO_2$ Storage In Saline Aquifers." Chemical Engineering Research and Design,2006;**84**(9): 764-775.

[8]. Nakanishi, S., Mizuno,Y.,Okumura, T., "Methodology of $CO_2$ aquifer storage capacity assessment in Japan and overview of the project." Energy Procedia,2009;**1**(1): 2639-2646.

[9]. Shafeen, A., Croiset, E.,Douglas, P. L., Chatzis, I."$CO_2$ sequestration in Ontario, Canada. Part I: storage evaluation of potential reservoirs." Energy Conversion and Management,2004; **45**(17): 2645-2659.

[10]. Soares, J. L., Oberziner, A. L. B., Jose, H. J., Rodrigues, A. E., Moreira, R. F. P. M., "Carbon Dioxide Adsorption in Brazilian Coals." Energy & fuels,2006;**21**(1): 209-215.

[11]. Xu, T., Apps, J. A. , Pruess, K., "Numerical simulation of $CO_2$ disposal by mineral trapping in deep aquifers." Applied Geochemistry,2004;**19**(6): 917-936.

Zhang, W., Li, Y., Xu, T., "Long-term variations of $CO_2$ trapped in different mechanisms in deep saline formations: A case study of the Songliao Basin, China." International Journal of Greenhouse Gas Control,2009;**3**(2): 161-180.

# DYE SENSITIZED SOLAR CELL BASED ON HYDROTHERMALLY SYNTHESIZED TITANIA NANOTUBES

J Akilavasan[1], P R Chandrasekera[1,2], J Bandara[1*]

[1]Institute of Fundamental Studies, Hanthana Road, Kandy, Sri Lanka.

[2]Uwa Wellassa University, Badulla, Sri Lanka.
*Corresponding author: jayasundera@yahoo.com

**Abstract**
Titania nanotubes were introduced into conventional Dye Sensitized Solar Cell (DSSC) as an electron conducting medium as they provide straight pathway to electron transport. Titania nanotubes were synthesized via hydrothermal treatment of commercially available $TiO_2$ powder and deposited onto the conducting substrate FTO (F doped $SnO_2$) via electrophoretic deposition technique. Thin film of $TiO_2$ nanotubes was found to be nearly 10 nm diameter and ~200 nm long. Solar cell fabricated with $TiO_2$ nanotube and sensitized with N3 dye showed an open circuit voltage (Voc) of 0.75 V and short circuit current (Jsc) of 3.81 mA/cm$^2$ under AM 1.5 G irradiation. The cell performance further improved by treating the titania nanotubes electrode with $TiCl_4$.
***Key words:*** *Dye Sensitized Solar Cell, nanotube, hydrothermal, nanocrystalline*

## 1. Introduction

In 1991, Michael Greatzel introduced a new type of solar cell called DSSC which is a promising and best alternative concept to conventional silicon based solar cells with very low material cost making economically viable[1]. These devices consist of a wide band gap semiconductor covered by a monolayer of sensitizing dye, an electrolyte and a counter electrode. The semiconductor is directly supported by a transparent electrode on one side, while the dye is connected to the back electrode via a liquid electrolyte or a solid hole conducting material. The initial step of the photovoltaic process is a light induced electron injection from the dye into the semiconductor material. This process yields an oxidized dye and an energetic electron. Rapid regeneration of the dye by the electrolyte prevents back transfer of the electron or degradation of the photo-oxidized dye. Meanwhile, the energetic electron diffuses away from the dye, passing through the electrode and an external load, finally reaching the counter electrode where it regenerates the redox couple.

In conventional DSSC, despite of the high surface area, random nature of the titania nanocrystalline particle network results in electron to recombine as they move through the surface of the titania nanocrystalline network reducing cell performance[2]. In order to facilitate direct electron transfer within the electron conducting material, 1-D nanomaterials such as arrays of nanowires and nanotubes can be introduced instead of particle network as they provide short direct pathways. In DSSC, high electron collecting efficiency has been observed when 1-D nanostructure such as ZnO, $TiO_2$ were employed[3]. Compared to other nanostructured materials, $TiO_2$ has better performance giving higher open circuit voltages and fill factors leading to high efficiency. There are several methods available to synthesis titania nanotube such as ZnO template method[4], anodization[2], hydrothermal method [5-7]. Among these methods, hydrothermal is being used as it provides easy synthesizing method, cost effectiveness and possibility of bulk production. Though, anodization method provides well aligned array of nanotubes which results in better performance of the cell,

the fabrication cost is high compared to hydrothermal method. In this investigation titania nanotubes have been synthesized via hydrothermal treatment of commercially available titania powder.

## 2. Material and methods

Titania nanotubes were synthesized via hydrothermal treatment of titania nanoparticles: 2 g of $TiO_2$ powder (Degussa P 25) dispersed in 10 M NaOH $_{(aq)}$ solution by stirring for one hour followed by transferring into a Teflon lined autoclave. The autoclave was kept at 150$^0$C for 48 hours. The resulted precipitate was washed with distilled water and 0.1 M HCl until pH reaches 8.5. $TiO_2$ nanotube working electrode was prepared by using this precipitate by electrophoretic deposition technique on a

FTO substrate (Sheet resistance 8 Ohm/cm$^2$). The electrolyte for electrodepositon was parpared by ultra sonicating the precipitate for 60 minutes to make a nanotube suspension. The electrolyte was prepared by mixing nanotube suspension and methoanol in 2:1 volume ratio. Pt was used as counter electrode and FTO substrate was kept as anode at optimized voltage of 40 V for 5 minutes in a two electrode system. After electrodeposition of $TiO_2$ nanotubes on FTO, the film was dried at 130 $^0$C for 10 minutes followed by sintering at 450$^0$C for 30 minutes. The electrode was immersed into the dye solution containing 0.3 mM cis-bis(isothiocyanato)bis(2,2'-bipyridyl-4,4'-dicarboxylato)-ruthenium(II) (N3, Solaronix) for 12 hours. Finally, DSSC was assembled using $TiO_2$ nanotube working electrode and Pt counter electrode using iodine/triiodine redox couple as an electrolyte in between these two electrodes. Cell performance was measured under AM 1.5 G solar simulation using home made I-V analyzer based on Keithley 2000 multimeter and photentiostat POT1000M. Scanning Electron Microscopy (SEM) and UV-Visible analysis were used to study the surface morphology and absorption measurements respectively of the film prepared. The nanotube based electrode was treated with $TiCl_4$ in some cases. For this, two drops of 0.04 M $TiCl_4$ was dropped onto the surface of the $TiO_2$ nanotube coated electrode followed by washing in distilled water then again the film was sintered at 450 $^0$C for 30 minutes.

## 3. Results and Discussion

The electrodeposition time was optimized using uv-visible absorption (transmittance) analysis. The titania nanotubes was deposited on to the FTO surface by varying deposition time from 1 to 10 minutes. With the increase in deposition time from 1 to 5 minutes, the film thickness increases. With further increase in deposition time even thicker film could be obtained however, these films tend to peel-off during sintering process. We noted formation of nearly uniform $TiO_2$ film in 5 minutes electrodpeosition time.

The formation of fairly uniform $TiO_2$ nanotube films on FTO glass substrate with the optimum electrodepositon conditions was confirmed by the SEM analysis. Figure 1 shows the SEM image of the $TiO_2$ nanotubes synthesized via hydrothermal method which confirmed the formation of nearly 10 nm diameters and average length of 200 nm nanotubes.

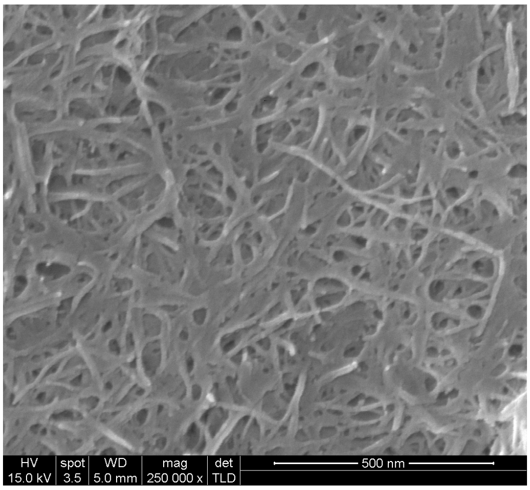



Figure 1: *SEM image of the hydrothermally synthesized titania nanotubes*

Also, it can be clearly seen in the SEM image, that there are few uncurled $TiO_2$ nano-structures which are similar to $TiO_2$ nanoribbon or nanobelt. In hydrothermal method, depending on the preparation conditions such as reaction temperature, reaction time, reactant concentration and the aciditic/basicity of the reaction medium, defects could be formed. Changchun et.al reported formation of only titnia nanoparticles with anatase and rutile phase following the same experimental conditions and procedure described above except the concentration of the NaOH (5 mol/L), indicating that at low basicity, formation of $TiO_2$ nanotube does not occur or if formed they are with defects[7].

Figure 2 and Figure 3 show the I-V characterization of the solar cells fabricated with $TiO_2$ nanotube films that are treated with $TiCl_4$ and untreated films respectively and which were sensitized with Ru-dye and the results are summarized in Table 1.

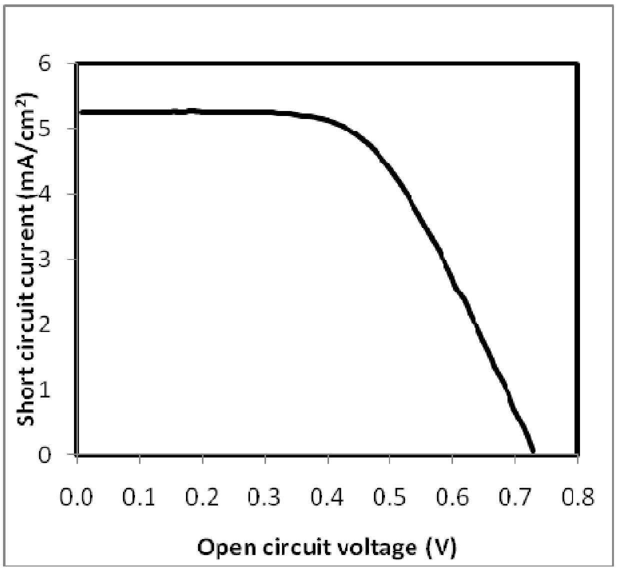



Figure 2: *The I-V characterization of the $TiO_2$ nantotube electrode after $TiCl_4$ treatment sensitized with N3 dye under AM 1.5 G irradiation.*

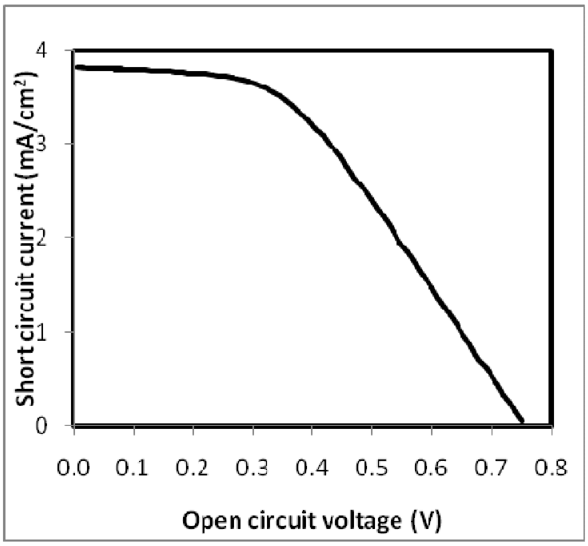



Figure 3: *The I-V characterization of the $TiO_2$ nantotube electrode before $TiCl_4$ treatment sensitized with N3 dye under AM 1.5 G irradiation..*

Table 1: *DSSC parameters for $TiO_2$ nantotube electrodes sensitized with N3 dye under AM 1.5 G irradiation.*

| | $J_{sc}$ (mA/cm$^2$) | $V_{oc}$ (mV) | FF (%) | $\eta$ (%) |
|---|---|---|---|---|
| Bare $TiO_2$ nanotube electrode | 3.81 | 750 | 45 | 1.28 |
| $TiO_2$ nanotube electrode treated with $TiCl_4$ | 5.24 | 728 | 58 | 2.21 |

It is clearly seen that the $TiCl_4$ treatment increases the overall solar cell performance compared to $TiCl_4$ untreated films. It has been shown that the $TiCl_4$ treatment of mesoporous $TiO_2$ layer enhances the overall solar cell performance of DSSC due to increase in dye uptake and change in surface properties of mesoporous $TiO_2$ layer.[8,9] Dye adsorption analysis and surface morphology study of pristine $TiO_2$ nanotube and $TiCl_4$ treated $TiO_2$ nanotube revealed the same trend. Figure 4 shows the dye absorption spectra of $TiCl_4$ treated and untreated $TiO_2$ nanotube arrays. As shown in Figure 4, the optical absorption of Ru-TPA-NCS dye coated $TiCl_4$ treated $TiO_2$ nanotube electrode is higher than that of the bare $TiO_2$ nanotube electrode indicating that enhanced dye uptake by $TiCl_4$ treated $TiO_2$ nanotubes which in turn resulted in higher $J_{sc}$ than untreated $TiO_2$ nanotube electrode. Furthermore, it has been reported that the $TiCl_4$ treatment of $TiO_2$ results in increased charge injection efficiency as a result of a positive shift in the flat-band potential of $TiO_2$. [8, 9] Also it has been suggested that the thin $TiO_2$ layer formed on mesoporous particles may acts as a charge recombination barrier preventing electron access to the surface. Therefore, we believe that a combination of effects of enhanced dye adsorption, efficient charge injection and reduced charge recombination, resulted in higher solar cell efficiency in $TiCl_4$ treated $TiO_2$ nanotube.

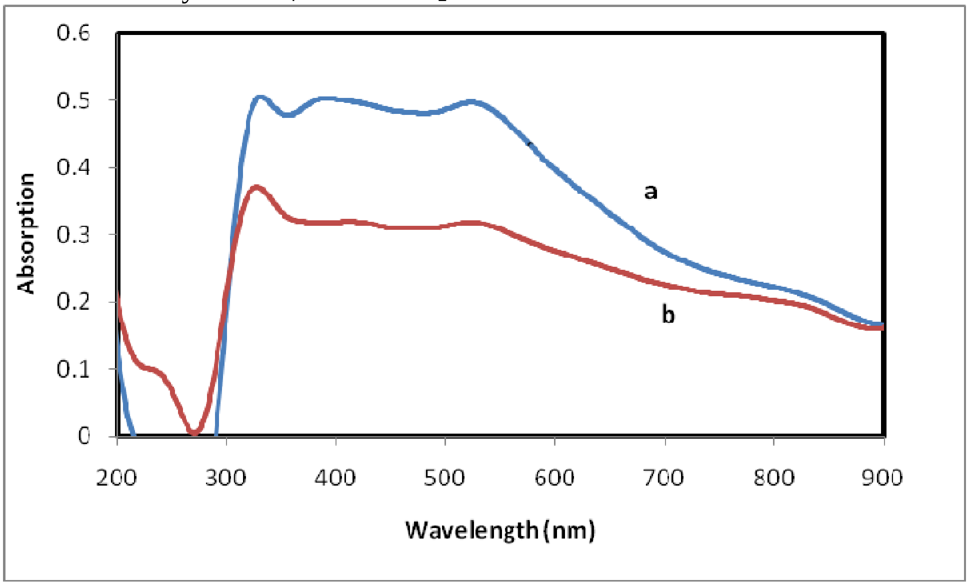



Figure 4: *UV-Visible absorption spectroscopy of titania nanotubes depositied on FTO substrate sensitized with Ru dye (N3). a) Electrode treated with $TiCl_4$. b) Untreated*

The solar cell performance reported in this investigation is low compared to that of reported values. We believe that this could be due to the defect present in our nanotubes such as nanoribbons and nanobelts. Furthermore, as shown in Figure 1, nanotubes were randomly oriented resulting in zig-zag movement of electrons in between nanotubes leading to enhance charge carrier recombination. Therefore increasing the orientation of nantoube would provide opportunity to improve the solar cell efficiency further and our research is been focused on this aspect.

## 4. Conclusion

In this investigation $TiO_2$ nanotubes were synthesized by hydrothermal method and fabricated in to Dye Sensitized Solar cell. Reaction conditions such as temperature and concentration are found to be very important in hydrothermal treatment for formation of titania nanotubes. Treatment of $TiCl_4$ with $TiO_2$ nanotubes increases the short circuit current and efficiency roughly 40 % and 72 % respectively. The performance of the cell largely depends on the ordered and vertical alignment of the nanotubes. Therefore, in order to improve the cell performance nanotubes should be deposited in a well ordered manner.

**References**

1. B. O'Regan, M.Gratzel, *Nature*, **1991**,353,737-740
2. Chengkun Xu, Paul H.Shin, Liangliang Cao,Jiamin Wu,and Di Gao *Chem.mater*,**2009**, 22(1),143-148
3. Elena Galoppini, Jonathan Rochford Hanhong Chen, Gaurav Saraf, Yicheng Lu, Anders Hagfeldt and Gerrit Boschloo, *J.Phys.Chem.B*, **2006**, 110(33), 16159-16161
4. Seok-In Na, Seok-Soon Kim, Woong-Ki Hong, Jeong-Woo Park, Jang Jo, Yoon-Chae Nah, Takhee Lee, Dong-Yu Kim, *Electrochemica acta*, **2008** ,53,2560-2566

5. Gil-Sung Kim, Hyung-Kee Seo, V.P. Godble, Young-Soon Kim, O-Bong Yang, Hyung-Shik Shin, *Electrochemistry communication*, **2006**,8,961-966
6. Gil-Sung Kim, Young-Soon Kim, Hyung-Kee Seo and Hyung-Shik Shin, *Korean J. Chem.Eng.*,**2006**, 23(6), 1037-1045
7. LI Zhenhua, LIU Zhongqing, YAN Qinghi,WANG Yichao, and GE Changchum, *rare metals*, **2008**,27(2),187
8. P.M. Sommeling, B.C. O'Regan, R.R.Haswell, H.J.P.Smit, N.J.Bakker, J.J.T.Smits, J.M.Koon, J.A.M. van Roosmalen, *J. Phys.Chem.B*, **2006** ,110,19191-19197
9. Brian C.O'Regan, James R. Durrant, Paul M. Sommeling, Nicolaas J.Bakker, *J. Phys. Chem. C*, **2007**,111, 14001-14010

# STUDIES ON BIOSYNTHESIS OF GOLD NANOPARTICLES BY *FUSARIUM OXYSPORUM*

Deepa K. and Tapobrata Panda*

Biochemical Engineering Laboratory, MSB 140A & MSB 235A, Department of Chemical Engineering,
Indian Institute of Technology Madras, Chennai - 600 036,
Tamil Nadu, India.

E-mail:*pandaiitm@gmail.com
*Corresponding author Tel.: + 91-9840708563 (T. Panda)

***Abstract***
Gold nanoparticles find applications in medicine, catalysis, electronics, and optics owing to their unique properties. Synthesis of gold nanoparticles by *Fusarium oxysporum* was studied as one of the efficient and eco-friendly alternatives. The extracellular fungal extract was analyzed for its bioreduction potential. The effects of changes in concentration of the precursor salt on the nanoparticle properties were investigated. The initial concentration of the gold ions influences the size of the nanoparticles. The results suggest the potential of employing biological extracts to synthesize nanoparticles and to control nanoparticle size and morphology.

***Keywords:*** Nanoparticles, gold, biosynthesis, *Fusarium oxysporum*

## 1. Introduction

Nanotechnology has revolutionized the modern world due to its various applications in simple domestic products such as toothpastes, clothing, etc., as well as core research areas of biomedical sciences, catalysis, electronics, and optics. One of the biggest challenges in nanotechnology is to develop the technique for the controlled synthesis of metal nanoparticles of characteristic size, shape, and its composition [1]. The limitations of the conventional routes of nanoparticles synthesis are the lack of monodispersity and instability of the resulting nanoparticles which makes it difficult to find useful applications. Other disadvantages include the usage of high temperatures, toxic reducing agents, and organic solvents. So researchers look for an alternative economically viable and ecologically safe procedure that could overcome such limitations. In this context, biological systems can be considered as a favorable option since organisms have the natural tendency to synthesize and/or to organize intricate nanostructures.

Several reports are available on the biosynthesis of nanoparticles. The biological sources employed for synthesizing the nanoparticles encompass a wide variety of organisms (or their extracts) ranging from viruses, bacteria, actinomycetes, yeasts, fungi, plants, etc., Microorganisms provide additional advantages concerning the ease of handling and manipulation of the biomass. Most of the experimental protocols use cells harvested after completion of growth of the microorganisms. Though this method yields nanoparticles of varying shapes and sizes, it is possible to attain better shape- and size-control by varying the microbial system used for the reaction [2], age of cells [1], etc.,

In this research, results are reported on the effect of varying initial concentration of the gold salt on the properties of gold nanoparticles synthesized using the fungal extract of *Fusarium oxysporum*.

## 2. Materials and methods

*Fusarium oxysporum* MTCC 284 was obtained from MTCC, IMTECH, Chandigarh, India. The organism was maintained on malt extract agar slants by subculturing at regular intervals. After proper incubation, the organism was transferred into the growth medium aseptically and incubated in a rotary shaker (100 rpm) at 30 $^{o}$C. After appropriate growth, cells were separated from the broth by centrifugation. The precursor salt, gold (III) chloride trihydrate, was then added to the cell-free supernatant to get a desired final concentration of gold ions in the solution. The initial concentrations of the gold ions used are in the range of 0.1 mM to 1 mM. A suitable control was included in all sets of experiments to ensure that no other salts present in the growth medium are responsible for the chemical reduction of the gold ions. The mixtures were then incubated in the dark at 30 $^oC$. Aliquots of samples were collected at regular intervals and analyzed by UV-visible spectroscopy, HRTEM imaging, and X-ray diffractometry.

UV-vis spectroscopy was performed on an ELICO SL 150 spectrophotometer. HRTEM analysis was carried out on a JEOL 3010 microscope. Samples were spotted on carbon coated copper grids and dried at room temperature. Images were collected at an accelerating voltage of 200 keV. X-ray diffraction analysis was carried out using a Bruker Discover D8 diffractometer. The results were used to account for the extracellular formation of gold nanoparticles obtained by the action of metabolic products secreted into the medium after the growth of cells.

## 3. Results

The formation of gold nanoparticles in the reaction medium was observed as visible color change from the original pale yellow color of the extract to a vivid purple. Figure 1 is a representative UV-visible spectrum of the reaction mixture (extract from resting cells) containing 1 mM $HAuCl_4{\cdot}3H_2O$. It shows a distinct absorption maximum centered around 540 nm, nearly 12 hours after the start of the reaction, whose intensity increases with time. This characteristic peak is the presumptive test for the formation of gold nanoparticles.

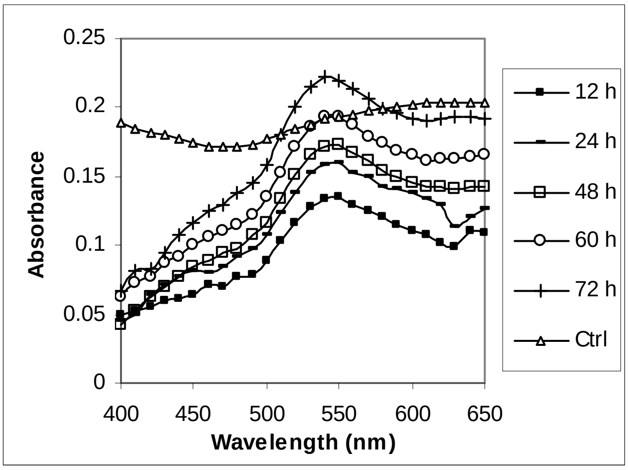



***Fig. 1*** *UV-visible spectrum of the extract obtained from resting cells of Fusarium oxysporum, exposed to 1 mM* $HAuCl_4$

The gold nanoparticles were further characterized using HRTEM imaging system (Figure 2). Nanoparticles of sizes between 10 nm and 50 nm were obtained using the extract from resting cells when treated with 1 mM $HAuCl_4$. Pentagonal and hexagonal bipyramidal shapes were obtained along with triangular and hexagonal nanoplates. The presence of twin boundaries was evident in some of the bipyramidal shapes as indicated by the arrows.

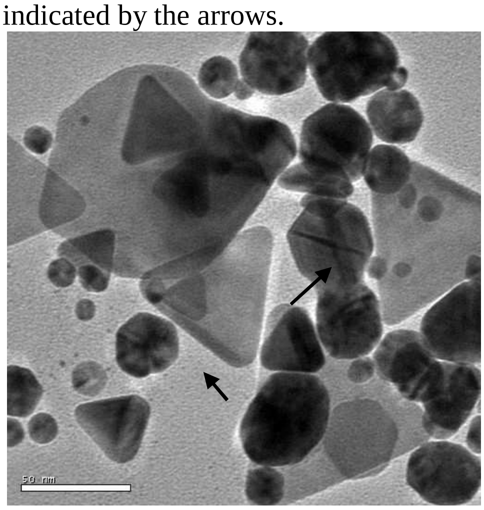



***Fig. 2*** *HRTEM image of gold nanoparticles obtained from the extract of resting cells exposed to 1 mM* $HAuCl_4$

Initial concentration of gold ions between 0.1 and 0.5 mM shows significant reduction in the sizes of the nanoparticles. The bipyramidal shapes of the nanoparticles were retained in all the cases. The XRD pattern shows the presence of peaks corresponding to facets of face-centered cubic (fcc) structure of gold.

## 4. Discussion

The change in color of the reaction mixture and the presence of the characteristic absorption peak around 550 nm, due to the surface plasmon absorption of gold nanoparticles, are considered as indications for the formation of gold nanoparticles. Further, HRTEM imaging gave information about the size and shape distribution of the nanoparticles. The particles were highly polydisperse at 1 mM $HAuCl_4$ concentration. Irrespective of the different concentrations of the gold ions, the pentagonal and hexagonal bipyramidal shapes was observed recurrently, which accounts for the reproducibility of the process. The initial concentration of the gold ions used in the experiments was found to be a factor capable of influencing the size of the nanoparticles. The XRD patterns obtained confirm that the nanoparticles are indeed crystalline.

## 5. Conclusions

We have demonstrated the synthesis of gold nanoparticles by the extracellular extract of resting cells of *Fusarium oxysporum*. With monodispersity still posing a distant goal to attain in the nanoparticles' synthesis protocols, the use of green technology to not only synthesize nanoparticles of unique shapes, but also achieve control over particle size seems to be a viable and convenient option. Furthermore, the unique bipyramidal shapes are known to have interesting properties which can find useful applications in electronics and sensing.

### *References*

[1] Gericke, M., Pinches, A., "Biological synthesis of metal nanoparticles", *Hydrometallurgy*, 2006;83: 132-140.

[2] Xie J., Lee J.Y., Wang D.I.C., Ting Y.P., "High-yield synthesis of complex gold nanostructures in a fungal system", *Journal of Physical Chemistry C*, 2007;111: 16858-16865.

# AIR FLOW THROUGH MACRO POROUS CERAMIC STRUCTURES

Dharmasena R.R[a] , Perera D.D.M[a] , Ekanayake P[a,b],Karunaratne B.S.B[a]

[a]Department of Physics, University of Peradeniya, Sri Lanka

[b]Department of Physics, Faculty of Science,
University of Brunei Darussalam, Jalan Tungku Link, Gadong BE1410, Brunei

Corresponding author: Ekanayake P,
E-mail: piyasiri.ekanayake@ubd.edu.bn , Telephone: +6738953093

**Abstract**
Macro porous ceramic bodies which are specifically designed for filtration purposes, cooling systems and porous medium burning were fabricated in order to study the three dimensional network of porous structures using Darcy Forcheinhimer equation. Fitting parameters obtained from Darcy Forcheinhimer equation could be used to fabricate macro porous ceramic bodies with required porosity.

## 1. Introduction

Solid materials that are permeated by an interconnected network of pores are known as porous materials. Natural and manufactured porous ceramic materials have broad applications in engineering processes, including filters, heat sinks, mechanical energy absorbers, heat exchangers, pneumatic silencers, high breaking capacity fuses, cores of nuclear reactors and gas burners.

Property evaluation, when fabricating porous ceramic structures, is important in order to assess the performances of the sintered products. Currently there are several methods to investigate the properties of porous bodies, such as, permeability measurements, porosity measurements, stiffness measurements and bulk density measurements. In this study, the dependence of the air permeability on porosity of the sintered samples was investigated. Macro porous ceramic bodies were fabricated according to a fabrication method introduced previously for gas burners (*R.R. Dharmasena, P. Ekanayake and B.S.B. Karunaratne).* Controlled porosity and pore connectivity were achieved inside the ceramic structure using this fabrication method. The internal structure (the interconnectivity of the pores) was studied in three dimension by applying an air flow through a sample. For this, Darcy Forchheimer's equation (*S.M.H.Olhero et al)* and a modified Darcy Forchheimer's equation (*W. Zhiyong et al)* were used in correlating the pressure drop and air flow velocity.

Pressure gradient along the Y direction $\Delta P/\Delta Y$ is given by the Darcy Forchheimer equation (*S.M.H.Olhero et al* );

$$\frac{\Delta P}{\Delta Y} = K_1 \mu U_o + K_2 \rho U_o^{\ 2}$$

where, $K_1$ is the coefficient at low air flow, $K_2$ is the coefficient at high air flow , $\rho$ is the density of air, $\mu$ is the viscosity of air and $U_o$ is the velocity of air flow. In general $K_1$ and $K_2$ give the resistance to the air flow through porous body.

A mean value of pore sizes can be obtained from a modified Darcy Forchheimer equation (*W. Zhiyong et al).* Using the modified Darcy Forchheimer equation it was attempted to correlate the expected pore sizes and observed pore sizes.

Pressure gradient along the Y direction $\Delta P/\Delta Y$, and mean pore size *d* is given by the modified Darcy Forchheimer equation;

$$\frac{\Delta P}{\Delta Y} = \frac{(1039 - 1002\varepsilon)}{d^2}\mu u + \frac{0.5138\varepsilon^{-5.739}}{d}\rho u^2$$

## 2. Experimental method

Polystyrene beads were used as the pore former. Nine cubic shaped green samples were fabricated under different compressing forces (10 N,20 N, 30 N) for three different pore former sizes (6 mm,7 mm, 8 mm) and sintered at 1250 °C for two hours to obtain porous samples. An air flow was applied through the sample in one direction at a time and the velocity of air and pressure drop across the sample were obtained (fig.1.0).

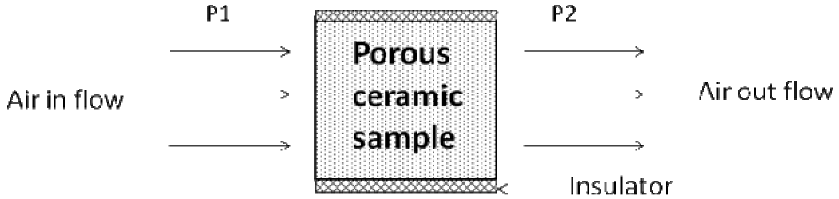



*Figure 1.0*

Observed readings were correlated with Darcy Forchheimer's equation and a modified Darcy Forchheimer equation.

## 3. Results and Discussions

Porosity values were determined for different pore sizes of porous ceramic cubes and it was observed that porosity has increased with the pore former size (table.1). The pressure gradient *vs* velocity of air through porous ceramic samples was plotted and the curve fitting was done according to the Darcy Forchheimer's equation and the modified Darcy equation separately. From the curve fittings, the constants ($K_1$, $K_2$) and mean pore sizes were obtained along each direction. In the fabricated porous ceramic body, the pore size is generally determined by the size of the pore formers. The mean pore sizes obtained by the fittings are in good agreement with pore former size (table.1.0).

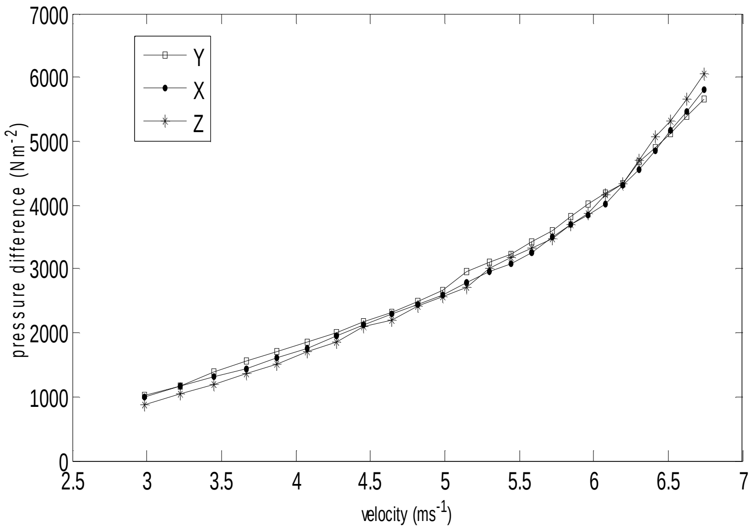



*Fig.1.1 Pressure difference vs velosity of air through 6 mm-10 N porous ceramic body in X, Y and Z direction (compresing force when fabricating the samples was in Y direction)*

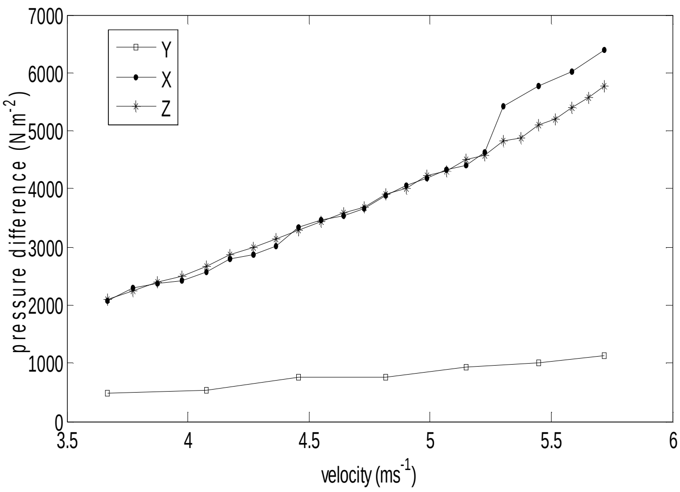



*Fig.1.2 Pressure difference vs velosity of air through 7 mm-10 N porous ceramic body in X, Y and Z direction (compresing force when fabricating the samples was in Y direction)*

Y direction is the compressing direction when samples were fabricated and X and Z directions are the perpendicular directions to Y direction

It was observed that the pressure difference along X, Y and Z directions are the same for 6 mm-10 N sample (fig. 1.1). Therefore, it can be concluded that 10 N force is not capable of making any difference in the connectivity of pores along all three directions in 6 mm pore sized sample. Moreover, for 7 mm-10 N sample, pressure difference along Y direction is the lowest and X, Z directions behave similarly in the velocity range of 3.5 - 5.5 ms$^{-1}$ (fig1.2).

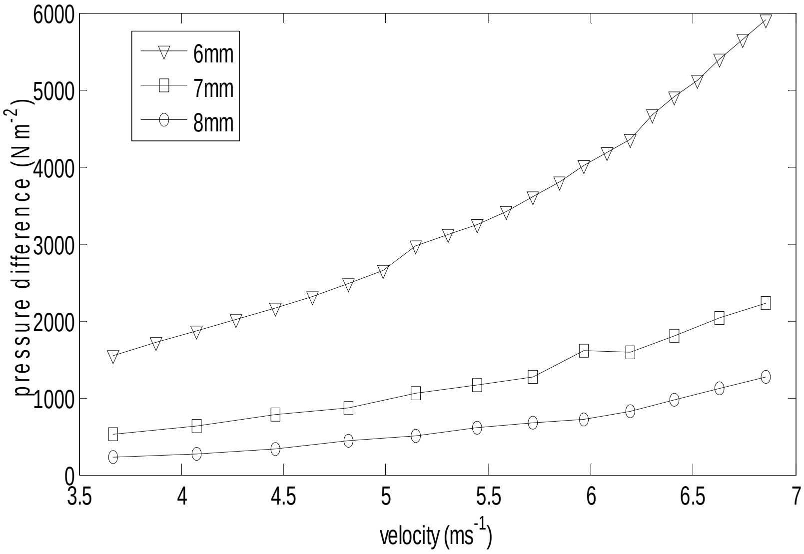



*Fig.1.3 Pressure gradient vs velosity of air through 10 N porous ceramic bodies of 6 mm, 7 mm, 8 mm pore forner sizes (compresing force when fabricating the samples was along Y direction)*

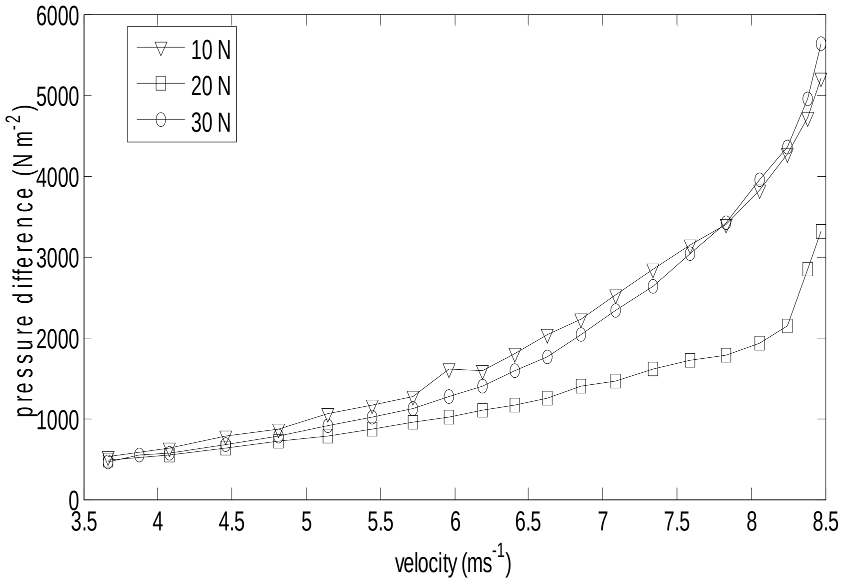



*Fig.1.4 pressure gradient vs velosity of air through 7 mm (pore former size) porous ceramic bodies of 10 N, 20 N, 30 N applied pressures (compresing force when fabricating the samples was along Y direction)*

It was also observed that the pressure gradient increases with decreasing pore former size (from 8 mm to 6 mm) in Y direction (fig 1.3). The porous ceramic body prepared with pore former size of 7 mm showed the expected behavior of pressure gradient at each applied compression .In this sample, the pressure gradient decreases with the compressing force (fig.1.4)

*Table 1: porosity of different porous ceramic materials made under different compressing forces*

| Sample | Ceramic volume (ml) | Y direction | | | Porosity |
|---|---|---|---|---|---|
| | | $K_1$ ($mm^2$) | $K_2$ (mm) | Mean cell size (mm) | |
| 6 mm-10 N | 24.0 | 3.91 | 2.22 | 5.2 | 0.65 |
| 7 mm-10 N | 15.5 | 20.83 | 0.71 | 7.1 | 0.76 |
| 8 mm-10 N | 12.5 | 10.10 | 0.31 | - | 0.81 |
| 6 mm-20 N | 19.5 | 200.00 | 1.23 | 3.2 | 0.68 |
| 7 mm-20 N | 11.0 | 20.83 | 0.60 | 9.1 | 0.79 |
| 6 mm-30 N | 24.5 | 50.00 | 12.35 | - | 0.63 |
| 7 mm-30 N | 15.0 | 9.62 | 0.68 | - | 0.76 |

## 4. Summary and Conclusions

It was attempted to obtain an idea about the 3D connectivity of pores in a porous ceramic sample which was fabricated according to a previously determined fabrication method (*R.R. Dharmasena, P. Ekanayake and B.S.B. Karunaratne).* The dependence of pressure gradient with the pore former size and compressing forces along the three directions were observed. The pressure gradient *vs* velocity along each direction was plotted and curve fitting was done with the Darcy Forchheimer's equation and the modified Darcy Forchheimer's equation for each sample. The permeability constants and mean cell sizes were obtained for each sample. 10 N compressing force is not capable of making any differences in the pore connectivity along the three directions in the case of pore former size 6 mm. A clear difference of the pressure gradients along the compressing direction was shown by the sample 7 mm-10 N. Pressure gradient decreases with increasing the compression and with increasing pore former size.

## 5. References

R.R.Dharmasena, P.Ekanayake, B.S.B.Karunaratne.(2009). Fabrication of macro porous ceramic bodies using polystyrene spheres . *Publication of PURSE 2009 University of Peradeniya*

B.S.B.Karunaratne and S.T.Kumaratunga. (2000). Fabrication of clay based ceramics for engineering applications. *A Journal of institute of physics-Sri –Lanka*

S.M.H.Olhero, J.M.P.Q.Delgado, J.M.F.Ferreira and C.Pinho .(2008). Development of porous ceramics for gas burners. *Defect and diffusion forum vols. 273-276(2008) pp 814-819*

S. Mößbauer, O. Pickenäcker, K. Pickenäcker and D. Trimis. Application of the porous burner technology in energy and heat-engineering. *Fifth international conference on technologies and combustion for a clean environment (Clean Air V), Lisbon (Portugal), 12-15 July 1999 Volume I, Lecture 20.2, pp. 519-523*

W. Zhiyong , C. Cyril, B. Fengwu, F. Gilles, W. Zhifeng, Z. Jinsong, T. Chong. Experimental and numerical Studies of the pressure drop in ceramic foams for volumetric solar receiver applications. *A journal of Elsevier Publications.(2009).*

# EXISTING CHALLENGE TO SUCCEED A SUSTAINABLE BUILT AQUA-ENVIRONMENT IN MISTY GREEN VALLEYS IN THE HILL COUNTRY OF SRI LANKA; A CASE STUDY

Shirantha, R. R. A. R[1]., Amarathunga, A. A. D[2], Weerasekara, K. A. W. S.[3]

[1]Research Officer, Inland Aquatic Resources and Aquaculture Division, National Aquatic Resources Research and Development Agency (NARA), Crow Island, Mattakkuliya, Colombo 15, Sri Lanka
[1]E-mail: r_shirantha@hotmail.com
[1]Telephone: +94-11-2521005; Fax: + 94-11-2521005

[2]Research Officer, Environmental Studies Division, National Aquatic Resources Research and Development Agency (NARA), Crow Island, Mattakkuliya, Colombo 15, Sri Lanka
[2]E-mail: Deeptha@nara.ac.lk
[2]Telephone: +94-11-2521009; Fax: + 94-11-2521009

[3]Research Officer, Environmental Studies Division, National Aquatic Resources Research and Development Agency (NARA), Crow Island, Mattakkuliya, Colombo 15, Sri Lanka
[3]E-mail: shyamalikaww@gmail.com
[3]Telephone: +94-11-2521009; Fax: + 94-11-2521009

**Abstract:** The upper watershed of Dambagastalawa River in Sri Lanka is a precious ecosystem of esthetic beauty that supplies direct drinking, bathing and agricultural water for the people in Pattipola-Ambewela and Agara Pathana areas. Nevertheless, is often subjected to multiple threats due to a range of human related activities. Therefore, a monthly bio monitoring study was carried out in 2007 and 2008 to assess current ecological conditions at five selected sites. The aim was to evaluate future sustainability of the catchments. The invertebrate fauna were collected through i*n situ* direct counting, stone lifting, kick, scope and core sampling methods and evaluation of faunal composition was done preferring to the Biological Monitoring Working Party (*BMWP*) scoring system. A parallel study was carried out to assess the water quality parameters following the *APHA* standard methods. Any visible change in the chatchment land use patterns was also monitored.

The highest faunal diversity (<10) was recorded at highly oxygenated (<7.33 mg/l) up stream sites with little/or no disturbance in Pattipola forest area where Ephemeropterans and Sumiliids were dominant. The second most diverse site (≤ 7) was at the disturbed Agara Pathana site where Hydrosphychids, Chironomids, Plannarians and Molluscs were dominant. The lowest faunal diversity (≤ 3) was recorded at the Ambewela reservoir which is largely colonized by poor water quality indicative taxa *Hydra* and Chironomid (~208 individuals/$m^2$). The Average Score Per Taxon (ASPT) of the upstream sties (6.8 and 7) were less comparable with reference stream site (8.2 out of 10) studied at the Piduruthalagala peak. ASPT was 4.57 in the down stream. However, significantly lower ASPT values (3.5 at sluice gate and 2.5 at uppermost outlet) were recorded at Ambewela reservoir evidently showing an associated ecological risk. The reservoir was found to received high levels of nutrients (ammonia 0.20- 0.25 mg/l, nitrite 0.09 -0.121 mg/l, nitrate 0.50 -1.35 mg/l and phosphate 0.050 -.075 mg/l) due to effluent run-off from the adjoined livestock and crop farms. It was found to have a periodic eutrophic condition especially during low rain and growing crop seasons. No biological/chemical evidence to prove self-purification in its down stream in Agara Pathana where the river encounters other additional threats. Converting of forest/tea plantations into annual crops, garbage dumping, loading tea dust/other waste is severe in this area thereby water become more turbid (TSS 459.90 mg/l) unpleasant and undrinkable. Present case study showed the agriculture based human activities in middle catchments of the Dambagastalawa river posed significant negative impacts on river quality. In the long run this might affect the entire aqua-environment in the Ambewela/Agra valleys. Actions should immediately be taken to bring these valleys into manageable levels otherwise we may face irreversible loss of sustainability.

**Key words:** hill country, Dambagastalawa river, bio monitoring, water pollution, ecological risk.

## 1 Introduction

### *1.1 Sustainable built aqua-environment*

When the phrase "sustainable built environment" is applied to natural environments it would fundamentally include conservation, biodiversity, and enhancement of the site ecology and enhance or safeguard human health as well as wellbeing (www.co.uk). In aqua ecosystems it is significant in many of ecological functions such as recycling of nutrients, purifying water, attenuating floods, recharging ground water and providing habitats for wildlife. It is also significant to use for human recreation and eco-tourism industry. It is known fact the health of an aquatic ecosystem is degraded when the ecosystem's ability to absorb a stress has been exceeded. Since human populations frequently impose excessive stresses on aquatic ecosystems through multiple use, pollution and habitat degradation, most of aqua environments of the world are under intense pressure (Mason 1996[1]). Therefore, the services they can provide to the society through their sustainability have been reduced and the biota is strongly affected disappearing of several aquatic species from some of eco-region (Stanford 1994).

### *1.2 Animal life in aqua-environments*

Stress on an aquatic ecosystem can be a result of alterations of its physical *i.e. changes in water temperature, water flow and light availability*, chemical *i. e. changes in the loading rates of bio stimulatory nutrients, oxygen consuming materials, and toxins* and biological environments *i. e. introduction of exotic specie*s (www.towards-sustainability.co.uk/issues/built) which are particularly conducive for composition of living organisms in it (Mason 1996[1]). If any water-body is contaminating with any pollutant, slowly loading any harmful substance or degrading due to any human activity it firstly affect on its living things then to the entire ecosystem. In initial stages, sensitive species would disappear or show decline in abundance while tolerant species become more prolific and dominant. Therefore, animals reflect the build up of impacts of environmental changes on the ecosystem such as the influence of surrounding land use or the effects of pollution. Therefore, faunal composition of any aquatic ecosystem logically expresses its long term condition with more precision than its chemical or physical parameters. Hence, they are important as bio indicators which can give early warning in potential pollution trend or effect of long termed stress conditions (Chessman 1995 and Mason 1996[1]).

### *1.3 Biological monitoring of pollution in freshwaters*

The quantitative studies of organisms and biological communities are known as biological monitoring through which the environmental integrity of many of freshwater ecosystems of the world has been assessed (Aarts & Nienhuis 1999; Anon 1998; Anon 2000 and Diersing 2009). The aquatic macro invertebrates such as insects, snails and worms that are widespread, easy to collect, relatively immobile and provide good information about the environment are very largely used in bio-monitoring studies (Mason 1996[2] and www.epa.vic.gov.au). In such studies data interpretation is commonly done with diversity indices which take account of species richness and evenness *i. e. the number of species and relative abundance of species* and biotic indices that take account of the sensitivities of different species to pollution. The value of diversity index is high in unstressed *i. e. pollution- free* environments while it is low in stressed environments. In biotic indices, a high score is given for the species which are sensitive to pollution while a low score is given for the tolerant species. It is more precise to compare diversity indices as it takes account of the tolerances of individual species to pollution. Hence, biotic indices are largely used and different scoring systems such as *BMWP* (*Biological Monitoring Party Score*), *MRHI* (*Monitoring River Health Initiative*) and *AUSRIVAS* (*Australian Rivers Assessment System*) have been devised and are employed to weigh out healthiness in many of freshwater systems of the world (Chessman 1995; Mason 1996[1] and Matcafe 1996).

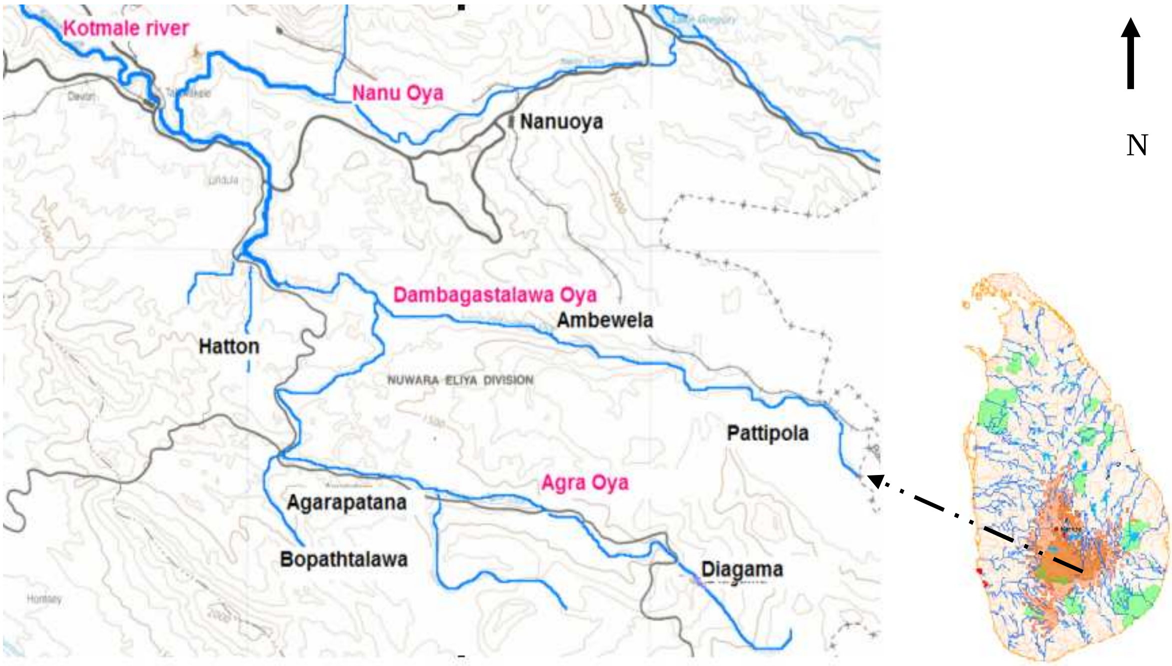
### *1.4 Rational for the study*

Freshwaters in the central hills of Sri Lanka are major suppliers of direct drinking, bathing, agricultural, irrigational and hydro-power generating water as, well as they are being good followers of the esthetic beauty of the Island (Arumugum 1969). Despite this, they are becoming constant victims of a number of human activities such as deforestation and excessive use of agrochemicals and fertilizer. Sometimes they have to act as waste sinkers (Silva 1996). Those fats are leading to oppose them from the standards of sustainable build environmental due to frequent changes in their physical and chemical environments in which long run it may result irreversible degradation.

The Dambagastalawa river catchment is the uppermost watershed of the Kotmale River which is one of the most productive and significant sub catchments with highly bio-diverse unique ecosystems of high floral and faunal endemism and precious ecosystems of esthetic beauty (Anon 2001). Since late 1970s its upper catchments area is under large scale crop farming, intensive animal farming, advanced tea plantation, and rapid urbanization (Arumugum 1969, Pethiyagoda 1991). Consequently, the catchment is more likely to have ecosystem degradation due to pollution, rapid seepage, heavy run off, high siltation etc. As such assessment of its healthiness is a timely requirement though it has not been exclusively carried out yet. Therefore, the present study was carried out from April 2007 to October 2008 through an exclusively evaluation of macro faunal composition and some water quality parameters to quantify presence of macro invertebrates in selected sites, identify habitat quality deterioration factors and to assess long term sustainability of the entire catchment.

## 2. Material and methods

### *2.1 Micro-catchments studied*



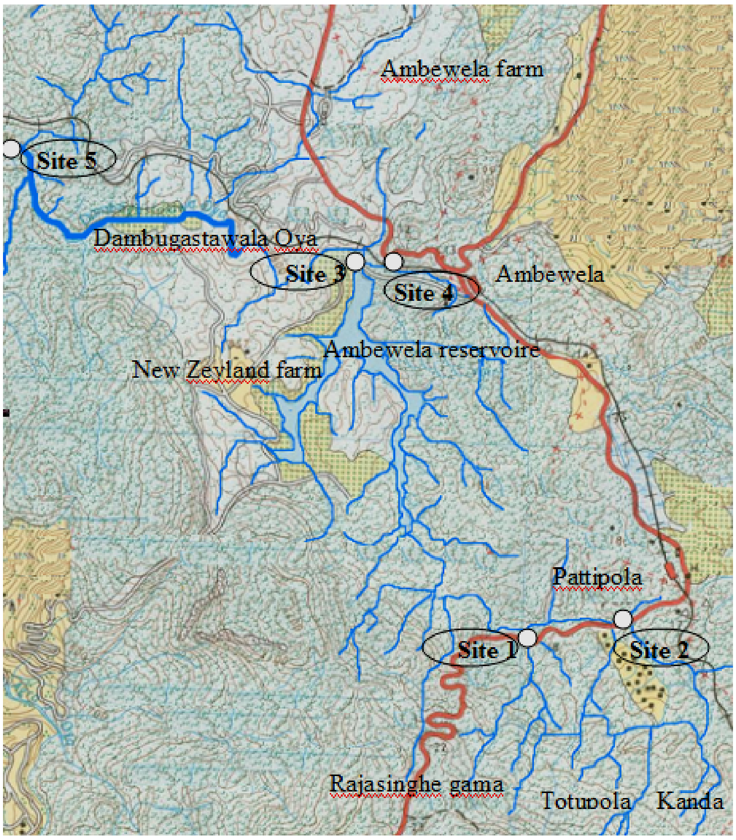

Scale 1: 1000 000

**Figure 2.1** Map of Sri Lanka showing the location of the uppermost catchments of the Mahaweli River and upper reaches of the Kotmale River; *Agra Oya, Nanu Oya and Dambagastalawa Oya*.

The Dambagastawala micro-catchment is located in the central highlands (above 2130 m MSL) of Sri Lanka (5°55’- 9°51’N and 79°41’- 81°54’E). It is one of the economically and ecologically significant uppermost catchments of the Kotmale River that tributes to the River Mahaweli (figure 2.1) (Anon 1988). The Dambagastalawa river harbours several perennial streams that originate in the Ambewela hills, Totupola Kanda and Pattipola area (figure 2.2). These uppermost streams are empty into a reservoir built by impounding the river with a across ridge at Ambewela. This Ambewela reservoir is of 14.25 $km^2$ catchment area and 113.28$m^3$ water storage capacity spreads over 60.7 hec. It supplies water to neighboring valleys for an extent of about 404.7 hec. through a link channel leading to a 137.16m tunnel underneath the railway line adjoining the Ambewela railway station (Arumugam 1969). Afterwards the Dambagastalawa river flows through Agra valley, encounters additional streams and subsequently meets Agra river and Nanu Oya to form the Kotmale River (figure 2.1).

### *2.1 Study site selected*

A total of six sampling sites representing the river catchment waterways *i. e. including relatively undisturbed sites and sites that are subjecting to the obvious point source pollution* were selected for the study (figure 2.2). One riverine site in protected forest area in the Pidurutalagala mountain range of the Nanu Oya catchment was selected as reference site as it was found with little or no disturbance.




Scale 1: 50 000

**Figure 2.2** Map of the upper reaches of the Dambagastalawa Oya showing the study sites.

The site 1 is a 3rd order perennial upstream flows through natural forest reserve at off Pattipola and its upper reaches in Rajasinghagama area. The site 2 is a 3rd order perennial downstream originates in the Totupola Kanda mountain region of the Horton Plains nature reserve and flows through forest plantation close to the Pattipola railway station. The site 3 and 4 are the reservoir sites respectively at the dam site (close to sluice gate) and at the uppermost outlet of the reservoir. These two sites are encountering obvious point sources of pollution due to agricultural run off, seepage of animal farming and siltation. The site 5 is a 4th order perennial more downstream flows through agricultural area especially tea and annual crops at Henfold estate at Agarapathana valley. At this point the Dambagastalawa Oya receives water from other animal farm areas as well as from other lands where annual crop farming was intensified in recent past.

### *2.2 Bio-monitoring procedure*

Each study site was monthly visited and mobile invertebrate fauna found on the water surface within a predetermined area, were studied through i*n situ* direct counting. The animals found in water column as well as benthos were collected into a kick sampler subsequently making them to be dislodged and swept into a net by the current. Five randomly selected stream bed boulders of approximately 3 cm × 4 cm in diameter were lifted into a white enamel tray and the anchored animals were then collected with a small paint bush. Aquatic fauna in water column as well as among the aquatic vegetation along the stream bankers were collected sweeping a fine meshed sweep net. Several number of mud samples were randomly collected using a corer and the animals were collected by wet sieving. Later all animals were preserved in 10% formalin for further analysis in the laboratory.

In the laboratory the animals were identified up to possible taxa using Fernando and Weerawardhana (2002); Manuel and Barton (2004); Manuel et al. (2004) and Needham and Needham (1962). Their abundance and generic richness were then calculated as follows.

Abundance = number of individuals/area of surveyed ($m^2$)

Species richness = number of species

### 2.3 Chemical monitoring procedure

Some water quality measurements; dissolved Oxygen (*Orion 830A* DO meter), pH (*Orion 260A* pH meter), temperature (thermometer incorporated in DO meter), electrical conductivity (*HANNA HI 8733t* conductivity meter) and turbidity (*HACH 2100P* turbidity meter) were made *in situ.* In the laboratory Total Suspended Solid (TSS), nitrite, nitrate, phosphorus and ammonia level were analyzed following the standards methods given in APHA (APHA, 1998). Bio-Chemical Oxygen Demand (BOD) and Chlorophyll-a content was measured using Aqua Lytic BOD sensors and a spectrophotometer (DR 4000) respectively.

### 2.4 Monitoring land use pattern changes

Any significant changes in river banks, within the water bodies as well as the land use pattern changes in the catchments were monitored.

### 2.5 Assessment of site condition

Finally assessment of invertebrate composition at each site was performed referring to the score system given to each taxon in *Biological Monitoring Working Party (BMWP) Score Table* (Mason, 1996[1]) (table 2.1). Then the total score was divided by the number of families recorded in the site to derive Average Score Per Taxon (ASPT) value which is less sensitive to sample size and sampling effort. According the BMWP scoring system high ASPT scores indicate good water quality while low values indicate poor water quality.

*Table 2.1 Biological Monitoring Working Party (BMWP) score system (Source: Mason 1996[1])*

| Invertebrate Families | Score |
|---|---|
| Siphlonuridae, Heptagenidae, Leptophlebidae, Ephemerellidae, Potamanthidae, Ephemeridae, Taeniopterygidae, Leuctridae, Capniidae, Perlodidae, Perlidae, Chloroperllidae, Aphelocheiridae, Phryganeidae, Molannidae, Beraeidae, Odontoceridae, Leptoceridae, Goeridae, Lepidostomatidae, Brachycentridae, Sericostomatidae | 10 |
| Astacidae, Lestidae, Agriidae, Gomphidae, Cordulegasteridae, Aeshnidae, Corduliidae, Libellulidae, Psychomyiidae, Philopotamidae | 08 |
| Caenidae, Nemouridae, Rhyacophildae, Polycentropidae, Limnephilidae, | 07 |
| Neritidae, Viviparidae, Ancylidae, Hydroptilidae, Unionidae, Corophiidae, Ganumaridae, Platycnemididae, Coenagriidae | 06 |
| Mesoveliidae, Hydrometridae, Gerridae, Nepidae, Naucoridae, Notonectidae, Pleidae, Corixidae, Haliplidae, Helodidae, Dytiscidae, Gyrinidae, Hydrophilidae, Clambidae, Helodidae, Dryopidae, Elminthidae, Chrysomelidae, Curculionidae, Hydropsychidae, Tipulidae, Simuliidae | 05 |
| Baetidae, Sialidae, Piscicolidae | 04 |
| Valvatidae, Hydrobiidae, Lymnaeidae, Physidae, Planorbudae, Sphaeriidae, Glossiphoniidae, Hirudidae, Erpobdellidae, Asellidae | 03 |
| Chironomidae | 02 |
| Oligochaeta (whole class) | 01 |

In addition to this the trophic status of the reservoir sites was evaluated referring to the Mason (1996[1]) eutrophication survey guideline values given for water quality parameters for lakes and reservoirs (table 2.2). The relationship between the changes in water quality and populations of some organisms also assessed preferring to figure 2.3 in order to get a precise idea on the existing quality status of the water bodies assessed.

*Table 2.2 Eutrophication survey guidelines for lakes and reservoirs (source: Mason 1996[1])*

| | Oligotrophic | Mesotrophic | Eutrophic |
|---|---|---|---|
| Total phosphorous (mg/l) | < 0.01 | 0.01-0.02 | > 0.20 |
| Total nitrogen (mg/l) | < 0.20 | 0.20 -0. 50 | > 0.50 |
| Secchi depth (m) | > 3.7 | 3.7 – 2 | < 2 |
| Hypolimnetic dissolved oxygen (% saturation) | > 80 | 10 – 80 | < 10 |
| Chlorophyll-a (mg/l) | < 4 | 4 – 10 | > 10 |
| Phytoplankton production (g $Cm^{-2}$ $d^{-1}$) | 7 -25 | 75 - 250 | 350 - 700 |

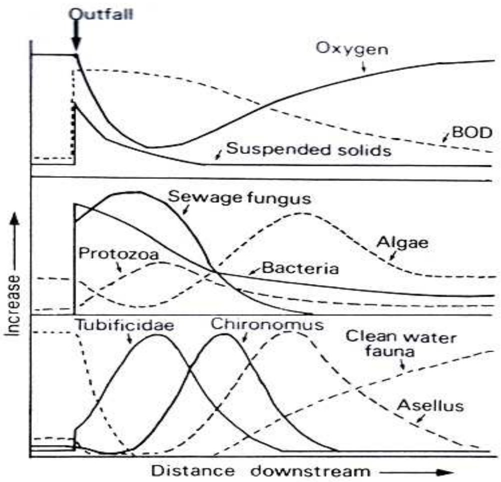



**Figure 2.3** Changes in water quality and populations of organisms in a river below a discharge of an organic effluent (Adopted from Mason 1996[1]).

## 3. Results

### *3.1 Results of bio monitoring*

The invertebrate taxa recorded at the study sites in the Dambagastalawa Oya micro-catchment during the study period are given in table 3.1. Majority of them were aquatic insects of which one genus of Trichopterans (caddis flies) of the family Hydropsychidae was found to more prolific in the down stream site studied at Agra valley. They were less dominant in the upstream sites studied in Pattipola forest area. The Ephemeropterans (May flies) were found in all upstream sites studied and none of them were recorded at the down stream site (site 5) and the reservoir sites. The Odonates (Damsel and dragonflies) were predominantly recorded at site 3 and site 5. There was a single record of Plecopterans (stone fly) in the reference stream site studied at the Piduruthalagala mountain peak.

The Chironomid larvae were recorded from all the riverine sites studied with relatively low abundance (<0.5 individulas/$m^2$). Its density was very high in the reservoir sites (average value was 208 individuals/$m^2$ in 2008) where occasional records of huge colonies of *Hydra* were found. The population density of *Hydra* was found to show negative relationship (r = 0.88) with monthly rainfall experienced in 2008 (figure 3.1). In addition to these the larval forms of several other invertebrate taxa such as hemipterans (water bugs/boatmen/striders), coleopterans (beetles) and Platyhelminthes (flat worms) as well as crabs and molluscs were occasionally recorded.

The highest invertebrate generic richness (13) was recorded at the stream sites studied in Pattipola forest area. The second most diverse site (5) was the down stream site studied at Agara valley. The lowest invertebrate diversity (3) was recorded at the Ambewela reservoir sites. The Average Score Per Taxon (ASPT) of the upstream sties were 6.8 and 7. It was 8.2 (out of 10) at the reference stream site studied at the Piduruthalagala peak. The ASPT was 4.57 in the down stream site (site 5). However, significantly lower ASPT values 3.5 and 2.5 respectively recorded at the sluice gate and at the uppermost outlet of the Ambewela reservoir indicating an associated ecological risk in it.

*Table 3.1 different invertebrate taxa recorded at the study sites at Dambagastalawa micro-catchment in 2007 and 2008 and calculated values for site generic richness and site ASPT (abundance: + less, ++ moderate, +++ high and – not recorded).*

| | Study site | | | | | |
|---|---|---|---|---|---|---|
| **TAXA** | **Site 1 (upstream)** | **Site 2 (upstream)** | **Site 3 (reservoir)** | **Site 4 (reservoir)** | **Site 5 (downstream)** | **Reference site** |
| **PLECOPTERANS** | | | | | | |
| **Perlidae** | | | | | | |
| *Neoperla* sp. | - | + | - | - | - | + |
| **EPHEMEROPTERANS** | | | | | | |
| **Hepatagenidae** | | | | | | |
| *Heptagenia* sp. | - | +++ | - | - | - | ++ |
| **Baetidae** | | | | | | |
| *Baetis* sp. | ++ | +++ | - | - | - | + |
| *Choroterpes* sp. | + | +++ | - | - | - | + |
| **TRICHOPTERANS** | | | | | | |
| **Hydrosphychidae** | | | | | | |
| *Cheumatopsyche* sp. | - | +++ | - | - | +++ | - |
| *Helicopsyche* sp. | + | +++ | - | - | - | + |
| **Molannidae** | | | | | | |
| *Molanna* sp. | - | - | - | - | - | + |
| **ODONATES** | | | | | | |
| **Gomphidae** | | | | | | |
| *Anisogomphus* sp. | + | - | - | - | - | + |
| **Libellulidae** | | | | | | |
| *Trithemis* sp. | + | ++ | - | - | ++ | + |
| **Lestidae** | | | | | | |
| *Lestes* sp. | + | - | ++ | - | + | + |
| **HEMIPTERANS** | | | | | | |
| **Naucoridae** | | | | | | |
| *Naucoris* sp. | + | - | - | - | - | + |
| **Notonectidae** | | | | | | |
| *Anisops* sp. | - | - | - | - | - | + |
| **Gerridae** | | | | | | |
| *Gerris* sp. | - | - | - | - | - | + |
| *Rhagodotarsus* sp. | - | - | - | - | - | + |
| **Gyrinidae** | | | | | | |
| *Aulonogyrus* sp. | + | + | - | - | - | + |
| **Dytiscidae** | | | | | | |
| *Cybister* sp. | + | - | - | - | - | - |
| **LEPIDOPTERANS** | + | + | - | - | - | - |
| **DIPTERANS** | | | | | | |
| **Simuliids** | + | + | - | - | + | + |
| **Chironomids** | + | + | +++ | +++ | + | - |
| **CNIDARIANS** | | | | | | |
| *Hydra* sp. | - | - | +++ | +++ | - | - |
| **CRUSTACEANS** | | | | | | - |
| Crab | + | + | - | - | - | - |
| **PLATYHELMINTHES** | | | | | | |
| Planarian | - | + | - | - | +++ | - |
| **MOLLUSKS** | - | - | - | - | ++ | - |
| **Generic richness** | **13** | **13** | **03** | **03** | **07** | **15** |
| **ASPT value** | **6.8** | **7** | **3.5** | **2.5** | **4.57** | **8.2** |

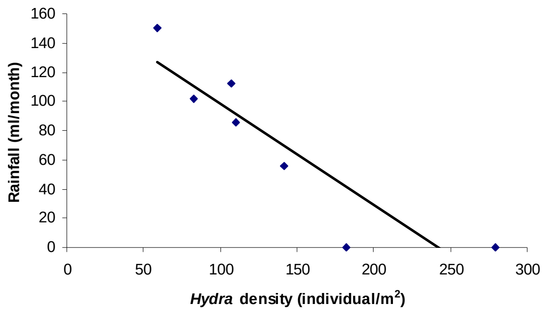



**Figure 3.1** relationship between *Hydra* population density and monthly rainfall in the Ambewela reservoir in 2008 (r = 0.888).

### *3.2 Results of chemical monitoring*

*Table 3.2 some physio-chemical parameters of water at the study sites average for the enter study period (AL -Average for lake).*

| Water quality parameter | Site 1 | Site 2 | Site 3 | Site 4 | AL | Site 5 |
|---|---|---|---|---|---|---|
| Water Temperature (ºC) | 16.30 | 17.20 | 19.97 | 18.40 | 19.19 | 19.20 |
| pH | 7.06 | 6.83 | 7.36 | 6.56 | 6.96 | 6.30 |
| DO (mg/l) | 7.63 | 7.33 | 5.05 | 4.30 | 4.68 | 7.40 |
| BOD (mg/l) | 4.00 | 4.40 | 7.60 | 9.00 | 8.30 | 7.70 |
| EC (mS/l) | 32.90 | 28.80 | 26.66 | 28.20 | 27.43 | 35.57 |
| Ammonia (mg/l) | 0.10 | 0.07 | 0.20 | 0.25 | 0.23 | 0.20 |
| Nitrite (mg/l) | 0.09 | 0.10 | 0.11 | 0.12 | 0.12 | 0.01 |
| Nitrate (mg/l) | 0.96 | 1.00 | 0.50 | 1.35 | 0.92 | 1.528 |
| Phosphate (mg/l) | 0.04 | 0.03 | 0.05 | 0.75 | 0.40 | 1.067 |
| Turbidity (NTU) | 0.96 | 1.53 | 5.38 | 6.77 | 6.08 | 6.23 |
| TSS (mg/l) | 2.38 | 3.67 | 4.96 | 3.25 | 4.11 | 459.9 |
| TDS (mg/l) | 15.80 | 12.68 | 11.52 | 13.08 | 12.3 | 16.49 |
| Chlorophyll- a content (mg/l) | 8.10 | 8.45 | 21.28 | 10.99 | 16.135 | - |

The average values for some water quality parameters measured during the study period are given in table 3.2. The highest values for nitrite (0.12 mg/l), Chlorophyll-*a* content (21.28 µg/l), turbidity (6.77 NTU), ammonia (0.25 mg/l) and BOD (9.00 mg/l) were recorded at the reservoir sites where the lowest DO value (4.3 mg/l) was recorded. More down stream site studied at Agra valley recorded the highest values for EC (35.57 mS/l), nitrate (1.528 mg/l), phosphate (1.067 mg/l) and TSS (459.9 mg/l).

### *3.3 Identified causes for low down of ecological integrity*

The identified causes for deterioration of ecological integrity of the study sites are given in table 3.3. According to table 3.3 it is clear that Ambewela reservoir area and the down stream studied are encountering more impacts that are leading to a rapid water quality depletion.

*Table 3.3 identified causes for low down of ecological integrity of the Dambagastalawa micro-catchments during the study period (possible effects are given parentheses).*

| | Study site | | | | |
|---|---|---|---|---|---|
| **Identified causes** | **Site 1** | **Site 2** | **Site 3** | **Site 4** | **Site 5** |
| Converting tea/grass land or forest area into annual crops (*soil erosion, high siltation and habitat loss*) | - | - | +++ | +++ | +++ |
| Excessive use of agrochemicals and fertilizer (*loading of organic pollutants, heavy metals and other toxic compound*) | - | - | ++ | ++ | ++ |

| | | | | | |
|---|---|---|---|---|---|
| Adding of effluent from dairy farms and unwanted vegetable matters (*loading of organic substances in excess, harmful microbs and pathogen*) | - | - | +++ | +++ | ++ |
| Dumping waste due to eco-tourism (*loading bio and non bio degradable substance*) | + | + | + | - | + |
| Dumping waste/tea dust (*loading bio degradable substance*) | - | - | - | - | ++ |
| Sand mining (*habitat damage*) | - | - | - | - | + |
| Mixing of house hold sewage/drainage channel | - | - | - | - | + |
| Cleaning river bankers for crop farming (*eroding of bankers and soil erosion* ) | - | -- | + | + | + |

(Severity: + low, ++ moderate and +++ high)

## 4. Discussion

The present study showed that few genera of Ephemeropterans, Plecopterans and Trichopterans (EPT) are predominately colonized in the upstream sties studied in the Dambagastalawa Oya micro-catchment. They are well known good water quality indicatives in tropical streams (Chessman, 1995). Due to presence of EPT insects, high generic richness and high ASPT values it is logical to conclude that the streams assessed are of good quality water as well as of good ecological integrity. This is in agreement with the results of parallel chemical monitoring studies since most of water quality parameters measured are within approval ranges for good quality water. However, the comparatively elevated level of nitrate at the site 2 (1.0 mg/l) might be due to decomposition of large amount of leaf debris as the stream flows through natural forest reserve and planted forest areas. At present these upstream sites are still not having any obvious impact to deplete sustainability except for non bio-degradable waste such as plastic containers and bottles that come through eco-tourism industry.

Nevertheless, the present bio- monitoring results showed a severe depletion in quality of water in Dambagastalawa Oya at its middle reaches where the river has given rise to the Ambewela reservoir. The reservoir sites recorded periodically intensified colonies of *Hydra* and a steady population of *Chironomid* larvae which taxa are among the well known indicatives of poor water quality (MaCafferty, 1981). This condition seems to be attributed by organic pollutants loading into the reservoir. According to the cause and effect studies (table 3.3), nearby dairies are critical problem to the reservoir as they bring plenty of organic waste due to intensified livestock rearing and the over wintering of animals in confined building, as well as the increased use of silage to feed them. The drainage canal systems of particular dairies directly connect to the reservoir and they transport plenty of unpleasant odor organic matters into the reservoir routinely. Of the waste effluents the silage seems to be a main contributor for its pollution condition as it can be 200 times as strong as settled sewages (Mason 1996[1]). In generally (see figure 2.3), when organic discharges load into a water body initially its oxygen level decreases and BOD level increases due to microbes activities to decompose the organic matter releasing large quantities of nitrates and phosphates that are stimulating massive algal growth (Mason 1996[1]). Therefore, it leads to increase BOD level, decrease DO and some deviation in other water quality parameter such as pH and conductivity from the accepted standards. Hence, decrease level of DO (4.30 mg/l), increase level of BOD (9.00 mg/l), nitrate (1.35 mg/l), phosphate (0.75 mg/l) and chlorophyll-*a* content (21.28 mg/l) in the reservoir sites are due to organic pollution. Above values exceed the meso-trophication indicative levels (Table 2.2).

In addition to animal farm wastes the Dambagastalawa catchments frequently encountered residues of different agrochemicals such as weedicides, fungicides, insecticides etc. that apply to potato, vegetables and flowering plants in the riparian areas. These probably increase level of toxic compound in water. In addition to agrochemicals, farmers constantly use crop manure in excess. This excess amount or sometimes all (if it is raining at the applying time) washed away into a nearby water bodies contributing to increased nutrient levels. The nitrogen and phosphorous are the two nutrients most implication in eutrophication (Mason 1996[1]). According to Mason (1996[2]) the most important sources of nutrients include phosphorous containing detergent, agricultural run-off and leaching of artificial fertilizers, the washing of manure from intensive farming units into water, the felling of forests which causes increasing erosion and run-off. The present study showed that the Ambewela reservoir is experiencing all these sources and have integrated to develop periodic eutrophic condition in the reservoir. Hence, the reason for huge colonies *Hydra* and *Chironomid* is obvious as the reservoir water does not get diluted in relative low rainy season and leads nutrient level to go up. The condition is favoured by *Hydra* and *Chironomid* build huge colonies that can live in extremely polluted waters (Dash 2001).

The pollution condition in downstream site studied is of significant as there was no any record of EPT insects except *Hydropsia* sp. which taxon has given low score in BMWP system. It means it is found in low quality waters. In addition to *Hydropsia* sp. some other poor quality indicative invertebrates such as Simuliid dipterans and Plannarians were also recorded. Most of them live in cases or possess external respiratory organs that are adaptive radiation to cope with rapid chemical environmental changes or rapid run off (Chessman 1995). Therefore, presence of large colonies of such invertebrates, relative low generic richness and relatively low ASPT value at the downstream site indicate lowering of water quality in the Dambagastalawa Oya at this reach too. The stream bed was of thick sediment layer of grass particles. This condition might be attributed to silage effluent encounters from Ambewela area. Although water gets diluted at this run, there is no sufficient purification process to bring river water into a good quality as the river encounter additional threats at this stretch. Furthermore, the landscape changes due to encroachment, illegal cultivation of tea and annual crops and converting of tea plantation into annual crop are attributing to elevated levels of turbidity, TSS and TDS due to high siltation. Thereby water becomes more turbid, unpleasant and undrinkable. Now it has already depleted the drinking and bathing water quality. Since the Dambagastalawa river catchment is a victim of several impacts returning to pristine conditions is a big challenge.

According to Dash (2001) the good quality water is highly significant to build sustainable aqua environments. If the catchments are facing chronic stress condition due to depletion of quality of water probably on organic pollution, it would cause hazards to human health, harm to living resources and ecological systems, damage to structure or amenity, or interference with legitimate uses of the environment. Since the studied catchment water is of significant use, it is extremely necessary bring the quality of water into a manageable level that depends on its local geology and ecosystem, as well as human uses. Therefore, action should be taken to protect geo-environment as well as to minimize the up loading of organic pollution from the reservoir area. Otherwise it will be a swerving menace to sustain good aqua environments in the Ambewela valley as well as Agra/Bopathtalawa valleys.

### 5. Conclusion

The uppermost reaches of the Kotmale river; the Dambagastalawa Oya micro-catchment is experiencing a severe water quality depletion trend clearly signifying eutrophic condition mainly on agricultural based activities. Therefore, actions should immediately be taken to bring the catchments into manageable levels. Otherwise we may face irreversible loss of sustainability of them with many adverse effects on water uses of downstream areas.

## Acknowledgements

We are very thankful to all staff members of Environment Studies Division and Mr. Janaka P. Kumara of Inland Aquatic Resources and Aquaculture Division of NARA, Sri Lanka for their assistance given through out this research project.

### References

Aart, B. G. W., Nienhuis, P. H. (1999), "Ecological sustainability and biodiversity", *International Journal of Sustainable Development World Ecology*, 6: 1-14.

Anon (1988), "Mahaweli Project Programme 1988: *A survey of the progress of work on the Accelerated Programme of Mahaweli Development in 1987 and the development proposed in 1988"*, Ministry of Lands and Land Development and the Ministry of Mahaweli Development, Colombo, Sri Lanka.

Anon (2001), "A Guide to National Parks of Sri Lanka", 1st Edition, Department of Wildlife Conservation, Sri Lanka.

Anon, (1998), "The health of streams in the Western port catchment", EPA Publication, Melbourne, Australia.

Anon, (2000), "The health of streams in the Campsite, London and Avoca catchments", EPA Publication, Melbourne, Australia.

APHA, (1998), "Standards Methods for the Examination of Water and Wastewater", 20th Edition, American Public Health Association, Washington, D.C, USA.
Arumugum, S. (1969), "Water resources of Ceylon", Water Resources Board, Colombo, Sri Lanka.
Chessman, B. C. (1995), "Rapid assessment of river using macro invertebrates: A procedure based on habitat-specific sampling, family level identification and a biotic index". *Australian Journal of Ecology* 20: 122-129.
Dash, M.C. (2001), Fundamentals of Ecology, 2nd Edition, Tata McGraw-Hill Publishing Company Limited, New Delhi, Indiea.
Diersing, N. (2009), "Water Quality: Frequently Asked Questions", *PDA*, NOAA. Published on the Internet; http://floridakeys.noaa.gov/pdfs/wqfaq.pdf. (accessed 24 July 2010).
Fernando, C. H., Weerwardhana, S. R. (2002), "Sri Lanka freshwater fauna and fisheries" Volumes Publishing, Kitchener, Ontario, Canada.
http//www.epa.vic.gov.au/sustainable Built Environment (accessed on 04 August 2010).
http://www.towards-sustainability.co.uk/issues/built/towards-sustainability (accessed on 8 August 2010).
MaCafferty, (1981), "Aquatic Entomology", Jones and Bartlett Publishers.
Manuel, L. P., Andrew, K. R., Steven, C. H. (2004), "Identification Manual for the Caddis fly (Trichoptera) Larvae of Florida", Revised Edition, Department of Environment Protection, Division of Water Resources Management, Tallahassee, Florida.
Manuel, L. P., Barton, A. R. (2004), "Guide to the Mayfly (Ephemeroptera) Nymphs of Florida", Department of Environment Protection, Division of Water Resources Management, Tallahassee, Florida.
Mason, C. F. (1996[1]), "Water Pollution Biology". In: *Pollution Causes, Effects and Control* 3rd Edition (Harrison, R. M. ed.) 66 – 91, The Royal Society of chemistry, University of Birmingham.
Mason, C. F. (1996[2]), "Biology of Freshwater Pollution". 3rd Edition, Longman Scientific and Technical, Harlow.
Matcafe, J. L. (1996), "Environmental Pollution", 60; 101
Needham, J. G., Needham, P. R. (1962), "A guide to the study of freshwater biology", 5th Edition, Holden-Day, Inc., San Francisco.
Pethiyagoda, R. (1991), "Freshwater fishes of Sri Lanka", The Wildlife Heritage Thrust of Sri Lanka, Colombo, Sri Lanka.
Silva, E. I. L. (1996), "Water quality of Sri Lanka, *A review on twelve water bodies*". Institute of Fundamental Studies, Kandy, Sri Lanka.
Stanford L. (1994), "Biological Monitoring of Aquatic Systems", CRC Press.

## About the Authors

**R. R. A. R. SHIRANTHA,** B.Sc. (Sp). Kelaniya, M. Phil. Kelaniya is a Research Officer at the Inland Aquatic Resources and Aquaculture Division, NARA, Sri Lanka. Her research interests are in the areas of freshwater conservation biology, biodiversity and use of aquatic macro-invertebrate as bio-indicators.

**A. A. D. AMARATHUNGA,** B.Sc. (Agri.) (Sp). Peradeniya, M. Sc. in Environment & Forestry Peradeniya is a Research Officer at the Environmental Studies Division, NARA, Sri Lanka. His research interests are in the areas of wetland ecology & management, water pollution & bio indicators, biodiversity & water quality in River basins & lagoons and environmental impact assessment.

**K. A. W. S. WEERASEKARA,** B.Sc. Ruhuna, M. Sc. in Natural Resource Management Peradeniya is a Research Officer at the Environmental Studies Division, NARA, Sri Lanka. Her research interests are in the areas of natural resource management, water quality management, development of bioassays and environmental impact assessment.

# RAINFALL FORECASTING FOR FLOOD PREDICTION IN THE NILWALA BASIN

Uditha Ratnayake, D.A. Sachindra, K.D.W. Nandalal

Faculty of Engineering, University of Peradeniya, Sri Lanka
udithar@pdn.ac.lk

**ABSTRACT**

Flooding is the major natural disaster in Sri Lanka and reliable forecasts with longer lead time is a way of reducing the damages. In this study a weather model was coupled with a hydrologic model and a hydraulic model for predicting floods in Nilwala river basin in southern Sri Lanka.

WRF 3.0 (Weather Research and Forecasting) weather model was configured and used to predict rainfall over the basin 24 h into future. The model was configured by investigating the impacts of its physics options on precipitation forecasting. The impacts of microphysics schemes, cumulus schemes, land surface schemes, long/shortwave schemes and boundary layer schemes on rainfall predictions were investigated. The predictions were compared with observed point rainfall data for three rainfall events to find reasonably good physics combination. It was seen that model physics combination; Ferrier microphysics scheme, Kain-Fritsch cumulus scheme, Rapid Update Curve land surface scheme, Rapid Radiative Transfer Model longwave radiation scheme, Dudhia shortwave scheme and Yonsei boundary layer scheme yields better precipitation predictions over the basin.

Output of the weather model was coupled with hydrologic model HEC-HMS 3.3 (Hydrologic Engineering Center-Hydrologic Modeling System) with Clark's, Snyder's and SCS transformation methods. In all model runs Green-Ampt loss model was executed with recession base flow method. Before using the model with the WRF output HEC-HMS model was calibrated for historical events and Snyder's method performed better than other methods in calibration and verification. Snyder's method produced Nash-Sutcliff efficiencies greater than 70% and 50% in calibration and verification respectively.

WRF predicted rainfall for May-2003 was introduced to HEC-HMS and the generated river discharges of sub basin were ingested to the HEC-RAS 4.0 (Hydrologic Engineering Center-River Analysis System) hydraulic model for water profile computations along the Nilwala main river. Output of HEC-RAS was exported to Arc-GIS 9.2 where it was two dimensionally visualized as a flood map. Model was capable of predicting the areas as inundated regions but with underestimation of inundation depth.

## 1. INTRODUCTION

Flooding has been one of the most costly disasters in terms of both property damage and human casualties in Sri Lanka. Records show that major floods have occurred in Sri Lanka in the years of: 1913, 1940, 1947, 1957, 1967, 1968, 1978, 1989, 1992, and 2003 with severe loss of human lives, public and private property and the environment. Sri Lanka has 103 major river basins. Of these, 17 rivers are associated with flood problems. These 17 rivers have a catchments area of about 1,600 $km^2$. Kalu, Kelani, Gin, Nilwala and Mahaweli are the major rivers causing floods in Sri Lanka (Jayasekera, 2009).

Historically floods have been the most prevalent cause of death from natural disasters (Jonkman, 2005). Most of the human losses are due to floods in the tropical regions of Africa, Asia, and Central America. A reliable flood forecasting can reduce the death toll associated with floods. (Guleid et al, 2007). Operational flood forecasting has traditionally been driven by a dense network of rain gauges or ground-based rainfall measuring radars that report in real time (Guleid et al, 2007).

The basic intention of the study was to develop a flood prediction tool for the Nilwala river basin. A model having following three basic components is proposed to introduce long lead flood forecasts.

1) Atmospheric model (To predict precipitation over the basin)
2) Hydrologic model (To predict the river flow at various locations)
3) Hydraulic model (To predict the river water profile, inundation area and depth)

### *1.1 Nilwala River basin*

Nilwala River originates at Panilkanda near Deniyaya at an altitude of 1,050 m and after traversing about 72 km the river flows to the Indian Ocean at Matara. Before falling into the sea it passes the Deniyaya town, Morawaka and Akuressa regions. Nearly 90 per cent of the area covered by the catchment of Nilwala River belongs to the Matara District. The area of the river basin is about 1,073 km$^2$. Figure 2 shows the location of the Nilwala river basin and the drainage network.


Figure1: Nilwala basin location and river network

The watershed is located in the wet zone of Sri Lanka and the upper part of the catchment is covered with rainforest. The mean annual rainfall of the upper basin is above 3000 mm while the lower areas receives about 1900 mm. The average monthly rainfall exceeds 200 mm during the March–June and August–December periods, but in other months it is about 150 mm (Elkaduwa et al, 1998).

## 2. METHODOLOGY

An atmospheric, hydrologic and inundation models coupled as shown in the Figure 2 forms the basis of the model. Following procedure is used in configuration and calibration of the model.

### *2.1 Weather modeling with WRF*

The Weather Research and Forecasting (WRF 3.0) model used to model the weather over the basin. 45 km/15 km/ 5 km domain configuration was used in the present study. Spatial extents of domains were maintained as 1800 x 1800 km/645 x 645 km/245 x 245 km, respectively for the 1st 2nd and 3rd domains. All the domains shared the same center. Figure 3 depicts the arrangement of the three domains nested for the model runs. Initial and lateral boundary conditions for the model runs were obtained from the GFS (Global Forecast System) for three rainfall events on 10/12/2008, 20/03/2009 and 06/04/2009.

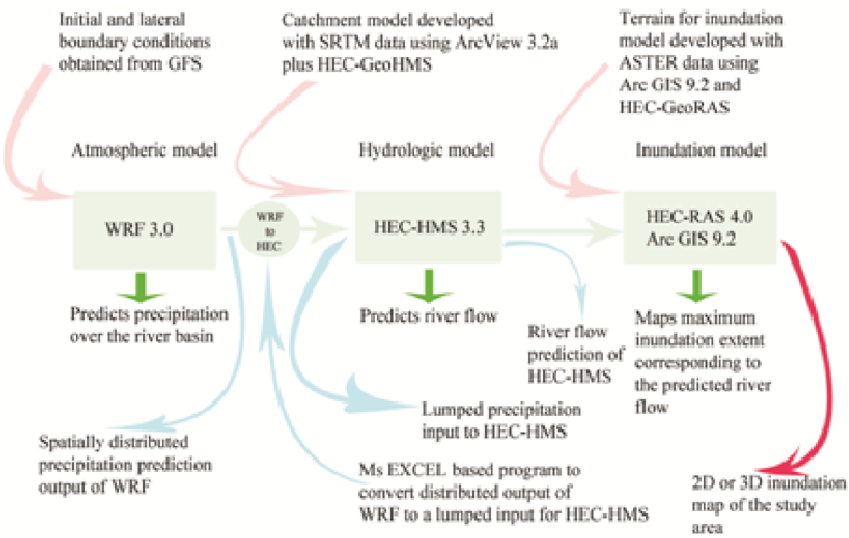



Figure 2. Flow diagram depicting the chain of model components

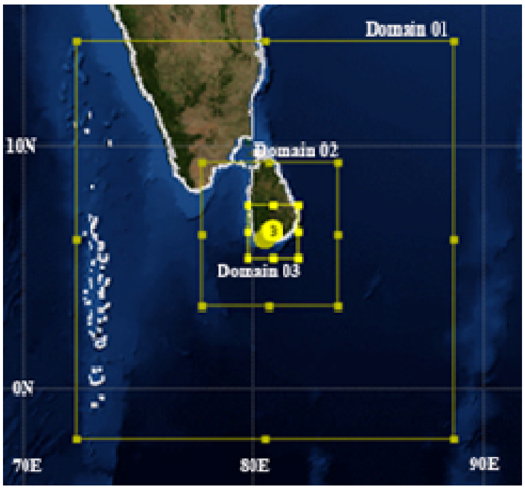



Figure 3 Arrangement of three domains in WRF Model

WRF model contains number of physics options. Under these physics options there are many schemes available for selection. This allows the modeler to use wide variety model physics combinations in predicting weather. These physics schemes can be configured to fine tune the model to produce best results for the study area. The physics options are microphysics schemes, cumulus schemes, land surface schemes, long/shortwave schemes and planetary boundary layer schemes which influence the precipitation predictions.

During the study the model predictions are compared with observed point rainfalls, obtained from the Department of Meteorology, Sri Lanka, for the rain gauging stations at Mapalana, Kekanadura tank, Thihagoda, Thelijjawila, Goluwatta, and Mawarella Estate. The observed point rainfall data were spatially distributed using inverse distance weighting on 5 km x 5 km horizontal grid for comparison with the predictions of WRF model for the same grid. Variation within +/- 5 mm range was considered as an acceptable forecast. Area inside the basin in which the predictions were within the above specified +/- 5 mm range was expressed as a % of the total area of the basin (Correctly Predicted Area %, CPA). This was taken as the measure of success of the predictions for different physics schemes.

### *2.2 Hydrologic Modeling with HEC-HMS*

Rainfall is converted to runoff using hydrologic modeling of the Nilwala basin with HEC-HMS 3.3 hydrologic model. HEC-HMS is a numerical model includes different methods to simulate runoff in a watershed predicting flow and stage variation with time (USACE, 2008).

Data needed for the hydrologic component of the study basically comprised of precipitation records of the Nilwala basin, discharge data of the river, digital elevation map of the basin, location data of the rain gauges and river gauges etc. Hydrologic modeling was performed on the upper part of the Nilwala basin upstream of Pitabeddara and once the model is calibrated the same is extended to lower basin. Such approach is needed as no reliable flow gauging station below this is available.

For model calibration and verification phases, as transformation techniques Clark's method, Snyder's method and SCS (US Soil Conservation Services) method were applied in conjunction with the Green Ampt loss model. The recession base flow method was used for modeling base flow in all the cases. For the model calibration three rainfall-runoff events were arbitrarily selected as given in the following Table1.

Table1 Rainfall-runoff events selected for the calibration of the HEC-HMS model

| **Start date of event** | **End date of event** | **Peak date of event** | **Peak discharge ($m^3/s$)** |
|---|---|---|---|
| 14-Sep-74 | 20-Sep-74 | 16-Sep-74 | 125.6 |
| 5-May-75 | 13-May-75 | 07-May-75 | 171.8 |
| 9-May-78 | 20-May-78 | 15-May-78 | 279.3 |

For the evaluation of model performance there are various different criteria are used. In this investigation Nash–Sutcliffe efficiency index, Q Simulated /Q Observed ratio and Peak Q Simulated/Peak Q Observed ratio were used to evaluate the model performances. For model verification another set of different flood events were selected as given in Table 2.

Table 2 Rainfall-runoff events selected for the verification of the HEC-HMS model

| **Start date of event** | **End date of event** | **Peak date of event** | **Peak discharge ($m^3/s$)** |
|---|---|---|---|
| 10-Jun-79 | 18-Jun-79 | 14-Jun-79 | 105.4 |
| 10-Jul-84 | 17-Jul-84 | 13-Jul-84 | 128.8 |
| 25-Sep-79 | 30-Sep-79 | 27-Sep-79 | 199.0 |

The Nilwala river basin was subdivided into 10 sub basins based on the major tributaries as shown in Figure 4. Flows generated in the sub-basins had to be routed in order to convey them downstream. The Muskingum-Cunge routing technique was selected in the study and the parameters were derived from details of river cross-sections. For all the reaches the Manning's 'n' was taken as 0.030 and in flood plains 0.035.(Dyhouse et al, 1996). The predicted rainfalls from the WRF were given as spatial average of rainfall over each sub-basin to HEC-HMS.

### *2.3 Inundation Mapping*

The flow prediction of the hydrologic model was used to map the inundation extent downstream of Pitabeddara up to Matara town. To obtain the water levels along the main river HEC-RAS 4.0 hydraulic model was used. Arc-Map was then used to prepare the inundation map.

Inundation mapping used digitized main river and a Digital Elevation Model from ASTER data. Along the main river cross-sections were defined. The lateral flows from tributaries were introduced to the main river at appropriate locations. The boundary condition at upstream the river at Pitabeddara was introduced as the hydrograph HEC-HMS for rainfall from WRF. The lower boundary condition was the normal depth with 0.001 energy gradient. Results from HEC-RAS were exported to Arc-GIS for two dimensional visualizations.

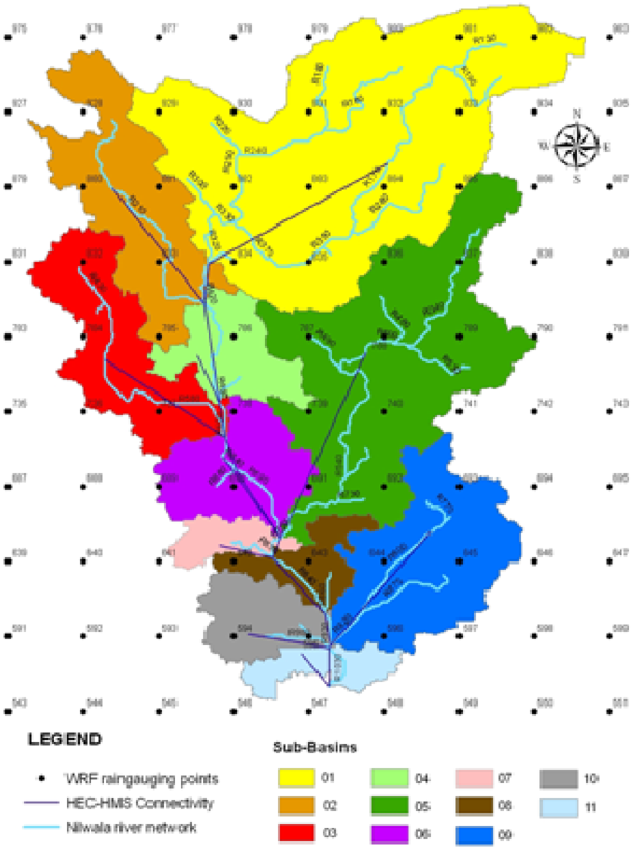



Figure 4 Sub-basin division of Nilwala Basin

Inundation corresponding to the flood event occurred on the 18-May-2003 was mapped. There flood maps were prepared for 16th,17th,18th and 19th of May-2003 with the discharges obtained from the HEC-HMS hydrologic model driven by the precipitation predicted by WRF weather model.

## 3. RESULTS AND DISCUSSION

Results of the investigation of impacts of microphysics schemes are given in Table 3. All the microphysics schemes (Lin et al, Kessler, Thompson, Morrison, WSM3, WSM6 and Ferrier) show high accuracy over the basin for event of 06/04/2009 while low accuracy for the event on 20/03/2009. The rain events on 10/12/2008 showed varying accuracy with different schemes. The Ferrier microphysics scheme is accepted as it was giving better results for all events.

Table 3. CPA % for different Microphysics schemes

| Rain event | 10/12/2008 | 20/03/2009 | 06/04/2009 |
|---|---|---|---|
| Microphysics scheme | CPA % | CPA % | CPA % |
| Lin et al | 66 | 55 | 88 |
| Kessler | 68 | 19 | 86 |
| Thompson | 40 | 37 | 88 |
| Morrison | 46 | 16 | 88 |
| WSM3* | 80 | 37 | 90 |
| WSM6 | 50 | 13 | 86 |
| Ferrier | 71 | 84 | 91 |

*WRF 3.0 default option

When it comes to the cumulus schemes a clear pattern of prediction accuracy over the basin was not observed. The prediction accuracy changed spatially from event to event with different cumulus schemes used. The model default Kain-Fritsch cumulus scheme produced reasonably good results and therefore selected for modeling. In the case of land surface options all schemes have produced good predictions in all the three rain events. The RUC is selected as it was the most consistent scheme among the three models tested. The RRTM longwave radiation scheme with Dudhia shortwave scheme produced good rainfall predictions for the three events considered. These are the model default longwave and shortwave radiation options in WRF. Mellor Yamada and YSU planetary boundary layer schemes have shown very little influence on the spatial distribution of the accuracy of the predictions. Therefore the default scheme is selected.

According to the results of hydrologic modeling performances, the Snyder's transformation technique in HEC-HMS produced the best results for the Upper Nilwala basin in calibration and verification phases. Results of model validation with Snyder's transformation technique are given in Table 4.

Table 4. Results of model validation with Snyder's transformation

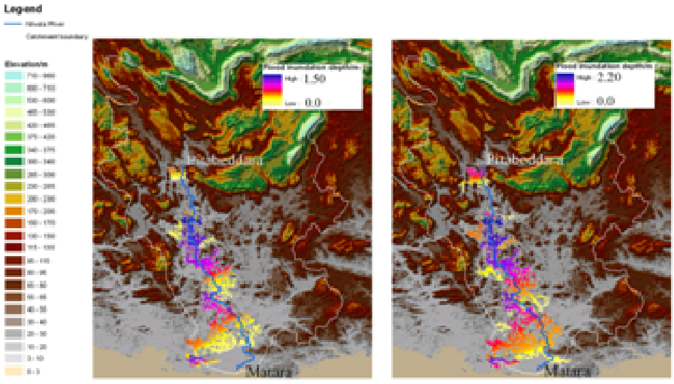
| Model performance evaluation criterion | Rainfall-Runoff event (date of peak) | | |
|---|---|---|---|
| | 14-Jun-79 | 13-Jul-84 | 27-Sep-79 |
| Nash–Sutcliffe efficiency % | 76.14 | 51.31 | 57.75 |
| Q Simulated /Q Observed | 1.29 | 1.27 | 1.09 |
| Peak Q Simulated/Peak Q observed | 1.03 | 0.78 | 0.76 |

The inundation maps developed for the stretch of Nilwala River from Pitabeddara to Matara are shown in figure 5. Depths of inundation and corresponding areas affected have been given in table 5.

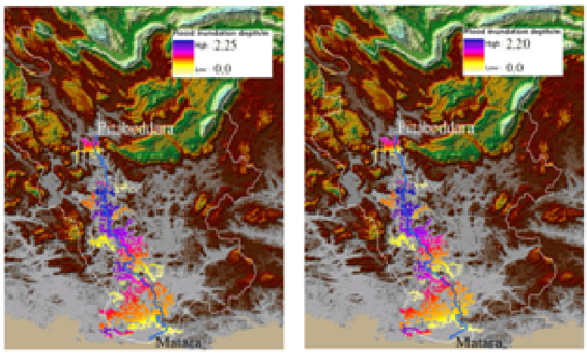
Table 5 Depths of inundation and corresponding areas affected

| Depth of inundation/ m | Inundated area km 2 | | | |
|---|---|---|---|---|
| | 16-May | 17-May | 18-May | 19-May |
| 0.0-0.5 | 28.3 | 16.9 | 16.2 | 16.6 |
| 0.5-1.0 | 15.7 | 30.1 | 30.1 | 31.1 |
| 1.0-1.5 | 12.2 | 14.7 | 15.3 | 14.4 |
| 1.5-2.0 | 0.0 | 13.1 | 13.8 | 10.5 |
| 2.0-2.5 | 0.0 | 0.5 | 1.2 | 0.4 |
| Total inundated area km $^2$ | 56.1 | 75.3 | 76.7 | 73.1 |

The model was capable of predicting the inundated areas correctly as shown in Figure 5. The combined WRF – HECHMS model has underestimated the river discharge which was about 1000 $m^3/s$ (Pacific, 2007) on the 18-May-2003 at Pitabeddara, according to the Department of irrigation but the corresponding discharge has been determined by the model as 664$m^3/s$. This is attributed to the model accuracies and improvement of the procedures is continuing.




(a) Maximum inundation on 16-May-2003 (b) Maximum inundation on 17-May-2003




(c) Maximum inundation on 18-May-2003 (d) Maximum inundation on 19-May-2003

Fig. 5 Inundation during the May 2003 flood

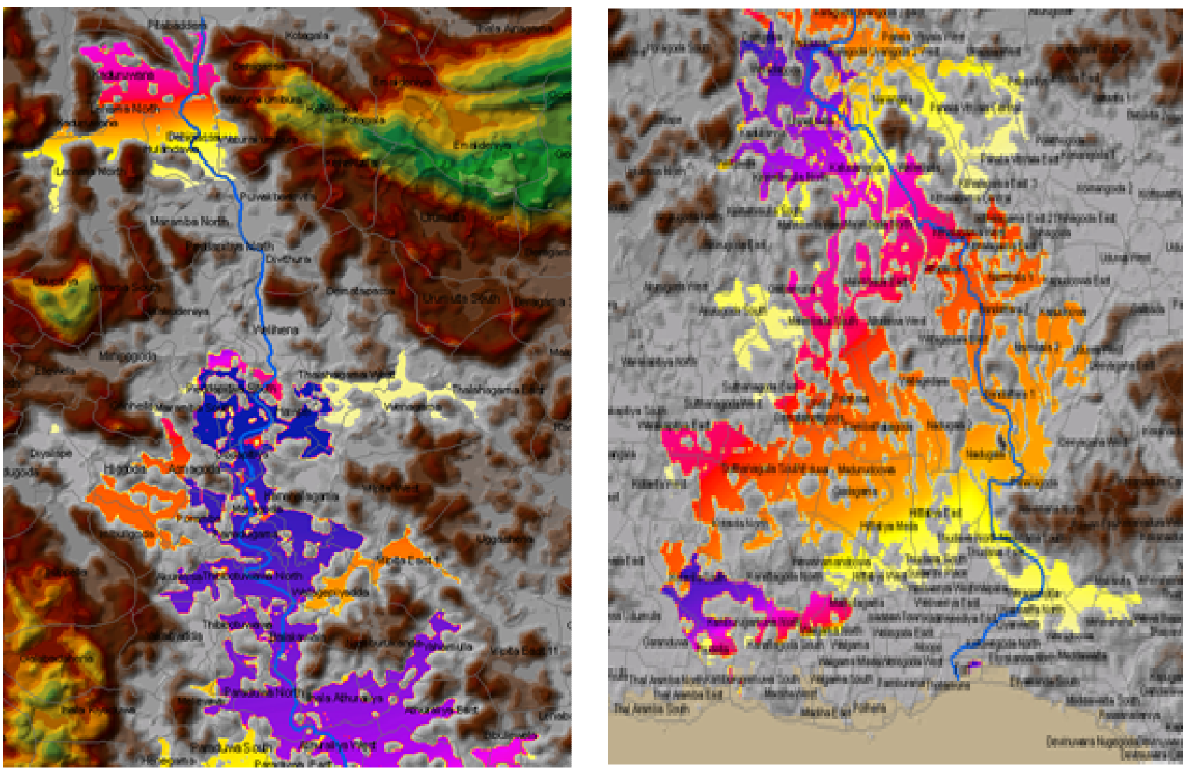


Fig 6 Maximum Inundation downstream of Pitabeddara

## 4. CONCLUSIONS

WRF Model and HECHMS model configuration for accurate flood prediction was thoroughly studied. It could be concluded that the model physics combination consisting of Ferrier microphysics scheme, Kain-Fritsch cumulus scheme, RUC land surface scheme, RRTM longwave radiation scheme, Dudhia shortwave scheme and YSU planetary boundary layer scheme has yielded better precipitation predictions over the Nilwala river basin. However, the total rainfall failed to generate the observed runoff indicating the model under estimated the total rainfall. The model was capable of predicting the inundation area with reasonable accuracy. This technique can be used to downscale GCM results to predict floods within reasonable accuracy.

## REFERENCES

Chen, G, Ning, Z, Hou, J, Xie, S and Deng, A, (2007), Using WRF-ARW in Operational Forecasts, WRF user's workshop

Dyhouse, G, Benn, J.R, Hatchett, P.E.J, (1996) Floodplain Modeling Using HEC-RAS, US Army Corps of Engineers, 1st edition

Elkaduwa, W.K.B, Sakthivadivel, R, (1998) Use of historical data as a decision support tool in watershed management: A case study of the Upper Nilwala basin in Sri Lanka. Research Report 26, International Water Management Institute, Colombo, Sri Lanka

Guleid, A, Hussein, G, Jodie, L.S, Kwabena, A, Christina, J. B, James, P. V. (2007) Adequacy of satellite derived rainfall data for stream flow modeling, Journal of Natural Hazards, Vol.43, pp.167-185.

Jayasekera, S.R (2009) Innovative Strategies towards Flood Resilient Cities in Asia-Pacific, Country report-Sri Lanka, Bangkok, 21-23 July 2009

Jonkman, S.N (2005) Global perspectives on loss of human life caused by floods, Journal of Natural Hazards, Vol.34, pp. 151–175.

Pacific Consultants International, (2007) Comprehensive study on disaster management in Sri Lanka, Progress report 1, March-2007, pp 29-30

USACE, (2008) HEC-HMS hydrologic modeling system user's manual, Hydrologic Engineering Center, Davis, CA.

# COMPARATIVE ANALYSIS OF THE RECONSTRUCTION PROCESS OF URBAN FACILITIES IN INDONESIA BASED ON RECOVERY CURVES AFTER THE 2004 INDIAN OCEAN TSUNAMI

K. Sugiyasu [1], O . Murao[2]

[1] Doctoral Student, Graduate School of Systems and Information Engineering, University of Tsukuba,
1-1-1 Tennodai, Tsukuba, Ibaraki, 305-8573 Japan
[1] E-mail: sugiya30@sk.tsukuba.ac.jp, Telephone & Fax: +81-29-853-5370

[2] Associate Professor, Graduate School of Systems and Information Engineering, University of Tsukuba,
1-1-1 Tennodai, Tsukuba, Ibaraki, 305-8573 Japan
[2] E-mail: murao@risk.tsukuba.ac.jp, Telephone & Fax: +81-29-853-5370

**Abstract:** Aceh in Indonesia was the most seriously damaged area due to 2004 Indian Ocean Tsunami, where more than 240,000 people were lost or killed. Following the event, Government devised a blueprint of urban recovery master plan, and lots of urban infrastructures related to the projects had been constructed in Aceh as of April 2009. The gap between the plan and reality that has some problems of recovery matter shows future challenge for post-disaster urban recovery and sustainable urban management. The authors conducted field survey in the damaged area to understand the recovery condition and obtained sets of data collected for 47 months since January 2005 by Badan Rehabilitasi dan Redonstruksi NAD-Nias (BRR), a recovery and rehabilitation agency. In this paper, recovery process in Aceh is analyzed using recovery curves for 14 indicators: department of housing (temporary and permanent housing), infrastructure (road, bridge, airport, and seaport), education (school and training of teacher), medical (hospital), economy (farmland, fishery, and enterprise support), cultural affairs (religious facilities) and Institutional development (government office). Then, the difference between the actual process of reconstruction and prepared recovery plans are discussed. In conclusion, the followings are clarified: (1) the progress of recovery of education, medical, and economy was hastened; (2) in other side, housing and infrastructure were delayed compared with other indicators; and (3) temporary housing was the earliest among all. Actually, the commencement of construction was delayed 7.6 months behind the scheduled recovery plan. The authors also discuss the reason of such problem based on its social context.

**Keywords:** recovery curve, Aceh, the 2004 Indian Ocean Tsunami, resettlement, urban infrastructure

## • Introduction

### *1.1 Background*

As of December 2010, six years have been since the 2004 Indian Ocean Tsunami. According to Reliefweb (2005) and WHO (2005), the catastrophic damage caused 300,000 people to die and 620,000 buildings to destroy in Southeast Asia and East Africa (Table 1). Aceh (Nanggröe Aceh Darussalam) in Indonesia was the heaviest of affected area where 240,000 people were lost or killed and 510,000 buildings were destroyed (Figure 1). The amount of damage in Indonesia was nearly 5 times more than that of Sri Lanka and 80% among all the affected countries. Especially, Kota (city) Banda Aceh and Kab (province) Aceh Besar was the main affected area and was also stricken by the impact of tsunami and earthquake. The range of damage in Indonesia reached to the inland area from the costal area. Therefore, Aceh had to recover almost all urban functions from the state of nothing. For example, the affected area that falls into a serious situation will have to spend a longer time and much more cost to complete the recovery. In addition, it may easily delay the recovery process comparing to the prepared recovery plan. However, the factor of delay is obvious because various stakeholders exist and they build a complicated relationship in the recovery process.
So, it is important to analyze the recovery process after disasters and to investigate the gap between actual process of reconstruction and prepared recovery plan of Aceh, concerning future post-disaster recovery initiatives. As a research method to analyze recovery process, this study applied Murao and Nakzato’s (2008) proposed “recovery curves” method based on the reconstruction situation data of building.

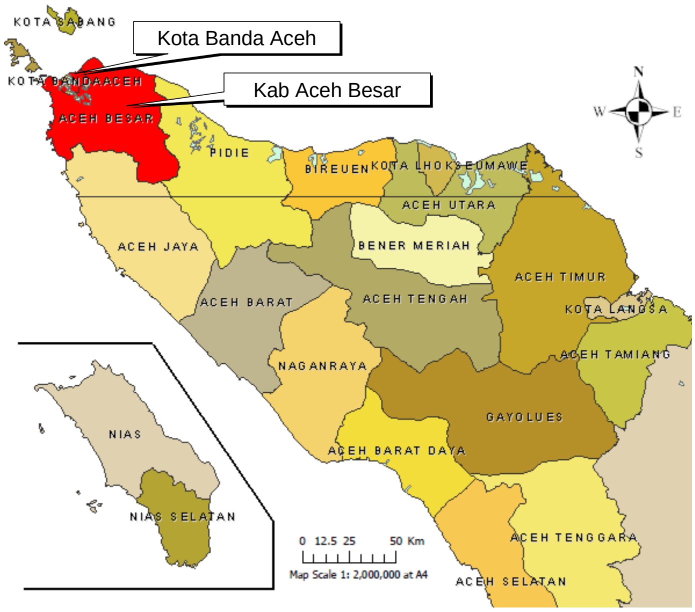



Figure 1: *Affected area in Indonesia (Nanggröe Aceh Darussalam)*

Table 1: *Damage of 2004 Indian Ocean Tsunami all over the world*

| Affected countries | Indonesia | Sri Lanka | India | Thailand | Somalia | Maldives | Malaysia | Myanmar | Sesel | Total |
|---|---|---|---|---|---|---|---|---|---|---|
| Damage of buildings | 514,150 | 103,753 | - | 4,806 | - | 3,997 | - | 592 | - | 627,298 |
| Displaced People | 417,438 | 500,668 | 112,588 | - | 2,320 | 11,568 | 8,000 | 2,592 | 160 | 942,746 |
| Deaths | 114,573 | 30,959 | 10,749 | 5,392 | 394 | 82 | 68 | 61 | 3 | 162,281 |
| Missing | 127,749 | 5,644 | 5,640 | 3,062 | 158 | 26 | 6 | - | - | 142,285 |
| Deaths +Missing | 242,322 | 36,603 | 16,389 | 8,454 | 552 | 108 | 74 | 61 | 3 | 304,566 |

### *1.2 Purpose*

In this paper, the authors analyzed recovery process based on recovery curves by 14 kinds of urban infrastructure indicators such a department of housing (temporary housing and permanent housing), infrastructure (road, bridge, airport and seaport), education (school and training of teacher), medical (hospital), economy (farmland, fishery and enterprise support), cultural affairs (mosque or church) and Institutional development (government office). In addition, they investigated the differences between the actual process of reconstruction and prepared recovery plans.

## • Methods

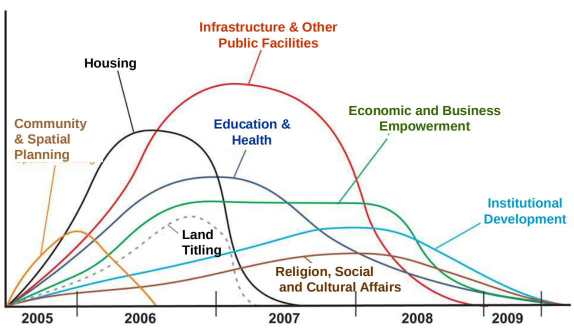
The procedure of this paper is shown as follows. The authors stepped on four stages in this study.

### *2.1 Data used*

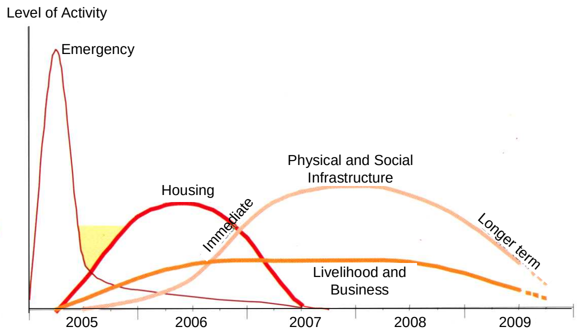
As of February 2008, the authors conducted field survey and interviews in the damaged area to investigate the recovery condition in Indonesia. During that time, they obtained a data set that had been compiled over 36 months since January 2005 by BRR (Badan Rehabilitasi dan Rekonstruksi NAD-Nias). Afterwards, the authors kept monitoring web contents of BRR and obtained new data set that had been compiled over 11 months since January 2008. Finally, the database was compiled over 47 months and gave the recovery condition of more than 19,000 traditional houses and 124,000 permanent houses reconstructed up to November 2008 in the damaged areas (BRR, 2007 and 2008). Moreover, they obtained six-monthly report (BRR, 2005b) and one year report (BRR, 2005a and 2006) that contained data on recovery condition and recovery plan.

### *2.2 Transition of recovery policy*

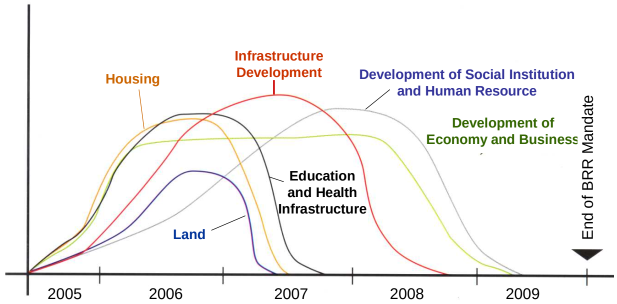
To understand the transition of the recovery policy, the authors arranged the conversion point of project period and the priority target according to recovery index.




(a) Recovery period (2005/10)




(b) Recovery period (2005/12)




(c) Recovery period (2006/4)

Figure 2: *Transition of recovery project period (2005/10-2006/4)*

Table 2: *The relationship between recovery section, index of infrastructure and planning period*

| Recovery section | Recovery indicaters sample | Years | 2005 | 2006 | 2007 | 2008 | 2009 |
|---|---|---|---|---|---|---|---|
| Housing section | Permanent houses built<br>Transitional houes built<br>Damaged house repaired<br>etc. | Planning period | Planned by 2005/10 (30 months)<br>Planned by 2005/12 (30 months)<br>Planned by 2006/4 (30 months) | | | | |
| | | Priority target | Recovery target ratio 30% | Recovery target ratio 90% | Recovery target ratio 100% | | |
| Infrastructure & Other Public Facilities section | Roads, bridges, airport, seaport rebuilt/constructed<br>Electric Power generated<br>Water production facilities reconstructed/rehabilitated<br>Houses supplied with sewage/sanitation<br>etc. | Planning period | Planned by 2005/10 (48 months)<br>Planned by 2005/12 (60 months or over)<br>Planned by 2006/4 (45 months) | | | | |
| | | Priority target | High priority period to recovery transportation | | Aceh: To recovery major road at west<br>To reinforce major road at east and center | | |
| | | | | Planned time period for major local road, seaport and airport | Nias: To connect major road at north with south/To reinforce major road at east and west | | |
| Education & health section | Schools, hospitals, built/repaired<br>classrooms provided<br>Teachers trained etc. | Planning period | Planned by 2005/10 (60 months or over)<br>Planned by 2006/4 (33 months) | | | | |
| | | Priority target | High priority period to recovery education and health facilities | | High priority period to reinforce education and health networks | | |
| Economic & Business empowerment section | Agricultural land and fish ponds rehabilitated, fishing vessels provided/replaced, mangrove restored, Microfinance supported etc. | Planning period | Planned by 2005/10 (60 months or over)<br>Planned by 2005/12 (60 months or over)<br>Planned by 2006/4 (54 months) | | | | |
| | | Priority target | Job creation period | Growing of small and medium enterprise period | Deveropment of small and medium enterprise period | | |
| | | | | | | Recovery of tourism | |
| Religion, Social and Cultural section | Churchs, mosques and temple built/repaired etc. | Planning period | Planned by 2005/10 (60 months or over)<br>Planned by 2005/12 (60 months or over) | | | | |
| | | Priority target | Under constructing at any time | | | | |
| Institutional development section | Government buildings built/repaired, civil servants training, expert provided, radio station established etc. | Planning period | Planned by 2005/12 (60 months or over)<br>Planned by 2006/4 (54 months) | | | | |
| | | Priority target | Providing of institutional capacity building | | | | |

## *2.3 Construction of recovery curve*

To express the recovery condition of infrastructure, the authors classified the recovery process into 6 categories of 14 items by obtained data set based on the classification method of BRR. After this process, the authors constructed recovery curve according to the method of Murao and Nakzato (2008).

## *2.4 Analysis of recovery process*

As a last step, the author compared the difference between the actual process of reconstruction and prepared recovery plans used by recovery curve and reported them in a Japanese paper (Murao and Nakazato, 2010). The method adds new information, the following sections explain how to develop the vulnerability functions and discuss the differences among the existed fragility curves.

- **Transition of recovery plan**

Up to now, the recovery project plan in Aceh has been published in three times. The first plan was published in October, 2005 (six-monthly report of BRR). Figure 2a shows the period of the first recovery project classified by recovery section. Table 2 shows the relationship between recovery section, index of infrastructure and planning period.
The longest recovery project period at the first plan was schemed to continue after 2010. Afterward, since the second plan was published in December, 2005 (one year report of BRR, Figure 2b), the third plan was published in April, 2006 (one year report of BRR, Figure 2c). The longest recovery project period at the third plan was schemed to finish in the first half of 2009. Finally, the recovery project period became half of a year. This reason is that BRR needed to be finished the main part of recovery project within their active period that had been decided until March, 2009.

- **Construction of recovery curve**

### *4.1 Recovery ratio calculation*

In order to plot the recovery curves, it is necessary to normalize the recovery condition of damaged areas of varying size. This was done by comparing the recovery ratio of the number of buildings constructed per month with the total number of completed buildings as of November 2008.

### *4.2 Selection of recovery curve*

For the time period of 50 months, the cumulative ratio of building completion is assumed to be fitted a sigmoid curve such as Cumulative Normal Distribution curve, Logistic curve, or Gompertz curve. Curves showing the highest correlation with observed data were considered to represent the most optimal recovery curve. However, according to previous research of Murao and Nakzato (2008), they concluded when the permanent housing was analyzed, Cumulative Normal Distribution curve is fitted. Also in this study, the permanent housing is the main analytical indicator. So the authors

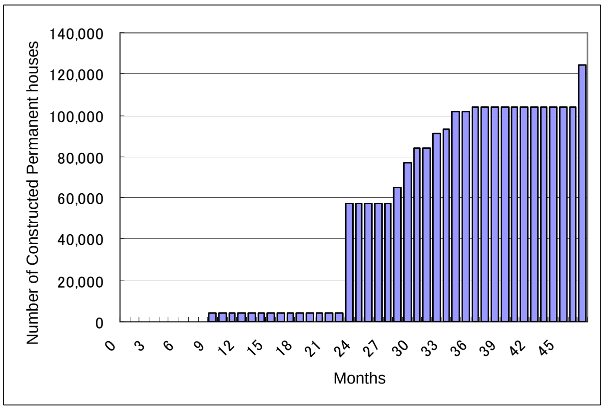



Figure 3: *Reconstruction of permanent houses*

Table 3: *Recovery process in Indonesia*

| Recovery section | | | Housing | | Infrastructure & Other Public Facilities | | | | Education & Health | | | Economic and Business Empowerment | | | Religion, Social and Cultural Affairs | Institutional development section |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Recovery indicaters | | | Transitional houes built (unit) | Permanent houses built (unit) | Roads built (km) | Bridges built(unit) | Airport built(unit) | Seaport built(unit) | Hospitals, built(unit) | Schools built(unit) | Teachers trained (person) | Fish ponds built (ha) | Agricultural land built, (ha) | Enterprises upport (unit) | Churchs and mosques built(unit) | Government buildings built(unit) |
| Amount of recovery target | Mar.2008 | | - | 132,928 | 3,000 | 1,628 | 11 | 17 | 923 | 1,750 | 8,999 | 27,593ha | 70,000ha | 100,000 | - | 450 |
| Recovery condition: Laying down the foundation for a better future | 2005 | Sep | 5,634 | 4,083 | | | | | | | | | 6,689 | | | |
| | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ |
| Tsunami Recovery Indicators Package | 2006 | Oct | | | | | 4 | 2 | | 556 | | | | | | |
| | | Nov | | | 1511 | 158 | | | 324 | | 5,429 | 6,800 | | 43,263 | | |
| QUICK STAT | | Dec | 15,000 | 57,000 | 1500 | 158 | 5 | 14 | 328 | 747 | 5,385 | 6,800 | 50000 | 43,263 | | |
| | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ |
| | 2007 | Apr | 17,159 | 64,971 | 1553 | 181 | 7 | 17 | 384 | 782 | 17,115 | 12,385 | 75483 | | | |
| | | May | 17,159 | 77,194 | 1553 | | 7 | 17 | 405 | 782 | 17,115 | 12,385 | 75483 | | 1364 | |
| | | Jun | 17,159 | 84,387 | 1586 | 181 | 10 | 17 | 405 | 804 | 21,962 | 27,593 | 63923 | 77,316 | 1472 | 332 |
| | | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ |
| | | Aug | 18,424 | 90,861 | 1586 | 216 | 10 | 17 | 515 | 822 | 22,436 | 12,935 | 64019 | 82,595 | 1477 | 367 |
| | | Sep | 18,424 | 93,629 | 1586 | 216 | 10 | 17 | 515 | 822 | 22,436 | 12,935 | 64019 | 82,595 | 1477 | 367 |
| | | Oct | 19,482 | 102,063 | 2006.8 | 216 | 10 | 17 | 534 | 837 | 22,548 | 12,935 | 64019 | 99,710 | 1477 | 367 |
| | | Nov | 19,482 | 102,063 | 2191 | 226 | 10 | 17 | 613 | 868 | 23,095 | 12,935 | 64019 | 99,903 | 1481 | 795 |
| | | Dec | 19,889 | 104,287 | 2191 | 226 | 10 | 17 | 613 | 888 | 23,270 | 13,570 | 78846 | 100,058 | 1512 | 808 |
| | 2008 | Jan | 19,889 | | 2475 | 253 | 10 | 17 | 757 | 922 | 24,369 | 14,589 | 93554 | 100,196 | 1620 | 933 |
| | | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ | ~ |
| e-aceh-nias.org | | Nov | - | 124,454 | 3,055 | 266 | 12 | 20 | 954 | 1,450 | 38,911 | | 103,273 | 139,282 | 1620 | 979 |
| Recovery ratio | at Nov.2008 | | - | 93.6% | 101.8% | 16.3% | 109.1% | 117.6% | 103.4% | 82.6% | 432.4% | 52.9% | 147.5% | 139.3% | - | 217.6% |

accepted to analyze recovery process by Cumulative Normal Distribution curve.

***4.3 Sample data of building construction***

Table 3 shows the recovery process in Indonesia classified by 14 kinds of urban infrastructure indicators such as department of housing (temporary housing and permanent housing), Infrastructure (road, bridge, airport and seaport), education (school and training of teacher), medical (hospital), economy (farmland, fishery and enterprise support), cultural affairs (mosque or church) and Institutional development (government office) based on statistical data. Figure 3 shows the recovery process in Indonesia calculated by the cumulative number of completed permanent houses.

***4.4 Plotting of recovery curves***

The factors of time (months) and the ratio of building completion were used to draft the recovery functions. The time period begun in December 2004, with January 2005 is regarded as month “1”, and extends over 50 months until February 2009. The ratio of building completion for a given time period is calculated based on the total amount of completed buildings. For a time period of $t$ (months), the cumulative ratio of building completion $R(t)$ can be described by the Cumulative Normal Distribution curves, using the following equations:

a. Cumulative Normal Distribution curve

$$R(t) = \Phi((t-\lambda)/\zeta) \qquad [1]$$

Where $\Phi$ represents the standard Normal Distribution, and $\lambda$ and $\zeta$ are the mean and standard deviation of $t$, respectively. The two parameters $\lambda$ and $\zeta$ are determined using the least squares method on probability paper.

## • Analysis of recovery processes

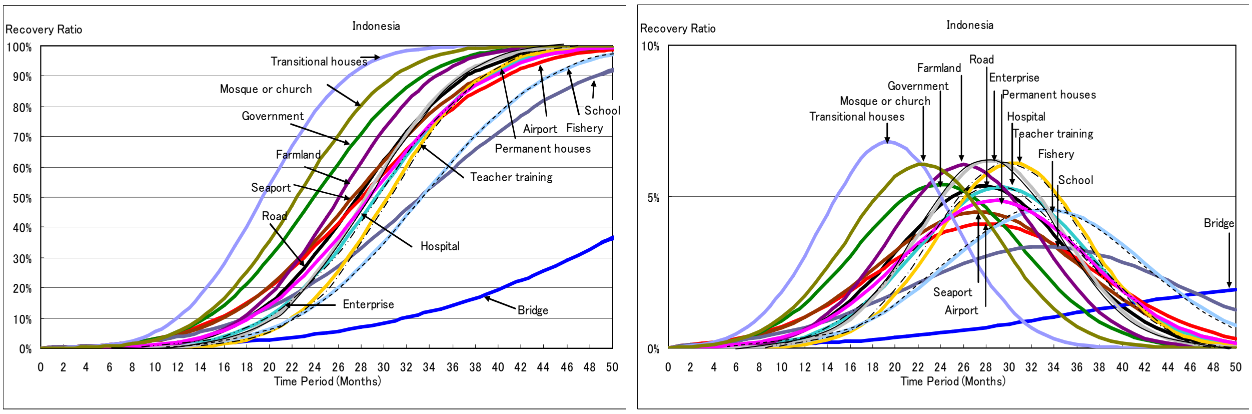
Figure 4 shows the recovery curves in Indonesia by 14 kinds of urban infrastructure indicators. Table 4 shows the parameters and recovery speed each of recovery curves. In this study, the authors assessed recovery curves by 2 point. First point was to analyze average completion months $\lambda$. It means the average period to complete recovery infrastructure. Second point was to analyze final completion months. It means the period of complete to recovery of all infrastructure.

***5.1 Comparison of completion months***

As a result of comparing the average completion months with the final completion months, the recovery process of transitional houses (Average: 19.4, Final: 37.6, Figure 5a) were the earliest all of indicators. And after followed by mosques and churches (Average: 22.4, Final: 42.8, Figure 5e), government buildings (Average: 23.9, Final: 45.2, Figure 5f), and Farm land (Average: 26.1, Final: 46.5, Figure 5d). These kept the same ranking in both average completion months and the final completion months. On the other hand, the recovery process of school (Average: 33.2, Final: 59.7, Figure 5c), fishpond (Average: 33.3, Final: 60.3, Figure 5d), bridges (Average: 56.7, Final: 83.2, Figure 5b) were slower than other indicators. And, bridge was the slowest all of indicators. In addition, permanent houses (Average: 28.8, Final: 54.2, Figure 5a), roads (Average: 27.7, Final: 50.8, Figure 5b), airports (Average: 28:1, Final: 53.1, Figure 5b), seaports (Average: 27.3, Final: 51.1, Figure 5b) drop the ranking from the average completion months to the final completion months.

***5.2 Difference in plan and actual***

In the last phase, the authors analyzed the difference between the plan and actual recovery process. Table 5 shows the data. As it is seen, the actual recovery process of transitional houses was delayed than that was planned by 7.6 months. Moreover, permanent houses delayed than the plan by 2 years (Figure 5a). It is thought that the confusion of recovery plan is a caused in this situation. On the other hand, mosques and churches, government buildings, Farm land kept early recovery speed (Figure 5d, e, f).




(a) Recovery curves (b) Probability density functions

Figure 4: *Recovery curves and probability density functions in Indonesia*

Table 4: *Recovery curve parameters and recovery speed in Indonesia*

| Recovery section | Recovery indicaters | $\lambda$ | $\zeta$ | $R^2$ | Average completion period | | Change Rank | Final completion period | | Change Rank |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Months | Rank | | Months | Rank | |
| Housing | Transitional houes built (unit) | 19.4 | 5.855 | 0.888 | 19.4 | 1 | | 37.6 | 1 | – |
| | Permanent houses built (unit) | 28.8 | 8.197 | 0.922 | 28.8 | 9 | | 54.2 | 11 | Down |
| Infrastructure & Other Public Facilities | Roads built (km) | 27.7 | 7.468 | 0.938 | 27.7 | 6 | | 50.8 | 8 | Down |
| | Bridges built(unit) | 56.7 | 19.417 | 0.845 | 56.7 | 14 | | 83.2 | 14 | – |
| | Airport built(unit) | 28.1 | 9.699 | 0.940 | 28.1 | 7 | | 53.1 | 10 | Down |
| | Seaport built(unit) | 27.3 | 8.889 | 0.861 | 27.3 | 5 | | 51.1 | 9 | Down |
| Education & Health | Hospitals, built(unit) | 29.3 | 7.508 | 0.946 | 29.3 | 10 | | 49.9 | 6 | Up |
| | Schools built(unit) | 33.2 | 11.919 | 0.890 | 33.2 | 12 | | 59.7 | 12 | – |
| | Teachers trained(person) | 30.3 | 6.519 | 0.890 | 30.3 | 11 | | 50.5 | 7 | Up |
| Economic and Business Empowerment | Fish ponds built (ha) | 33.3 | 8.718 | 0.945 | 33.3 | 13 | | 60.3 | 13 | – |
| | Farm land built (ha) | 26.1 | 6.601 | 0.907 | 26.1 | 4 | | 46.5 | 4 | – |
| | Enterprise support(unit) | 28.4 | 6.423 | 0.879 | 28.4 | 8 | | 48.3 | 5 | Up |
| Religion, Social and Cultural Affairs | Churchs and mosques built(unit) | 22.4 | 6.579 | 0.928 | 22.4 | 2 | | 42.8 | 2 | – |
| Institutional development section | Government buildings built(unit) | 23.9 | 7.380 | 0.933 | 23.9 | 3 | | 45.2 | 3 | – |

Table 5: *Difference of plan and actual recovery process in Indonesia*

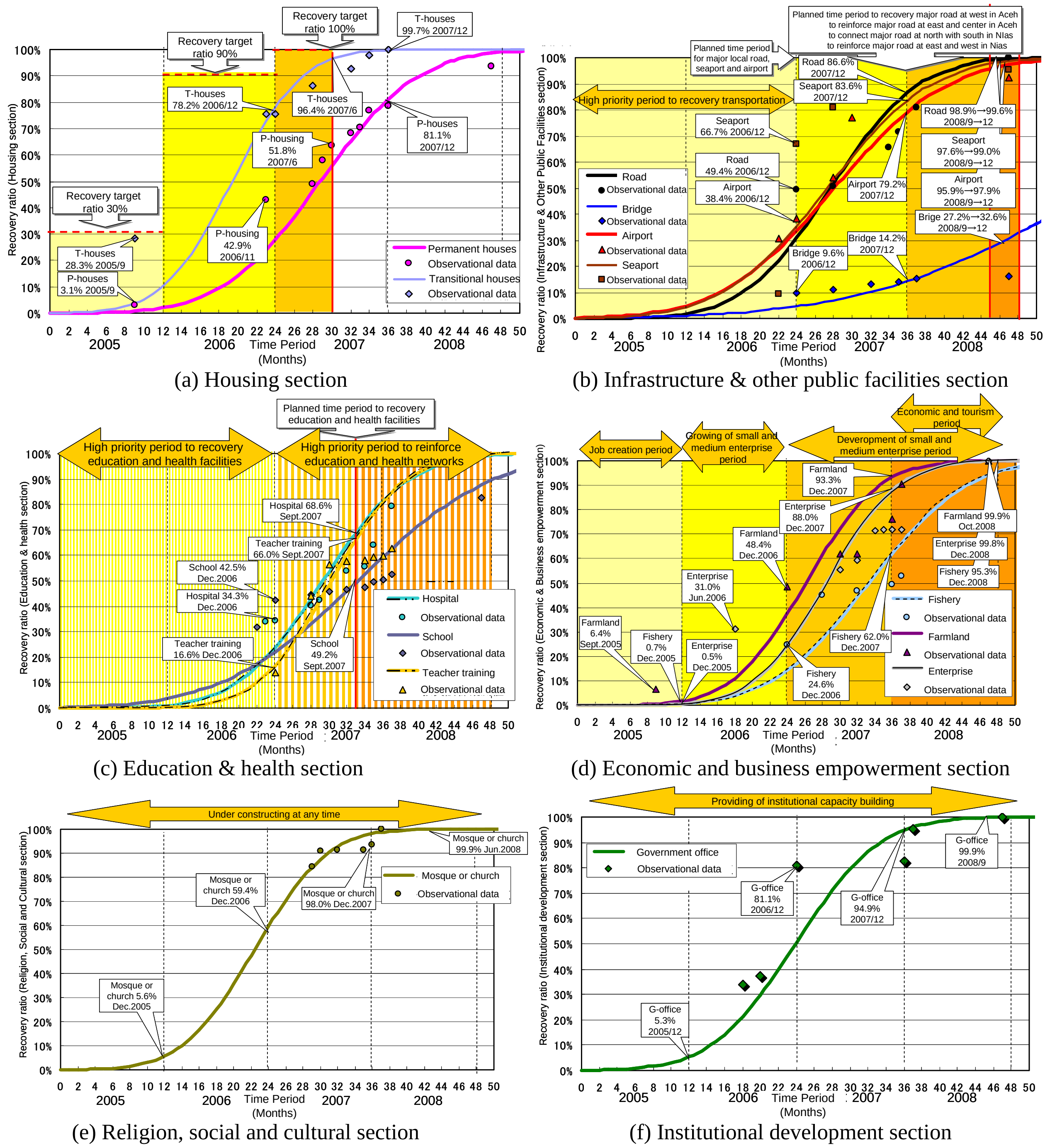
| Recovery section | Recovery indicaters | Final completion period | | Period of recovery plan | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Months | Rank | First plan Oct.2005 (months) | Difference of plan (months) | Rank | Second plan Dec.2005 (months) | Difference of plan (months) | Rank | Third plan Apr.2006 (months) | Difference of plan (months) | Rank |
| Housing | Transitional houes built (unit) | 37.6 | 1 | 30 | Δ7.6 over | 11 | 30 | Δ7.6 over | 9 | 30 | Δ7.6 over | 7 |
| | Permanent houses built (unit) | 54.2 | 11 | | Δ24.2 over | 12 | | Δ24.2 over | 11 | | Δ24.2 over | 11 |
| Infrastructure & Other Public Facilities | Roads built (km) | 50.8 | 8 | 48 | Δ2.8 over | 8 | 60 or over | 9.2 early | 5 | 45 | Δ5.8 over | 4 |
| | Bridges built(unit) | 83.2 | 14 | | Δ35.2 over | 13 | | Δ23.2 over | 10 | | Δ38.2 over | 13 |
| | Airport built(unit) | 53.1 | 10 | | Δ5.1 over | 10 | | 6.9 early | 7 | | Δ8.1 over | 8 |
| | Seaport built(unit) | 51.1 | 9 | | Δ3.1 over | 9 | | 8.9 early | 6 | | Δ6.1 over | 5 |
| Education & Health | Hospitals, built(unit) | 49.9 | 6 | 60 or over | 10.1 early | 4 | | | | 33 | Δ16.9 over | 9 |
| | Schools built(unit) | 59.7 | 12 | | on schedule | 6 | | | | | Δ26.7 over | 12 |
| | Teachers trained(person) | 50.5 | 7 | | 9.5 early | 5 | | | | | △17.5 over | 10 |
| Economic and Business Empowerment | Fish ponds built (ha) | 60.3 | 13 | 60 or over | on schedule | 7 | 60 or over | on schedule | 8 | 54 | Δ6.3 over | 6 |
| | Farm land built (ha) | 46.5 | 4 | | 13.5 early | 2 | | 13.5 early | 3 | | 7.5 early | 2 |
| | Enterprise support(unit) | 48.3 | 5 | | 11.8 early | 3 | | 11.8 early | 4 | | 5.8 early | 3 |
| Religion, Social and Cultural Affairs | Churchs and mosques built (unit) | 42.8 | 2 | 60 or over | 17.2 early | 1 | 60 or over | 17.2 early | 1 | | | |
| Institutional development section | Government buildings built (unit) | 45.2 | 3 | | | | 60 or over | 14.8 early | 2 | 54 or over | 8.8 early | 1 |

## 6 Conclusion

In this study, recovery curves were developed to assess recovery from the 2004 Indian Ocean Tsunami using the construction ratio of urban infrastructure in Indonesia. In addition, the authors analyzed the difference between the actual process of reconstruction and prepared recovery plans.

It found the recovery process in average and final completion months of urban infrastructures in Indonesia. Recovery curves could be used to quantitatively assess the differences in recovery efforts of various urban infrastructures.

However, some problems remain in this study. The authors analyzed amount of infrastructure and period of recovery plan. But they did not refer to the cost problem of managing money and man power. These indicators are important to analyze the recovery process. So, it needs to consider combining. Moreover, this paper focused to capture the whole image of the recovery process in Indonesia. At the next stage, it is necessary to analyze the differences between regions.




(a) Housing section

(b) Infrastructure & other public facilities section

(c) Education & health section

(d) Economic and business empowerment section

(e) Religion, social and cultural section

(f) Institutional development section

Figure 5: *Recovery curves for different infrastructure in Indonesia*

## References

BRR. (2005a), *Aceh and Nias One Year After The Tsunami (The Recovery Effort and Way Forward).*
BRR. (2005b), *Laying Down the Foundation for a Better Future (Six-monthly Report of the Executing Agency for the Rehabilitation and Reconstruction of Aceh and Nias).*
BRR. (2006), *Building a Land of Hope (One Year Report Executing Agency for Aceh and Nias).*
BRR. (2009), "e-aceh-nias.org" , http://e-aceh-nias.org/home/ , 2009.1.31
BRR Information analysis section. (2007), "Tsunami Recovery Indicators Package For Aceh And Nias English Edition".
Murao, O., and Nakazato, H. (2008), "Recovery Curves for Housing Reconstruction in Sri Lanka after the 2004 Indian Ocean Tsunami", *Proceedings of the International Symposium on the Restoration Program from Giant Earthquakes and Tsunamis*, 191-196, Phuket, Thailand.
ReliefWeb Map Centre. (2005), "South Asia Earthquake and Tsunami : Affected population.WHO", *Situation report 32*.

## Acknowledgements

This paper is supported by "Restoration Program from Giant Earthquakes and Tsunamis, granted by Ministry of Education, Culture, Sports, Science and Technology, Japan. The authors are grateful for the assistance of Indonesia Governments, Mr. Rivadsyah, Ms. Sarah as a guide and translator.

## About the Authors

**K. SUGIYASU,** Doctoral Student at the Department of Risk Engineering, the University of Tsukuba, Japan, had been engaged in research on the post-tsunami urban recovery in Indonesia, Thailand and Sri Lanka with Professor Murao.

**O. MURAO,** an Associate Professor at the Department of Risk Engineering, the University of Tsukuba, Japan, had been engaged in research on building collapse risk at Institute of Industrial Science (IIS), the University of Tokyo for four years since 1996. Using the actual building damage data due to the 1995 Kobe earthquake, he analyzed them to clarify the relationship between seismic ground motion and building damage. As a result, he constructed building vulnerability functions based on the Kobe earthquake. The research paper was submitted to the University of Tokyo as his doctoral dissertation, *Study on Building Damage Estimation based on the Actual Damage Data due to the 1995 Hyogoken Nanbu Earthquake*, and he received the doctorate degree in 1999. His recent research field is post-disaster urban recovery and urban safety planning.
Dr. Murao had researched on "Comparative Study of the Policy and Urban Planning for Disaster Management in the USA and Japan." in the US as a Fulbright Visiting Scholar 2009-2010 at Graduate School of Design, Harvard University, and Pacific Tsunami Museum managed by the University of Hawaii since July 2009. His recent research interest is the relationship between architectural/urban design and disaster mitigation.

# VULNERABILITY FUNCTIONS FOR BUILDINGS BASED ON DAMAGE SURVEY DATA IN SRI LANKA AFTER THE 2004 INDIAN OCEAN TSUNAMI

Murao, O.[1], Nakazato, H.[2]

[1]Graduate School of Systems and Information Engineering, University of Tsukuba, 1-1-1 Tennodai, Tsukuba, Ibaraki, 305-8573 Japan
E-mail: murao@risk.tsukuba.ac.jp, Telephone & Fax: +81-29-853-5370

[2]Okinawa Electric Company, Japan

***Abstract***
The authors investigated building damage conditions as a result of the 2004 Indian Ocean Tsunami in five areas of Galle, Matara, and Hambantota, Sri Lanka. This paper presents tsunami vulnerability functions for the buildings, a relationship between building damage and inundation, in the country. In order to develop the functions, the authors used 1,535 building damage data in terms of structural types, solid (mainly reinforced concrete) and non-solid (masonry and timber-frame), and 166 inundation height data obtained by the field surveys. The developed fragility curves are compared with other curves previously developed by other researchers, and those future usages are discussed.

***Keywords:*** *fragility curve, Sri Lanka, the 2004 Indian Ocean Tsunami, building damage, inundation*

## 1. Introduction

Sri Lanka was the most affected country by the 2004 Indian Ocean Tsunami after Indonesia. Following the tsunami disaster, the Department of Census and Statistics (DCS) (2005a) reported that approximately 40,000 people were killed, and 96,000 houses in coastal areas were destroyed in the country. Those devastating experiences should be utilized for future urban safety and disaster reduction not only in the affected countries by the 2004 tsunami but also in other tsunami-prone areas. Building damage data can be used to develop tsunami vulnerability functions, which are helpful for building damage estimation in urban disaster management, if adequate inundation data is prepared. Aiming at developing vulnerability functions for building damage for tsunami in Sri Lanka, the authors conducted field surveys to collect building damage and inundation data after the 2004 tsunami, and consequently the vulnerability functions in terms of structural type were developed. Explaining how the authors developed the fragility curves, this paper discusses some characteristics and difference between the developed fragility curves and other curves constructed by other researchers. It also presents those usages for sustainable urban development in Sri Lanka.

## 2. Literature Review and Methods

### *2.1 Literature Review of Tsunami Vulnerability Functions for Buildings*

The relationship between building damage and inundation height was quantitatively examined by Hatori (1984) for the first time in Japan, based on damage data due to the 1896 and the 1933 Sanriku Tsunami, and the 1960 Chilean Tsunami. Shuto (1994) deduced the relation between the damage ratio of housings and tsunami height in the past tsunamis in Japan by numerical analysis and proposed "tsunami intensity and damage" table.

After the 2004 Indian Ocean Tsunami, Koshimura et al. (2007) developed fragility functions "by an integrated approach using the numerical modeling of tsunami inundation and GIS analysis of post–tsunami survey data" for Banda Aceh Indonesia. However, the consideration was not given to building structure because the used data were obtained by satellite images.

As for Sri Lanka, Peiris (2006) developed vulnerability functions based on the statistical data released by DCS (2005b), and Kimura et al. (2006) proposed other vulnerability functions through an investigation in Matara. However, those functions are not distinguished by building structural type. Then, the authors constructed other ones in terms of structural type using the actual damage data obtained by field surveys and reported them in Japanese paper (Murao and Nakazato, 2010). Adding new information, the following sections explain how to develop the vulnerability functions and discuss the differences among the existing fragility curves.

### *2.2 Methods*

This paper has two main parts: outline of the vulnerability functions developed by the authors, and comparison of tsunami vulnerability functions based on building damage in Sri Lanka.
The authors' vulnerability functions are developed in the following procedures: (1) collecting damage data by field surveys in Galle, Matara, and Hambantota, (2) defining building damage rank, (3) classifying building structural types, (4) collecting inundation height data by field surveys, (5) arranging the obtained data, (6) applying the dataset to normal distribution, and (7) constructing fragility curves. The procedure is outlined in Section 3.
In Section 4, the developed vulnerability functions mentioned above are compared with those by Peiris (2006) and Kimura et al. (2006). For the comparative study, differences of collecting data and developing methods are considered first. Then the developed curves by three research groups are examined in a same figure. Finally, meanings and usage of the vulnerability functions are discussed.

## 3. Development of Tsunami Vulnerability Functions for Buildings

With some additional information, this section explains how Murao and Nakazato (2010) developed the tsunami vulnerability functions.

### *3.1 Collecting Damage Data*

Field surveys were carried out twice in February and November 2005 in the Coastal Conservation Zone for five characteristic districts chosen from Galle, Matara, and Hambantota based on the regional map of Sri Lanka Survey Department, which contained polygonal building-shaped data. The total coastline length of the investigated areas were approximately 7km, and the total number of buildings was 1,535.

### *3.2 Defining Building Damage Rank*

Building damage due to the tsunami was classified into four categories defined in Table 1. The authors visually investigated each building damage condition on the ground first and then interviewed residents to confirm the adequacy through the field surveys.

*Table 1 Building damage rank and definition*

| Damage Rank | Complete Damage | Heavy Damage | Moderate Damage | No/Slight Damage |
|---|---|---|---|---|
| Definition | Complete structural damage | Structural damage and unusable | No visible structural damage and reusable | No visible mentionable damage |
| Image |  |  |  |  |

### *3.3 Buildings Structural Types*

Most of the houses were one or two storied buildings in the districts, and most of the one-story houses were brick-built, block-built, or wooden. On the other hand, two or more storied houses including apartments in the residential area and public, commercial, or office buildings with several stories in the central area of Galle were mainly reinforced concrete or steel buildings. Building structure is one of the important factors to differentiate the damage condition for tsunamis in general. Therefore, the objective buildings were classified into two structural types in this research: non-solid buildings and solid buildings, shown in Table 2.

*Table 2 Building structural types*

| Type | Non-solid buildings | Solid buildings |
|---|---|---|
| Structure (material) | Brick-built, block-built, or wooden | Reinforced concrete, steel |
| The number of floors | One or two | Two or more |
| Usage | Housing (commercial) | Public, commercial, or office |
| Image |  |  |

### *3.4 Collecting Inundation Height Data*

In the field surveys, the authors collected inundation height information by interviews with residents or checking water level sign remaining on buildings or concrete block walls. According to regional characteristics, the inundation data was collected at an interval of dozens or hundreds of meters.

### *3.5 Arranging the Obtained Data*

In order to analyze the relationship between building damage and tsunami inundation height, the investigated areas were divided by grid of 100m x 100m first. The total number of grids was 166. Then the representative inundation height of each district was determined based on the acquired data by the surveys gathering with information from contour models the authors made and existing materials about tsunami height in the areas. An example is shown in Figure 1. Finally, the building data—1,202 non- solid buildings, 333 solid buildings, and 1,535 all buildings—were sorted into appropriate groups by inundation height respectively. As a result of the arrangement, necessary data was prepared for the next step.

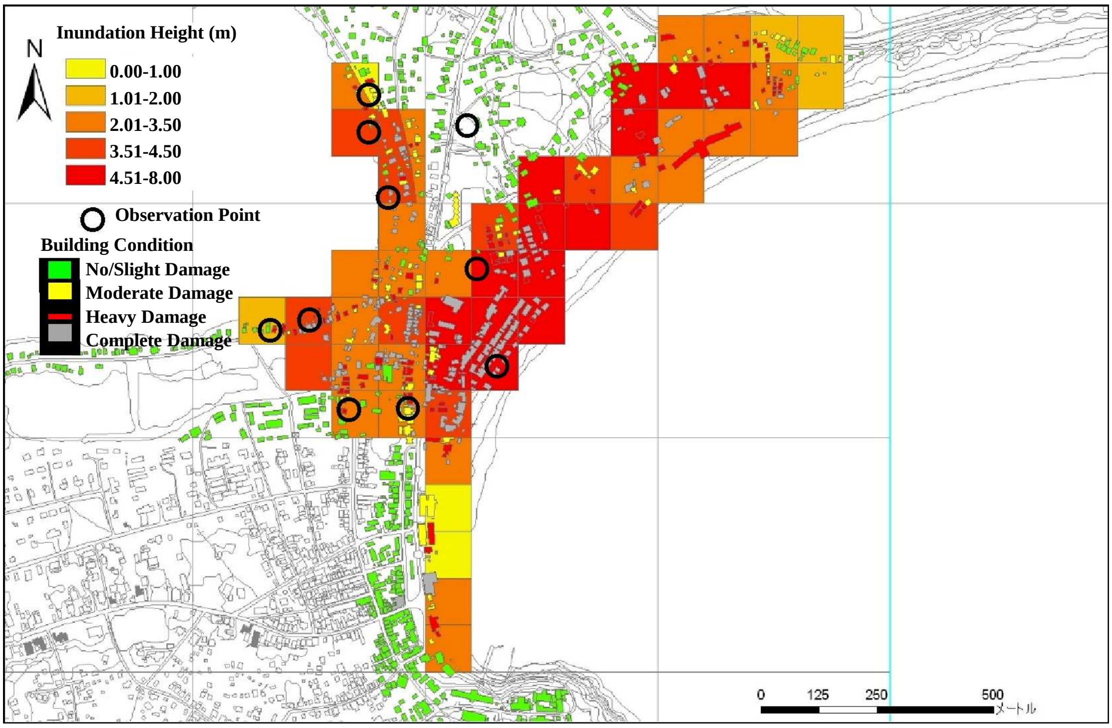



*Fig. 1 Inundation height and observation points on grids in Hambantota*

### *3.6 Applying the Dataset to Normal Distribution*

Such grouping was adopted to obtain reliable damage statistics that correspond to the inundation height. For an inundation index *x*, the cumulative probability $P_R(x)$ of the occurrence of damage equal or higher than rank *R* is assumed to be normal as follows:

$$P_R(x) = \Phi[(x - \lambda)/\varsigma] \tag{1}$$

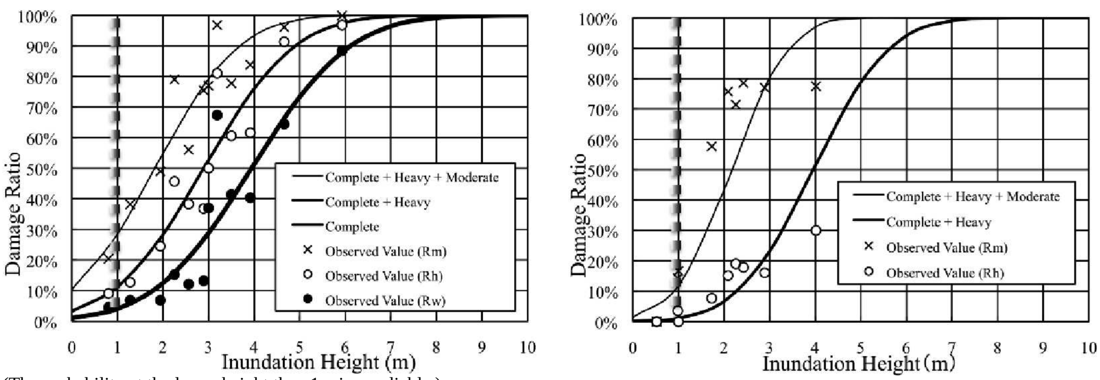
In which Φ is the standard normal distribution, and and are the mean and the standard deviation of *x*. The two parameters of the distributions, and , were determined by the least square method on logarithmic normal probability paper as shown in Table 3.

*Table 3: Parameters of tsunami vulnerability functions for buildings in Sri Lanka*

| | Damage Rank | | | $R^2$ |
|---|---|---|---|---|
| Non-solid Buildings | Complete ($R_c$) | 3.94 | 1.69 | 0.846 |
| | Complete + Heavy ($R_h$) | 2.89 | 1.56 | 0.908 |
| | Complete + Heavy + Moderate ($R_m$) | 1.82 | 1.45 | 0.859 |
| Solid Buildings | Complete ($R_c$) | - | - | - |
| | Complete + Heavy ($R_h$) | 3.96 | 1.31 | 0.684 |
| | Complete + Heavy + Moderate ($R_m$) | 2.16 | 0.98 | 0.641 |
| All Buildings | Complete ($R_c$) | 4.25 | 1.74 | 0.931 |
| | Complete + Heavy ($R_h$) | 3.19 | 1.60 | 0.971 |
| | Complete + Heavy + Moderate ($R_m$) | 1.87 | 1.65 | 0.901 |

### *3.7 Constructing Fragility Curves*

By the above means, fragility curves were constructed. The curves for non-solid buildings and solid buildings are shown in Fig. 2, and those for all buildings are shown in the next section to be compared with others. Herein the probability of the lower height than 1m is unreliable because the damage in the shallowly inundated areas was too light to distinguish moderate/slight damage from non-damage.




(The probability at the lower height than 1m is unreliable.) (The probability at the lower height than 1m is unreliable.)

(a) Non-solid buildings (b) Solid buildings

*Fig. 2 Fragility curves with respect to inundation height for different structural types*

## 4. Comparison between the Developed Fragility Curves and Other Curves in Previous Research

The vulnerability functions developed by the authors are compared with those of Peiris (2006) and Kimura et al. (2006) in this section. Table 4 shows the differences focused on data source and methods.

*Table 4: Comparison of the developing methods by three research groups*

| | Murao and Nakazato (2010) | Peiris (2006) (southwest) | Kimura et al. (2006) |
|---|---|---|---|
| Objective Area(s) | Galle, Matara, and Hambantota (7km-coast line) | 11 DS Divisions | Matara |
| Building Data Source | Field surveys (1,535 buildings) | Department of Census and Statistics (2005b) | Questionnaire survey (586 buildings) |
| Data Unit | Building | Division | Building |
| Inundation Data Source | Estimated by altitude and tsunami height, and obtained by field surveys | Department of Census and Statistics (2005b) | Numerical simulation |
| Data Unit | 100m x 100m grid | Division | 50m x 50m grid |
| The median (mean) value (m) | Complete: 4.25<br>Complete + Heavy: 3.19<br>C +H +Moderate: 1.87 | Complete: 2.80<br>Partial (Unusable): 2.05<br>Partial (Usable): 1.55 | Heavy: 2.70<br>Partial: 0.6 |
| Characteristics | Classified by structural type as well as all buildings | Using statistical macro data | Maximum inundation height is 4.1m |

### *4.1 The vulnerability functions developed by Peiris (2006)*

The vulnerability functions by Peiris (2006) were developed based on the statistical data reported by DCS. The functions were approximated as follows:

$$P\left[ds|H\right] = \Phi\left[\frac{1}{\beta_{ds}}\ln\left(\frac{H}{\overline{H}_{ds}}\right)\right] \tag{2}$$

In which, " $\overline{H}_{ds}$ is the median value of tsunami water height corresponding to the damage state, ds…. Φ is the standard normal cumulative distribution function and $\boldsymbol{\beta}_{ds}$ is the standard deviation of the natural logarithm of tsunami water height, *H* for damage state, ds."

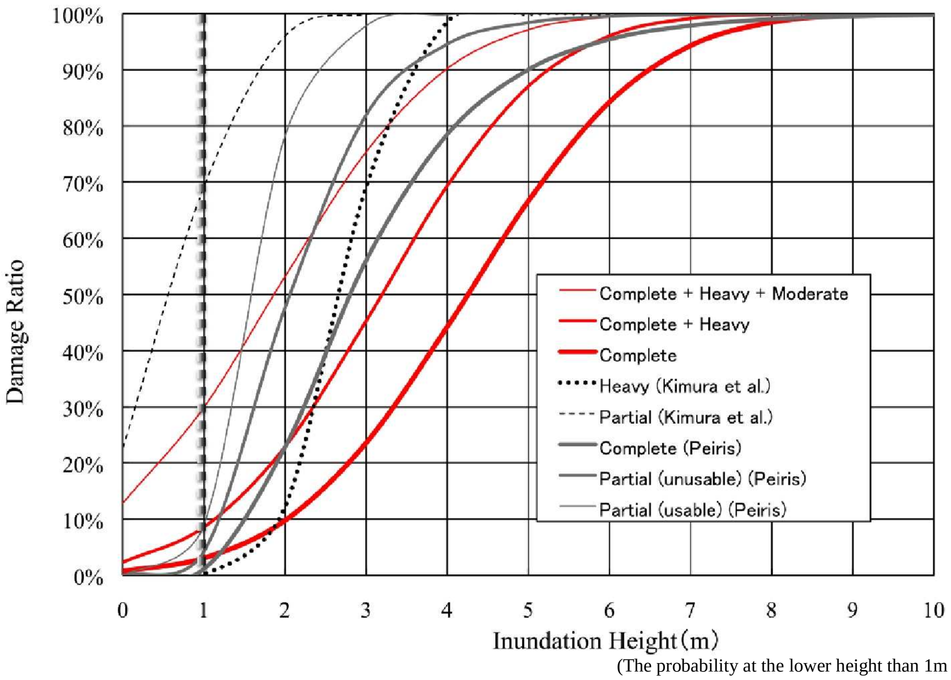
Although the research comprehensively covers the damage in whole Sri Lanka, there is a problem to represent the actual relationship between building damage and inundation height with the dataset. One reason is that the used value is DS unit data, which only shows several representative values for one DS district. Another one is that the data does not contain non-damage building data. It means that the denominator for the damage ratio consists of only damaged buildings, so the ratio at one inundation height value tends to be higher than actual condition.

### *4.2 The vulnerability functions developed by Kimura et al. (2006)*

Kimura et al. (2006) obtained the dataset of buildings by questionnaire survey in Matara and calculated inundation data by numerical simulation. The concept of approximation is same as ours as shown in Sec. 3.6. However, it cannot demonstrate the actual damage incurred in higher inundation areas such as Hambantota, since the maximum inundation height in their research is 4.1m.

### *4.3 Comparison of the fragility curves*

The fragility curves for all buildings developed by the authors are compared with other two-research based curves in Fig. 3. As mentioned in the above subsections, the curves by the other researchers show higher damage ratio than ours: for example, the curve for "heavy damage" by Kimura et al. reaches to 100% damage at 4m, and the curve for "complete damage" by Peiris demonstrates 80% at the same height, where our curve for "complete damage shows 45%.




(The probability at the lower height than 1m is unreliable.)

*Fig. 3 Comparison of fragility curves for buildings in Sri Lanka*

We saw some non-collapsed buildings in 4m estimated inundation areas. In order to solve the problems pointed out in the above subsections, our approach in the survey was to collect data of various building condition and inundation by ourselves along the 7km coastline areas in five districts with different characteristics, including slight damaged blocks or atrocious areas, in Galle, Matara, and Hambantota. The fragility curves in our research appropriately demonstrate the building vulnerability based on the surveys.

## 5. Conclusion

This paper presented how authors obtained building damage and inundation data in Galle, Matara, and Hambantota in Sri Lanka after the 2004 Indian Ocean Tsunami and how the vulnerability functions were developed based on the dataset. Also, the developed fragility curves were compared with those by other two research groups focused on the data and methods.

The developed functions in this research have two advantages for damage estimation over the previous ones. The first one is adequacy. The dataset acquired by the authors' elaborate surveys covered various districts in the country ranging from slightly damaged area to seriously damaged area. The surveys were conducted in order to solve the problems in the previous research, so the developed vulnerability functions represent the actual damage more adequately than the others. The second one is structural classification. The authors investigated the damage condition considering structural types. As a result, the vulnerability functions for non-solid buildings (brick-built, block-built, or wooden) and solid buildings (reinforced concrete, or steel) were developed as well as for all buildings.

Those vulnerability functions, which were built on the catastrophic experience in Sri Lanka, can significantly contribute to building damage estimation for future urban safety. As Nakazato and Murao (2007) pointed out, several regional problems such as the building regulation in the Coastal Conservation Zone appeared in the post-tsunami recovery process in the country. According to regional situation and usable data, the vulnerability functions can be used for tsunami damage estimation and proper tsunami management toward the future sustainable society.

**Acknowledgements**

This paper is supported by "Restoration Program from Giant Earthquakes and Tsunamis, granted by Ministry of Education, Culture, Sports, Science and Technology, Japan. The authors are grateful for the assistance of Sri Lankan Governments, Mr. Nihal Rpasinghe at Central Engineering Consultancy Bureau, Dr. Srikantha Herath at United Nations University, and Navindra De Silva as a guide and translator.

**References**

Department of Census and Statistics, Sri Lanka (2005a), "Census of persons, housing units and other buildings affected by the tsunami, 26th December 2004".

Department of Census and Statistics, Sri Lanka (2005b), "Reports on Census of Buildings and People Affected by the Tsunami - 2004".

Hatori, T. (1984), "On the damage to houses due to tsunamis", *Bull. Earthq. Res. Inst.*, Vol.59: 433-439 (in Japanese).

Kimura, K., Ishimi, K., Sato, S., Fukuda, T., Mikami, T., and Kikuchi, S. (2006), "Verification of tsunami wave source model and formulation of a vulnerability function for buildings based on the investigation of actual damage due to the Sumatran earthquake and tsunami: subject area being the damage at Matara city, Sri Lanka", *Coastal Engineering Committee*, Vol.53: 301-305 (in Japanese).

Koshimura, S., Oie, T., Yanagisawa, H., and Imamura, F. (2009), "Developing fragility functions for tsunami damage estimation using numerical model and post-tsunami data from Banda Aceh, Indonesia", *Coastal Engineering Journal*, Vol.51, No.3: 243–273.

Murao, O., and Nakazato, H. (2010), "Development of fragility curves for buildings based on damage survey data in Sri Lanka after the 2004 Indian Ocean Tsunami", *Journal of Structural and Construction Engineering*, Vol.75, No.651: 1021-1027 (in Japanese).

Nakazato, H. and Murao, O. (2007), "Study on regional differences in permanent housing reconstruction process in Sri Lanka after the 2004 Indian Ocean Tsunami", *Journal of Natural Disaster Science*, Vol.29, No.2: 63-71.

Peiris, N. (2006), "Vulnerability functions for tsunami loss estimation",

*Proceedings of 1st European Conference on Earthquake Engineering and Seismology*, No.1121.

Shuto, N. (1993), "Tsunami intensity and disasters", *Tsunami in the World*, Kluwer Academic Publishers: 197–216.

# SETTING UP OF INDICES TO MEASURE VULNERABILITY OF STRUCTURES DURING A FLOOD

H.K. Nandalal[a], U.R Ratnayake[b]

[a]Department of Civil Engineering, University of Peradeniya, Sri Lanka
hemalin@pdn.ac.lk, +94 81 2393532, +94 77 5548469, fax +94 81 2393500

[b]Department of Civil Engineering, University of Peradeniya, Sri Lanka
udithar@pdn.ac.lk +94 81 2393561, +94 77 5548469, fax +94 81 2393500

**Abstract**
Computation of flood extents and identification of vulnerable elements help to take mitigatory measures effectively and efficiently during a disaster. This is a baseline study carried out to examine how effectively the vulnerability of a small administrative division that depend on the type and facilities of existing structures can be assessed. In this study the flood extent of upper reaches of Kalu-Ganga River in Sri Lanka for a rainfall of 100 year return period was derived using HEC-HMS and HEC-RAS software. Building vulnerability within the administrative divisions affected by floods were derived using census data. Subsequently, flood risk maps with respect to structures due to 100 year return period rainfall were developed based on vulnerability derived from census data and the developed flood hazard maps. This approach could be used to identify high risk administrative divisions during a disastrous flood event to organize relief aids. It further helps in taking post disaster mitigatory measures according to the vulnerability levels of the buildings in each administrative division. Identification of crucial factors that influence vulnerability helps to reduce vulnerability and to mitigate the risks involved in future flood events.

***Keywords:*** *structural vulnerability; flood vulnerability; flood hazard; flood risk maps; flood modeling;*

## 1. Introduction

Most disasters are characterized by short reaction/response times, and present a significant strain on the resources of the affected communities. Therefore, responding effectively to post-disaster events requires a rapid and coordinated response to save lives and enables communities to get back on their feet. Most often, this response is predicated on access to good information (Lembo et al., 2008). Recent studies have indicated that the need for information in post-disaster response for first responders, command and control managers, public information managers, and eventually, recovery workers (Gunes & Kovel, 2000).

The concept of vulnerability has been a powerful analytical tool for describing states of susceptibility to harm, powerlessness, and marginality of both physical and social systems, and for guiding normative analysis of actions to enhance well-being through reduction of risk (Adger, 2006). Knowingly or unknowingly new houses are built in areas at risk from flooding making more people are being placed at risk by moving into these ‘unsafe’ areas.

Before any study on the social distribution of risk can be made, the areas at risk need to be defined. In this research the flood plain maps for 100 year return for flood was used to identify area at risk.

Kalu-Ganga river basin in Sri Lanka was selected as the study area since flooding in this river basin is the most frequent natural hazard in the country. Due to its geographical location the Kalu-Ganga river basin receives a large amount of rainfall during monsoon seasons. This causes flooding almost every year in the Kalutara and Ratnapura districts. As stated by (Churchill & Hutchinson, 1984) relief and aid programmes have been essentially the only collective adjustment to flooding in Sri Lanka, which is still in practice. Increase in human population and migration to cities has made the flood risk management an inevitable step that should be taken to reduce the vulnerability of human population to frequently occurring floods in Sri Lanka.

## 2. Materials and Method

### *2.1 Study Area*

Kalu-Ganga river catchment covers 2658 $km^2$ and is dominant by forest, residential and agricultural cropland land use types. It experiences an average annual rainfall of 4000 mm, which varies from

2800 mm in lower reaches to 5300 mm in higher elevations. Geographically the catchment lies between 6.32°N and 6.90°N latitude and 79.90°E and 80.75°E longitude as per WGS84 coordinate system and the river flows from a height of about 2,250 m MSL (Figure 1).

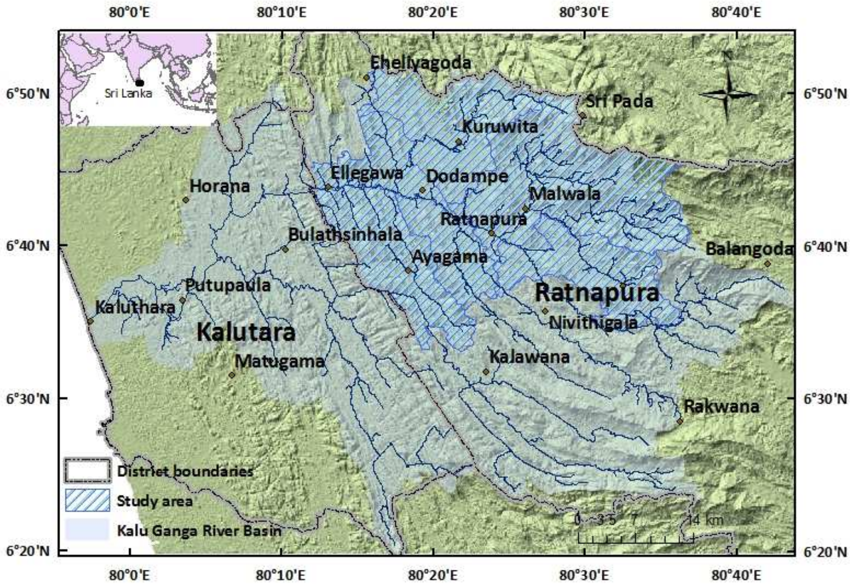



*Figure 7. The Kalu-Ganga catchment and the area under investigation*

The Kalu-Ganga river passes through two administrative districts, Ratnapura and Kalutara, as shown in Figure 2. This paper focuses on analyzing floods in upper reaches of the Kalu-Ganga river in the Ratnapura District. In Sri Lanka administrative divisions within a district are named as Divisional Secretariat (DS) divisions while as the lowest administrative division a DS division is divided into several Grama Niladhari (GN) divisions. In the study six DS divisions in the Ratnapura district namely, Kiriella, Kuruwita, Ratnapura, Pelmadulla, Elapatha and Ayagama were considered and the GN divisions were taken for unit-wise risk assessment.

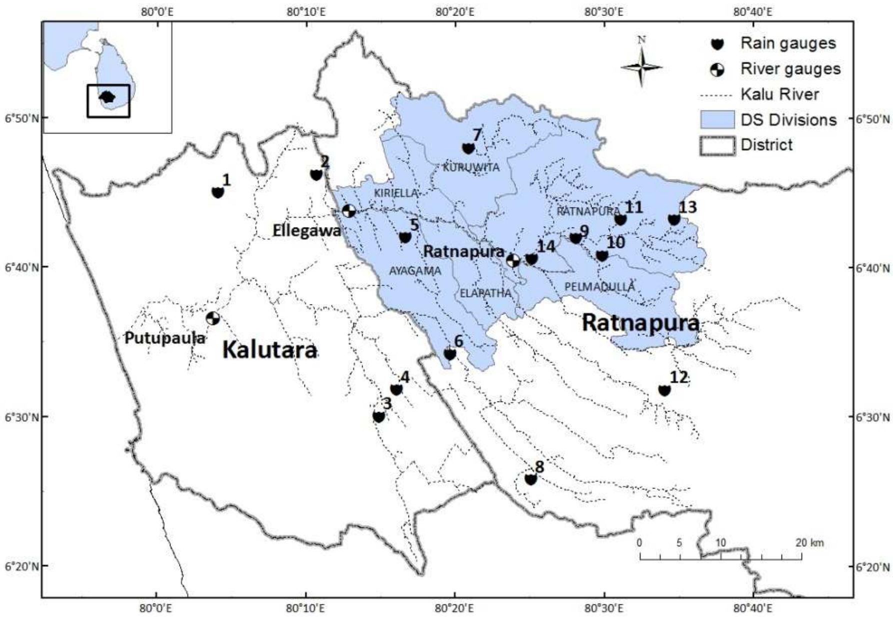



*Figure 8. Hydrological gauging stations*

## *2.2 Methodology*

Risk-oriented methods and risk analyses are gaining more and more attention in the fields of flood design and flood risk management since they allow us to evaluate the cost effectiveness of mitigation measures and thus to optimize investments. The most common approach to define flood risk is the definition of risk as the product of hazard, i.e. the physical and statistical aspects of the actual flooding (e.g. return period of the flood, extent and depth of inundation), and the vulnerability, i.e. the exposure of people and assets to floods and the susceptibility of the elements at risk to suffer from flood damage (Apel et al., 2009).

Hazard analyses give an estimation of the extent and intensity of flood scenarios and associate an exceedance probability to it. The usual procedure is to apply a flood/rainfall frequency analysis to a given record of discharge/rainfall data and to transform the discharge associated to defined return periods, e.g. the 100-year event into inundation extent and depths. Vulnerability analyses are normally restricted to the estimation of detrimental effects caused by the floodwater like fatalities, business interruption or financial/economic losses.

To determine the peak discharge due to a rainfall of 100 year return period Hydrologic Engineering Center's Hydrologic Modelling System (HEC-HMS) developed by US Army corps of Engineers (USACE, 2009a) was used. To obtain the flood extent HEC-GeoRAS (USACE, 2009b) and Hydrologic Engineering Center's River Analysis System (HEC-RAS) (USACE, 2010) software were used.

Cutter et al. (2003) found that eleven composite factors that differentiated U.S. counties according to their relative level of social vulnerability. Out of them they found that eleven percent of the variation in counties was captured by density of the built environment. Dominey-Howes & Papathoma (2006) identified 'attributes' (indicators) that are reported to affect the degree of damage from, or protection to, tsunami flooding for individual buildings and structures. (Dall'Osso & Dominey-Howes, 2009) had selected seven factors to assess the vulnerability of buildings to a tsunami flood namely; number of stories, building material and technique of construction, ground floor hydrodynamics, foundations, shape and orientation of the building footprint, movable objects and preservation condition.

Flood water can damage residential property in at least four ways: building materials and contents are damaged by immersion; mud, sediments and other contaminants in the flood water can cause corrosion or other decay; Dampness promotes the growth of mildew; the physical force of the water and objects swept along in the flow may damage the building structure. While the depth of overfloor inundation is usually seen as the most important control on residential damage, other factors may also be important – for example, duration of inundation, sediment content, water velocity, building materials, interior construction, building age, content location, and warning time ( (N'Jai et al., 1990) as cited in (NHRC, 2000)).

Vulnerability of built in environment within the administrative divisions affected by floods were derived using census data. Due to the limited data availability building material used for construction of walls and floor and density of buildings per GN division were used to assess the vulnerability.

Subsequently, flood risk maps due to 100 year return period rainfall were developed based on vulnerability derived from census data and the developed flood hazard maps at GN division level based on built envioronment.

## 3. Theory/calculation

### *3.1 Flood-hazard assessment*

Flood hazard assessment is the estimation of overall adverse effects of flooding for a particular area. It depends on many parameters such as depth of flooding, duration of flooding, flood wave velocity and rate of rise of water level. One or more parameters can be considered in the hazard assessment depending on the characteristics of study area and floods (Tingsanchali and Karim, 2005). Considering the characteristics of the study area, two major parameters, namely depth of flooding and percentage area of flooding, were considered for the assessment of hazard of land units considered.

A hazard index, HI, was introduced to represent degree of hazard corresponding to different flood depths. As recommended in past studies (e.g. Chowdhury and Karim, 1997), four hazard categories were used and each category was represented by a hazard index. To devise a scale for HI, flooding areas were divided into four depth categories based on three critical flood depths 0.6, 1.0 and 3.5 m. Based on these three critical values of flood depth (D), hazards were classified as low ($D \leq 0.6$ m), medium (0.6 m < D ≤ 1.0 m), high (1.0 m < D ≤ 3.50 m) and very high (3.50 m < D) as presented in Table 2.

Table 3. *Hazard index for depth of flooding*

| Depth (D) of flooding (m) | Hazard category | Hazard Index (HI) |
|---|---|---|
| 0 < D ≤ 0.6 | Low | 1 |
| 0.6 < D ≤ 1.0 | Medium | 2 |
| 1.0 < D ≤ 3.5 | High | 3 |
| 3.5 < D | Very High | 4 |

For the practical application of predicted results, the hazard was estimated for a land unit and represented by a number, hazard factor *(HF)*. The *HF* for a land unit was taken to represent hazards due to flood depth ($HF_D$) and inundated area ($HF_A$). A land unit here is the lowest administrative division, while a piece of land of 200 m × 200 m is the computational grid cell. The DEM of 200 m × 200 m grid cells was generated from the 1:10000 contour maps (contour interval of 5 m). A land unit is several times larger than a computational grid cell and therefore, flooding of more than one depth category may occur in the same land unit. Exposure of a land unit to the flood hazard is taken as 1 assuming that all land units are equally exposed to it.

The hazard factor $HF_D$ for flood depth of each land unit was computed based on the fraction area under each depth category and the corresponding hazard index using equation 1.

$$HF_D(i) = \frac{\left(\sum_{j=1}^{N_d} A(i,j) HI_i(j)\right)}{\sum_{j=1}^{N_d} A(i,j)} \qquad (1)$$

Where, $i$ is the land unit identification number and j represents the depth category; $N_d$ is the total number of depth categories; $HI_i(j)$ is the hazard index for the area under depth category $j$ in land unit $i$ and $A(i,j)$ is the area under depth category $j$ in land unit $i$.

The hazard factor $HF_A$ for flood area of each land unit was computed as the percentage area under flood irrespective of depth using equation 2.

$$HF_A(i) = \frac{Arae\ under\ flood\ in\ land\ unit\ i}{Total\ area\ of\ land unit\ i} \times 100 \qquad (2)$$

As each of the above hazard factors were measured on a different scale, they were standardized as an index using equation 3.

$$HF_K^s(i) = \frac{HF_K(i)}{(HF_K)_{max}}; \quad (K = D\ or\ A) \qquad (3)$$

Where, $HF_K^s(i)$ is the standardized hazard factor of the land unit, $HF_K$ ($i$) is the original hazard factor for land unit and $(HF_K)_{max}$ is the maximum *HF* in the range. Finally giving the same weight for both of these factors, average value was taken as the hazard factor of the land unit as in equation 4.

$$HF(i) = (HF_D^s(i) + HF_A^s(i))/2 \qquad (4)$$

### *3.2 Flood-vulnerability assessment*

Vulnerability is a measure of the intrinsic susceptibility of an element at risk exposed to potentially damaging natural phenomena. The vulnerability is expressed on a scale from 0 (no damage) to 1 (total damage). The vulnerability factor (*VF*) of each land unit was assessed using the parameters, type of material used for floor and walls and density of the buildings per land unit. Thus the vulnerability indices with respect to type of material used for floor ($VI_F$), walls ($VI_W$) and density of the buildings ($VI_D$) are;

Vulnerability indices of land units (GN divisions) were calculated using the census data. Vulnerability rankings were assigned to each building category based on the material type used for construction of walls and floor as illustrates in Table 2 and 3, respectively.

Table 4. *Assignment of ranks according to construction materials used in walls*

| *Category* | Construction Material of walls | Ranking [$R(i)$] |
|---|---|---|
| 1 | Brick | 1 |
| 2 | Kabok | 3 |
| 3 | Cement Blocks/ Stones | 2 |
| 4 | Pressed Soil Blocks | 4 |
| 5 | Mud | 6 |
| 6 | Cadjan / Palmyrah | 7 |
| 7 | Planks/Metal Sheets | 5 |

Table 5. *Assignment of ranks according to construction materials used for floor*

| *Category* | Construction Material of floor | Ranking [$R(i)$] |
|---|---|---|
| 1 | Cement/Tiles/Terrazzo | 1 |
| 2 | Clay/Wood/Sand | 2 |

Therefore, vulnerability index of a land unit *j* with respect to construction materials of wall of a housing unit ($VI_W(j)$) was calculated as;

$$VI_W(j) = \sum_{i=1}^{n} F_W(i) R_W(i)/n \tag{5}$$

Where, $F_W(i)$ is the fraction of the buildings of a category from the total number of houses in the land unit *i* and $R_W(i)$ is the ranking assign to that vulnerability category based on construction material used for the walls.

Vulnerability index of a land unit *j* with respect to construction materials of floor of a housing unit ($VI_F(j)$) was calculated as;

$$VI_F(j) = \sum_{i=1}^{n} F_F(i) R_F(i)/n \tag{6}$$

Where, $F_F(i)$ is the fraction of the buildings of a category from the total number of houses in the land unit *i* and $R_F(i)$ is the ranking assign to that vulnerability category based on construction material used for the floor.

Vulnerability index of a land unit *j* with respect to density of housing units ($VI_D(j)$) was calculated as;

$$VI_D(j) = \frac{Total\ number of\ housing\ units\ in\ the\ land\ unit\ j}{Area\ of\ the\ land\ unit\ j} \tag{7}$$

However, scale of this $VI_D$ differs from above two vulnerability indexes, therefore this vulnerability index was standardised using the flowing equation for summing up the vulnerability of buildings of a particular land unit.

$$VI_{DS}(j) = \frac{(VI_D(j)) - Min(VI_D(j))}{Max(VI_D(j)) - Min(VI_D(j))} \tag{8}$$

Finally, vulnerability factor of the land unit *j (VF(j))* with respect to built environment was calculated using;

$$VF(j) = \frac{VI_{DS}(j) + (VI_W(j) + VI_F(j))/2}{2} \tag{9}$$

### *3.3 Flood Risk Assessment*

In general, risk as a concept that incorporates the concepts of hazard and vulnerability. It is customary to express risk (R) as a functional relationship of hazard and vulnerability. The magnitude of risk for a land unit was estimated by a risk factor, *RF*(*i*), which was computed as the product of the hazard factor and the vulnerability factor as in equation 10.

$$RF(j) = HF(j) \times VF(j) \tag{10}$$

## 4. Results

### *4.1 Flood Flow Simulation*

A frequency analysis was performed using the Gumbel distribution for all fourteen rainfall gauging stations. Average value of the rainfall with 100 year return period was selected as the rainfall to be used in the generation of river flows. This rainfall was used in the calibrated HEC-HMS based model for the basin to generate river discharges that are required for the hydrodynamic model to obtain flood. Using the HEC-RAS based model the inundation extents with corresponding depths for a flood expected to occur due to a rainfall of return period 100 years were developed. Figure 3 depicts the inundation area.

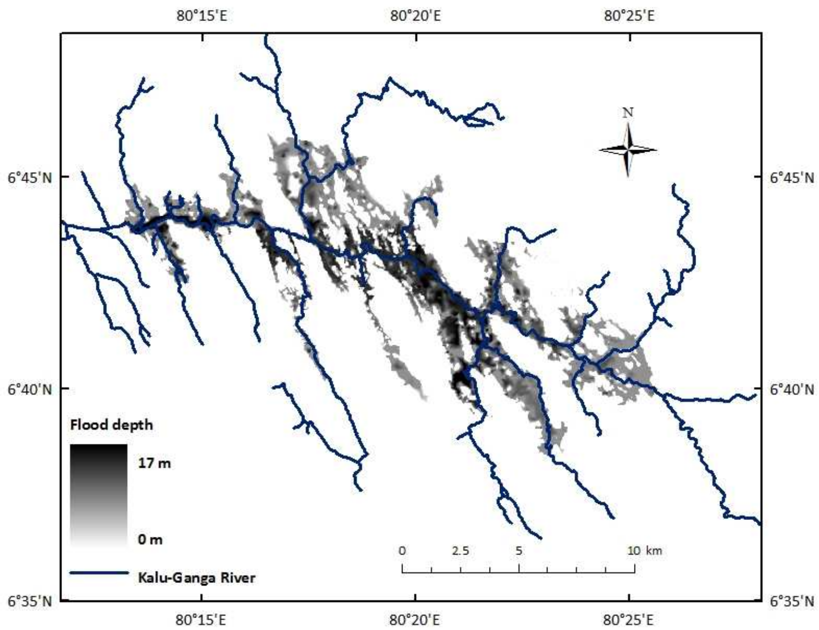



Figure 9. *Flood Inundation area along the Kalu-Ganga river*

### *4.2 Risk Assessment*

Risk due to flooding was computed at GN division level. The inundation areas and depths obtained from the hydrodynamic model provide exposure that needed for the calculation of risk.

Sixty seven GN divisions were identified within the 100 year return period floodplain area as shown in Figures 3. The numbers appear in the figure are the identification numbers given for the GN divisions within the Ratnapura district by the administration. These GN divisions were categorized into five risk zones based on scores of five equal intervals. The risk areas were named as very low-risk zone for $0.001 < RF \leq 0.1$, low risk zone for $0.1 < RF \leq 0.2$, medium risk zone for $0.2 < RF \leq 0.3$, high risk zone for $0.3 < RF \leq 0.4$ and very high risk zone for $0.4 < RF \leq 1.0$. According to this analysis 37 GN divisions were rated as very low risk, 21 GN divisions as low risk, 6 GN divisions as medium risk, 2 GN divisions as high risk and 1 GN division (Ratnapura, No.284) as very high risk divisions . Figure 4 presents the spatial distribution of risk levels of the sixty seven GN divisions.

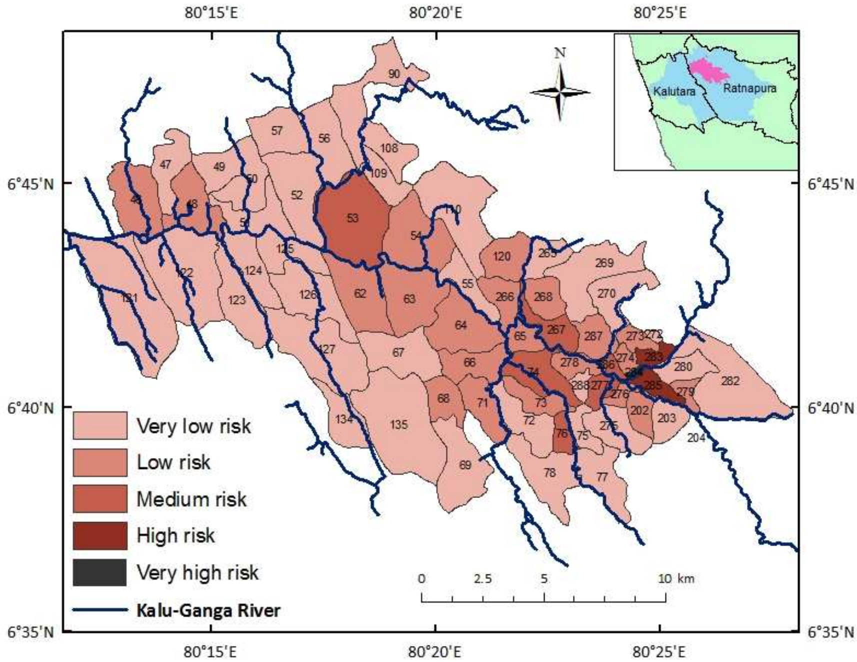



Figure 10. *Flood risk map based on built environment*

## 5. Discussion and conclusions

A flood risk analysis of a flood prone area immensely supports the decision makers to take correct decision at the right time. It helps in all phases of a flood related disaster i.e., pre, post, and during the disaster. In Sri Lanka, during a disaster, all relief activities are carried out always at GN division level and therefore, flood risk analysis taking the land units as GN divisions is the most suitable. The HEC-HMS and HEC-RAS based models were found to be very suitable in the derivation of flood inundation area in the Kalu-Ganga river basin.

Among the different hydraulic parameters that determine hazard due to a flood, the flood depth and inundation are found to be two major parameters that can be used to determine hazard level sufficiently.

Vulnerability of an area always depends upon the population of that area and also the type of built environment of the area. There are many studies related to the study of damages due floods on structures. However, very little emphasis has been given for the studies related to the flood risk based on built environment. Vulnerability factors based on type of material used for floor and walls and density of the buildings per land unit, were found to be effective in the determination of risk in the area.

Flood risk analysis conducted based on census data found to be effective in taking decisions during pre and post disaster due to floods. However, if GIS data on built environment were available enabling explicit consideration of building vulnerability, mitigating measures to be taken with regard to built environment such as flood proof techniques to buildings would be very easy.

GIS data of buildings (building foot prints) including all necessary secondary data is not available in Sri Lanka at present. Building such a data base would facilitate disaster mitigation immensely.

### Acknowledgements

Authors acknowledge the University Grant Commission of Sri Lanka for providing necessary funds to carry out this research.

### References

Adger, W.N., 2006. Vulnerability. *Global Environmental Change*, 16(3), p.268–281.

Apel, H., Aronica, G.T., Kreibich, H. & Thieken, A.H., 2009. Flood risk analyses—how detailed do we need to be? *Natural Hazards*, 49(1), pp.79-98.

Chowdhury, J.U. & Karim, M.F., 1997. *A risk based zoning of storm surge prone area of the Ganges tidal plain*. UNCRD Proceeding Series, 17(2). Nagoya, Japan: United Nations Centre for Regional Development.

Churchill, R.R. & Hutchinson, D.M., 1984. Flood hazard in Ratnapura, Sri Lanka: Individual attitudes vs collective action. *Geoforum, 15(4), pp.517-524.*, 15(4), pp.517-24.

Cutter, S., Boruff, B. & Shirley, W.L., 2003. Social venerability to environmental hazards. *Social Science Quarterly*, 84(3), pp.242-61.

Dall'Osso, F. & Dominey-Howes, D., 2009. *A method for assessing the vulnerability of buildings to Catastrophic (tsunami) marine flooding*. Report prepared for the Sydney Coastal Councils Group Inc. Sydney: Unpublished report University of New South Wales.

Dominey-Howes, D. & Papathoma, M., 2006. Assessment Model (the. *Validating a Tsunami Vulnerability Assessment Model (the PTVA Model) Using Field Data from the 2004 Indian Ocean Tsunami*, 40(1), pp.113-36.

Fielding, J. & Burningham, K., 2005. Environmental inequality and flood hazard. *Local Environment*, 10(4), p.379 -395.

Gunes, A.E. & Kovel, J.P., 2000. Using GIS in emergency management operations. *Journal of Urban Planning and Development*, 126(3), pp.126-49.

Lembo, A.J., Bonneau, A. & O'Rourke, T.D., 2008. Integrative Technologies in Support of GIS-Based Postdisaster Response. *Natural Hazards Review*, 9(2), pp.61-69.

N'Jai, A. et al., 1990. *Flood Loss Assessment Information Report*. Middlesex: Flood Hazard Research Centre Middlesex Polytechnic.

NHRC, 2000. Whats the damage. *Natural Hazards Quardterly*, 6(3), p.1.

Tingsanchali, T. & Karim, M.F., 2005. Flood hazard and risk analysis in the southwest region of Bangladesh. *Hydrological Processes*, 19(10), pp.2055-69.

UNDP, 2004. *Reducing disaster risk. A challenge for development.* A Global Report. New York: UNDP Bureau for Crisis Prevention and Recovery.

USACE, 2009a. *Hydrological Modeling System HEC-HMS'' User's manual*. 34th ed. Davis, California: Hydrologic Engineering Center.

USACE, 2009b. *"HEC-GeoRAS: GIS tools for support of HEC-RAS using ArcGIS'' User's manual*. 42nd ed. Davis, California: Hydrologic Engineering Center.

USACE, 2010. *"HEC-RAS River Analysis System.'' User's manual*. 41st ed. Davis, California: Hydrologic Engineering Center.

# IDENTIFY DAMAGED BUILDINGS FROM HIGH-RESOLUTION SATELLITE IMAGERY IN HAZARD AREA USING DIFFERENTIAL MORPHOLOGICAL PROFILE

C. D. K. Parape[1], M. Tamura[2]

[1]Doctor Student, Department of Urban and Environmental Engineering,
Graduate School of Engineering, Kyoto University, Japan
[1]E-mail: dinesh.c@ky4.ecs.kyoto-u.ac.jp ; dineshchandana@gmail.com
[1]Telephone: +94-11-2650567-8 (Ext.2006 - 2001); Fax: + 94-11-2651216
[2]Professor, Graduate School of Global Environmental Studies,
Kyoto University, Japan

**Abstract:** This paper presets a methodology and results of evaluating damaged building detection algorithms using an object recognition task based on Differential Morphological Profile (DMP) for Very High Resolution (VHR) remotely sensed images. The proposed approach involves several advanced morphological operators among which an adaptive hit-or-miss transform with varying size, shape and gray level of the structuring elements. IKONOS Satellite panchromatic images consisting of pre and post earthquake site of Sichuan area in China were used. Morphological operation of opening and closing with constructions are applied for segmented images. Unsupervised classification ISODATA algorithm is used for the feature extraction and the results comparison with ground truth data, complex urban area before the earthquake gives 76% and same area wracked after the earthquake gives 88% buildings detection on object based accuracy. This work is being extended to extract shadows and non building objects for better classifications of building roof footprints.

**Keywords:** Differential Morphological Profile, IKONOS, Building Extraction

## 1. Introduction

The very high spatial resolution satellite images offer the opportunity to recognize features such as road, vegetation, buildings and other kind of infrastructures. In this paper, we focus on automatic damaged building detection method which is helpful to optimize recognize, rescue, management and recover tasks when an event of hazard. In the past decade, many kinds of methods have developed especially to classification and feature extraction using high-resolution imagery. According to difference manners, these methods can be summarized in to difference kinds: automatic and semi-automatic according to the iteration extent of human; single view and multi views, according to difference principles; region-based and edge-based according to the principle elements acquired manners. Among these methods, mathematical morphology has already proved to be effective for many applications in remote sensing [1] - [9]. Classification and Feature Extraction for Remote Sensing Images From urban Area Based on Morphological Transformations and Classification of Hyper spectral Data From Urban Areas Based on Extended Morphological Profiles were presented by Benediktsson et al [10]. Similarly, Aaron K. Shackelford et al were investigated a method for Automated 2-D Building Footprint Extraction from High-Resolution Satellite Multispectral Imagery [11]. This research also focused on region based classifications.

## 2. Objectives and Methodology

### *2.1 Differential Morphological Profile*

The Here Differential Morphological Profile (DMP) was developed by feature detectors attempt to identify buildings, shadows, roads and so on of the high-resolution panchromatic images and it is constructed using morphological opening and closing by reconstruction operators. Mathematical morphology employs a set of image operators to extract and analyze image components based on shape and size of quasi-homogeneous regions in the image. This concept is used to create a feature vector from a single image, *I* and it is based on the repeat use of the opening and closing operators, which are commonly used in mathematical morphology [12]. Opening and closing by reconstruction is obtained following by erosion and dilation under the original image [10]. The gray-scale reconstruction $\rho^{*f(p)}$ of image *I* could be defined as follows. Opening $\gamma$ is defined as the result of erosion followed by the dilation.

$$\gamma^*_{\mathrm{N}}\ f(p) = \rho^{*f(p)}\ (\varepsilon_N\, f(p)) = \mathrm{Re}\, c(\varepsilon_N\, f, f) \qquad (1)$$

In a similar fashion, closing f by reconstruction can be defined as

$$\varphi^*_{\mathrm{N}}\ f(p) = \rho^{*f(p)}\ (\delta_N\, f(p)) = \mathrm{Re}\, c(\delta_N\, f, f) \qquad (2)$$

Here in the Euclidean transforms assume that flat structuring element that corresponds to the neighborhood SE= $N_G(p)$.The erosion $\varepsilon_N$ of the grey level function using the structuring element *N* is defined by the infimum of the values of the grey level function in the neighborhood

$$\varepsilon_N\, f(p) = \{\wedge f(p^{/}) \big| p^{/}) \in N_G(p) \cup f(p)\} \qquad (3)$$

And the Dilation $\delta_N$ is similarly defined by the supremum of the neighboring values and the value of as

$$\delta_N\, f(p) = \{\vee f(p^{/}) \big| p^{/}) \in N_G(p) \cup f(p)\} \qquad (4)$$

Opening and closing by reconstruction can be considered as lower-leveling opening and upper-leveling closing operations [13]. The idea of the multi-scale segmentation based on the derivative of the morphological profile was developed as

Let γ*λ be a morphological *opening operator by reconstruction* using structuring element SE = λ and Πγ(x) be the *opening profile* at the pixel x of the image *I* . Πγ(x) is defined as a vector

Πγ(x) = { Πγλ : Πγλ = γ*λ (x), ∀λ v [0,n]}  (5)

Also, let f*λ be a morphological *closing operator by reconstruction* using structuring element SE = λ . Then, the *closing profile* Πfλ at pixel x of the image is defined as the vector

Πf(x) = { Πfλ : Πfλ = f*λ (x), ∀λ v [0,n]}  (6)

In the above, Πγ0(x) = Πf0(x) = *I*(x) for λ=0 by the by the definition of opening and closing by reconstruction [3]. Given (1) and (2), the opening profile can also be defined as a granulometry [1] made with opening by reconstruction, while the closing profile can be defined as antigranulometry made with closing by dual reconstruction. The derivative of the morphological profile is defined as a vector where the measure of the slope of the opening-closing profile is stored for every step of an increasing SE series. The *derivative of the opening profile* Δγ(x) is defined as the vector

Δγ(x) ={ Δγλ : Δγλ = |Πγλ – Πγλ-1 |, ∀λ v [1,n]}  (7)

By duality, the *derivative of the closing profile* Δf(x) is the vector

Δf(x) ={ Δfλ : Δfλ = |Πfλ– Πfλ-1 |, ∀λ v [1,n]}  (8)

Generally, the *derivative of the morphological profile* $\Delta(x)$or the DMP can be written as the vector

$$\Delta(x) = \begin{cases} \Delta c : \Delta c = \Delta f \;\; \lambda = n-c+1, \; \forall c \vee \;\; [1,n] \\ \Delta c : \Delta c = \Delta \gamma \lambda = c-n, \; \forall c \vee \;\; [n+1,2n] \end{cases} \qquad (9)$$

with equal to the total number of iterations, c=1,…..,2n, and , and |n-c|=the size of the morphological transform.

***2.2. Used Data***

The morphological filter theory was designed for a series of gray-level images. In this paper, we used pre and past IKONOS panchromatic (PAN) imagery of Sichuan earthquake in China in 2008. The gray color image of high-resolution IKONOS panchromatic images that consist of 1m resolution band (450-900 nm) is used. The images were smoothed using median convolutions filter for removal post classification procedures like "salt-and-paper" and other visual enhancement procedures. IDL programming language and ENVI 4.7 commercial software package is used for image processing and classification on this research.

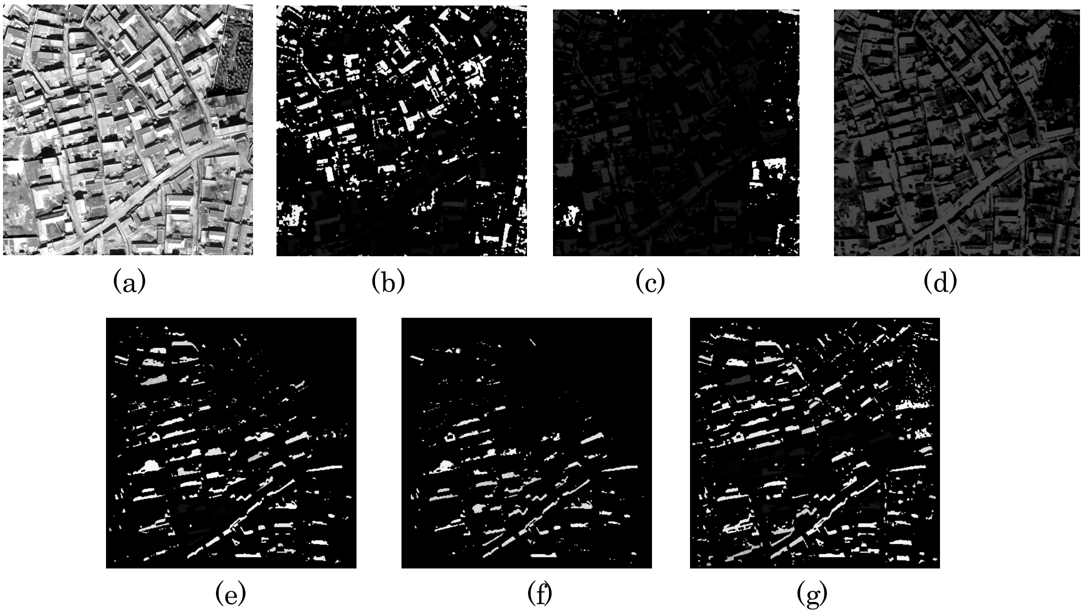



Figure 1. *The image (a) shows the original image before the earthquake event and (b)-(g) represents Structural decomposition of the image using differential morphological profile. The images have been visually enhanced. The derivative has been calculated relative to a series generated by six iterations of the elementary SE with radius from 7-19m. Derivative of the opening profile with r=(b)7, (c)11, (d)15 and closing profile with r=(e)7,(f)11,(g)15 are shows above respectively.*

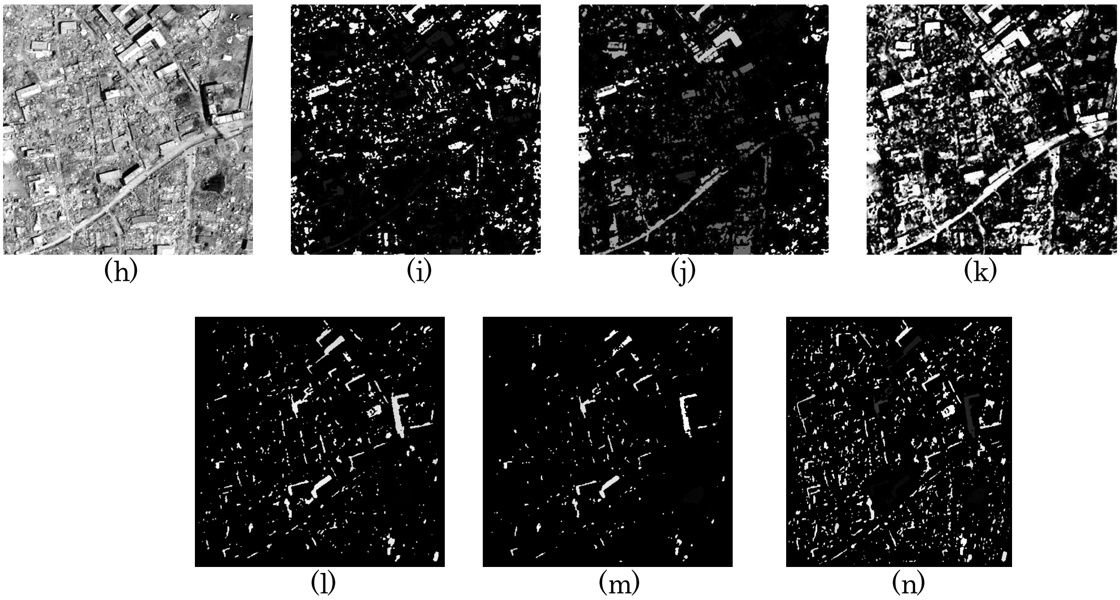



Figure 2. *The image (h) shows the original image after the earthquake event and (h)-(n) represents Structural decomposition of the image using differential morphological profile. The images have been visually enhanced. The derivative has been calculated relative to a series generated by six iterations of the elementary SE with r (radius) from 7-19m. Derivative of the opening profile with r=(i)7, (j)11, (k)15 and closing profile with r=(l)7,(m)11,(n)15 are shows above respectively.*

## 3. Building Extraction

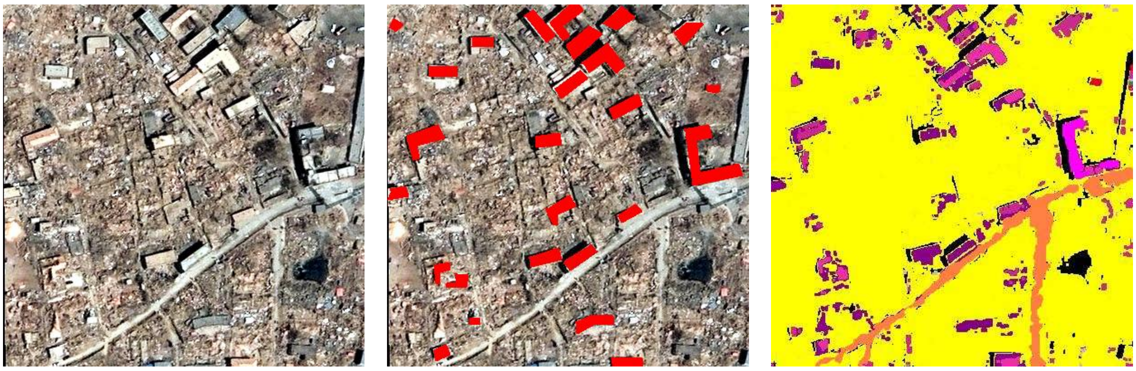
The building shadows are easy to extraction using their low reflection value. Musk for the shadows is built up using the reflection value between 0 and 60. The structures with similar scale to the SE diameter give high response when SE in DMP value with bright structures in opening portion and dark structures in closing portion of the profile. For each pixel in the image, the position of the maximum response within the DMP vector ($\Delta(x)$), indicate both the SE size that best characterize the structure that the pixel resides within and whether the pixel is part a structure that is brighter or darker than the surrounding region. The maximum DMP response indicates with well match SE value that the pixel resides within. There are 8 differential morphological profiles were created using disc shaped morphological elements with radius (r) increasing 7 to 19m (step size is equal to 4m). The SE that less than 7m are not reliable for use because of it consists of small shadows, trees and wracked of buildings. Those fingers give noise for the classification results, we used that SE more than 7 to detect for remain Buildings. Most of the bright building roof is gives the maximum response with opening differential profile and dark color roof, shadows are with closing differential profile. Unsupervised ISODATA Classification way is used for the classification and identification the structures. Combining morphological operations is carried out for remove pixel errors that occurred due to delineation of image objects with DMP before classification.

## 4. Results

The results have shown the usefulness of the proposed method during detection of various types of building, as illustrated by the portions given this paper. The image patches used to train the Unsupervised ISODATA classifier. The shadows of the building were masked when classification using their low spectral value. The candidate area contained various kinds of roofs with difference colors and shapes before the earthquake. There are some building structures that complex and combine together were classified as a one building. Although this result appeared to be high accuracy, the confidence measures produced by the ISODATA training suggested a reliability of post event gives 88.46%. The error extraction of building structure could be due to over fitting of the decision surface to the data. The totally collapsed building in the applied area is identified as 89 according to manually labeled buildings as ground truth and the result of the above algorithm views as 65.

(o) (p) (q)

Figure 3. *Building extraction results. (o) IKONOS image of the pre-earthquake area. (p) Manually labeled buildings as ground truth. (q) Result of the building extraction according to approached method.*



(r) (s) (t)

Figure 4. *Building extraction results. (r) IKONOS image of the post-earthquake area. (s) Manually labeled buildings as ground truth. (t) Result of the building extraction according to approached method.*

Table1. *Before the earthquake*

| | Object base | Pixel base |
|---|---|---|
| Correctly Extracted Buildings | 88 | 62572 (39.11%) |
| Total | 115 | 79240 (49.56%) |
| | 76.52% | Dismiss pixels 16668 |

Table2. *After the earthquake*

| | *Object base* | *Pixel base* |
|---|---|---|
| *Correctly Extracted Buildings* | *23* | *12886 (08.05%)* |
| *Total* | *26* | *7308 (04.56%)* |
| | *88.46%* | *Overlap pixels 5578* |

## 5. Conclusions

We applied a method for extraction of urban structures and hazard estimation using very high-resolution satellite images. The first step was to segmentation structural information using morphological opening and closing by reconstruction operators. IKONOS satellite panchromatic gray level images of pre and post earthquake event were applied to morphological operators. Then, the building footprint were extracted in candidate region using connected components analysis to the pixels selected according to their morphological profiles, obtained using increasing structural element sizes for 7 to 19m for opening and closing operators. From the analysis of the extraction results, complex urban area before the earthquake gives 76% and same area wracked after the earthquake gives 88% buildings detection on object based accuracy percentage according to the applied method. Further work is required to increase the accuracy of building detection and determine if damage ratio of the structure can be estimated.

### References

56. H. J. A. M. Heijmans and B. T. M. Roerdink, Eds. Dordrecht, The Netherlands: Kluwer, “From connected operators to levelings,” in Mathematical Morphology and Its Applications to Image and Signal Processing, 1998, pp. 191–198.
57. Gonzales, R. C. and R. E. Woods, 2002, “Digital Image Processing", 2nd ed., Upper Saddle River, NJ’ Prentice Hall, 2002.
58. P.Soille, “Morphological Image Analysis- Principle and Applications”, 2nd ed. Berlin, Germany’ Springer Verlag, 2003.
59. N.Jiang, J.X. Zang, H.T. Li, X.G.Lin , Semi-Automatic Building Extraction from High Resolution Imagery Based on Segmentation
60. Sébastien Lefèvre, Jonathan Weber, David Sheeren, Automatic Building Extraction in VHR Images Using Advanced Morphological Operators
61. I. Destival, “Mathematical morphology applied to remote sensing,” Acta Astronautica, vol. 13, no. 6/7, pp. 371–385, 1986.
62. F. Laporterie, G. Flouzat, and O. Amram, “Mathematical morphology multi-level analysis of trees patterns in savannas,” in IEEE International Geosciences And Remote Sensing Symposium, 2001, pp. 1496–1498.
63. P. Soille and M. Pesaresi, “Advances in mathematical morphology applied to geoscience and remote sensing,” IEEE Transactions on Geoscience and Remote Sensing, vol. 40, no. 9, pp. 2042–2055, September 2002.
64. S. Derivaux, S. Lefèvre, C. Wemmert, and J. Korczak, “Watershed segmentation of remotely sensed images based on a supervised fuzzy pixel classification,” in IEEE International Geosciences And Remote Sensing Symposium, Denver, USA, July 2006.
65. Jon Atli Benediktsson, , Martino Pesaresi, and Kolbeinn Arnason, “Classification and Feature Extraction for Remote Sensing Images From Urban Areas Based on Morphological Transformations”, IEEE TRANSACTIONS ON GEOSCIENCE AND REMOTE SENSING, VOL. 41, NO. 9, SEPTEMBER 2003
66. Aaron K. Shackelford, Curt H. Davis, IEEE International Geosciences And Remote Sensing Symposium, 2004, pp. 1996–1999.
67. Martino Pesaresi, J.A. Benediktsson, “A New Approach for the Morphological Segmentation of High-Resolution Satellite Imagery”, IEEE International Geosciences And Remote Sensing Symposium, Vol.39, No. 2, 2001, pp. 309–319.

### Acknowledgements

We would like to thank for access and imagery provided by GeoEye (space imagine) and Global Land Cover Facility (GLCF).

### About the Authors

**C. D. K. PARAPE,** B.Sc. Kobe University., M.Sc. Kyoto University, Ph.D. candidate in Environmental Informatics Laboratory, Faculty of Urban and Environmental Engineering, Kyoto University.

**M. TAMURA,** Professor, Graduate School of Global Environmental Studies,
Kyoto University, Japan.

# ADVANCEMENTS IN SUSTAINABLE HOUSING PRACTICES IN AUSTRALIA

Saman de Silva[1]

[1]Senior Lecturer, School of Civil, Environmental and Chemical Engineering, RMIT University, Melbourne Australia.
[1]E-mail: saman.desilva@rmit.edu.au
[1]Telephone: +61-3-99253235; Fax: + 61-11-2651216

**Abstract:** Australian housing industry practices, especially in the single and low to medium density housing development sectors are predominantly driven by the volume builders. Here the term, volume builders, refers to a practice of high volume of cloning. Individually designed owner builder house are very limited in Australia. Within such an industry environmentally unsustainable practice could multiply significantly at much faster rate. Australian living standard is resource intensive and the life style is high on the consumption of water and energy. For example living space, energy consumption, water consumption and embodied energy usage per capita are, relatively, at a much higher level. It is therefore envisaged that the positive influence on the sustainable practices can be affected efficiently through policy innovations and regulatory measures. Since early 2000, the topic of sustainable housing has been a very active area of discussion. Therefore, over the last ten years, notable progress was made in this sphere. Attention was drawn by the media through public debates and significant improvements have been observed in policy improvements, R&D activities and in engineering curricula. This paper presents the positive outcomes of current practices, policy innovations, research and education.

**Keywords:** Sustainable housing, Policy and practices, Engineering education

## Introduction

Housing industry is a culturally sensitive practice which has established over many generations. It is acknowledged that influencing such a practice to change its course requires a concerted and coordinated response from the stake holders. It also needs to have a well informed customer base and a sustained leadership.

The housing industry generates about 15% of the GDP in Australia. Australia is expected to have 1.1 million new houses built between 2002 and 2011[1]. Minnery [2] identifies the present pattern of Australian metropolitan development as low density housing; an observation supported by the Australian Bureau of Statistics [3] reporting that 73% of all Australian housing was comprised of single dwellings. Current domestic construction is predominantly site-built, timber-framed, brick-veneered construction with tiled roof. This method has been utilized to great effect and the industrial infrastructure in place is vast. Any attempted shift to a fundamentally different approach is often met with strong resistance, frequently due to questions of its viability.

The energy efficiency practices in the building industry introduced some ten years ago. However, this was primarily limited to condition of minimum insulation requirements, a deem-to-comply approach specified in the Building Code of Australia (BCA) [4]. The objective was to minimize the energy losses through building's envelope and roof in order to increase the efficiency of heating and cooling system. More quantifiable mandatory sustainability benchmarks were set out to regulate the industry towards an environmentally sustainable practice in 2005. As given in Table 1, although there are some disparities among the standards adopted in different states of Australia, an encouraging positive trend is clearly evident (Refer Table 1). Most commonly adopted mandatory requirements are the provision of rainwater tank per house hold and also achieving a minimum energy star rating. As can be seen from Table 1, the state of Victoria is providing leadership in forcing industry towards a more sustainable practice in which RMIT University has become a proactive stakeholder over the last ten years.

## Sustainable hosing practices in Australia, 2000 - 2010

Among developed nations, Australia has progressed well in sustainable housing sector during this period. The areas where progress has been achieved include a number of good examples of sustainable houses, development of rating tools to administer mandatory requirements, legislative and financial incentives, institutional capability development and sustainable technologies and products.

Sustainable housing practices in early 2000 were isolated rather than in the mainstream. There were example houses of best practice [5, 6]. Technical guidelines of best practices were also being developed and at their preliminary stages [7, 8]. Relatively higher cost of off-the-shelf products was a barrier to uptake of technologies, particularly solar PV panels, given the marketing potential was limited at the early stages, and no government subsidies were made available. However, public awareness, industry readiness and media discussion towards sustainable practices were on a high, which subsequently drove changes to policy and legislation. There are a number of good case studies of sustainable houses in Australia [9, 10]. Many of the example homes were part of research projects. In other words, their performances are evaluated and measured in terms of energy and water efficiency, ecological footprint etc.

A number of rating tools are available in Australia currently being use to evaluate energy efficiency of house design. Top tier rating tools include *FirstRate*, *BERS* and *AccuRate*. These are widely used rating tools in Australia. Some other rating tools include *NABERS*, *BASIX*, *ABGR* and *Greenstar*.

There are a number of key organizations promoting sustainable housing in Australia. Australian Greenhouse Office (AGO) is a statutory body responsible for GHG emissions inventory. AHURI (Australian Housing and Urban Research Institute) conducts research on social and economic aspects of housing. CSIRO (Commonwealth Scientific and Industrial Research Organization) developed the *AccuRate* rating tool. Landcom in NSW and VicUrban in Victoria play a special role by developing master planned sustainable communities. Department of Housing in Queensland actively promotes sustainable housing through the ‘smart housing’ project. Similar state bodies in other states are also actively promoting sustainable housing. Green Building Council of Australia (GBCA) developed the ‘Greenstar’ rating tool. Sustainability Victoria is a statutory body in Victoria promoting sustainability among local government, schools and community groups.

Currently 5 star standards are mandatory in ACT, Victoria, South Australia and Western Australia. Table 1 summarizes the current status according to Building Commission [9]. Rebates are available for solar photovoltaic panels, solar hot water systems and rainwater tanks in most of the States and territories. Australia is currently tradeoff incandescent light bulbs with energy saving compact fluorescent bulbs at no cost to the house hold.

A comprehensive on-line guide *Your Home Technical Manual* [11], managed by Australian Greenhouse Office, made available to public which is also a great resource for educational institutions. This technical manual is a great resource for professionals, students, researchers and members who are interested in the current trends in sustainable housing. It includes major issues such as passive design, energy use, materials use, water use, site impacts and other impacts. It also includes a comprehensive discussion of case studies around Australia. A growing market of sustainable products available for buildings include solar PV panels, solar hot water systems, rainwater tanks, insulation products, plantation timber, evaporative coolers, energy efficient bulbs, energy and water efficient home appliances to name a few.

Table 1: *Energy rating regulations across Australia*

| State | 5 star building fabric | 5 star building fabric plus renovations and alterations |
|---|---|---|
| Victoria | Yes from 2005 | Yes from 1 May 2008 |
| ACT | Yes from 2006 | Yes from 2006 |
| Western Australia | Yes from 2006 | Yes from 2006 |
| South Australia | Yes from 2006 | Yes from 2006 |
| New South Wales | NSW uses BASIX | NSW uses BASIX |
| Queensland | 4 star energy rating | - |
| Tasmania | 4 star energy rating | 4 star energy rating |

## Methods and tools available in measuring sustainability

The practice of measuring sustainability is a recent development and the housing industry is rapidly getting adjusted to the task and should be treated as work in progress. Methods of measurements, tools and skilled personnel are at the developing stage. Minimum energy ratings and use of rainwater tanks became mandatory in 2005 and 2006 respectively as discussed. Industry response is really encouraging as some volume builders are now promoting their products achieving 7.5-8 energy star ratings. Also, it is noteworthy that much innovative water conservation devises and apparatus are currently flooding the market. Through such industry participation and competition the products and processes can only improve to better serve the course. However, it is important in ensuring that the current limited capabilities in assessing sustainability is not over rated and over used for purposes other than the set objective.

In true sense sustainability rating tools of the housing industry must be capable of assessing a green rating encompassing,

1. Solar and water sensitivity of the subdivision/plot within which the house is located
2. Energy sensitivity based on, both, embodied energy and operational energy of the house
3. Water sensitivity based on conserving potable water, harnessing rain water and recycling gray and black water

In this light the current status of the commercially available rating tools, despite exceptional progress within a relatively short period, can only be described as at their early stages of the development. The three key elements of sustainability mentioned above are currently being treated as standalone independent parameters which are not necessarily true. For instant, when a tool classifies the home having a five star energy rating; it means that rating is based only on the operational energy component. Widely used commercially available energy rating tools indicated above calculate star energy rating only using operational energy. Also, each of the above elements must be assessed not only at the development and occupancy stage of the house but over the full life cycle. The LCA tools currently in use only focus on maintenance intervention. This is a potential future research area where very little work has been done to date.

## Concept of Bill of Embodied Energy (BEE)

As mentioned before a full evaluation of energy efficiency of a domestic building should include, both, embodied energy and operational energy. Embodied energy is a measure of environmental impact, which can be quantified and optimised through proper selection of materials and construction processes during the planning stage. Embodied energy is defined here as the cumulative energy embedded in the process of bringing a domestic unit into being. This includes the energy embedded in the processing and manufacturing of materials, transporting and handling and construction [12]. Figures 1(a) and 1(b) illustrate the distribution of embodied energy within a typical single dwelling in Australia and a comparison between embodied energy and operational energy breaking even.

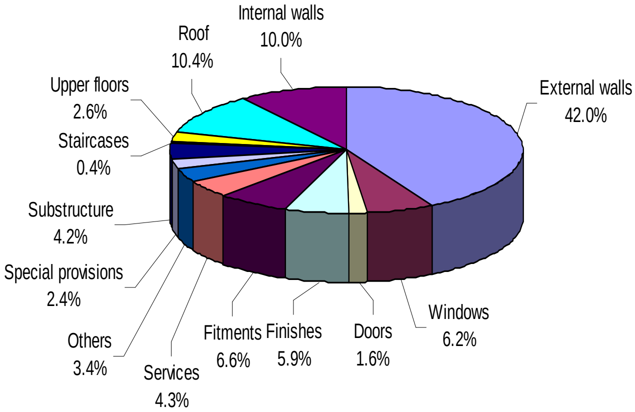



Figure 1(a): *Embodied Energy (EE) distribution of a single dwelling*

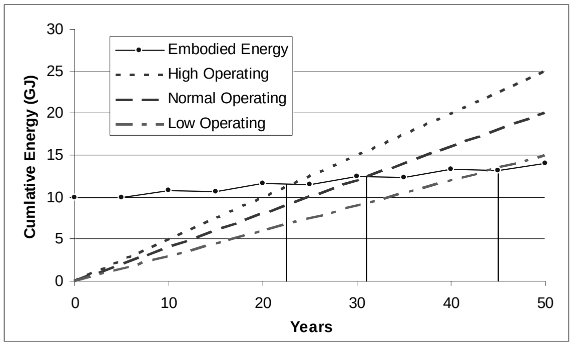



Figure 1(b): *Comparison of EE and operational energy breaking- even during life cycle*

Objective of estimating embodied energy can be beneficial in two ways. Firstly, in practice, achieving a better carbon foot print of the final product. Secondly, in research, it has the capacity to highlight the energy intensive elements of the building and the materials which are contributing to that outcome. For example, as Figure 1(a) illustrates, 42% of the embodied energy reside in the building envelope. The typical single dwelling assessed above has a building envelope which comprises a timber load bearing frame, brick venire, and 10mm internal plaster board finish with standard insulation. Out of all these products used to construct the building envelope the brick venire contributes the most. Obviously, production of kiln baked brick venire is highly energy intensive. Such measures and observations of environmental impact can influence the brick manufacturing industry to improve their production processes to be more environmentally friendly. As a result, less embodied energy bricks and smarter wall paneling systems are starting to emerge.

BEE

| *Components* | *Material/Dimension/Methods of Construction* | *Unit* | *Qty* | *Cost ($)* | *Embodied Energy(GJ)* |
|---|---|---|---|---|---|
| Sand | 50mm thick | $m^2$ | 180 | 547.20 | 2.340 |
| Membranes | 200um | $m^2$ | 180 | 486.00 | 0.522 |
| Stiffened Raft Slabs | 300mm deep x 300mm wide @ 7.0 meter spacing; F11TM3 and F72 mesh | $m^2$ | 180 | 13,114.80 | 41.472 |
| Clay Brickwork | Walls - 110mm thick | $m^2$ | 344 | 24,668.78 | 374.070 |
| Upper Floor Framing - Hardwood F8 | 200 x 50mm @ 450mm C/C | $m^2$ | 180 | 4,951.80 | 20.232 |
| | | | | TOTAL COST | $xxxxxx.xx |
| | | | | TOTAL EMBODIED ENERGY | xxxxxx GJ |

Figure 2: *Sample of Bill of Embodied Energy (BEE) output*

To further leverage the embodied energy measurements toward a more sustainable construction industry, including housing, the concept of Bill of Embodied Energy (BEE) was introduced. The initiative stemmed from extending a traditional practice widely adopted by the industry. It is an age old practice to produce a Bill of Quantities (BOQ) prior to commencement of a project to estimate the optimum cost. The idea was to develop and test a web based decision support tool [web link], which has embodied energy data presented next to bill of quantities. The embodied energy unit rates were aligned to the items and unit rates of the standard building cost guides. Figure 2 gives a part of a report generated by this tool to illustrate the idea. The reason for developing a web base tool was to disseminate this knowhow to a wider community at no cost. Full account of this research undertaking, development of web based tool and research outcomes can be found in reference [12] which was conducted under the supervision of the author.

It is envisaged that such tools would not only assist the developers and the home owners to understand and appreciate their environmental duty of care but also the planning authorities and local governments to make informed decisions with respect to carbon foot print within their constituencies.

## Influencing volume builders market

As mentioned at the beginning, especially in Australia, the key is to positively influence the volume builders market towards a more sustainable practice. This section briefly outlines couple of recently concluded research projects conducted at RMIT University supervised by the author [10, 13].

### *5.1 EcoHome, Cairnlea, Victoria, Australia*

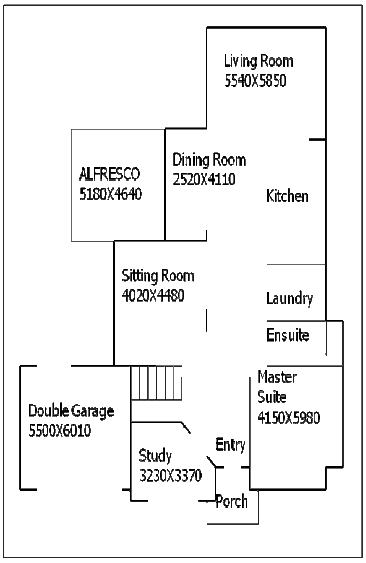



Figure 3: *EcoHome, Cairnlea, Victoria, Australia*

This project is the fist-of its kind in Australia. A standard 280 $m^2$ floor plan from the volume builder, Metricon Homes, was selected and improved with state-of the art sustainability features including full assessment and against passive design principles. Project was a joint collaboration between the industry stakeholders, two Universities which was partly funded by the Australian Research Council. Project duration was 2002-2007. The project is strongly supported by a consortium of industry partners including the Urban and Regional Land Corporation, Metricon Homes, Building Commission, Origin Energy, City West Water, Melbourne Water, Sustainable Energy Authority of Victoria and Hassell Architects. The main objective was to investigate the sustainability outcomes that are possible in outer suburban project homes using current building and design technologies, and the barriers to the uptake of these technologies more broadly in outer suburban project home developments. Building was designed and constructed incorporating instrumentation to measure and monitors its performance during first two years of occupancy. Water and energy usage was zoned for monitoring purposes. Full account of this research undertaking, development of web based tool and research outcomes can be found in reference [10].

Key findings were,

1. The cost overshoot due to the incorporation of state-of-the art sustainable features were around 15% of the total construction cost of the house.
2. Use of passive design principles (solar orientation) in the design and proper orientation of the house paid dividends in terms of maintaining thermal comfort with notable energy savings especially during winter time.
3. Based on the comments during display period and the occupant's comments (over two years of occupancy) no negative perceptions were noted.

### *5.2 Sustainability of pre-fabricated modular construction*

This project investigated the feasibility of the use of pre-fabricated modules replacing the traditional and current practice of on-site construction. The investigation took into consideration not only the direct benefits in achieving better environmentally sustainable housing but also the secondary benefits such as affordability, speedy response to shortage of housing, better quality control, measurement and certification of sustainability. Study revealed the modular construction, for volume builders market, was not only found feasible but attractive.

Layout flexibility and avoiding repetitiveness of external façade identified as the deterrents needing innovative thinking. This is because home is perceived as a way of expressing individuality and there is a stigma attached to living in community housing environments.

So the key is the ability of the modular system to produce an adequate number of different floor plans. Obviously this is possible when utilizing stick construction, because the components are small enough that they can be combined in any number of possible ways. In practice of course houses look similar, regardless of their technical uniqueness. Many trends and standards exist in home design; for example 80% of Australian homes have 2-4 bedrooms. This and other trends provide a useful framework and benchmark from which new designs can be formulated. This fact allows high-volume builders to produce a finite number of house designs for a development and then build them repetitively, making sure to spread them out so no one homeowner gets the feeling of living in a carbon copy home. This same method is applicable to a modular approach. If one set of modules can be designed in such a way that they can be arranged into several different designs, then the system utilises the benefits of replicate construction while providing a developer and homeowner with the necessary range of different designs.

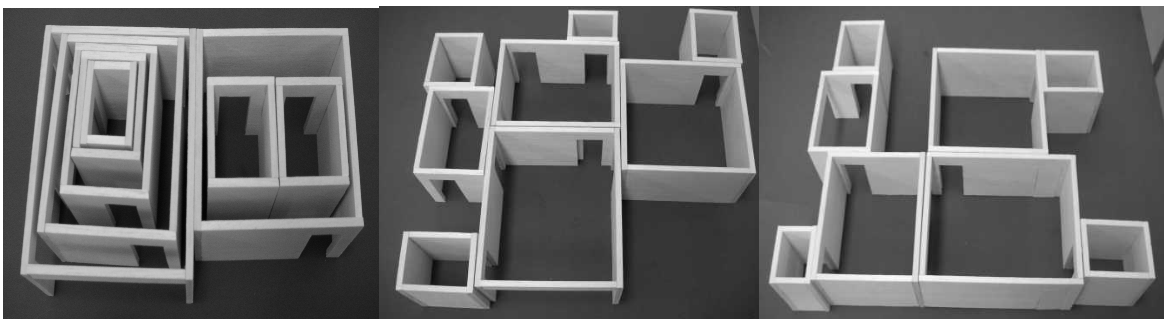


Figure 3: (a) Modules (b) Layout option 1 (c) Layout option 2

Table 2: *Module sizes and numbers in set*

| Unit | Size (m) | # |
|---|---|---|
| A | 1.5 x 1 | 1 |
| B | 2 x 1.5 | 3 |
| C | 3 x 2 | 1 |
| D | 4 x 3 | 1 |
| E | 4 x 4 | 1 |
| F | 5 x 4 | 1 |

Initial investigations into the potential flexibility of modular systems indicate the necessary performance is achievable. Simple single-bedroom floor plans have been developed demonstrating that a single eight module set (which contains six distinct module sizes) is capable of delivering twenty or more different layouts of varying uniqueness. Table 1 provides the sizes, which are also compatible with current volume builders' product range, and number of modules that the set contains. Full account of this research undertaking, development of web based tool and research outcomes can be found in reference [13].

## Conclusions

Promotion of sustainability in the housing industry requires the commitment and proactive participation of the stakeholders. Effectiveness of the process cannot be driven from top nor could it be expected to spring from the grassroots. An informed concerted effort from community, industry and research community can only be expected if the need is felt. This requires a significant effort in education and capability development. From a technical and commercial view point, sustainable housing industry is viable. It is imperative to develop measurable outcomes to ensure opportunities are not carried away to become a promotional and propagandist style campaign. No environmentally sustainable measure can be regarded as independent. Each measure is part of an interwoven system which must be treated as an eco system in balance where one measure feeds from the other. In this light, current rating tools and methods of measurement has a greater potential for research, innovation and development.

### References


68. McDonald, P., Medium and long term projections of housing needs in Australia. *Australian Housing and Urban Research Institute*, 2003.
69. Minnery, J. R., Urban Form and Development Strategies: Equity, Environmental and Economic Implications. Canberra, AGPS, 1992.
70. Australian Bureau of Statistics, *2001 Census of Population and Housing.* http://www.abs.gov.au/Ausstats/abs@.nsf/e8ae5488b598839cca25682000131612/a758e92a2d80c819ca256cf4007e1dcf!OpenDocument, visited on 10th Nov, 2010.
71. Building Code of Australia, Australian Building Commission, 1994
72. Mobbs, M., *Sustainable House*, Choice Books, Sydney, 1998
73. Owen, *Clematis House*, ABEC Case study for ABEC, http://www.netspeed.com.au/abeccs/clematis/clematis.htm, visited on 10th Nov, 2010.
74. Building Designers Association of Victoria, *Designing in Waste Minimisation*, BDAV and EcoRecycle Victoria, Melbourne, 1998.
75. Property Council of Australia, *Sustainable Development Guide: A Roadmap for the Commercial Property Industry*, Property Council of Australia, Sydney, 2001
76. Your Home Technical Manual. Fourth Edition, 2008, http://www.yourhome.gov.au/technical/index.html, Lasaccessed 12 July 2010.
77. Rahman, S.M.S., Sustainable Housing and the Outcomes of the Cairnlea Ecohome, PhD Thesis, RMIT University, January 2010.
78. Your Home Technical Manual, http://www.yourhome.gov.au/, visited on 10th Nov, 2010.
79. Ting S.K., Optimisation of Embodied Energy in Domestic Construction, Master Thesis, RMIT University, January 2007.
80. Oxley, D. R., Promoting Sustainable Culture: Could Prefabricated Modular Housing Systems Could impact on Current Domestic Construction Practices?, Master Thesis, 2006.

**Acknowledgements**

This paper is based on three post graduate research project outcomes conducted by Dr. S. M. S. Rhaman, Miss. S. K. Ting and Mr. David Oxley under the supervision of the author.


**About the Author**

**SAMAN DE SILVA,** B.Sc. (Hons), Moratuwa, Ph.D., Melbourne, MIE (SL), C.Eng., is a Senior Lecturer at the School of Civil, Environmental and Chemical Engineering, RMIT University, Melbourne, Australia. His research interests are in the areas of earthquake engineering, sustainable infrastructure, and computational mechanics.

# ECOLOGICAL PARADIGMS IN PLANNING THE LIVEABLE CITY

Prof. Harsha Munasinghe

Department of Architecture, University of Moratuwa, Moratuwa, Sri Lanka.

***Abstract***
Sustainability is currently the most pressing, complex, and challenging agenda faced by the city. The focus of sustainability has turned on to wider issues of environment, ecology and people. Eco-city as a concept has played a vital role in designing new habitats in the city. In the context of promoting nature as bio-centric entity that controls itself, the eco-city has become a life-territory, a place defined by its life forms, topography and biota, rather than by human activities. As a result the concept of eco-city is over-dependent on the carrying capacity of land, thus paralyzing the growth of the city and its social evolution. We intend to test the strength of the evolved city culture as a reliable tool in shaping strengthening the liveability of the city. In Sri Lanka, cities have been built as political cum religious polis since antiquity and they pretty much yet remain politically-centred, fragmented and nature-represented. Although, the Sri Lankan city is full of trees, it is not *green* as a living space because the illegible city form is distancing from society. Our research, through a literature review and a field study carried out in the city of Panadura, intends to test new paradigms that make the city a strengthened living container. We find that the socio-culturally defined ecological footprint is a useful tool in reinforcing city's liveability.

***Keywords:*** Eco-city, ecological-footprint, cultural-planning, carrying-capacity, Sri Lanka

## INTRODUCTION

Sustainability is currently the most pressing, complex, and challenging agenda faced by the city. Expanding urban population across the globe has turned the focus of sustainability from being simple concerns such as global warming or depletion of non-renewable energy into a wider issue of environment, ecology and people. Idea of sustainable development launched by the World Summit on Environment and Development of 1983,[28] and the Earth Summit of 1992, redefined ecological-sustainability as a key word in city planning. Eco-city and Green Architecture have come to play a vital role in designing new habitat of the urban society as a result.[29]

Cities are living containers designed to promote good life. Liveability is the most man-oriented scale to measure the quality of life in the city. Ecological sustainability improves the liveability of the city for linking man and his society with the environment. Our attempts shall aim at evolving the *correct* balance between the nature where the city is located and human action that impinge on the city.[30] The linking of *good city life* with ecology gave birth to the idea of Eco-city defined with nature, its resources and their continuity in pristine form without sacrificing the will and strength of an evolving urban life. In the context of promoting nature as a bio-centric entity that controls itself, the Eco-city has become a life-territory, a place defined by its own life forms, topography and biota, rather than by any human activities. This tendency to make Eco-city over dependent on the carrying capacity of land hampers the evolutionary process of the human habitat. The concept of carrying capacity, having defined from usability of land in agriculture, places undue emphasis with materialistic values of

[28]. *Our Common Future* (1987) World Commission on Environment and Development, London: Oxford Press

[29]. These concepts intend to create habitats with minimum environmental impact: minimized requirement of input resources and controlled waste out. Richard Register coined the term Eco city in his book, *Eco-city Berkley: Building Cities for Healthy Futures*, published in 1987.

[30]. One shall not confuse the idea of Eco-city as bringing the elements of countryside to the city. Lefebvre (1996:87) notes the differences, "the countryside, both practical reality and representation, will carry images of nature, of being, of the innate. The city will carry images of effort, of will, or subjectivity, of contemplation, without these representations becoming disjointed from real activities".

natural resources at the expense of qualitative aspects such as culture, society, threatening city's continuity and diversity.[31]

Management of natural resources, use of ecologically-friendly materials, use of renewable energy sources, and such quantitative aspects of ecological sustainability are being taken care of at the expense of qualitative aspects such as city form, lead-built forms, land use, potential growth patterns that necessarily represent the living society. As a result, one may observe the degradation of city's capacity to contain its life thus becoming sterile and stereotyped. Society failing to find its place in the city is forced to inhabit the space rather than dwelling it. Having noted the need to balance the quantitative-bias approach with a qualitative-oriented catalyst for growth, we intend testing the strength of the evolved city culture as a tool in shaping a new paradigm shift that would strengthen the liveability of the Eco-city.

Cultural dimension in city designing notes a critical question in pursuing sustainability. Culture and cultural activities have been displayed in the city since antiquity reinforcing the continuous evolution of the both city and society. As such, a lived city, with its layers of cultural deposits, attests to a diversified city culture. Culture is the whole set of values, ideas, meanings, symbols, and organisational rules of a society reflected in its institutions, and using the environment to support its social relationships. The lack of orientation to accommodate the cultural dimension in shaping cities has resulted in the loss of place-identity and the representation of evolving society as it is clearly visible in many Sri Lankan cities. We observe the degradation of city life as a result of paying attention on the growth or its control rather than its pattern.

City may further lose its liveability if the universally-defined concept of Eco-City is enforced without integrating socio-cultural interests. By bringing the society and culture into the centre of decision making with regards to city planning, a holistic approach could be formulated to protect the uniqueness of the city. Scott (2000:30) notes, "Place and culture are persistently intertwined with one another, for any given place… is always a locus of dense human relationships, and culture is a phenomenon that trends to have intensely local characteristics thereby helping to differentiate places from one another". The significance of the culture in the planning the Eco-city has been discussed in many forums. Since the carrying capacity of land is mostly decided upon geo-physical conditions of the land, the strength of the socio-cultural context has been largely neglected and city development has been attempted on a piecemeal basis. The point of departure of our research is the resulted *place-less* city. We intend to look at the practice of the concept of Eco-city to prepare the ground works for integrated planning that may bring about a liveable city.

Our aim is looking at the new paradigm called Eco city and to assess its strengths in reviving the Sri Lankan city as a living place.[32] We scrutinized the concepts of Ecological Footprint, Smart Growth, New Urbanism, and PLACE3 that complement the strengthening of the liveability of the city. Our primary objective has been carrying out an in-depth analysis of the Sri Lankan city in terms of its liveability and growth, and collecting and processing data in relation to the concept and practice of Eco-city. Then using, testing, and disseminating the processed data we intend, as an end result, to outline a method of analysis to study the liveability of city form.

The other objectives realized are: Reviewing literature and producing them into a useable format: Preparing a checklist for assessing city planning: Shaping design briefs to inform prospective developers and designers of the possible Eco-appropriate growth patterns: Enlarging awareness of the developed framework through a set of stakeholder meetings, and as an end product we also aimed at

---

[31]. One could easily see that how these conventional means of shaping sustainable environments are becoming rather *stereo-typed* thus risking the making of the Eco-city into a mere *type.*

[32]. Munasinghe (2004) 'Ecological Housing in Helsinki: Case study of Viikki', *Sri Lanka Architect*, Journal of Sri Lanka Institute of Architects, 4/2004

preparing the ground works for the development of educational programmes of modules at primary, secondary and tertiary levels.

## SRI LANKAN CITY FORM

"City is a fact in nature, like a cave, a run of mackerel or an ant-heap. But it is a conscious work of art, and it holds within its communal framework many simpler more personal forms of art" (Mumford 1938). No city can stand still and neither has it been dropped down from the sky. It is a dynamic representation of diverse understandings between flesh and stone (Sennett, 1994). The particular socio-spatial matrix of the city shows an evolving value system of man, his society, and the environment. Learning to respect and to respond to the volatile forces of the place, and then evolving designing criteria of less-destructive additions to that place may lay the foundation to the ecologically-sustainable city design. In the case of the city, ecology has more to do with a built environment, functions of its elements, accommodating a society, and its possible growth/change, that together mirror the evolution of a living society. Here one shall pay due attentions to the socio-spatial matrix rather than the individual elements that is the case of *natural* environment, and as such to the involvement of the city life. As such, it is necessary to pay due attention to the society in assessing the impact of designing and building any new extension to the city.

In Sri Lanka, cities have been built as political cum religious polis since antiquity.[33] Although the city form has evolved from antiquity to medieval, then to industrial and post industrial resulted along with the social evolution and diversification, the Sri Lankan city form pretty much yet remains politically or religious centred, and as a space, remains more fragmented and nature-represented. One could thus argue that our cities are already Eco-cities, emphasizing the existence of extensive green patches and low density. This particular state of our city is a reaction to our way of life- majority of us living in the city yet leading a rural way of life, and as such the city has not remained an Eco-city but a mere village. Also, most of us were attracted to the city by pull factors such as employment, infrastructure facilities, etc not through our intention to be urbanized. Having come to live in the city we have re-adapted our rural way of life in a more dense and diversified surrounding but have not paid due attention on the public realm for collective human activities. This is why have not evolved urban activities in the city.

The best example for this type is the fortified city of Galle. The city plan is based on a grid-iron street layout of a defined hierarchical order dominated by the Queen Street, thus depicting the influence of political power. This street provided access to the main trading point and the port. The city form was divided into three quarters: administrative, commercial and residential, and buildings were designed to fit the particular functions and users. Later during the British rule, the commercial quarter was shifted outside the ramparts. British imperialists willing to live in much grandeur, moved out of the fort, allowing the locals to settle down in the residential quarter of the fort. This resulted in a more significant division of the fort between administrative quarter and residential quarter. Identification of the universal heritage values of the Galle fort and its enlisting on the World Heritage List eventually turned it into a place of leisure filled with museums, art galleries, hotels, and holiday homes. The administrative function has been ejected from the fort and the residential function is declining. As a whole, the declaration of Galle fort and the adopted development regulations and restrictions have resulted in degradation of city life.[34] The new urban quarter that accommodates the ejected functions took its reference from the old city and retained the sense of belonging over a long period of time mostly due to its street layout and plot division pattern. However, the new built forms have not responded to the existing character. The city centre trapped in between the two quarters attests to a typical medium size town and does not refer to the glamorous past. Moreover, Galle is full of underused, unused as well as misused built urban spaces, which could easily facilitate the demands of the evolving society without losing ecological balance.

[33]. Refer Ellawela (1969) to understand the emergence of human settlements in early Ceylon (Sri Lanka) and how they developed into urban settlements (pp 115-116).

[34]. Munasinghe (1998) notes the degradation of city life in Galle fort as a result of urban conservation.

In the Sri Lankan city, development is misinterpreted as improvements of economics, thus paying less attention to the social capital. Today, the Sri Lankan city centre is disintegrated due to urban sprawl along the main trunk road, and losing its identity mostly due to mediocre place-less architectural forms that form the enclosure of the said trunk road. With new buildings being built without responding to the true character and identity of the city, the container quality of the urban space is degrading. Furthermore, most of the cities have extensive green spaces, attractive water fronts, many spaces for public activities, socio-cultural resource that could draw an income. The existence of layers as well as the underutilized or unused urban spaces suggests that Sri Lankan city needs a comprehensive integrated approach that identifies its urban precincts and strengthens their particular container quality to restore the liveability of the city.

In fact, the urban society in Sri Lanka has not evolved any collective activities, making the city a living setting nor have we evolved social structures or organizations to support such activities. City is full of *dead* spaces and thus has become a threatening place- littered, covered in graffiti, polluted, congested, plagued, and filled with many non-urban mediocre buildings. Although, the city is full of trees, it is not *green* in terms of liveability.[35] As such, the city has not been able to foster comfortable dwelling. The sprawl in the Sri Lankan city along manor traffic artilleries has been unstoppable hence threatening the basic definition of city as a bounded space and attesting to the fact that urban designers have failed to facilitate city's growth. This attests to the grave need to strengthen city's role as a living container to rebuild links between urban society and city.[36]

Preliminary observations had surfaced the possible contradictions of enforcing this concept in the Sri Lankan context. The major conflict comes up with the definition of city in Sri Lanka. It had been noted that the concept of Eco-city had mostly been used in Western contexts. There is a marked contrast in socio-cultural values, social context, social order and organization, and social surroundings once compared with Sri Lanka. Also, the city as a living space as well as the types of urbanism has been different in Sri Lanka. Case study surfaced the need for case-base shaping of the concept and implementation tools of the Eco-city concept.[37] We focused onto a case to learn more of its processes than end results using Strategic Planning, Eco-city Zoning, Proximity Law, Increased Diversity, and Clustered Development as tools to redefine urban ecology. The tools used to implement such criteria are city governance and communicative planning that involves the society in decision making. The strength of these criteria is supporting sustainability and social empowerment, and facilitating responsive environments. These criteria, through an understanding of the unique relationship between life patterns of individuals, their social context and reality, their way of ascribing values with the environment, could revive city's liveability. Our intention is to promote a building process that will necessarily upgrade the living standards of the city by paying due attention on those links between man, society, and environment.

## CONCEPTUAL DEVELOPMENT IN ECO-CITY

Ecological Footprint is a concept adopted in the cities in UK. It is a system of comparing human demand with the ecological capacity of the place, especially in the case of regeneration. It is a further development of the representation of amount of biologically productive land and sea are needed to regenerate the resources a human population would consume and absorb and render harmless corresponding waste.[38] We used this analysis to check the capacity of a city and its built elements in

---

[35]. Eco-city is defined as the one that enhances the well-being of citizens and society through integrated urban planning and management that harness the benefits of ecological systems.

[36]. Munasinghe (2001) for a discussion on the city and society.

[37]. The Green Finger plan adopted in the developing of the urban district called Viikki in Helsinki is an instructive example for integrated development in which the public participation too was integrated in decision making. Viikki was a *tabula rasa* and a predominantly residential quarter but clereverly intertwined with employment generation and a socio-cultural diversity.

[38]. Wackernagel M. and Rees W.E. (1996) *Our Ecological Footprint: Reducing Human Impact on the Earth*

accommodating the needs and demands of the living society thus testing the morphology of the city: city plan, land utilization and the building stock: solid-void ratio, building types, indoor-outdoor urban spaces, and spatial structure criteria of the city against the demands of the living society that are depicted in the built environment. This analysis prepares the grounds for shaping an integrated development scenario that is place-oriented in scale and in type to determine the reuse and recycling of the built fabric and urban land. The report, *Towards an Urban Renaissance* that notes the economic, social and environmental context as dependable surroundings in designing the footprints is an instructive example such an attempt.[39] Key themes of the report are recycling lands and buildings, improving the urban environment, achieving excellence in leadership, participation and management, and delivering regeneration. As a whole, the report finds the strength of *Ecological Footprint* in making a city of higher quality of life.[40] Our investigation is mostly framed by these themes.

The European Commission adopted the Communication COM (2004) 60, *Towards a Thematic Strategy on the Urban Environment*, of January 2004, setting out the Commission's idea for a thematic strategy on the Urban Environment in summer 2005.[41] The four themes; environmental management, urban transport, sustainable construction, and urban design are at cross cutting with nature and possess many strong links with environmental issues. It must be noted that these principles and approaches may not fit the Sri Lankan conditions as social values, orders, and organizations vary significantly yet would prepare the grounds to develop our own system.

The definition of principles of sustainable development for policy makers given in the Brundtland Report (1987) could be developed for Eco-city planning. They are:

1. Changing current patterns of economic growth, technology, production and management which may have negative impact on the environment and population:
2. Ensuring employment, food, energy, safe water, and sanitary services for all populations:
3. Protecting natural resources for future generations:
4. Integrating economic, environmental and population considerations in policy decision-making and population growth.

Strengths in these principles are referring to place-oriented approaches that may instigate the growth of the city and promote a holistic approach, linking development to society, culture as well as to environment.

We also tested the concept of *Smart Growth*, which has the vision to build an Eco-community through designing extensions in the city. It reinforces the liveability of the city without distorting its conceived images yet guiding the necessary changes demanded by evolutions and development of the society. Smart growth aims at managing the *bounded space* – the city, thus arresting sprawl and directing growth patterns, and allowing one to develop local solutions that respond to the place: the physical as well as cultural context. This can be used to assess and further build up the *social capital* that is not exclusively geared to note the material gains but mostly to include the enhancing spiritual well-being, sense of identity and belonging, social status, honour and prestige (DFID, 1999).[42] This concept as such would respect the evolved morphology of the city. *Smart Growth* is a useful tool for planning the ecologically sustainable city. It advocates that the growth itself should be tied to the quality of life and how and where it should be persuaded and, as such *Smart Growth* touches the crux of the said principles. This can be defined as the practice of integral quality: including economic, social and

[39]. www.eukn.org/unitedkingdom/themes/Urban _Policy/Towards-an-urban-renaissance-final-report, The report was written by the Urban Task Force headed by Lord Richerd Rogers

[40]. Our data suggests that most of urban lands and buildings (about 43%) in Sri Lankan cities are either unused or underused. The reasons being buildings outliving their functions and societies, and uncertain political visions beside the uneven urbanization patterns.

[41]. Ec.europa.eu/environment/urban/pdf/Sec_2006_16_en.pdf

[42]. DFID (1999)- *Social Capital: Overview of the Debate,* www.oneworld.org/odi/keysheets

environmental performances in a broader way. The rational use of natural resources and appropriate management of the city will contribute to saving scares resources, reducing energy consumption and improving environmental quality. *Smart Growth* can be used to essentially involve the entire life cycle of the city, environmental quality, functional quality and future values. It is important that the authorities note the qualitative aspects of the built environment. If properly planned as representations of an evolving socio-cultural discourse, the built environment could be resource efficient, energy efficient, pollution preventing, harmonising with the natural environmental.[43]

*New Urbanism*, the other concept that we investigated, is known for rebuilding the degrading city since it emphasizes on the creation of public realm and not reserving space for motor car. Harvey (2000:169) notes “It (new urbanism) attempts intimate and integrated forms of development that by-pass the rather stultifying conception of the horizontally zoned and large-platted city. … It also permits new way of thinking about the relation between work and living, and facilitates an ecological dimension to design that goes beyond superior environmental quality as a consumer good”. However, there are many who challenge this movement as nostalgic. They are sceptical about the other similar attempt, Urban Village, led by Price Charles in Britain, as the locus of urban regeneration. Among the issue put forward by these sceptics is the dwindling growth and change as a result. There is no guarantee that the people are that keen to live in communities with strangers who live in their neighbourhoods, and as such the urban environment cannot facilitate community formation to the extent of the village. As a whole such attempts may again lead to urban utopia, including some people while excluding the others. *Smart Growth*, as a concept, seems to have learnt from these lessons to create a balance in the city by facilitating a diversity that is unique to the place. As such, our aim is to test the strength of this concept in the context of Sri Lankan city in order to managing, reinforcing or even rebuilding the liveability of those city centres focussed through case studies.

Our intention has been to test the strength of these concepts in achieving the eco-city. We have tested a living city in terms of its capacity to cater the demands of the evolving society. It has been noted as important to discuss the sustainability of the city in terms of its lived life and the lives that will come in the future.

## ECOLOGICAL FOOTPRINT OF SRI LANKAN CITY

Sri Lankan cities are either colonial-found or emerged on major traffic artilleries. The colonial-found cities are typical medieval city centres where trade was given priority along with political power. They were mostly planned around a main street where trade activities took place, but the political hub has been at the core of the main street.[44] The colonial city was planned as an extract-point for the surplus made in the countryside that was noted as a hostile landscape. The resident colonial community demanding a higher sense of security, the city was protected with physical barriers such as ramparts, moats, etc. The city forms evolved with the change of colonial powers and their relationship with the local community.

Panadura is a town centre that was developed as a way-side stop-over on the Galle road. It may have been a port-town as the river meets the sea there, and there was a famous ferry. The Dutch built main street proves that this town would have been largely shaped by the Dutch as an extract point to gather cinnamon. The town has a particular morphology with a city plan dominated by the main street used for trading activities and there are secondary streets starting off the main street. This main-street dominated linear city form is a typology of a wayside town in which trading was the primary function. The urban wall of the main street is dominated by a duel-function two story built form, in which the lower floor was used for trading and the upper floor was the residential quarters. The particular form

---

[43]. This integrated planning shall be achieved at different levels: policy level, planning level, spatial development level, and at last building level. The built environment as a whole shall note the uniqueness of the place where they are built and then strengthen the particular genius loci in order to strengthen the container quality of the city space.

[44]. Munasinghe (1992)

of the building type, that still dominates the main street, further attests to city's evolution into a main trading port. More importantly, this trading town was not a haunted one in the night with a permanent population residing within the city centre. The Dutch origin of the built form and its abutting the main street marks another era of urbanisation in Sri Lanka. The spatial structure criteria of the city form have accumulated layers of living experience of post-Dutch societies. The commercial centre of the city started to diversify as a result, thus relegating the cinnamon trade to a minor activity. The said shop houses became just commercial building, ejecting the residential function towards the east of the city. As a result the city form transformed from a linear one to a centric one adding new urban quarters. The main street dominated the spatial structure criteria of the city. Panadura retained its significance during the British as a mini administrative centre. The effluent society emerged during this period, and they built manor houses outside the city, thus further diversifying the city centre.

As the main street was not sufficient to accommodate the growing traffic needs, a four lane road was added by cutting through the edge of the city in the post-independent era. Later, this *new* Galle road became more significant for touching the major transport hub: the Bus station, thus attracting the commercial activities out of the main street. The main street was returned to simple and quiet trading activities that are sought by the local residents while Galle road becoming the *busy* central business district. The city began to evolve between the two artilleries, acquiring a new identity without losing its original sense of belonging on which the city life could depend. Many public facilities such as Hospital, Library, Town Hall, Bus station, Police station, etc were added to the urban district between the two roads while banks and other commercial institutions establishing themselves along Galle road. This development necessarily created a rather busy character along the Galle road. The meeting points of the two streets importantly started to dictate the bounded nature of the city as the visitors always identify the arrival in the city with these two points. Most of the local citizens frequently move between the two streets: the main street and Galle road. As a whole this new layering of Pandaura diversified its urban character.

The overpowering nature of the new economic activities located and the changes in social values have shifted the residential functions out of the city centre. Simple specific trading activities of the main street do not demand the occupation of the upper floors of the duel function buildings. At the same time, some traders sought replacing these buildings with new taller structures. Today, the upper floors of the old buildings are mostly empty. With Panadura becoming a major urban hub that would retain the over spilling populations of Colombo and with the travelling of Galle road becoming more diversified, the city started to sprawl. The city form started surpassing its identified edges, residential function growing towards east thus colonising more new lands and commercial functions causing a ribbon development along Galle road. This ribbon development challenges city's bounded nature to extent of distorting Panadura's unique identity. Today, the city centre that evolved around Galle road and its shopping district depict a typical wayside town, built to pass-by rather than to contain social and living demands. There is a grave need to identify an integrated development strategy based on the transforming morphological elements: city form, land utilization and built forms. Retaining the quality of a bounded space and restructuring the forcefully-emerged spatial structure are also essential.

Among the most detrimental to Panadura has been the sprawling along Galle road- the most powerful urban element. As a result, urban centre is being compartmentalized and therefore is at the brink of becoming a non-city. Its bounded nature is disappearing with the two celebrated points becoming insignificant. Their sense of belonging has been threatened by the mediocre architectural forms, which do not respond to the place. All cities seem to be acquiring layers that are shaped by extrinsic values such as quick financial gains, tourism, etc, and these layers eventually turn the city into typical life-less space. The city is losing its identity, failing to sustain life and becoming a haunted place, especially after day-to-day activities are concluded. Overpowering financially-driven activities and motor traffic have swallowed up the public realm and as such the container quality of the city. The traffic has become a more addressed topic than the society or quality of life in urban development. Building regulations or any other legal documents that deal with urban development do not refer to social responses or to any type of impact assessments unless there is a bio-centric environmental issue. Ecological Footprint, as an assessment for built environment, has potential to test urban development. It will emphasise an alternative way for development regulations such as height restrictions, plot coverage, floor area ratio. Today, these regulations and guides are mostly political, financial or security oriented rather than socio-cultural. Ecological Footprint as a way of analysis would first enhance the gestalt of the city form and then prescribe the development guides for individual plots and buildings.

The strength of ecological footprint of the existing built stock was assessed in the context of evolving society. We also attempted at identifying functions and users in these buildings to revive the city centre. At the same time it has been necessary to ensure that the city form will not lose its bounded nature and the current urban identity. It is essential to identify the urban districts within the city and designate them appropriately as commercial, residential, mix, etc that together compose what Panadura is. Also, the post-industrial city is often in demand for light industries and they can be integrated within the mix zone. The strength of ecological footprint is reinforcing the designated zones so that the city will not be compartmentalized. The residential zone will demand certain amount of commercial functions but more importantly residential function should make a return to the commercial zone. The duel function built form is the key to facilitate the continuity of city image and to accommodate the returning society. Our assessment shows the possibility of turning the upper floor to a successful living space as over 60% of those who were interviewed considered the city centre as habitable. The ground floor of the building, which is a more open hall, could accommodate new types of commercial activities as well as the said light industries. In addition, it is also found that there is demand for financial activities such as banks, insurance, etc as well as for cultural activities such as galleries, studios, theatres, etc to be accommodated within the city centre. The two story buildings thus can be saved for their *soft environment* factor that attracts people, facilitates society forming, and as such strengthening the liveability of Panadura.

Typological analysis of the built fabric in the city centre assists us understanding the means of inserting the said activities and in-filling the urban grain with new built forms that would be compatible with the lead types. Educational and religious institutions located in the city centre bring many people. It is necessary to manage the solid-void ratio in the educational and commuter sectors in order to facilitate the smooth functioning of the city. The link between bus station and railway station is the most used during the day. There are many under-used built forms that could accommodate formal as well as casual public activities around this link. The most crucial problem in the city would be facilitating parking in the case of inserting new activities. It is possible to manage the type and scale of activities in order to control the need of vehicles and then to use the vacant plots in which the old buildings have been removed, and the land available behind the buildings to satisfy this controlled demand. It is also found that the new facilities would be mostly used by the local residents who may not bring vehicles if the public transport is provided. It is needless to emphasize the importance of renewing the railway and its related activities as Panadura has become one of the most popular spill-over of Colombo. Thus establishing links between the local buses and the train station would be essential. The development of a railway square thus would be important to sustain an informed ecologically-sustainable society.

Most importantly, the city plan, land use pattern, and built forms of the city inform the possibility of bringing in the revivals. Using the concept of new urbanism as a guide, we may be able to add new built forms to all the quarters of the city. The current lead building type and the typologies of elements could be exploited to enhance the current image of the city and as such links between citizen and the city. The accommodating nature of the existing buildings through their veranda, balcony and the inviting character could be used extensively in designing new urban quarters and their built elements. Urban briefs, including such information to give directions to the prospective developers and educating them on sustainability, will facilitate the making of an ecologically-sustainable urban society.

Today, Pandaura is not a flourishing life world but its extensive water bodies, mashes, and paddy fields along with the inhabited lands could support the rebuilding of a life world. The life cycle of city’s economics changing in an unprecedented speed but can be comprehended by the societies and be facilitated in city’s morphology. The growth pattern of the city could be made more place-oriented: for example extrinsic values such as tourism that now take precedence could instigate the city life and the job market.[45] The relationship between city space and its users could also be strengthened as more place-oriented activities are inserted into urban spaces that are today under-used or unused. As such the community spirit could be resurrected, strengthening the container quality of the city space.

## CONCLUDING REMARKS

We find that Ecological Footprint, as a concept, could be used at policy making level to determine the scale and type of development, to empower communities and to accommodate their values into the centre of decision making, to establish grass root movements or supporting their existence in order to build up a closer rapport between the community and decision making. Furthermore, at planning level the concept controls the sprawl by declaring Environmental zones, urban precincts, etc, and integrating them in Master Plan/ Structure Plans, by screening new development, by promoting new economic activities, by promoting mix development yet without losing the perceived identity of the precinct rather than using the conventional zoning, and by turning cities into cultural diversities, and then at spatial development level to understand the particular Spatial Quality of the built environment, to design a spatial structure criteria, to develop concepts similar to the Urban Village, to promote the city spaces as a mixed used and socio-culturally diversified liveable containers, to share non-renewable resources such as water, energy, and above all the social responsibilities, Recommending spatial design tools such as veranda, courtyards, etc, to make the dwellings liveable and less expose to the hazardous environmental conditions.

Concept of Sustainable Building has been realised in many ancient civilisations. Sri Lankans, being Buddhists and Hindus respected the nature as the source of inspiration and their designs were like considered as meaningful extensions to nature. Some argue that they were an agro-based society and therefore were forced to respect the natural elements such as water, trees, etc. Their designs clearly show that their response to the location where the villages were built was more than this forced respect. Their responding to the nature was more to do with natural production of energy and respecting the energy cycle in order to lead a healthy and happy living condition. The expansion of the village was not preferred at the expense of the sustaining of the village society. The emerging of several modest villages closer to each other as spill-over and then forming of a town as the central location for trade and barter would be a notable solution to urban sprawl. The existence of such villages around the town centre is today reinterpreted as the establishment of cluster towns as a solution for the sprawl.

Traditional way of site selection for human settlements is an important next lesson for us to plan ecologically-fit living spaces. The villagers' dependence on the strength of the location brought in the limits of expansion, thus turning the village into a sustainable one. However, transformation of man's habitat from village to city and the resulted demands of the urban society cannot be met by merely following the said traditions. Since the development of technology and diversified economy were not desired by the rural society, the limitations of the village could be overcome. Their dependence on water and land also was helpful in controlling the growth. Yet, the new opportunities sought by the urban society caused the loss of control in the human habitat and its links with the environment. Our particular investigation of a living city, one can argue, cannot be inspired by the traditional village. However, such inspirations could bring parallels to the living city if we consider the city as the environment- not natural but built, thus considering the new changes as designing new buildings in nature. Site selection, controlled development in particular sites, they all can be readopted in the case of adding new functions in the used built forms.

Through development of urban design guides/ briefs in which the four basic principles: Solution grown from place: Making nature visible: Design with nature: Ecological accounting informs design, could be integrated. Moreover, they will result in the making of an informed society. We intended testing a form of retrofitting more than new developments so that these cities and towns would

[45]. Jensen-Verbeke (1995) notes how tourism activated socio-economics in Bruges.

become better living environments. They would grow without losing their own nature and culture, or without losing their bounded space. As a whole, the city will continually reflect the nature where the city is built, the man who came to that nature, society evolved by him to survive, and institutions, networks and values developed by that society to be a true Eco-community.[46]

It is noted that the Eco city would mark a true continuity of a culture by responding to the present as well as future demands. It will be a highly futuristic human setting that learns lessons of the past and project those lessons towards the future.

## REFERENCE


1. Berleant, Arnold (2000)- 'The Wilderness City: An Essay on Metaphorical Experience' in *Place and location* Kaija Lehari & Virve Sarpik (eds.), Tallinn: Academy of Arts
2. Edmonson, Susey (2008) *Ten Tips for Producing top quality research,* Boston: Pearson Allyn & Bacon
3. Ellawela, H (1969)- *Social History of Early Ceylon,* Colombo: Department of Cultural Affairs
4. Evans Gaeme (2001), *Cultural Planning: An Urban Renaissance,* London & New York: Routledge
5. Greed Clara H (ed.) (1999), *Social Town Planning,* London & New York: Routledge
6. Habermas Jurgen (1987), *The Theory of Communicative Action: The Critique of Functionalist Reason,* Cambridge: Polity Press
7. Harvey David (2000) *Spaces of Hope* Edinburgh: Edinburgh University Press
8. Jensen-Verbeke (1995), 'Involving People' in *Historic Cities and Sustainable Tourism,* Conference proceedings, Bath pp. 39-51
9. Larice, Michael, and MacDonald, Elizabeth (eds.) (2007) *The Urban Design Reader*, New York: Routlegde
10. Lefebvre Henry (1996), *Writing on Cities*, trans. & ed. by Eleonore Kofman & Elizabeth Lebas, Oxford: Blackwell
11. Mumford, Lewis (1938)- *The Culture of Cities,* New York: Harcourt Brace
12. Munasinghe Harsha (1991)- *Transformation of Colonial Urban Space in Sri Lanka,* Helsinki: Helsinki University of Technology
13. Munasinghe Harsha (2001) Aesthetics of Urban Space through Collaborative Urban Planning' in *Built Environment 3/01*
14. Munasinghe Harsha (2004) 'Ecological Housing in Helsinki: Case study of Viikki', *Sri Lanka Architect,* Journal of Sri Lanka Institute of Architects, 4/2004
15. PLACE3 (1997) Centre for Excellence for Sustainable Development, US Department of Energy
16. Scott, A (2000)- *The Cultural Economy of Cities,* London: Sage
17. Sennette Richard (1994) *Flesh and Stone: The Body and The City in Western Civilization,* New York: Norton
18. Thompson John B (1970), *Ideology and Modern Culture: Critical Social Theory in the era of Mass Communication,* Cambridge: Polity Press
19. Wijedasa, K.H.J. (1994)- *Towards Sustainable Growth: The Sri Lanka Experience,* Colombo: Central Environmental Authority
20. www.habitat.jgc.org/agenda21/


[46]. Doxiadis has noted five elements that make the city what it is: Nature, Man, Society, Networks and Institutions. We acknowledge his thinking in developing our urban design guides.

# USE OF RAINFALL DATA TO CALCULATE INCIDENT SOLAR RADIATION IN TROPICAL COUNTRIES

[1]S. Sendanayake, [2]M. T. R Jayasinghe

[1]Postgraduate Student, Department of Civil Engineering, University of Moratuwa, Moratuwa, Sri Lanka.
[1]E-mail: sisuru@civil.mrt.ac.lk
[1]Telephone: +94-11-2650567-8 (Ext.2021) ; Fax: + 94-11-2651216

[2]Professor, Department of Civil Engineering, University of Moratuwa, Moratuwa, Sri Lanka.
[2]E-mail: thishan@civil.mrt.ac.lk ; mtrjayasinghe@yahoo.com
[2]Telephone: +94-11-2650567-8 (Ext.2006 - 2001) ; Fax: + 94-11-2651216

**Abstract**: Determining the incident solar radiation for a given location is an important aspect of any solar related application. Though solar radiation data are available at weather stations, localized nature of solar radiation due to topographic and climatic parameters demands measured or calculated solar radiation values for a given location for accurate results. Many correlations have been developed over the past few decades yielding solar radiation values from various weather parameters such as daily sunshine duration, ambient temperature difference, relative humidity, cloud cover etc.

However, most of the weather data are practically difficult and costly to record hence requiring a simplistic approach to the issue. For any geographical location the cloud cover plays a major role in controlling the incident solar radiation. In tropical countries, where the climate is highly humid, cloud cover is closely related to rainfall. Therefore, day time rainfall data can be taken as representative of overcast and clear days, paving the way to calculate the clearness index, $K_T$ using equations developed on cloud cover data.

***Key Words**: Solar radiation, Rainfall, Cloud cover, Clearness index*

## 1. Introduction

Determining of average global solar radiation incident at a given location is usually carried out by long term direct measurement or using correlations developed through research using weather data. Of the two methods, long term direct measurement of solar radiation, other than that carried out at weather stations, are found to be low in accuracy due to cost of maintenance of equipment and requirement of skilled labor over an extended period of time. A relatively new development is the prediction of incident solar radiation using satellite technology, combining both the direct measurement and correlation methods, where collected data are simulated over a long period of time. However, the predicted values are found to be accurate only within a limited geographical region of 50 km radius from a given weather station (Cano) (11), hence rendering the values obtained from satellite technology useful only in gross calculations.

Correlations developed to predict solar radiation using weather parameters on the other hand are much more practical provided the relevant weather data are easily obtainable. Of the correlations developed, Angstrom (3) type is the most widely used with daily sunshine duration as the input. For tropical countries, the most widely used correlation is one developed by Black (8) based on Angstrom's correlation with regression coefficients of 0.28 and 0.47 generalized for the tropical belt $0^0 – 60^0$ N & S.

However, once again the long term measurement of daily sunshine duration accurately is a costly exercise requiring skilled labor, rendering it impractical for many remote locations. In Sri Lanka, several radiation correlations have been employed but a general radiation model which can be considered reliable for estimation of solar energy for the three climatic zones (wet, dry and intermediate) does not exist. As the incident terrestrial solar irradiation at a given location varying with geometrical parameters (such as latitude and altitude) and meteorological parameters (sunshine duration, relative humidity, ambient temperature and cloud cover amount), an approximate generalized model has to be selected from models developed for similar climatic conditions and validated for Sri Lanka identifying the parameters which most impact the outcome. It is also important to identify a model which will rely on easily obtainable data without using complex instrumentation and the resultant inaccuracies that can arise in measurements. As cloud cover and atmospheric turbidity having a major impact than the latitudinal effect on the incident solar radiation in the tropics, it is appropriate to develop clearness index $K_T$ based on sky conditions with the overcast and clear skies in the extremes (Bindi) (6). As rainfall is closely related to cloud cover in warm and humid tropical weather, this study attempts to develop a methodology to relate rainfall to incident solar radiation for a given location.

## 2. Correlations based on cloud cover data

The solar radiation that arrives at ground depends on the day of the year, the latitude of the location and on the atmospheric transmittance, also termed as the clearness index $K_T$. On reaching the earth's surface, the incoming radiation is partly reflected and partly absorbed. Net radiation, corresponding to the overall balance of absorbed solar radiation and long-wave exchange, is converted to the sum of sensible heat, latent heat and ground heat fluxes. During day time the earth's surface receives irradiative energy and both air and soil temperatures are expected to increase. At night, the surface loses energy by emitting radiation, especially during clear sky conditions. Hence, a clear day is expected to be generally characterized by an increased difference between night and day temperatures. On overcast days, the cloudiness reduces the incoming radiation during day time and also reduces the outgoing radiation at night. The difference between night and day temperatures is therefore expected to be reduced. Accordingly, the difference between the thermal ranges of two consecutive days is expected to be related to the difference in the mean sky transmittance (mean value for $K_T$) of the same two days (Bindi) (6). However, in the tropical countries this phenomenon is not so profound due to frequent convective cloud movements trapping heat into the atmosphere making the temperature differences between night and day minimal. Therefore, a closer relationship between solar radiation and cloud cover exists in the tropics.

As the cloud formation over tropical islands with a relatively small land mass is limited, most of the rain events occur from low pressure atmospheric conditions in the surrounding ocean. It is also observed that most of the rain events in Sri Lanka occur from Low-family clouds (Nimbostratus and Altostratus) and therefore it can be assumed that rain events (rainfall > 0.3 mm per day) in tropical islands occur on overcast days. Conversely non-rainy days can be assumed to be clear sky days. Further, research conducted in Pnom Penn in tropical Asia (Bindi) (6) has shown that the difference in incident solar radiation on rainy and clear days is lower than in high latitude countries. This fact is strengthened by the low difference of night and day time temperatures in the tropics.

### *2.1 Predicting mean sky transmittance of clear days $(K_T)_C$*

The solar radiation that reaches the earth's surface on a clear day is a function of the solar constant, of the sine of the solar elevation, the relative air mass and the turbidity factor of the air mass. Turbidity, in turn, depends on the transmittance due to molecular scatter (Rayleigh), to ozone absorption, to the uniformly mixed gases, to water vapor and to aerosols (Justus and Paris) (3).

If a constant air pressure of 1013 hPa at 0 m elevation is assumed, the relative air mass is approximately calculated for given location, day of the year and time of day as the reciprocal of the sine of solar height. The turbidity factor (*TI*) is normally calculated from measured incoming radiation by means of Linke's method but it can be also estimated on the basis of an existing correlation between the water content of the atmosphere, i.e. its perceptible water (w), and the turbidity coefficient (β) by means of the empirical equation developed by Dogniaux and Lemoine (18)

$$TI = \{(\alpha+85)/(39.5e^{-w} + 47.4) + 0.1\} + (16 + 0.22w)\,\beta \qquad (1)$$

Where α = solar elevation (in degrees)

In absence of direct observations, the parameters w and β of equation 1 can be derived from the following classification of different types of radiation climates by neglecting the effect on these values of air mass conditions:

| | |
|---|---|
| - polar and desert climates (dry air) | w = 0.5 to 1 |
| - temperate climates | w = 2 to 4 |
| - tropical climates (humid air) | w = 5 |
| - rural site | β = 0.05 |
| - urban site | β = 0.1 |
| - industrial site | β = 0.2 |

(Dogniaux and Lemoine) (18)

When the value of *TI* is estimated for a given location for a given day of the year and for a given solar elevation, the sky transmittance of a clear sky $(K_T)_C$ is calculated, according to the modified Beer's law equation (Kasten & Czeplak) (33):

$$(K_T)_{Ch} = 0.83e^{(-0.026TI/\sin h)} \qquad (2)$$

Where $(K_T)_{Ch}$ is the sky transmittance calculated for the solar elevation h. The mean daily values of $(K_T)_C$ can be found by integrating and averaging $(K_T)_{Ch}$ over the length of the day.

### *2.2 Predicting mean sky transmittance of overcast days $(K_T)_O$*

The sky transmittance on an overcast day mainly depends on the thickness and type of clouds and on the sun elevation (Lumb) (40). It is known that high, middle and low clouds attenuate the solar radiation in different ways (Haurwitz (24), Bennet (5), Kimura and Stephenson (35)). A distinction between the fraction of total sky cover (TSC), often recorded in synoptic weather stations, and the fraction of cloud cover (cc), that takes into account the attenuation effect different cloud type groups, was made by Turner & Abdullaziz (65). The relationship between these two fractions is given as:

cc = TSC for low clouds, middle clouds or low and middle clouds
cc = 0.5 TSC for high clouds
cc = TSC – 0.5(Amount of high clouds) for mixed clouds

Since the model developed sets the condition that the overcast days are also rainy days, the rainfall probability of a given day is to some extent related to the cloud type being maximum for Low-Family clouds (Nimbostratus and Stratocumulus) for Middle clouds (Altocumulus and Altostratus) and for Vertical clouds (Cumulus and Cumulonimbus). Hence, the cloud cover fraction (cc) on days selected as overcast by the model is assumed to be equal to the maximum sky cover fraction (cc = 1)

Turner & Abdulaziz (65) developed an empirical equation to calculate the sky transmittance of overcast days as a function of the solar elevation and the cloud cover fraction. The equation has the following form:

$$(K_T)_{Oh} = a + b(cc)^2 \mathrm{Sinh} + c\,(cc)^2 + d\mathrm{Sinh} \qquad (3)$$

Where, $(K_T)_{Oh}$ is the sky transmittance of an overcast day calculated for the solar elevation h and a, b, c and d are regression coefficients calculated for different solar elevation (Table 1). The value of the mean daily sky transmittance $(K_T)_O$ is calculated by integrating over the day and averaging.

Table 1: *Regression for different solar heights*

| *Range of* α | *a* | *b* | *c* | d |
|---|---|---|---|---|
| $0^0 \le \alpha \le 20^0$ | 0.3080 | -1.165 | -0.0586 | 1.0743 |
| $20^0 \le \alpha \le 40^0$ | 0.5695 | -0.1065 | -0.4755 | 0.2809 |
| $40^0 \le \alpha \le 60^0$ | 0.7862 | 0.2736 | -0.6943 | -0.0467 |
| $\alpha > 60^0$ | 0.6423 | 0.9109 | -1.2873 | 0.1222 |

## 3. Methodology and Calculations

In this study, Sri Lanka is taken as representing a tropical country and daily rainfall data are collected at four locations in close proximity to weather stations. Locations are selected to represent the main climatic characteristics of the country with Colombo representing the coastal wet region, Nuwara Eliya the high altitude wet region, Anuradhapura and Hambantota representing the dry region. The monthly average daily radiation values for each location are obtained from SWERA TMY data base, calculated from direct measurement of solar irradiance, adjusted for system inaccuracies through the use of correlations based on weather parameters.

The clearness indexes for clear and overcast days are calculated as follows;

From equations 1 to 3 and taking w = 5 representing the tropical humid conditions and $\beta = 0.1$ to represent the urban nature of the weather station location clearness index for a clear day $(K_T)_C$ is calculated to be 0.68. Taking cc = 1 for low and middle clouds which are the most prevalent and rain causing in Sri Lanka, clearness index for an overcast day $(K_T)_O$ is calculated to be 0.28. The clearness index, $K_T$ was calculated using equations 1, 2 and 3 for all locations using rainfall data where a rainy day is considered when rainfall in 24 hours is greater than 0.3 mm. Using the calculated clearness index values for overcast and clear days the monthly average daily solar radiation for a particular month can be obtained by calculating $K_T$ by simply averaging corresponding clearness index values for rainy and non rainy days for the respective month. However, it can be seen that much accurate predictions can be made if the data on the number of rainy days per month can be calculated over a period of minimum 5 years. Chart 1 to 4 show monthly averaged daily values of incident solar radiation $G_{m\text{-}h}$ calculated with monthly average $K_T$ values (RF model) averaged over 5 years against monthly average daily solar radiation values from SWERA TMY data for the four stations. The Charts also show the $G_{m\text{-}h}$ for $K_T$ values calculated using RF model with data obtained during a single year (2008).

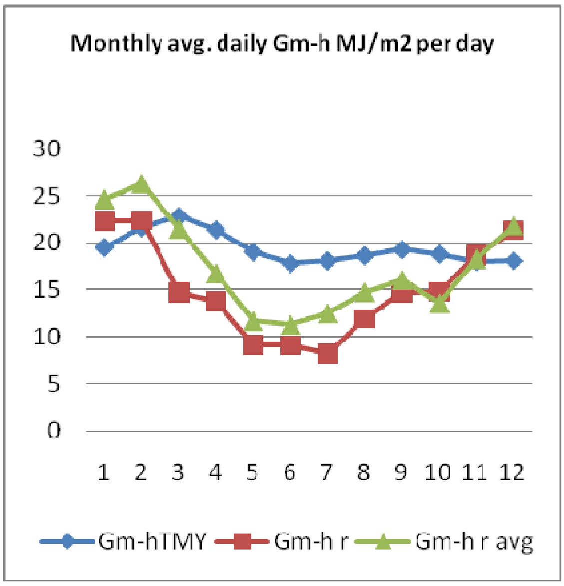



Chart 1: *Comparison of GSR(RF), Col.*

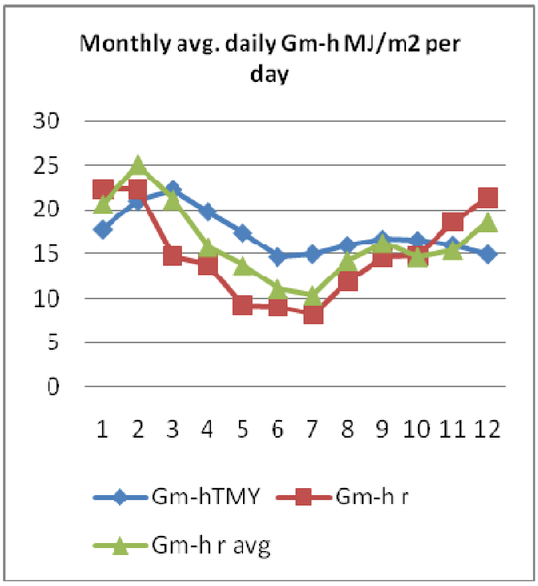



Chart 2: *Comparison of GSR(RF), NE.*

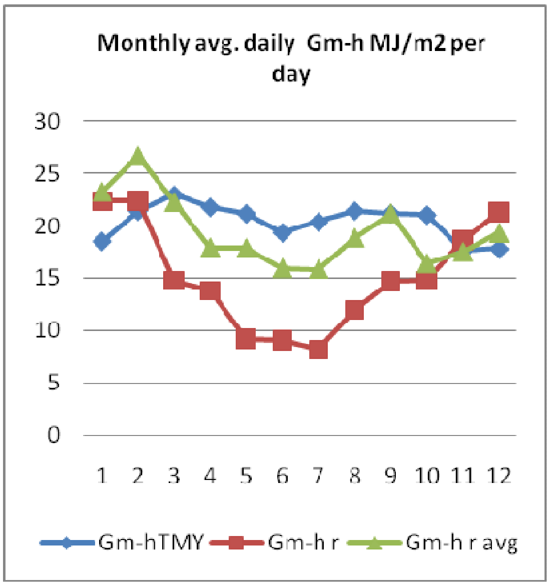



Chart 3: *Comparison of GSR(RF), A'pura*

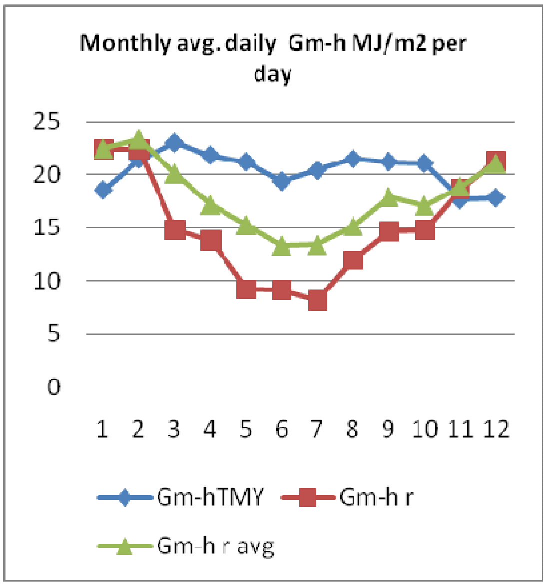



Chart 4: *Comparison of GSR(RF), H'tota*

In the Charts 1 to 4 $G_{m-h}$TMY, $G_{m-h}$ r and $G_{m-h}$ r avg denotes the monthly average daily global radiation on a horizontal surface obtained from SWERA TMY data base, average rainfall data for one year (RF model) and 5 rainfall data averaged over a period of 5 years (ARF model).

Table 2: *Percentage deviation of Gm-h (ARF) from corresponding SWERA TMY data*

| *Station* | *Jan* | *Feb* | *Mar* | *Apr* | *May* | *Jun* | *Jul* | *Aug* | *Sep* | *Oct* | *Nov* | *Dec* |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Colombo | -26.1 | -21.3 | 5.9 | 21.8 | 38.6 | 36.5 | 30.8 | 20.7 | 16.7 | 27.9 | -1.4 | -20.7 |
| N'Eliya | -15.9 | -19.4 | 5.0 | 20.2 | 20.9 | 24.5 | 30.9 | 10.7 | 2.6 | 11.2 | 3.6 | -24.1 |
| A'pura | -25.2 | -24.3 | 3.14 | 17.9 | 15.8 | 17.7 | 22.2 | 12.2 | 0.18 | 22.1 | 0.92 | -8.2 |
| H'tota | -8.3 | 1.48 | 15.1 | 20.5 | 27.0 | 30.0 | 32.3 | 29.2 | 18.4 | 17.8 | -1.4 | -10.4 |

Charts 6 and 7 show the monthly average daily global radiation for the four locations obtained from SWERA data and ARF model indicating that in both cases sites located in the wet region displaying lower radiation levels after the end of the North- East monsoon period, i.e. from March to October.

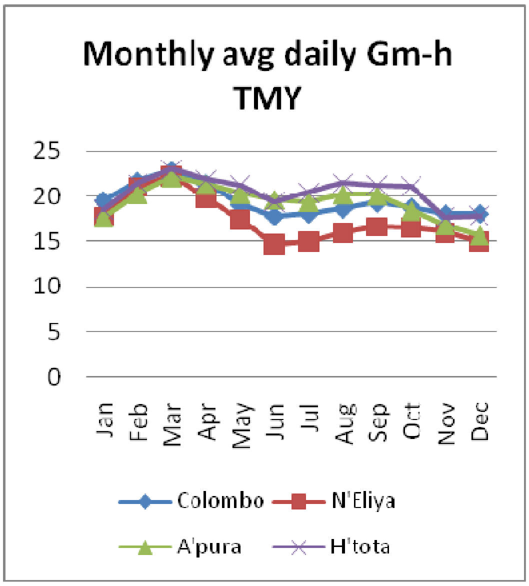



Chart 6: *Gm-h (SWERA) for all locations*

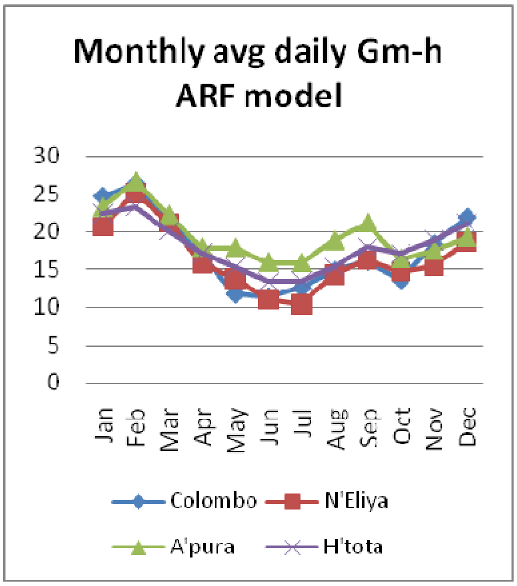



Chart 7: *Gm-h (ARF) for all locations*

## 4. Discussion

It can be seen from Table 2 that the percentage deviation of solar radiation values obtained through rainfall data from that of SWERA data are displaying a similar pattern over a calendar year for all four locations. Further, from Charts 1-4, it can be seen that the solar radiation values obtained through average rainfall (ARF) are more compatible with SWERA data, indicating the importance of collecting rainfall data over a longer period of time for more accuracy. However, as the shape of the graphs obtained for solar radiation values through ARF model for all four locations show consistency, it is reasonable to assume that the particular shape is due more to the amount and duration of rainfall and thus should be compatible with the type and extent of cloud cover observed.

The model can be further improved by closely examining the cloud formation patterns, wind directions and seasonal variations of weather in Sri Lanka. Though Sri Lanka is located close to the equator, as a country located in the northern- hemisphere it still experiences summer and wintry conditions albeit mildly. As such from December to February the day length is 3% shorter than the average of 12 hours and humidity is relatively low leading to higher percentage of high clouds formation in the cooler upper atmosphere. These high clouds, though mostly producing no rain or insignificant rainfall as trace precipitations or rain events less than 1 mm, still prevent significant amount of solar radiation penetration particularly during morning hours. Therefore when calculating the number of days in which rainfall events occur for the RF model, trace precipitation events as well as the rainfall events less than 1mm should also be taken into account during December – February period. The summer period from June to August on the other hand is 3% longer in day length from the average and the south-westerly wind with high humidity forms a higher percentage of isolated low and middle clouds, though causing minor rain events not blocking solar penetration for a prolonged period of time. Therefore, when the rain event is less than 1 mm per day, such days can be generally considered as clear days with considerable accuracy. As such, during the period from June to August only the days that produce more than 1mm of rain per day can be counted as rainy days for the RF model. For the in-between seasons precipitations more than 0.3 mm per day can be considered as rain events.

Further, as Sri Lanka is an island in the tropics, it is observed that more than 50% of the rain events during March to October occuring in the night time due to increased ground temperatures and the resultant wind direction from ocean the to inland, causing more rain events in the night and early morning. Therefore a considerable improvement in the RF model can be envisage if only the day time rain events are considered. A further improvement can be envisaged if the adjusted RF model can be provided with data from a longer historical time series of 5 or 10 years of day time rain events.

In the calculations, $K_T$ values are taken at either overcast or clear sky conditions. However, there exist days where the cloud cover is partial or prevailing for a particular period of time during the day, necessitating in-between values for $K_T$ using an interpolative technique. If such an interpolative method can be developed to define $K_T$ values for days in between clear and overcast days, the variations from measured radiation data can be minimized enabling the ARF method to be widely used.

## References


1. Ahrens C.D. (1998), "Esential of meteorology, an introduction to the atmosphere", 2nd ed. New York: Wadsworth Publishing Company.

2. Allen, R.G. (1989), "A penman for all seasons," J.Irrig. and Drain Engr., ASCE, 112(4): 349-368.

3. Angstrom, A., (1924), "Solar and terrestrial radiation," Quart., J.R. Met. Soc., 50, 121-126.

4. Atwater, M. A., Ball, J.T. (1978), "A numerical solar radiation model based on standard meteorological observations," Solar Energy 21: 163-170.

5. Bennet, I. (1969), "Correlation of daily insolation with daily total sky cover, opaque sky cover and percentage of possible sunshine," Solar Energy 13: 391-393.

6. Bindi, M., Miglietta, F., Maracchi, G. (1988), "Estimating daily global radiation from air temperature and rainfall measurements," Boll. Geof., VI: 141-147.

7. Bird R., Hulstrom R.L. (1981), "Review evaluation and improvement of direct irradiance model," J. Sol Energy Eng,103:183.

8. Black, J. N., Bonython, C. W., Prescott, G. A. (1954), "Solar radiation and the duration of sunshine," Q.J.R. Meteorol. Soc. 80: 231-235.

9. Blaney, H.F, Criddle, W.D. (1950), "Determining water requirements in irrigated areas from climatological and irrigation data," USDA/SCS-TP 96.

10. Bristow, K. L., Campbell, G. S. (1984), "On the relationship between incoming solar radiation and daily maximum and minimum temperature," Agric. For. Meteorol. 31: 159-166.

11. Cano, D., Monget, J. M., Albuisson, M., Guillard, H., Regas, N., Wald, L. (1986) "A method for the determination of the global solar radiation from meteorological satellite data," Solar Energy 37: 31-39.

12. Cengiz, H.S., Gregory, J. M., Sebaugh, J. L. (1981), "Solar radiation prediction from other climatic variables," Trans. A.S.A.E. 24(5):1269-1272.

13. Collins, W.D. et al. (2001), "Simulating aerosols using a chemical transport model with assimilation of satellite aerosol retrievals: Methodology for INDOEX," J. Geophys. Res., 106D, 7313-36.

14. Collins, W.D. et al. (2002), "Simulating of aerosol distributions and radiative forcing for INDOEX: Regional climate impacts," J. Geophys. Res., 107D, 8028.

15. Daneshyar M. (1978), "Solar radiation statistics for Iran," Sol Energy,21: 345-9.

16. DeBoer D.W., Olson D.I., Skonard C.J. (2005), "Evaluation of solar radiation estimation procedures for eastern South Dakota," J Proceedings of the South Dakota Academy of Science, Vol.84: 265-275.

17. Diak, G.R., Gautier, C., Masse, S. (1982), "An operational system for mapping insolation from GOES satellite data," Solar Energy 28: 371-376.

18. Dogniaux, R., Lemoine, M. (1982), "Classification of radiation sites in terms of different indices of atmospheric transparency in Palz. W.," Solar radiation data. Series F. Vol.2, Reidel Publishing Company, Dordrecht, p. 94-105.

19. Frulla, L. A., Gagliardini, D. A., Grossi, G., Lopardo, R., Tarpley, J. D. (1988),"Incident solar radiation on Argentina from the geostationary satellite GOES: comparison with ground measurements," Solar Energy 41: 61-69.

20. Gregory S. (1993), "Statistical methods and geographer," New York: Wiley.

21. Hammer, A., Heinemann, D., Hoyer, C., Kuhlemann, R., Lorenz, E., Muller, R., Beyer, H. G. (2003), "Solar energy assessment using remote sensing technologies," Remote Sens. Environ., 86, 423-433.

22. Hargreaves, G.H, Samani, Z.A. (1985), "Reference crop evapotranspiration from temperature," Transaction of ASAE 1(2): 96-99.

23. Hargreaves, G.H. (1994), "Simplified coefficients for estimating monthly solar radiation in North America and Europe," departmental paper, dept. of boil. and Irrig. Engrg., Utah State University, Logan, Utah.

24. Haurwitz, B. (1948), "Insolation in relation to cloud type," J. Meteorol. 5: 110-113.

25. Hay, J. E., Suckling, P. W. (1979), "An assessment of the networks for measuring and modeling solar radiation in British Columbia and adjacent areas of Western Canada," Canadian Geogr., 23: 222-238.

26. Hoogenboom G. (2000), "Contribution of agro-meteorology to the simulation of crop production and its applications," J agriculture and forest meteorology, 103: 137-157.

27. Iqbal M. (1983), "An introduction to solar radiation," London: Academic Press.

28. Iziomon, M. G., Mayer, H. (2002), "Assessment of some global solar radiation parameterizations," Atmos. Solar Terres. Phys., 64, 1631-1643.

29. Jafarpur K., Yaghoubi M.A. (1989), "Solar radiation for shiraz, Iran," Sol. Wind Technol. 6(2): 177-179.

30. Jagtap, S.S. (1991), "Spatial pattern of reference evapotranspiration in Africa," ASAE paper no.91-2644, ASAE national meeting, Chicago, IL.

31. Justus, C. G., Paris, M.V. (1985). "A model for solar spectral irradiance and radiation at the bottom and top of a cloudless atmosphere," J. Appl. Meteorol, 24(3): 193-205.

32. Kandirmaz, H. M., Yegingil, L., Pestemalci, V., Emrahoglu, N. (2004), "Daily global solar radiation mapping of Turkey using Meteosat satellite data," Int.l. J. Remote Sens., 25: 2159-2168.

33. Kasten, F., Czeplak, G. (1980), "Solar and terrestrial radiation dependent on the amount and type of cloud," Solar Energy, 24: 177-189.

34. Keulan, H., Van, S., Van, H. D. J. (1986), "Meteorological data," Wageningen, p. 35-46.

35. Kimura, K., Stephenson, D. G.,(1969),"Solar radiation on cloudy days," ASHRAE Trans., 75: 227-234.

36. Klein S.A. (1977), "Calculation of monthly average insolation on tilted surfaces," Sol Energy, 43(3): 153-68.

37. Kondo J, Nakamura T, Yamazaki T. (1991), "Estimation of the solar and downward atmospheric radiation," Tenki, 38: 41-48.

38. Li, F., Ramanathan, V. (2002), "Winter to summer monsoon variation of aerosol optical depth over the tropical Indian ocean," J. Geophys. Res., 107D, doi: 10.1029.

39. Li, Z., Garand, L. (1994), "Estimation of surface albedo from space: A parameterization of global application," Journal of Geophysical Research, Vol. 99, pp.8335-8350.

40. Lumb, F. E. (1964), "The influence of cloud on hourly amounts of total solar radiation at sea surface,"Q.J.R. Meteorol, Soc. 90:383.

41. Magal, B. S. (1993), "Solar Power Engineering," Tata McGraw-Hill Publishers.

42. Mahmoud, E., Nather, H. (2003), ”Renewable energy and sustainable developments in Egypt, Photovoltaic water pumping in remote areas,” Applied Energy, 74 Elsevier.

43. Marion, W., George, R. (2001), “Calculation of solar radiation using a methodology with worldwide potential,” Solar Energy, 71, 4: 275-283.

44. Marion, W., Urban, K. (1995), Users manual for TMY2s-typical meteorological years derived from the 1961-1990 National Solar Radiation Data Base, NREL/SP-463-7668. National Renewable Energy Laboratory, Golden, CO.

45. Maxwell, E. L. (1998), “METSTAT- The solar radiation model used in the production of the NSRDB,” Solar Energy, 62.4: 263-279.

46. McCaskill, M. R. (1990), “ Prediction of solar radiation from rain day information using regionally stable coefficients,” Agric. For. Meteorol., 51:247-255.

47. Meinel A.B., Meinel M.P. (1977), “Applied solar energy,” New York: Addison-Wesly Publ.inc.

48. Nunez, M. (1987), “A satellite-based solar energy monitoring system for Tasmania, Australia,” Solar Energy, 39: 439-444.

49. Perenc, K., Jozsef, B., Marianna, V. (2002), “Changes in solar radiation energy and its relation to monthly average temperature,” Acto Montanistica Slovaca J.,7: 164-166.

50. Plantico, M.S., Lott, J.N. (1995), “Foreign weather data servicing at NCDC,” ASHRAE Transactions, Vol 1, Part 1, pp. 484-490.

51. Polo, J., Zarzalejo, L. F., Ramirez, L., Espinar, B. (2006), “Iterative filtering of ground data for qualifying statistical models for solar irradiance estimation from satellite data,” Solar Energy, 80: 240-247.

52. Randel, D.L, Thomas, H., Vonder, H., Mark, A. (1996), “A new global water vapor dataset,” Bull.Amer.Meteor.Soc., 77(6):1233-1246.

53. Richardson, C.W, (1981), “Stochastic simulation of daily precipitation, temperature and solar radiation,” water Resour. Res., 17(1): 182-190.

54. Rimoczi-Paal, A. (1983), “Determination of global radiation from satellite pictures and meteorological data,” Solar Energy, 31: 79-84.

55. Samani, Z., Bawazir, A. S., Bleiweiss, M.,Sskaggs, R., Tran, V.D. (2007), “Estimating daily net radiation over vegetation canopy through remote sensing and climatic data,” J. Irrig. Drainage Eng., ASCE, 133: 291-297.

56. Samimi J. (1994), “Estimation of height dependent solar irradiation and application to the solar climate of Iran,” Sol Energy, 52: 5401-9.

57. Samuel, T.D.M.A. (1991), “Estimation of Global radiation for Sri Lanka,” Solar Energy, 47(5): 333-337.

58. Samuel, T.D.M.A., Srikanthan, R. (1982), “Solar radiation estimation for Sri Lanka,” transactions of the Institution of Engineers, Sri Lanka, pp.15-19.

59. Spencer J.W. (1971), “Fourier series representation of the position of the sun,” Search 2(s): 172.

60. Squires, M.F. (1995), “The demographics of worldwide weather data,” ASHRAE Transactions, Vol.1, Part 1, pp. 484-490.

61. Suckling, P.W. (1985), “Estimating daily solar radiation values in selected mid-latitude regions by extrapolating measurements from nearby stations,” Solar Energy, 35: 491-495.

62. Tabata, S. (1964), “Insolation in relation to cloud amount and sun’s altitude,” In. Yohida K. (ed:) Studies on oceanography. University of Tokyo., p.202-210.

63. Tarpley, J.D. (1979), "Estimating incident solar radiation at the surface from geostationary satellite data," J. Appl. Meteorol., 18: 1172-1181.

64. Tiwari, G.N. (2002), "Solar Energy: Fundamentals, Design, Modeling and applications," Alpha Science Publishers.

65. Turner, W.D., Abdulaziz, M. (1984), "The estimation of hourly global solar radiation using a cloud cover model developed at Blytheville, Arkansas," J. Appl. Meteorol., 23: 781-786.

## Acknowledgements

The technical staff of the Department of Civil Engineering, Messers S.P.Madanayake, S.L Kapuruge assisted this project at the concept development stage.

## About the Authors

**S. SENDANAYAKE,** B.Sc. Hons. Moratuwa., M.Sc. Moratuwa is currently reading for his Ph.D. at the Department of Civil Engineering, University of Moratuwa.

**M. T. R. JAYASINGHE,** B.Sc. Hons. Moratuwa., Ph.D. Cambridge., MIE (SL), C.Eng. is a Professor at the Department of Civil Engineering, University of Moratuwa. His research interests are in the areas of tall buildings, energy efficiency, sustainable development and disaster resistant structures.

# DEVELOPMENT OF DARRIEUS-TYPE VERTICAL AXIS WIND TURBINE FOR STAND-ALONE APPLICATIONS

[1]Samaraweera K.K.M.N.P., [2]Pathirathna K.A.B., [3]De Silva H.E.D, [4]Sugathapala A.G.T.

Department of Mechanical Engineering, University of Moratuwa, Moratuwa, Sri Lanka
[1]*[U.G.Student] nalaka@mech.mrt.ac.lk*, [2]*[U.G.Student] apathirathna@gmail.com*,
[3]*[U.G.Student] dushan569@gmail.com*, [4]*[Senior Lecturer] thusitha@mech.mrt.ac.lk*

Corresponding Author: K.K.M.N.P. Samaraweera (0094)718366763, Department of Mechanical Engineering, University of Moratuwa, Moratuwa, Sri Lanka. Email: nalaka@mech.mrt.ac.lk

**Abstract**

A theoretical model for the design and performance simulation of Darrieus-type vertical axis stand alone wind turbine for small scale energy applications was developed. The model is based on application of momentum theory and blade element theory to multiple stream-tubes. Software was developed to solve the resulting non-linear equations for the flow-field. Results were used to analyze the effects of blade profile, rotor solidity, Reynolds number and aspect ratio on the maximum power and torque coefficients, optimum tip speed ratio, and ability to self start, which lead to design of optimum rotor configurations.

***Keywords:*** *Wind energy, Vertical axis wind turbine, Darrieus rotor, multiple stream-tube*

**Nomenclature**

| | |
|---|---|
| $A$ | Swept area |
| $A_f$ | Frontal area of the struts |
| $AR$ | Aspect ratio |
| $a$ | Induction factor |
| $c$ | Chord length |
| $C_D$ | Infinite span drag coefficient |
| $C_{Dmax}$ | Finite span aerofoil maximum drag coefficient |
| $C_L$ | Infinite span lift coefficient |
| $C_p$ | Power coefficient |
| $C_T$ | Thrust coefficient |
| $D$ | Rotor diameter |
| $h$ | Blade height |
| $N$ | Number of blades |
| $n$ | Number of support arms |
| R | Radius of the rotor |
| $Re$ | Reynolds number |
| $U$ | Ambient air velocity |
| $U'$ | Air velocity enters to the rotor |
| $\alpha$ | Angle of attack |
| $\Delta\theta$ | Magnitude of a stream-tube |
| $\varepsilon$ | Surface roughness |
| $\theta$ | Azimuth angle |
| $\lambda$ | Tip speed ratio |
| $\rho$ | Air density |
| $\sigma$ | Solidity |
| $\omega$ | Angular velocity |

**Abbreviations**

| | |
|---|---|
| HAWT | Horizontal Axis Wind Turbine |
| MSM | Multiple Stream-tube Model |
| NACA | National Advisory Committee of Aeronautics |
| TSR | Tip Speed Ratio |
| VAWT | Vertical Axis Wind Turbine |

## 1. Introduction

In the context of harnessing resources of wind energy, decentralized stand alone systems have unique benefits. Such systems could be a more viable option in rural areas where grid electricity cannot be implemented due to lack of infrastructure. Moreover, in an interruption or a breakdown of the grid, stand alone systems could act as a backup. In un-electrified areas, basic electric energy needs are usually obtained by batteries, which are charged by grid-electricity available in city centers. Therefore stand alone systems could play an important role as an alternative and cost effective source of battery charging.

Within the broad context of harnessing energy from wind, wind turbine rotors could be considered as the key component. Basically, based on the driving force, they can be divided as lift driven and drag driven. They are also categorized based on orientation of the axis of rotation as vertical axis and horizontal axis turbines (VAWT and HAWT, respectively), and both of which have intrinsic advantages and drawbacks. In the category of lift driven, VAWTs consist of variety of rotor configurations such as troposkine, straight bladed (or cylindrical) Darrieus, delta Darrieus, etc. VAWTs are Omni directional and, unlike HAWTs, variations of wind direction do not influence the performance. Therefore yawing mechanisms are not needed and the rotor could operate in a gust wind without much efficiency reduction. One disadvantage of lift driven VAWT is its difficulty of self starting. However there are some successful methods proposed to improve this situation. Normally VAWTs are operated in low angular velocities comparing to HAWTs and therefore aerodynamic noise is less. Further, the electrical generator load mountings are easy in VAWTs since the end of its axis is near to the ground level. Nevertheless VAWT has inherent difficulty in theoretical modeling of its performances and more research and developments are needed on this aspect.

## 2. Performance Characteristics of VAWT Rotors

Several parameters should be considered in designing a wind turbine rotor for a particular type of application. These parameters include solidity of the rotor, length, chord and aspect ratio of the blades, diameter of the rotor, number of blades, surface roughness of the blade, operational range of Reynolds number, blade profile, etc. (see Equation 1). As a result, modeling and design of a rotor become difficult. Moreover, as the requirements differ depend on the type of application, some general design criteria have been established such as high power coefficient, wide range of operation, high self-starting capability, reliability, cost effectiveness, strength and rigidity, optimum rotor inertia etc. One of the difficulties of matching the performance characteristics with the design requirements is the existence of paradox among some design requirements such as high power coefficient versus high self-starting capability, strength and rigidity versus low rotor inertia, reliability versus cost etc. Major challenges in the design of wind turbine rotors include the complexity in the relationships between the performance parameters and the lack of availability of aerofoil data. In particular, the aerofoil of the blade section is subjected a wider range of angle of attack during the operation and aerofoil data is usually available for limited range near the optimum angle of attack. However, a satisfactory performance model could be derived with carefully selection of appropriate techniques and reasonable assumptions.

As there are several parameters involved in the flow around wind rotors, basic approach in theoretical modeling is to use the concept of dimensional analysis. This technique simplifies the initial functional relationship to a lesser number of parameters appeared in non-dimensional forms. For example, the power output of the rotor $P_{rotor}$ could be expressed in non-dimensional form as the power coefficient $C_P$ in the form

$$Cp = g\ (\sigma, c/h, c/D, blade\ profile, \varepsilon, Re, \lambda), \qquad (1)$$

Where $C_P = P/\ (0.5\ \rho A U^3)$ is the power coefficient, $P$ is the power, $\rho$ is the air density, $A$ is the frontal area of the rotor, $U$ is the ambient air velocity. $\sigma = Nc/D$ is the rotor solidity, $N$ is the number of blades and c is the chord length. $D$=2$R$ is the rotor diameter, $\varepsilon$ is the relative roughness of the blade surface, $Re$ is the Reynolds number, $\lambda = R\omega/U$ is the tip-speed ratio, $\omega$ is the angular velocity of the rotor, $AR = h/c$ is the aspect ratio, $h$ is the blade length. This equation could be further simplified for the case of relatively high $Re$ as

$$Cp = g\,(\sigma, c/h, c/D, blade\,profile, \varepsilon, \lambda) \quad \ldots\ldots(2)$$

The torque coefficient of the rotor could be derived from above relationships in the form

$$C_T = C_P/\lambda \quad \ldots\ldots(3)$$

Usually the performance characteristics of a wind turbine rotor are represented in non-dimensional form as variation of power and torque coefficients with tip speed ratio at specified values of the other governing parameters. Note that the effect of blade profile on the performance is basically through the variation lift and drag coefficients $C_L$ and $C_D$ with angle of attack $\alpha$. In other words, $C_L$ and $C_D$ depend on the blade profile and $\alpha$. Based on experimental data, such relationships could be obtained in the form

$$C_L = lift\,force/(0.5\,\rho A U^2) = h_1\,(blade\,profile, \alpha)$$

$$C_D = drag\,force/(0.5\,\rho A U^2) = h_2(blade\,profile, \alpha),$$

for relatively high $Re$.

The above relationships basically represent the performance characteristics of a wind turbine rotor.

## 3. Theoretical Modeling of VAWT Rotors

The aim of a theoretical model is to convert the qualitative functional relationships given in expressions (1), (2) and (3) to quantifiable ones. In the present study, the momentum theory (which considers the rate of change of momentum of the flow in relation to the force acting on the fluid as it passes through the rotor) and the blade element theory (which considers the forces acting on the blades of the rotor in relation to the flow and geometrical characteristics) are applied to a suitably selected elementary flow region around the rotor to quantify the above functional relationships. Among the approaches available, the concept of multiple stream-tubes is employed in the present study, as it is capable of capturing the key design parameters, such as $D$, $N$, $AR$, $\sigma$, $Re$, blade profile, etc. .. This method is described in the following sub-section Software was developed to solve the governing equations derived from momentum theory and blade element theory for selected set of appropriate conditions and the output results were represented graphically. The two set of basic curves, $C_P$ vs. $\lambda$ and $C_T$ vs. $\lambda$, are in non-dimensional form and therefore applicable to series of geometrically similar turbine rotors. But geometrical similarity is only a necessary condition but not a sufficient condition due to the variation of $Re$. Therefore the effects of $Re$ should also be investigated.

The main input parameters of the theoretical model are free stream air velocity, density of air, viscosity of air, number of blades, diameter of rotor, height of rotor, blade profile, chord length and blade pitch angle. Further effect of blade profile is analyzed by considering lift and drag data relevant to four NACA 4 digits aerofoils. These are NACA 0012, NACA 0015, NACA 0018 and NACA 0021.

### Implementation of Multiple streamtube model (MSM)

In this method, flow through the entire rotor is separated into adjacent, aerodynamically independent sufficiently high number of parallel stream-tubes (refer Figure 1). This is a better approach for the non uniformity of inflow. It was proved that total decrement of flow velocity in upstream and downstream is same as proved by Bernoulli theorem in the Betz limit. Even though some of the assumptions behind the MSM are contradicted the basic fluid dynamic laws, it simplifies the complex flow pattern of the VAWT and thus makes logical mathematical model for VAWT (Paraschivoiu 2002). This method assumes that cross section of each stream-tube remains unchanged. Path of the inflow is assumed to be a straight line and air velocity of each stream-tube is supposed to be decreased before entering to the upstream circular path. Furthermore on leaving the downstream circular path, velocity of the air flow decreases and settles down in far away. Another assumption used is that the velocity does not change while passing through the circular path (Strickland 1975). Firstly the momentum theory is applied for each stream-tube and dimensionless thrust coefficient is obtained as.

$$C_{thrust} = \frac{momentum\ loss\ rate}{\frac{1}{2}\rho U^2(\mathrm{hR}\Delta\mathrm{esine})} = 4\frac{U'}{U}\left(1-\frac{U'}{U}\right) = 4a(1-a)$$

where $a = U'/U$ is the induction factor $U$ is free wind velocity and $U'$ is air velocity enters to the rotor. This equation is only valid when $a$ is less than 0.5. When $a$ exceeds 0.5, Glauert empirical formula is used. For $0.4 < a < 1.0$

$$C_{thrust} = \frac{26}{15}a + \frac{4}{15}$$

Secondly, the blade element theory is applied to obtain another expression for thrust coefficient and these two set of equations are solved by a root finding algorithm to obtain induction factor for each stream-tube. This leads to the required expressions for torque and power coefficients.

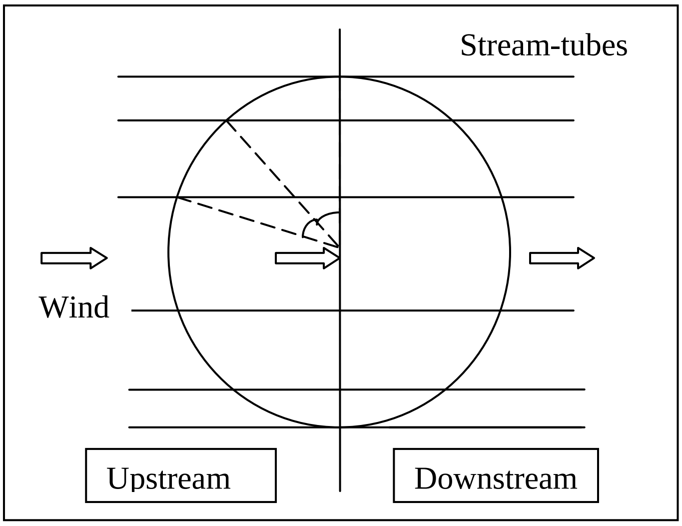



**Figure 1** - *A systematic view of multiple stream-tube model*

<u>Finite span effect and Strut losses</u>

To evaluate the finite span effect, modifications and corrections are incorporated to the aerofoil drag and lift coefficients that are relevant for infinite spans. These corrections vary according to the type of coefficients and whether the angle of attack exceeds the stall angle or not. Before stalling, lift and drag coefficients are evaluated by using Lanchester and Prandtl theory (Bertin 2006). After the stall angle, coefficients are evaluated by using Viterna and Corrigan model (Chua 2002). Support arm drag losses are also included with some simplifications. The expressions related to the general element are integrated to obtain the resultant effects. Average torque coefficient loss and power coefficient loss are estimated in the form

$$(C_T)loss = \frac{1}{4}(\lambda^2+1)\frac{n \times A_f}{A_s}c_D$$

$$(C_P)loss = \frac{1}{4}\lambda(\lambda^2+1)\frac{n \times A_f}{A_s}c_D$$

Where $n$ is the number of support arms, $A_f$ is the frontal area of the struts, $A_s$ is the frontal area of the rotor. Here $c_D$ is taken as 0.9.

## 4. Results and Discussion

Aerofoil's lift and drag coefficients are obtained corresponds to the infinite blade or a bounded blade (Sheldahl and Klimas 1981). Aerodynamic characteristics of finite span blades are different from those of infinite span blades or bounded blades. When the blade span is finite, high pressure air spills out to the low pressure region and circulation along the span is not uniform. This alters the lift, drag and moment coefficient of the aerofoil and there should be adjustments for data taken from bounded blade (Anderson 2007). Figure 2 presents the effects of aspect ratio on the power coefficient of the rotor. NACA 0021 profile is used for the analysis and the rotor diameter is kept at 2 m. Wind velocity is taken as 6 m/s and chord length is kept at 0.2 m. Note that the chord length is kept constant in order to omit the effects of change of Reynolds number and only the blade height is changed. As *AR* is increased, the maximum power coefficient is also increased and the operating range becomes wider. Furthermore it was seen that the self starting ability is improved as the aspect ratio increases.

Solidity is influenced by three design parameters of the rotor that are number of blades, chord length and rotor diameter. Eventhough turbines cannot be strictly classified as low, medium and high solidity, for electrical generation lower solidity turbines are very usable. The main reason is that electrical generation applications normally deal with lower torque. In addition high maximum power coefficients can be achieved and electrical unit cost may be lower owing to lower construction and material cost. Moreover high solidity results for narrow operating range of tip speed ratios and it effects of performance reduction in gusts and variation of wind. However higher solidity improves the self starting performance, and therefore is better for VAWTs (Kirke 1998). The effect of rotor solidity is presented in Figure 3. The profile selected for this analysis is NACA 0021 and chord length, blade height, wind speed and number of blades are selected as 0.2 m, 1.8 m, 6 m/s and 3, respectively. Comparing values of the Solidity are 0.6, 0.4, 0.3 and 0.24 and it is varied by changing the value of rotor diameter. Variation of the power coefficient with the tip speed ratio is demonstrated and unlikely to maximum power coefficient ($C_{Pmax}$), maximum thrust coefficient ($C_{Tmax}$) increases when solidity lowers. When solidity increases, it can be easily seen a drop of $C_{Pmax}$ and optimum $\lambda$. But range of operating tip speed ratios is high for low solidity. However, there is an increment of self starting ability with high solidity. Number of blades primarily affects the solidity ($\sigma$) of the rotor. Two blades or three blades are frequently used in wind turbines. Number of blades affects not only power coefficient but also structural stability. As illustrate in Figure 4, four graphs are drown using NACA 0021 aerofoil and keeping blade diameter, chord length, blade height and ambient air velocity as 2 m, 0.2 m, 1.8 m, 6 m/s respectively. Torque performance is presented and it is apparent that higher torque can be achieved by increasing turbine blades. Furthermore that average torque of two bladed rotors is lower than the three bladed ones (DeCoste, et al. 2005). Also it was noted that as number of blades decreases, range of operating tip speed ratios increases and tip speed ratio correspond to the maximum power coefficient shifts towards to higher values. But it should be noted that there is no visible difference of value of $C_{Pmax}$ when the turbine consists with one blade, two blades or three blades. These facts were confirmed in literature (Kirke 1998) by compareing one bladed rotor with two and three bladed rotors. However there is a slight drop of value of $C_{Pmax}$ if the turbine consists of four blades. Moreover there is an improvement of self starting ability, when number of blades increase and it is also confirmed in literature (Kirke 1998).

The frequently used aerofoils for standalone VAWTs are NACA 0012, 0015, 0018, and 0021. These are preferred due to its simplicity for manufacturing and availability of performing data (Claessens 2006). The basic difference in these blade profiles are variation of thickness. A thick aerofoil improves the self starting capability of VAWT and there is another benefit for the structural strength. But there is no apparent change in $C_{pmax}$ and tip speed ratio ($\lambda$) where $C_{p\,max}$ occurs. Even though most of the researchers argued that thicker the aerofoil, higher the self starting capability, some of them show it is not relevant to low Reynolds number applications such as small scale vertical axis wind turbines (Kirke 1998). In Figure 5, ambient air velocity, number of blades, rotor diameter, chord length and blade height are kept as 6 m/s, 3, 2 m, 0.2 m, 1.0 m respectively. Improvement of self starting ability can be identified while profile goes to higher numbers that is thickness of the aerofoil increases. But there is no significant variation of rang of operating $\lambda$. Eventhough gradual increment of $C_{Pmax}$ and decline of optimum $\lambda$ can be seen in blade profile NACA 0012, NACA 0015, NACA

0018 respectively, NACA 0021 behaves in a slightly different way. Its $C_{Pmax}$ is less than the $C_{Pmax}$ of NACA 0018 and optimum $\lambda$ is slightly higher than the optimum $\lambda$ of NACA 0015 blade profile.

The most influential geometrical parameter for the Reynolds number is chord length which influences solidity, flow curvature and aspect ratio further. Ratio of $C_L/C_D$ is high for higher Reynolds numbers and it effects for better performance of the turbine. Hence high torque and power performance can be achieved. But contribution of the Reynolds number for the self starting is controversial and still it is not completely solved. In Figure 6, NACA 0021 is selected and ambient air velocity, rotor diameter, blade height are kept as 6 m/s, 2 m, 1.8 m respectively. Effect of Reynolds number for the performance of the turbine is examined by omitting variation of solidity and without including finite span effect. Thus variation of chord length only cause for variation of Reynolds number. By Figure 6, it can be easily seen that higher Reynolds numbers cause for higher $C_{Pmax}$ and shifting of optimum $\lambda$ towards to low tip speed ratios. Moreover self starting ability increases and operating range of tip speed ratios increases.

Combined effect of Reynolds number and solidity is evaluated without including finite span effect. Here only chord length is changed. Although there can be seen a gradual increment of optimum $\lambda$, it cannot be seen a gradual increment of $C_{Pmax}$ when chord length is increased. But it was noted a gradual increment of $C_{Tmax}$ as chord length is increased. Moreover it is apparent that self starting ability is increased with increasing chord length. In Figure 7, diameter is varied by keeping swept area constant. Decline of $C_{Tmax}$ can be noticed with increasing rotor diameter. Increment of turbine diameter results decline of $C_{Pmax}$ and shifting of optimum $\lambda$ toward low speed ratios. Nevertheless there is a slight increment of range of operating tip speed ratios with the turbine diameter. There can be seen an enhancement of self starting ability with decline of diameter.

## 5. Conclusion

Major factors needed to be concerned for designing small scale VAWTs for battery charging applications and influence of dimensionless parameters for the major factors are concluded.

### Ability to self start

From the theoretical modeling, means and their success on improving ability to self start can be evaluated. It is hard to see any influence of aspect ratio on self starting. But When the number of blades of the turbine is increased, it can be seen an apparent improvement of self starting. But going for more than four blades is not feasible and there are number of factors such as power performance, economy, rotor inertia etc. need to be considered. Even though higher solidity and Reynolds number improve self starting, economic aspect of these factors has to be considered. Moreover, Blade profile has significant effect on self starting that is thickness improves it. Constructing aerofoils with camber is difficult task and due to the lack of data on lift and drag coefficients, mathematical prediction is difficult. But implementing thicker aerofoil is easy and another benefit of thick aerofoil is high strength. Even so, blade inertia increases owing to thicker blades.

### Power performance

$C_{Pmax}$ is the most important value in power performance and it can be easily achieved by increasing aspect ratio. But higher solidity lowers the $C_{Pmax}$ and high Reynolds numbers give high $C_{Pmax}$. Although increment of chord length increases Reynolds number, it as well causes for higher blade inertia. Effect of number of blades on $C_{Pmax}$ is not subtle and when turbine consists with one, two or three blades, it is hard to notice any change of $C_{Pmax}$. Under the topic of power performance, width of the power curve is crucial inasmuch as it measures sensitivity for short term changes of wind speed. Peaky power curve causes for extreme sensitivity for changes of wind speed and it should be avoided. As number of blades lower, wider the power curve and one bladed turbine gives widest power curve. But implementing one bladed turbine is not possible due to unbalanced centrifugal forces and three bladed turbines are more appropriate than two bladed turbines for it gives more stability.

### Optimum tip speed ratio

The tip speed ratio which gives $C_{Pmax}$ is also significant in designing small scale VAWTs in that it affects for noise and life of the turbine. High tip speed ratio means high rotational speed and it causes for aerodynamic and structural noise. Furthermore vibrations can be high since unbalanced centrifugal forces develop with rotational speed and it may reduce the life of the turbine. Normally optimum tip speed ratio shifts towards low tip speed values with number of blades.

### Torque performance

Consideration of torque performance of the turbine is crucial with respect to the load and transmission system. Normally load can be classified according to the stating torque and variation of torque with respect to the rotational speed. Normally turbine connected with multi pole generator requires low stating torque. Thus it can be seen a convergence with the turbine and the multi pole generator. Most influential dimensionless parameter for the torque characteristic of turbine is solidity. Turbines with one blade and two blades shows low torque characteristics and three bladed turbine gives sufficient torque characteristics for small scale VAWTs.

## Acknowledgements

At Mechanical Engineering department of University of Moratuva, we would like to thank Dr. U. Kahangamage, Mr. Janaka Mangala, and Mr. Saman Chandrasiri whom guided us in various stages of the research. It is glad to recall the contribution given by Mr. Prasad Nissanka of preparing the software on theoretical modeling. He gave his JAVA knowledge freely and devoted his valuable time without showing any difficulty.

## References

Anderson, J., *Fundermentals of* Aerodynamics. New Delhi: Tata McGraw-Hill publishing company, 2007.

Bertin, J., *Aerodynamics for Engineers.* Dorling Kindersley (India) Pvt. Ltd., 2006.

Claessens, M., "The Design and Testing of Airfoils for Application in Small Vertical Axis Wind Turbines." M.Sc thesis, Department of Aerodynamics, Delft University of Technology, 2006.

DeCoste, J., McKay, D., Robinson, B., Whitehead, S., Wright, S., "Vertical Axis Wind Turbine." B.Sc Thesis, 2005.

Chua, K.L., *Finite aspect ratio effects on airfoil coefficients.* 11 6, 2002. http://www.Darrieus windturbine-analysis.com (accessed 11 10, 2008).

Kirke, B.K., *Evaluation of self starting vertical axis wind turbine for stand alone applications.* PhD Thesis, School of Engineering Griffith university Gold Coast campus, 1998.

Paraschivoiu, I., *wind turbine design with emphasis on Darrieus concept.* Presses Internationles Polytechnique, 2002.

Sheldahl, R.E., Klimas, P.C., "Aerodynamic Characteristics of Seven Airfoil Sections Through 180 Degrees Angle of Attack for Use in Aerodynamic Analysis of Vertical Axis Wind Turbines." New Maxico, 1981.

Strickland, J.H., *The Darrieus turbine:A performance prediction model using multiple streamtubes.* New Mexico: Sandia laboratories, 1975.

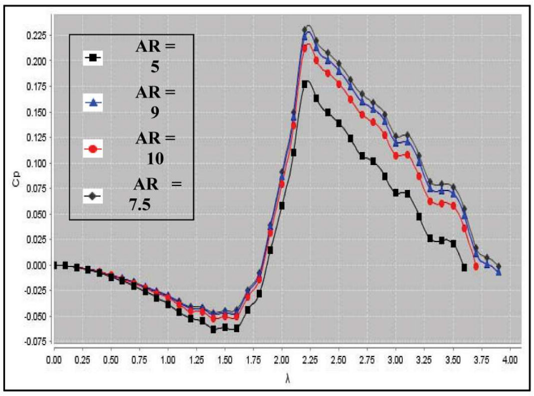



**Figure 2 -** ***The effect of aspect ratio***

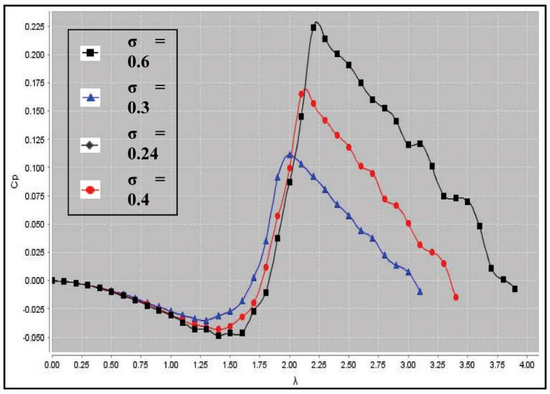



**Figure 3 -** ***Effect of solidity***

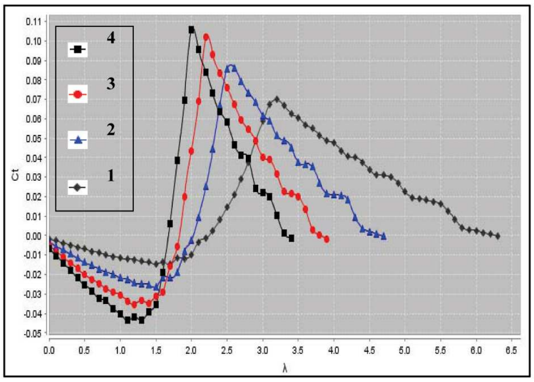



**Figure 4 -** ***Effect of number of blades***

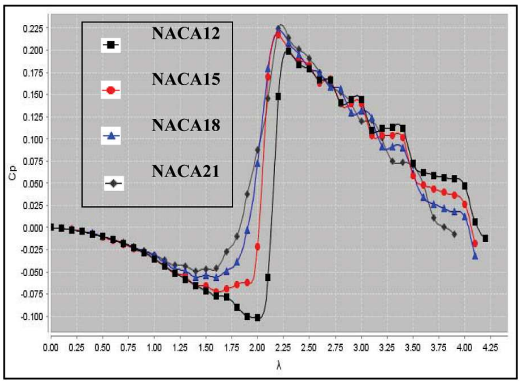



**Figure 5 -** ***Effect of blade profile***

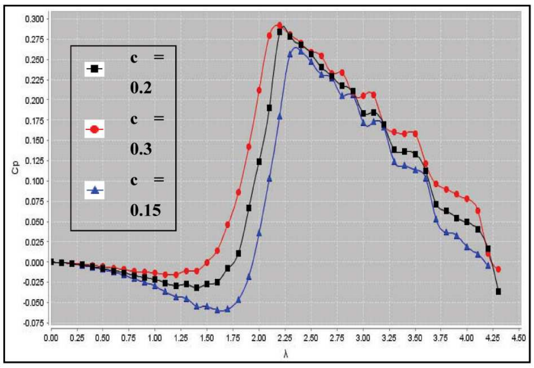



**Figure 6-** ***Effect of Reynolds number***

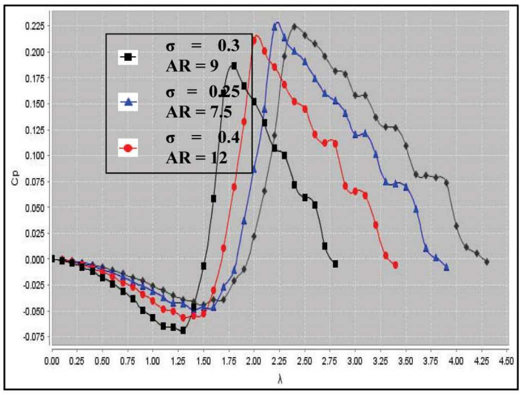



**Figure 7 -** ***Changing aspect ratio while keeping chord length as a constant***

# MODELING AND SIMULATION OF TEMPERATURE VARIATION IN BEARINGS IN A HYDRO ELECTRIC POWER GENERATING UNIT

[1]C.G.S.Gunasekara, [2]Primal Fernando, [3]Joachim Claesson

[1]PO Box 42, Mahaweli Complex, Ceylon Electricity Board, Kandy, Sri Lanka,
[1]E-mail: cgsgu@kth.se

[2]University of Peradeniya, Peradeniya, Sri Lanka
[2]E-mail: primal@kth.se

[3]Royal Institute of Technology, Stockholm, Sweden
[3]E-mail: claesson@energy.kth.se

***Abstract**:* Hydroelectric power contributes around 20% to the world electricity supply and is considered as the most important, clean, emission free and an economical renewable energy source. Hydro electric power plants operating all over the world has been built in the 20$^{th}$ century in many countries and running at a higher plant-factor. This is achieved by minimizing the failures and operating the plants continuously for a longer period at a higher load. However, continuous operation of old plants have constrained with the failures due to bearing overheating. The aim of this research is to model and simulate the dynamic variation of temperatures of bearing temperature of a hydro electric generating unit.

Multi-input, multi-output (MIMO) system with complex nonlinear characteristics of this nature is difficult to model using conventional modeling methods. Hence, in this research neural network (NN) technique has been used for modeling the system.

*Keywords*— Hydro electricity, Bearing Temperature, Plant factor, Neural network, Simulation

## 1.0 Introduction

Hydro power contributes around 20% of the world electricity generation [1]. As a renewable source of energy it has become more important economical resource compared to other renewable sources as far as the scarcity of fossil petroleum fuel deposits, environmental threats, climate change due to green house gas emissions, and acid rains global warming, etc. are concerned. Hydro power produces no direct waste and contribution to $CO_2$, green house gases compared to fossil fuel plants. The global installed capacity of Hydro-electrical power generation is approximately 777GW with a production of 2998TWh/year [1]. It is around 88% of the renewable sources [2].

In Sri Lanka about 40% of electricity is generated from hydro electricity. At present almost all hydro potential available in the country has been utilized for electricity generation and few remaining are under construction. The deficit between electrical power generation and demand is met by thermal power generation.

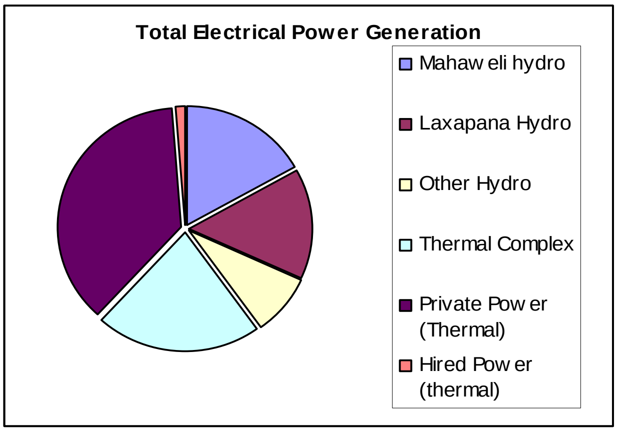



Figure 1. *Hydro electric contribution in 2009* (Source: Ceylon Electricity Board, Statistics 2009)

The electricity power generation by different sources in the year 2009 is shown in Fig. 1. Electricity generated in three major hydro power complexes (Mahaweli Hydro complex, Laxapana

Hydro Complex, Other Hydro Complex) in Sri Lanka, contributes 40% to the national energy supply while the rest is coming from thermal power generation. Hence, getting the maximum possible share from hydro would be great saving to the national economy.

Around 95% of the existing hydro power plants have passed the 25 year limit of their life span. Many developing countries in the region are not in a situation to replace all old-hydro power plants, within a short period and also their energy production are mainly depends on hydropower. In Sri Lanka all most all available hydro plants have the 25 year limit. Age analysis of the hydro plants in Sri Lanka is shown in Table 1.

TABLE 1: *Age analysis of Hydro electric Plants in Sri Lanka (Source: Ceylon Electricity Board, generation Data)*

| Name of the Station | Installed capacity / MW | Commissioned year | Age (Years) |
|---|---|---|---|
| Inginiyagala | 11.25 | 1950 | 65 |
| Norton | 50 | 1950 | 65 |
| Udawalawe | 6 | 1955 | 60 |
| Old Llaxapana | 50 | 1955 | 60 |
| Polpitiya | 75 | 1960 | 50 |
| Ukuwela | 40 | 1976 | 34 |
| Bowetenna | 40 | 1981 | 29 |
| New Laxapana | 100 | 1984 | 26 |
| Canyon | 60 | 1984 | 26 |
| Kothmale | 201 | 1985 | 25 |
| Victoria | 210 | 1985 | 25 |
| Samanalawewa | 120 | 1985 | 25 |
| Randenigala | 122 | 1986 | 24 |
| Nilambe | 3.2 | 1988 | 22 |
| Rantambe | 50 | 1990 | 20 |
| Kukule | 70 | 2002 | 8 |

Therefore, it is essential to obtain the maximum capacity from the existing plants by minimizing the down time through proper operations. In that context predicting the availability of hydroelectric generating units for fault free operation is one of the crucial factors for achieving this.

Bearing oil temperature plays a vital role in continues operation of hydro power plants. Stability of bearing temperatures in turbine and generators are essential for their successful continues operations. All hydraulic and lubricating fluids have practical limits on the acceptable higher operating temperatures. The machine loses its stability and experiences conditional failures whenever the system’s fluid temperature violates this limits. Violations of the temperature limit could occur due to inadequate heat transfer rate, operating under higher ambient temperatures and longer duration of operation at higher mechanical loads. The power plant staff should closely monitor the bearing oil and metal temperatures in order to ensure a safe operation of the plant and the bearings life time [4]. Typical acceptable bearing temperatures of a vertical shaft hydro electric turbine generator unit are shown in Table 2.

Monitoring the bearing temperature is an important task for continues running of a hydro electricity generating unit and maximizing the plant-factor. In this context, old hydro power plants continuous operations have been constrained with the failures due to the bearing temperatures rise.

TABLE 2: *Bearing Metal/Oil Temperature Limits*

| Bearing Type | Temperature / Deg C (Alarm) | |
|---|---|---|
| | Metal | Oil |
| Upper Guide (UGB) | 85 | 50 |
| Lower Guide (LGB) | 85 | 65 |
| Thrust Bearing (THB) | 85 | 65 |
| Turbine Guide (TGB) | 70 | 70 |

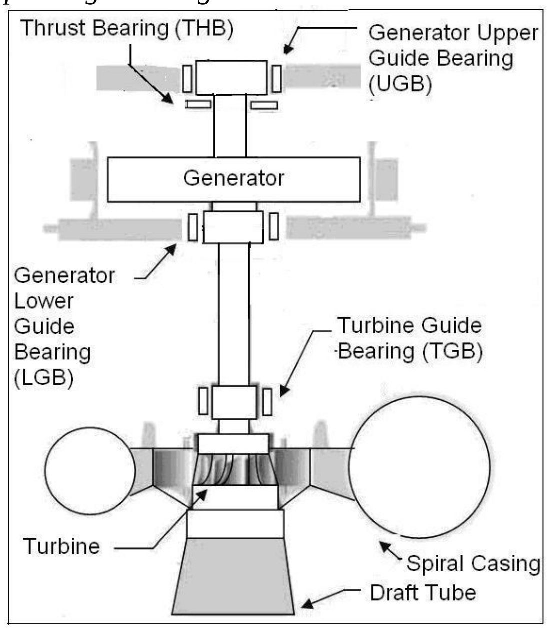
The aim of this research project is to model and simulate the dynamic variation of bearing (generator upper guide bearing UGB, generator Lower guide bearing LGB, turbine guide bearing TGB, thrust bearing THB) temperatures of a hydro electric generating unit which depends on multiple variables such as ambient air temperature, cooling water temperature, cooling water flow rate, initial bearing temperatures and generating unit electrical load and duration of operation etc.

### *1.1 The hydro electricity generating unit*

Hydro electricity generating plant utilizes the potential energy of the water stored in a higher elevation and the turbine and the generator converts the potential energy in to kinetic energy and electric energy respectively. [5].

In this study real data is taken from the Kotmale hydro power generating system for simulating the proposed methodology. Kotmale hydro electric power generating unit consists of four main bearings namely upper guide bearing (UGB), lower guide bearing (LGB), thrust bearing (THB), and turbine guide bearing (TGB) as shown in Fig 2.

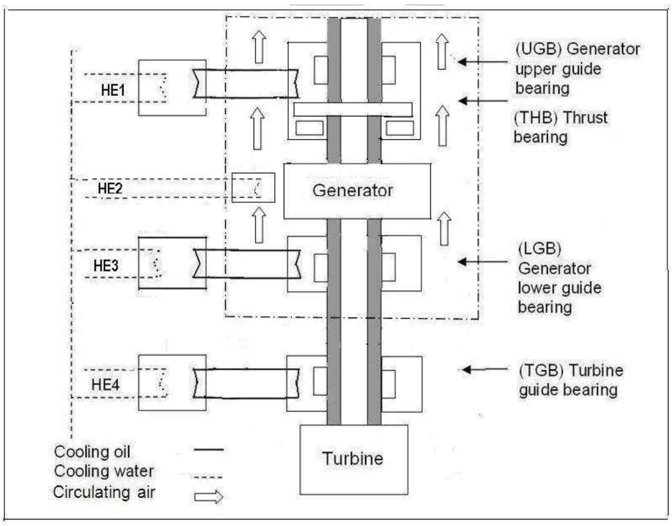
Figure 2. *Bearing arrangement of the hydro-electric power generating unit*




## 2.0 Determination of NN ARCHITECTURE

### *2.1 Overview of modeling*

The investigated system is a multi dimensional system with multiple-input, multiple-output (MIMO). The physical arrangement of the different types of heat exchangers which transfers the heat generated by heat sources (bearings and generator stator) is shown in Fig 3.0.




HE3, HE4 – LGB, TGB oil coolers,
HE1 – THB and UGB oil cooler,
HE2 – Stator cooler
Figure 3. *Physical arrangement of bearing system*

A simplified diagram illustrating the heat transfer taking place within the system is shown in Fig 4. Bearings (UGB, LGB, THB, TGB) and generator stator are considered as heat sources, and cooling water as well as ambient air act as heat sinks. A detailed diagram of heat transfer is shown in Fig 5.

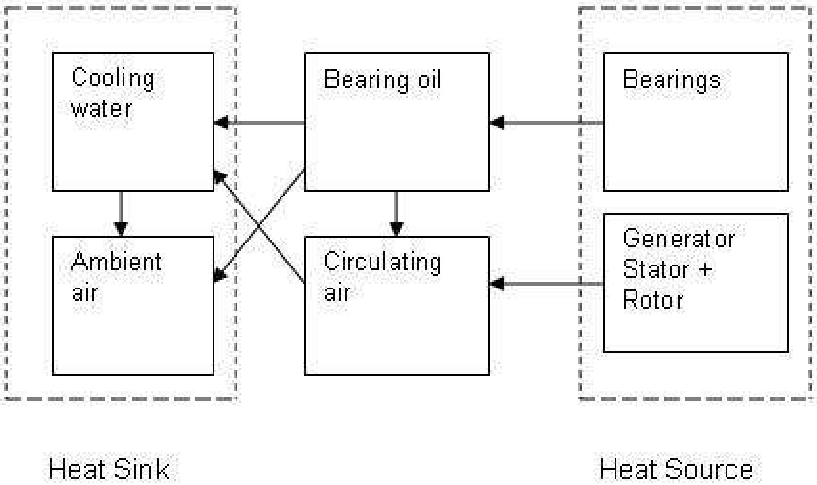



Figure 4. *Simplified heat transfer diagram*

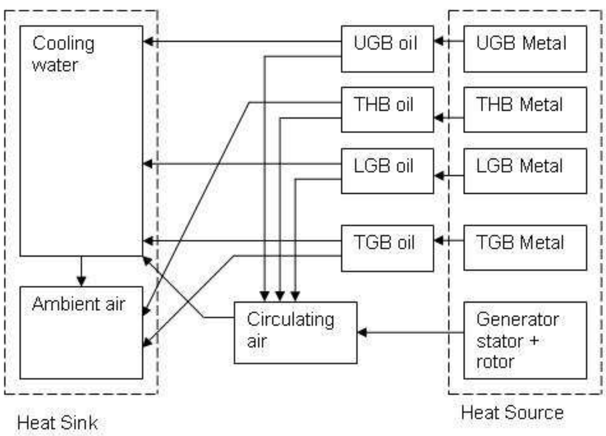



Figure 5. *Detailed heat transfer diagram*

### 2.2 General Framework for ANN model

A multi-layer feed forward network consists of input layer, out put layer and several hidden layers. The input layer passes their output to the first hidden layer or (with skip layer connection) to directly to output layer. Each of the hidden layer units takes a weighted sum of its inputs, adds a constant (the bias) and calculates a fixed function $\Phi_h$ of the result. This is then passed to the hidden units in the next layer or to the out put unit(s).The fixed function is given by

$$f(z) = \exp(z)/(1+\exp(-z)) \qquad (1)$$

The output units apply a threshold function $\Phi_0$ to the weighted sum of their inputs plus their bias. If the input are $p_i$ and outputs are $a_k$ for one hidden layer ,

$$a_k = \phi_o(b_k + \sum_i^k w_{ij} p_h + \sum_j^k w_{jk} \phi_h (b_i + \sum_i^j w_{ij} p_i)) \qquad (2)$$

*i* ,*j*,*k* denotes number of input ,hidden layer and output layer units. Following equation gives general form of a multi layer neural network.

$$a_i = \phi_0(b_k + \sum_j w_{ij}^{(1)} p_j + \sum_j \sum_k w_{ijk}^{(2)} p_j p_k + \sum_j \sum_k \sum_l w_{ijkl}^{(3)} p_k p_k p_l + \ldots) \qquad (3)$$

(1),(2),(3) denotes the layer numbers and others are usual notations.

## 3 TRAINING THE NN

### 3.1 Developing the model

This section describes the approach and steps followed to develop a dynamic model to simulate hydro-electric power generating unit bearing temperature variation with time, electrical load, with the duration of operation and other environmental factors.

### 3.2 Selection of input/outputs

Input variables which affects to the characteristics of the system under investigation can be shown as given below in Table 3.

TABLE 3: *INPUTS/OUTPUTS*

| Notation | Description |
|---|---|
| $T_{LGBm}$ | Lower guide bearing metal temperature |
| $T_{UGBm}$ | Upper guide bearing metal temperature |
| $T_{TGBm}$ | Turbine guide bearing metal temperature |
| $T_{THBm}$ | Thrust bearing metal temperature |
| $T_{LGBoil}$ | Lower guide bearing oil temperature |
| $T_{UGBoil}$ | Upper guide bearing oil temperature |
| $T_{TGBoil}$ | Turbine guide bearing oil temperature |
| $T_{THBoil}$ | Thrust bearing oil temperature |
| $T_{cooling\ water}$ | Cooling water temperature |
| $T_{air}$ | Circulating air temperature |
| $mdot_{CW}$ | Cooling water flow rate |
| $m_{BCW}$ | Bearing cooler water flow rate |
| $L_e$ | Electrical load (MWs) |
| $L_{vars}$ | Electrical load (Vars) |

*TLGB*, *TUGB*,*TTHB*, *TTGB*, *TLGBoil*, *TUGBoil*, *TGBoil* and *TTHBoil*. But, values of the above variables depend not only on the instantaneous values of them , but current values as well as the previous values. It can be illustrated more general form as shown in the fig. 6. where, *Xi* as temperature related inputs, mi as inputs related to flow rates, *Li* inputs related to load variables.

Where,

Xi = { $T_{UGBm}(0)$, $T_{THBm}(0)$, $T_{LGBm}(0)$, $T_{UGBO}(0)$, $T_{LGBO}(0)$, $T_{TGBO}(0)$, $T_{UGBm}(t-2T)$, $T_{UGBm}(t-T)$, $T_{UGBm}(t)$, $T_{THBm}(t-2T)$, $T_{THBm}(t-T)$, $T_{THBm}(t)$, $T_{LGBm}(t-2T)$, $T_{LGBm}(t-T)$, $T_{LGBm}(t)$, $T_{TGBm}(t-2T)$, $T_{TGBm}(t-T)$, $T_{TGBm}(t)$, $T_{UGBo}(t-2T)$, $T_{UGBo}(t-T)$, $T_{UGBo}(t)$, $T_{LGBo}(t-2T)$, $T_{LGBo}(t-T)$, $T_{LGBo}(t)$, $T_{TGBo}(t-2T)$, $T_{TGBo}(t-T)$, $T_{TGBo}(t)$, $T_{CW}(t-2T)$, $T_{CW}(t-T)$, $T_{CW}(t)$, $T_{CA}(t-2T)$, $T_{CA}(t-T)$,$T_{CA}(t)$,}

Mi = {$m_{dot1}(t-2T)$, $m_{dot1}(t-T)$, $m_{dot1}(t)$, $m_{dot2}(t-2T)$, $m_{dot2}(t-T)$, $m_{dot2}(t)$}

Li = { $L_{mw}(t-2T)$, $L_{mw}(t-T)$, $L_{mw}(t)$, $L_{mv}(t-2T)$, $L_{mv}(t-T)$, $L_{mv}(t)$ }

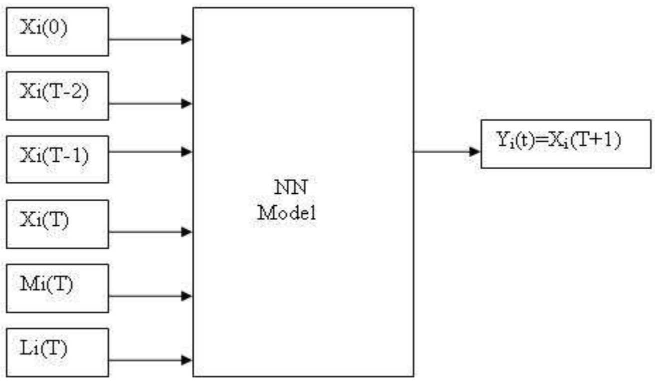



Figure 6. *Inputs / outputs for training the NN model*

### *3.3 Approach*

As discussed earlier, in section 3 and as shown in Fig. 6, there are two types of input variables to the model, viz temperature dependent variables (bearing metal temperatures, bearing oil temperatures, cooling water temperature and circulating air temperature) as denoted by Xi. Second, type of inputs is the bearing water flow rates that do not change due to the performance of the system and the electrical load that directly affect to the bearing metal and bearing oil temperatures.

The variables that interact with system can also be classified into two categories. They are external variables and internal variables. Electrical load, cooling water and circulating air temperatures act as external factors while initial bearing metal temperature, bearing oil temperature act as internal variables. In a system of this nature, output values depend on the present status as well as previous status of the system.

In mathematical form, general behavior of the system can be defined as,
State equation,

$$S(t+T) = f(S(t), X(t), w) \qquad (4)$$

Output equation,

$$y(t) = h(S(t), w) \qquad (5)$$

Where, *S* represents the state vector, *x* external input vector and *w* neural parameter vector synaptic connection vectors and operational parameters, *f*(.) is the function that represents the structure of the neural network, and *h*(.) is a function that represents the relationship between state vector *S*(t) and output vector *y*(t) [10].

Some times in order to get a reasonable accuracy several previous states have to be considered. Therefore, some sort of memory capability has to be introduced to the model. The variation of temperatures are continues varying functions. But, as we consider sample inputs at a chosen time interval the model becomes a discrete system. Hence, the memory capability can be incorporated by giving a series of time delay inputs. Equations (8) and (9) describe behavior of a first order system which takes into account the previous state (with one step time delay) of the variables. In generally n$^{th}$ order system can be described as,

State equation,

$$S(t+T)=f(S(t),S(t-T),S(t-2T)......S(t-[n-1]T)X(t),w) \qquad (6)$$

Output equation,

$$y(t)=h(S(t),w) \qquad (7)$$

We have developed two models of second order and third order in order to select the one that gives the best performance.

In a third order system we have to consider the three previous states. Therefore, in order to predict the bearing temperature value at t, bearing temperature at t, (t-T) and , (t-2T) also has to considered. Then, with the bearing metal temperature, bearing oil temperature, cooling water temperature, circulating air temperature and electrical load MWs, MVars altogether makes 32 inputs to the model. Our intention is to predict the four bearing metal temperatures but as bearing oil temperatures, cooling water temperature and circulating air temperatures also affect to it, altogether the number of out puts become 9 ($T_{UGBm,}$ $T_{THBm,}$ $T_{LGBm,}$ $T_{TGBm,}$ $T_{UGBOil,}$ $T_{LGBOil,}$ $T_{TGBOil,}$ $T_{cw,}$ $T_{CA}$) So that, the initial architecture of the NN takes shape of 32 input nodes, and 9 output nodes as shown in Fig. 4.2 shown below. Let's arbitrarily select two hidden layers, this can be changed if necessary during the process of training the network. Number of nodes in the hidden layers also could be selected as an average number of nodes of the two adjacent layers of the network [11][12].

Then, the initial architecture becomes (32, 24, 15, 9), where number of inputs and outputs are a fixed value and the number of input also can be changed according to the consideration of previous status of inputs at interval such as t-T, t-2T, etc depending on the accuracy or the error of training. Training, validation and testing errors explain to what extent that the model fit to the actual system behavior.

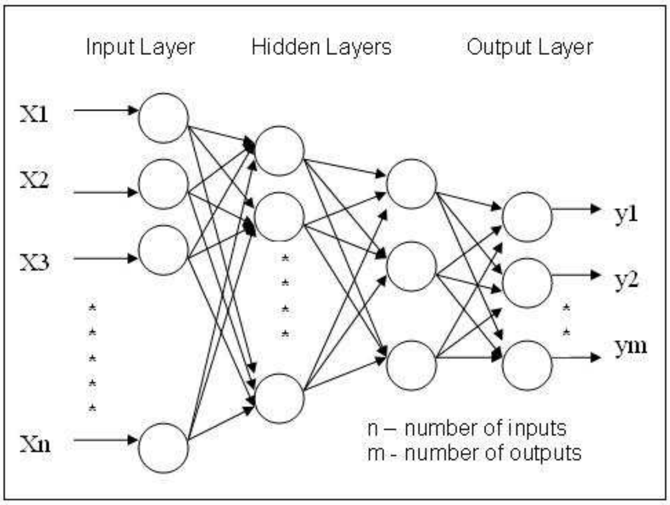



Figure 7. *Inputs / outputs for training the NN model*

Input and output data was fed into the network and trained in the MATLAB environment, Fig. 8 shows the training performance of the model. The mean squared error (Mse) was converegd to 3.10263e-007 , for a model with (32,40,26,9) aechitecture.

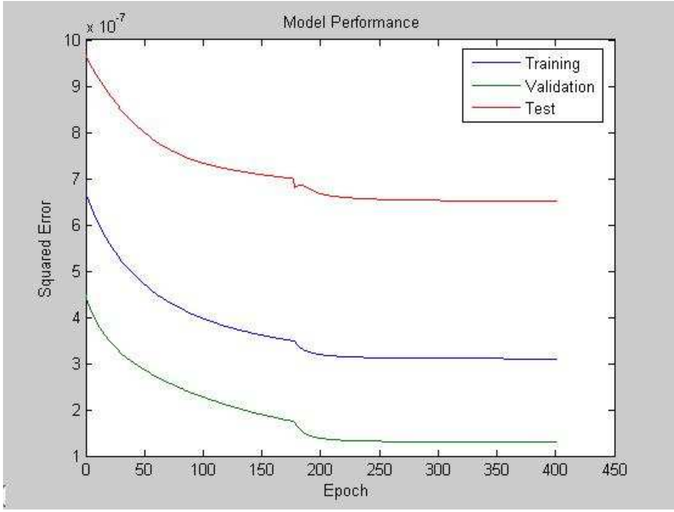



Figure 8. *Inputs / outputs for training the NN model*

### *3.4 Developing the dynamic model*

As described in the previous section in order to model the temporal nature of the system as well as the effect of the internal variables the general architecture of the model should be as in Fig 9 shown below where Xi(0) denotes the initial conditions.

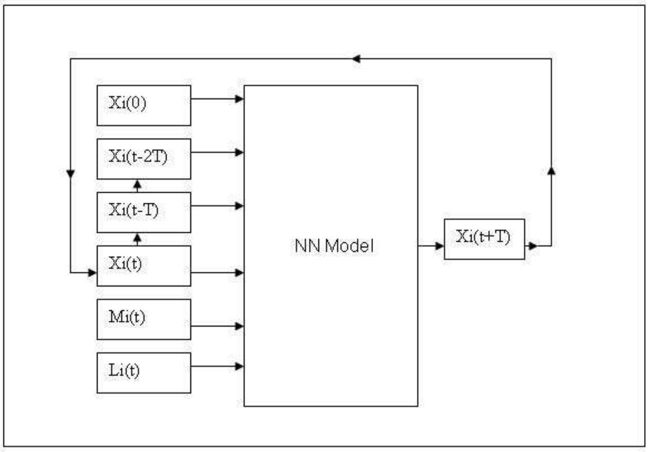



Figure 9. *Dynamic model*

### *3.5 Developing the dynamic model*

Simulation was carried out according to the following algorithm.

Algorithm of the simulation:

```
Read

Xi(0), initial conditions  (bearing metal
                     and oil temperature)
read Xi(t),Xi(t-T),Xi(t-2T), bearing metal
                     and  oil temperature
Mi(t),Li(t)   cooling water flow rates,
                     circulating air
                     temperature and
                     electrical load,
```

```
make input matrix
load trained neural network
decide time duration n
loop up to n records
      simulate and get output of Xi(t+T)
      update inputs
      record output
end
```

## 4 Simulation results

### *4.1 Static model Performance*

Our approach is to develop (training) a static model to simulate the behavior of the real system and then to convert it to a dynamic model by arranging a feedback of internal variables as inputs to the model. The simulated out puts were compared with the actual outputs to evaluate the performance of the static model. Then the correlation of simulated outputs with actual targets was compared. Static simulation results and corresponding correlation results of temperature variation for *UGBm, THBm, LGBm, TGBm* are shown in Fig 10, and 11 respectively.

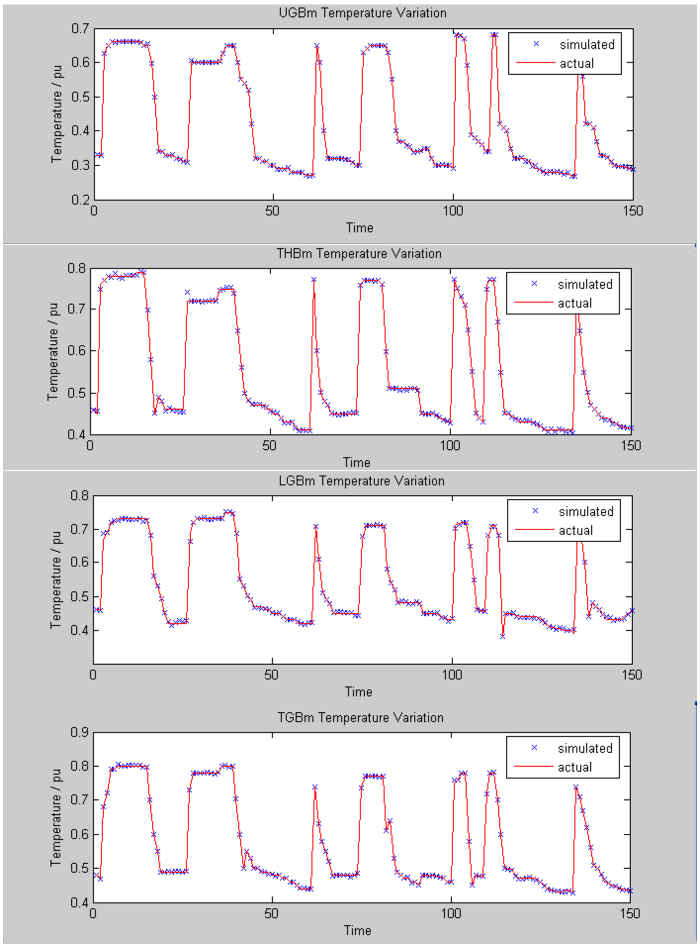



Figure 10. *simulaed results for static model*

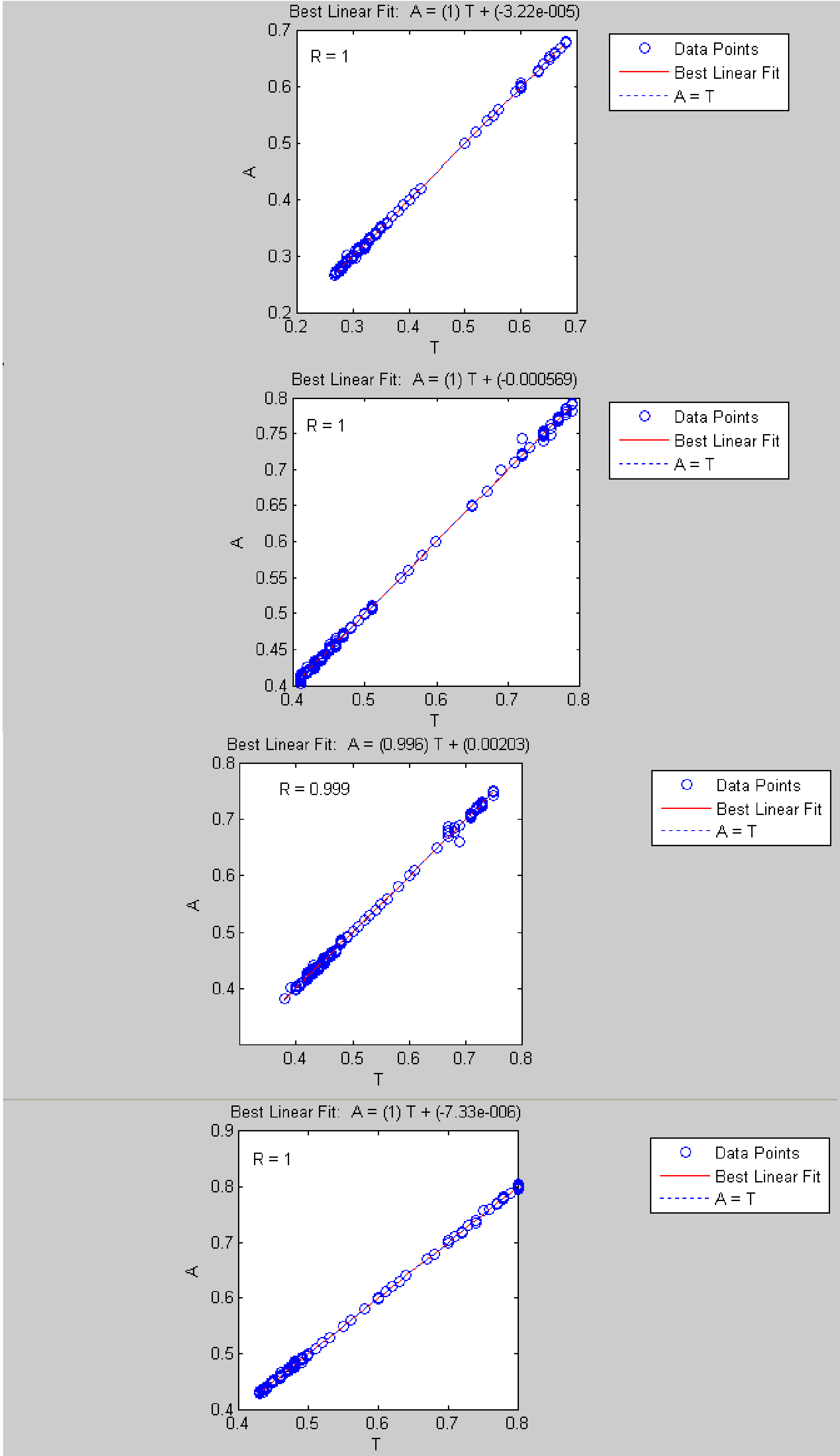



Figure 11. *corelation coefficient for static model*

### 4.2 *Dynamic Model Performance*

A set of unused data test data was fed into the trained model and simulated according the algorithm given in section III, E and the performance of the model was evaluated. The regression analysis results in Table II shows the correlation between actual and the simulated results.

## 5 CONCLUSION

The model developed in this research to simulate bearing temperatures of a bearing heat exchanger system was successful giving promising results. Initially a static model was developed and it was extended to simulate the dynamic behavior of the system. According to these results, neural network models are more capable of modeling non linear, multi-dimensional MIMO systems rather than using conventional methodologies using first principles. The model developed and methodology used provides and serves as a good initiative for others for modeling problems of this nature.

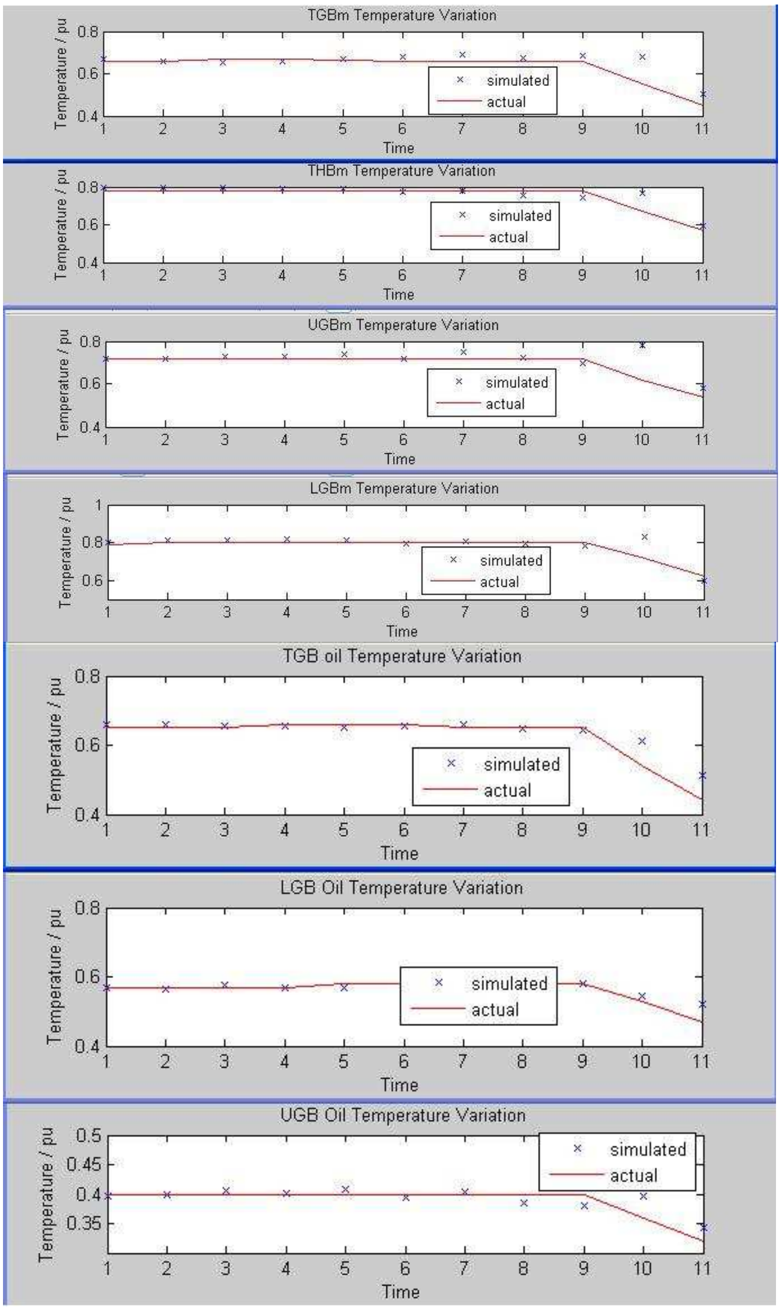



Figure 12. *Dynamic simulation results of the dynamic model*

TABLE 4: CORRELATION RESULTS OF THE OUTPUTS

| OUTPUT NO | CORELATION COEFFICIENT |
|---|---|
| 1 | 0.82 |
| 2 | 0.87 |
| 3 | 0.71 |
| 4 | 0.84 |
| 5 | 078 |
| 6 | 095 |
| 7 | 0.97 |

## References

1. http://www.eia.doe.gov/ Energy Information Administration international statics database, as accessed 04-03-2010
2. Renewables Global Status Report 2006 Update, *REN21*, published 2007, accessed 04-03-2010; see Table 4, p. 20
3. Statistical Digest 2009, Ceylon Electricity Board, Sri Lanka
4. http://www.machinerylubrication.com/Read/367/temperature-stability, *Machinery Lubrication*, *as* accessed 2010-03-02

[5] R.K.Sharma, T.K.Sharma, A text book of Water Power Engineering, S.Chand & Company Ltd, 2000

[6] Girish kumar Jar, Artificial neural networks and it's applications, IARI, New Delhi- 100-012,

[7] Raůl Rojas, Neural network a systematic introduction, Sprinter, Berlin, Heidelberg, Network, March 1996.

[8] Heichi Neilson R. [1990], Neurocomputing, Addison-Wesley, Reading, Mass.

[9] Matlab Neural Network Use's Guide, The Mathworks inc, 1992-2010

[10] Madan M Gupta, Liang jin and Noriyasu Homma, John Wiley& Sons Inc, Hobokan New Jersey, 2003

[11] James A Freeman, David M Skapura, Neural networks Algorithms, Applications, and programming techniques, Pearson Inc, 1999

[12] Stuart Russel, Peter Norvig, Artificial Intelligence A Modern Approach Persons Inc, 1995,

# STRATEGIES FOR ADOPTING NEW TRENDS IN WIND LOAD EVALUATION ON STRUCTURES

Weerasuriya A.U. [(a)], Jayasinghe M.T.R. [(b)]

[(a)]Research Assistant, Department of Civil Engineering, University of Moratuwa, Sri Lanka
E-mail: asiriuw@civil.mrt.ac.lk
Tel: +94 11 2650568 (Ext 2021) Fax: +94 11 2651216
[(b)]Professor, Department of Civil Engineering, University of Moratuwa, Sri Lanka
E-mail: thishan@civil.mrt.ac.lk
Tel: +94 11 2650568(Ext 2006) Fax: +94 11 2651216

**Abstract:** The advancement of knowledge of wind engineering introduces lots of changes to wind loading standards. There are many differences in old codes of practices compare to the newer standards by means of factors, methods and ultimately wind induced forces in structural members. Since tall buildings are more susceptible for wind loads and thus, require more close consideration when they are designed for wind loads. In this study, five major wind loading standards, CP 3 Chapter V – Part 2:1972, BS 6399.2:1997, AS 1170.2:1989, AS/NZS 1170.2:2002 and EN 1991-1-4:2005 are compared with respect to the CP 3 Chapter V – Part 2, for designing of a 183 m tall building. From one standard to another, factors like basic wind speeds, terrain height multiplier and procedures like analysis methods are different because of strategies set by the conditions and requirements of the country of its origin. The serviceability limit state behaviour of tall building is equally important like ultimate limit state behaviour and hence it discussed with this paper by the means of drift index and along and cross wind accelerations.

***Key Words:*** *Wind loading, Dynamic response*

## 1. Introduction

The assessment of wind loads on buildings requires knowledge of complex interaction between meteorological, aerodynamic and structural aspects of the problem. The physical modelling of the structure is only viable mean to obtain information about along-wind, cross-wind and torsional effects of the structure resulted from wind loads. However, requirements time, cost and resources discourage the designers to carry out physical modelling. Therefore, as an alternative, wind loading standards have included empirical relationships to produce an estimation procedure to evaluate the dynamic wind response (Kareem and Kijewski, 2001). Basically wind loading standards enable estimating safety and serviceability of a structure with information supplied by meteorology and aerodynamics together with basics of structural theory (Kulousek, 1984). However, due to continuous research work done on wind engineering during last two to three decades lots of improvements have been included in wind loading standards. Old codes were suppressed by the new standards, which are capable of supporting dynamic analysis rather than assuming only quassi –static behaviour of the building and better strategies to assess risk for different types of buildings. The evolution of the tall building design and construction may also enforce the wind loading standards to have methods to predict complex behaviour of the building not only at the ultimate limit state but also at the serviceability limit state as well.

The first mandatory document on wind engineering, the design manual "Design buildings for high wind – Sri Lanka" was published by the Sri Lankan Government in 1980. The design manual was based on the previous code of practise CP 3 Chapter V – Part 2: 1972; this extensively covers the design and construction of low rise buildings (Clarke et al, 1979). However, in recent times, there has been a national trend to build tall, slender tower type high rise buildings in Sri Lanka especially in Colombo city limit (Karunarathne, 2001). Not Only due to their heights but also light materials used as building materials for both super structure and the inner partition walls and some complex architectural features, these buildings may be prone to excessive dynamic motion induced by winds. Neither design manual nor CP 3 Chapter V-Part 2: 1972, adequately address this kind of complex situations. This means that there is a need to go for a wind loading standard, which can cover more complex wind spectrum as well as dynamic effects arising from the wind. Therefore, designers and structural engineers of Sri Lanka have been looking for advance wind loading standards, which are capable to evaluate more complex dynamic behaviours. International wind loading standards such as Australian, British, American, Japanese and Euro codes have been used by Sri Lankan engineers.

However, use of different wind loading standards in a given design may lead to severe problems such as poor understanding about the use of country specify factors in conjunction with Sri Lankan context, some inconveniences about understanding and comparing wind load calculations, lack of harmonization among wind load design of structures, etc. Therefore, it is necessary to have a broad and clear idea about strategies adopted by different wind loading standards before carrying out any wind load design of a building.

## 2. Different strategies adopted by wind codes and standards

### *2.1 Selecting codes and standards for the study*

Due to the incapability of design manual to address the issues of tall building design, many Sri Lankan engineers used different international wind loading standards as their preferred options. These preferred options may vary from some old code of practise like CP 3 Chapter V – Part 2 to newest codes like Euro code. By considering all of current practises that are found in Sri Lankan civil engineering sector, following codes and standards were chosen for the comparison purpose. The selected codes and standards are CP 3 Chapter V – Part 2:1972, BS 6399.2:1997, AS 1170.2:1989, AS/NZS 1170.2:2002 and EN 1991-1-4:2005.

There are different strategies that can be clearly identified from these selected codes and standards. CP 3 Chapter V-Part2:1972 uses quasi-static method to calculate wind loads on a building, this quassi static approach is more suitable for evaluating wind loads on low rise buildings rather than to evaluate the performance of a high rise building. Many like to continue to with this code because of its simplicity and familiarity of the code. BS 6399.2:1997 is the newer version of the British standard and capable to handle both static and dynamic behaviour of a building. Gust Load factor is a more popular method to calculate wind load by considering both fluctuating wind speeds and dynamic behaviour of a structure. AS 1170.2:1989 use Gust factor method and it generally uses 3 second gust velocity as basic wind speed. AS/NZS 1170.2:2002 has changed some factors and methods used in previous Australian standard and made it as a simple document to use. Apart from these reasons, Australian standards cover wide spectrum of wind, including cyclones and it is used by many island nations such as Fiji, Solomon Island, etc. EN 1991-1-4:2005 is the newest code and not only it compromises many aspects present in other codes such as BS 6399.2:1997, AS/NZS 1170.2:2002. However, it allows to adjust the methods and factors which are suitable for own country by means of a national annex.

### *2.2 Basic wind speeds*

The design manual defined two types of 3 – second gust wind speeds for three wind zones in Sri Lanka as shown in Table 1. The basic wind speed can vary with different average times. The averaging time depends on some facts such as long enough to allow the non stationary phenomena to decrease to a minimum, long enough to allow recording of steady vibration during a simultaneous examination of the structural response, short enough to provide a true picture of wind gusts of short duration and long enough to allow the application of the wind speed measurement methods used in meteorology. The conversion between two averaging times can be done by using some graphical method like Durst method or by using some empirical relationship like one proposed by Cook (1999) to convert 3 second gust wind speed to mean hourly wind speeds or use a constant value as a conversion factor as ICEUK proposed 1.06 for convert mean hourly wind speeds to 10 minute mean wind speed. The basic wind speeds used for different standards is shown in Table 2.

Table 1. Three second gust velocities used for different areas of Sri Lanka (Design manual "Design building for high winds", 1978)

| Wind Zone | Post disaster structures ($ms^{-1}$) | Normal structures ($ms^{-1}$) |
|---|---|---|
| Zone 1 | 54 | 49 |
| Zone 2 | 47 | 42 |
| Zone 3 | 38 | 33 |

Table 2. Basic wind speeds with different averaging time

| | Zone 1 ($ms^{-1}$) | | Zone 2 ($ms^{-1}$) | | Zone 3 ($ms^{-1}$) | |
|---|---|---|---|---|---|---|
| | Normal structure | Post disaster structure | Normal structure | Post disaster structure | Normal structure | Post disaster structure |
| CP 3 : Chapter V : Part 2 : 1972 (3 second gust wind speed) | 49 | 54 | 43 | 47 | 33 | 38 |
| BS 6399 - 2:1997 (Mean hourly wind speed) | 27 | 30 | 24 | 26 | 18 | 21 |
| BS EN 1991-1-4:2005 (10 minutes mean wind speed) | 28 | 32 | 25 | 28 | 19 | 22 |
| AS 1170.2 -1989 (3 second gust wind speed) | 49 | 54 | 43 | 47 | 33 | 38 |
| AS/NZS 1170.2:2002 (3 second gust wind speed) | 49 | 54 | 43 | 47 | 33 | 38 |

### *2.3. Pressure coefficient*

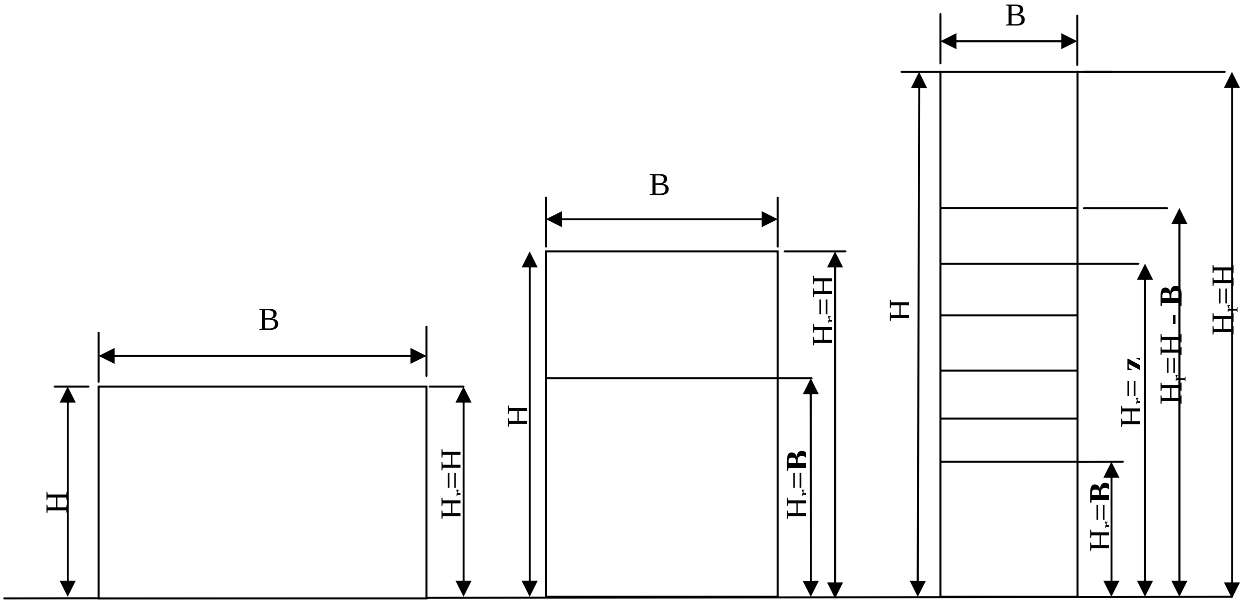
The total pressure mainly depends on three factors namely external pressure coefficients, internal pressure coefficients and wind speed at that height. The external pressure coefficients ($C_{pe}$) used in international standards are different from one another due to their own methods of determinations and the policies adopted by the country. Internal pressure coefficient values are also not same in codes due to their national practices. The external and internal pressure coefficient values used for 183 m high building with rectangular plan dimension of 46 m x 30 m are shown in Table 3.

Table 3. External and internal pressure coefficient

| Standard | | $C_{pe,windward}$ | $C_{pe,leeward}$ | $C_{pi}$ |
|---|---|---|---|---|
| CP3 Chapter V-Part 2: 1972 | 46 m side | +0.70 | -0.40 | +0.2 or -0.3 |
| | 30 m side | +0.80 | -0.10 | |
| BS 6399.2:1997 | 46 m side | +0.80 | -0.30 | -0.3 or +0.2 |
| | 30 m side | +0.80 | -0.30 | |
| BS EN 1991-1-4:2005 | 46 m side | $C_f$= 1.3 | | |
| | 30 m side | +0.80 | -0.65 | -0.3 or 0.0 |
| AS 1170.2:1989 | 46 m side | +0.80 | -0.50 | -0.2 or 0.0 |
| | 30 m side | +0.80 | -0.39 | |
| AS/NZS 1170.2:2002 | 46 m side | +0.80 | -0.50 | -0.2 or 0.0 |
| | 30 m side | +0.80 | -0.39 | |

### *2.4 Pressure distribution along the building height*

Newer codes like British and Euro codes use 'division – by - parts' rule to distribute wind pressure along the building height as shown in Figure 1. However, in CP 3 Chapter V and Australian standards use dynamic wind pressure continuously change with the building height. The suction pressure variation in leeward side is not defined in many codes except Australian codes defined it as uniform pressure and value is equal to the pressure at top of the building.




One part when H<B Two parts when B<H<2B Multiple parts when H>2B

Figure 1. Division of buildings by parts for lateral loads (BS 6399.2:1997)

### *2.5 Analysis methods used in different standards*

CP 3 Chapter V – Part 2:1972 uses quassi –static method to assess the wind loads on the building, which is more suitable When the structure is very stiff, the deflections under the wind loads would not be significant and the structure is said to be 'static' (Dyrbre, 1999). However, slender structures are more susceptible to dynamic motion in both parallel and perpendicular to the directions of the wind. Dynamic analysis used in AS 1170.2:1989 is the gust factor method, which uses stochastic dynamics theory to translate the dynamic amplification of loading, caused by turbulence and the dynamic sensitivity of the structure, into an equivalent static loading (Kijewski and Kareem, 2001). However, new version of the Australian Standards AS/NZS 1170.2:2002 uses a dynamic factor which encounters factors such as background factor, resonant factor, etc. The dynamic analysis method used in BS 6399.2:1997 is based on equivalent static method with dynamic augmentation factor which depends on building type and this method limits to use with building less than 200 meter high. Euro code defines a factor called structural factor should take into account the effect on wind actions from the non simultaneous occurrence of peak wind pressures on the surface together with the effect of the vibrations of the structure due to turbulence.

## 3. Case study for comparison

A 183 m high rectangular shaped building was modelled and analysed by using SAP 2000 software, in order to determine dynamic behaviour of tall building and the effect of using various standards to calculate the wind induced behaviour. The plan dimensions of the building are 46 m x 30 m (Figure 2(a)). The building is typical column - beam frame structures with service core of shear walls. Within the service core, all lifts, ducts and toilets are located. The hard zoning lift system was used for the building to simulate a more actual scenario. The diaphragm constraint was used for slabs to move all points of the slabs together. Other than the dead load of the structural members, super

imposed and live loads were applied in the model according to the BS6399: Part 1: 1996. Wind loads on the building were calculated for all three wind zones as given in the design manual and applied with respect to two orthogonal directions as joint loads at the column - beam junctions on the wind ward and leeward faces separately as shown in Figure 2 (b).
Wind forces were calculated as provisions given in different wind loading standards by encountering different factors and methods. For British and Euro codes, wind loads were calculated according to the division –by –parts rule. Only for wind zone 1, importance factor 1.1 has used with special terrain – height multiplier as given in AS 1170.2:1989, but only higher terrain –height multiplier used for cyclonic region to calculate wind loads according to the AS/NZS 1170.2:2002.

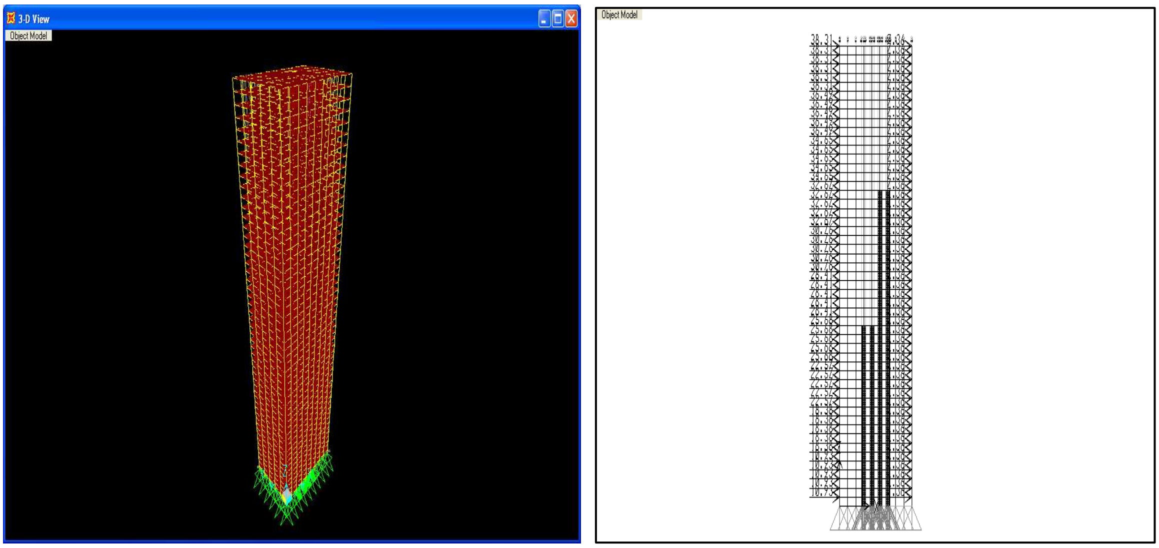



Figure 2: (a) Finite element 3 – D model of 183 m height building (b) Wind loads applied in windward and leeward sides of the 183 m high building

## 4. Wind Pressures and Wind induce forces

### *4.1 Comparison of wind pressure*

The calculated wind pressure values by using different standards are showing in Table 4 and the difference of those pressure values compared with respect to the CP 3 Chapter V – Part 2:1972, which uses quassi –static approach to calculate wind pressure.

Table 4: Comparison of wind pressure at 183m height in zone 3

| | CP3 Chapter V-Part2:1972 | BS 6399.2 : 1997 | BSEN 1991-1-4 :2005 | AS 1170.2: 1989 | AS 1170.2: 2002 |
|---|---|---|---|---|---|
| Basic wind speed ($ms^{-1}$) | 38 | 21 | 22 | 38 | 38 |
| Terrain height multiplier | 1.172 | 2.18 | 1.71 | 0.806 | 1.23 |
| Design wind speed ($ms^{-1}$) | 44.54 | 45.78 | 37.69 | 33.69 | 46.74 |
| Dynamic wind speed ($Nm^{-2}$) | 1190 | 1257 | 1227 | 680 | 1311 |
| Dynamic response factor | - | 1.125 | 0.972 | 2.085 | 0.918 |
| External pressure coefficient-Windward | +0.7 | +0.8 | +0.8 | +0.8 | +0.8 |
| External pressure coefficient-leeward | -0.4 | -0.3 | -0.65 | -0.5 | -0.5 |
| Internal pressure coefficient | 0.2 | 0.2 | -0.3 | -0.2 | -0.2 |
| Total Pressure at 183 m height ($kNm^{-2}$) | 1.309 | 1.335 | 1.663 | 1.802 | 1.564 |
| % difference at the top with respect to CP3 Chapter V-Part 2:1972 | 0 | 2.0 | 27.0 | 37.7 | 19.5 |

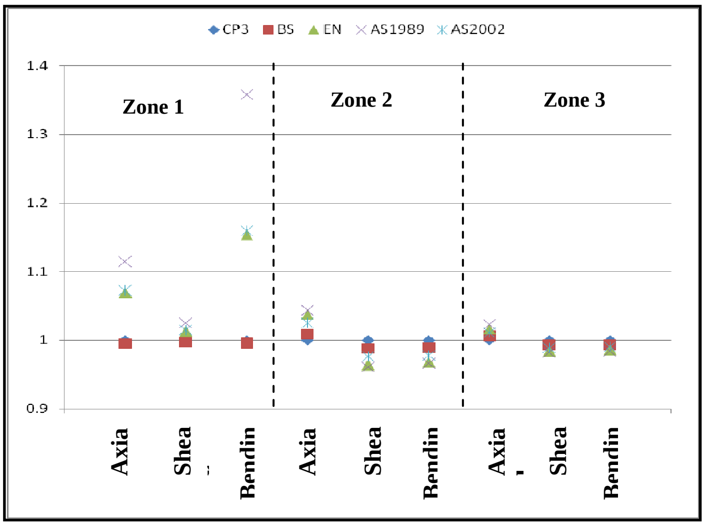
According to the Table 5, it can be seen that there is only 2% difference in pressure, when it calculated with CP 3 Chapter V-Part 2 and BS 6399.2:1997. The AS 1170.2:1989 standard has much larger pressure difference because the use of importance factor for its calculation. A higher percentage of pressure difference in Euro code is primarily resulted due to its higher negative pressure coefficient in leeward face of buildings. The pressure difference at the top most level of the building for AS/NZS 1170.2:2002 is about 20%, compared with the CP3 Chapter V-Part 2:1972, where both codes use 3 second gust wind speed for its calculations.

### *4.2. Comparison of structural member forces*

Wind loading standards only facilitate to calculate wind pressures at different heights of the building. Multiplying these values by contributory areas will enable the calculation of wind forces at a particular height. However, it is not the actual force experienced by the structural members such as beam, columns, etc. due to the various behaviours of a structure like load sharing among structural members. These actual member forces are necessary to design structural members against lateral loads such as wind load. Actual member forces can be obtained by using finite element 3-D model by applying forces derived from different standards. For the purpose of comparison, the obtained results are shown as normalised forces. This is the ratio between a force obtained from a particular standard and the same force obtained from CP 3 Chapter V- Part 2:1972, most common practice in Sri Lanka.
The member forces used for comparison in this study are maximum values of axial forces, shear forces and bending moments in columns, shear forces and bending moments in beams, base moment and base shear at the support level and maximum compressive stresses in shear wall.
The member forces are calculated for the following load combinations:

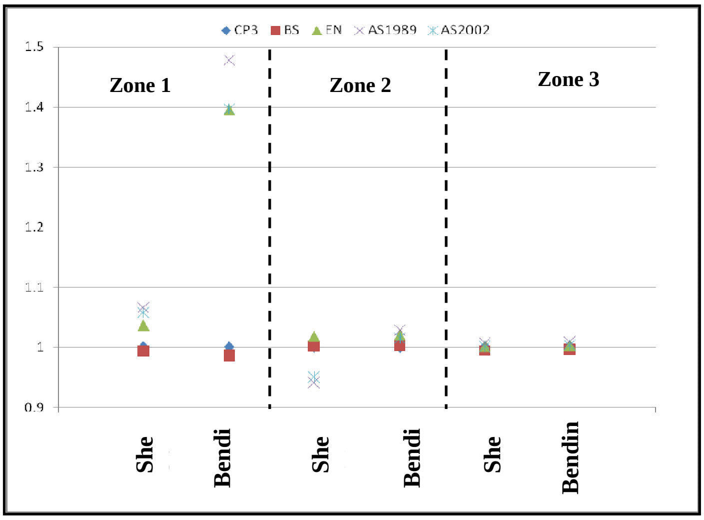
1. 1.2(Dead loads)+1.2(Live load)+1.2(Wind load)
2. 1.0 (Dead loads) + 1.4(Wind load)
3. 1.4 (Dead loads) + 1.4(Wind load) and
4. Wind load only.

Wind induced forces in columns and beams, on 183 m high building for governing load case 1.2G+1.2Q+1.2W in all three zones are shown in Figures 3(a) and (b), respectively.




(a)

Figure 3(a): Column loads for load combination 1.2G+1.2Q+1.2W
(wind flow perpendicular to 46 m long side)

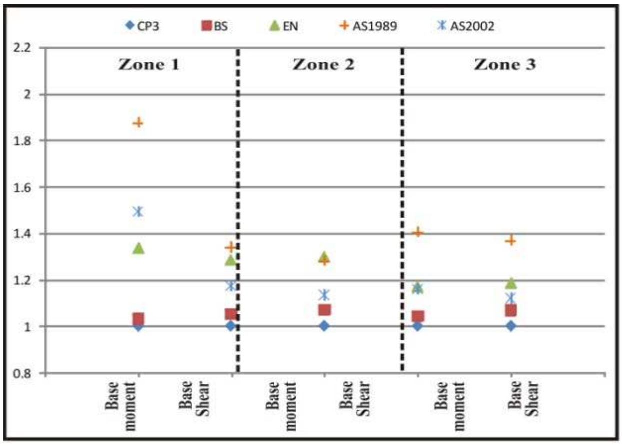



(b)

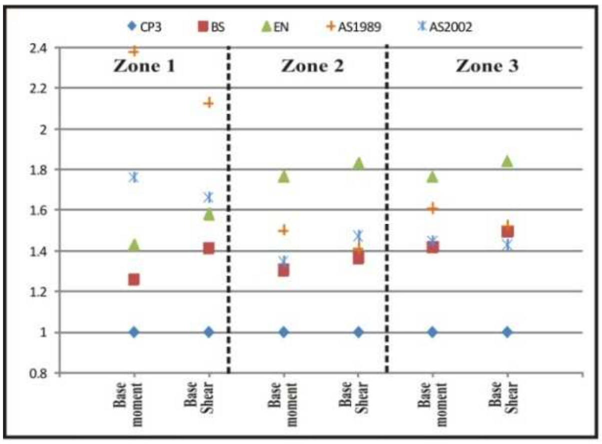
Figure 3(b): Beam loads for load combination 1.2G+1.2Q+1.2W
(wind flow perpendicular to 46 m long side)

The 183 m tall building is more susceptible to wind loading due to its exceptional height. However, the governing load can be observed for load combination 1.2G+1.2Q+1.2W. The variation in the zone 1 is much larger due to higher wind loads derived from Australian standards, especially for AS 1170.2:1989 which uses as importance factor 1.1 in zone 1. Normalised bending moment has maximum variation about 35% in column and about 48% for the beams. However, column maximum axial load variation is in the range of 10%. This value is as high as 17% when wind load is governing as in load combination 1.0G+1.4W. The bending moment value is higher as 50% for the column and more than 55% for beam bending moments for load combination 1.4G+1.4W. For wind load only case, the variation is much larger as 200% to 250% in zone 1 for Australian standards. It should be noted that although this variation is obtained with 1.0G + 1.4W and 1.4G + 1.4W cases, still the significance of this variation on design calculation would be to a lesser degree.

## 5. Base reactions




(a)

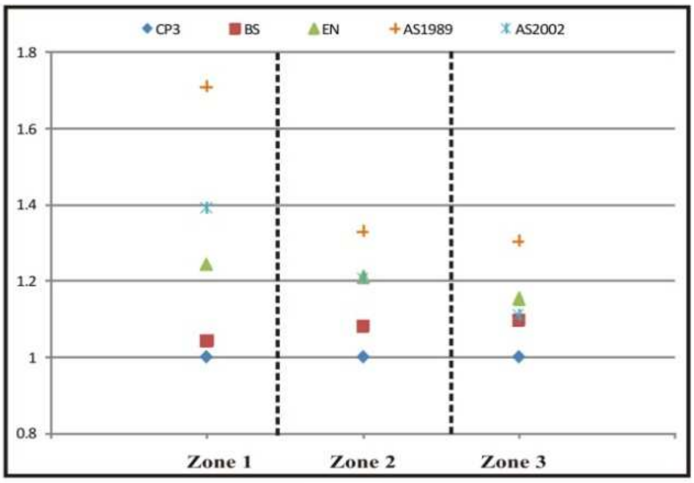



(b)

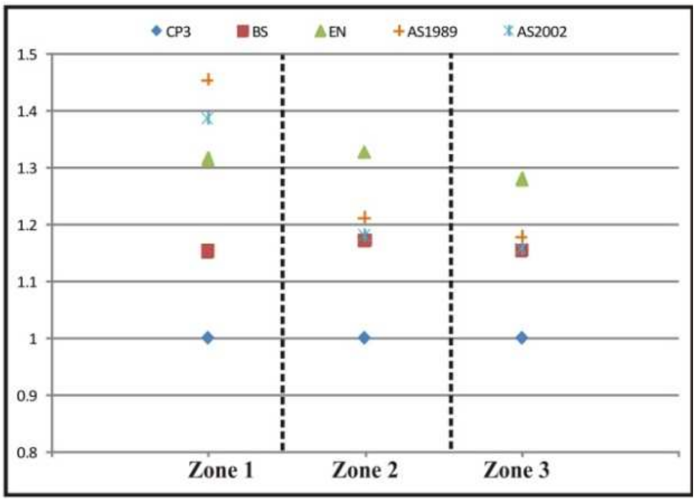
Figure 4: Base moment and base shear of the 183m building (a) wind flow perpendicular to 46 m wall (b) wind flow perpendicular to 30 m wall.

According to the Figures 4(a) and 4(b) maximum base moment and base shear can be observed for Australian codes, because of their higher wind speeds resulting from special terrain-height multiplier used in zone1. These values are almost twice the valued derived from CP 3 Chapter V: Part2:1978. However, these codes have a difference in 183 m building due to importance factor used by AS 1170.2:1989. In the zone 2 and zone 3, Euro codes yield higher base moment as well as base shears values. The maximum value 1.6 can be observed in zone 2 for wind flow perpendicular to 30 m side of the building. BS 6399.2:1997 has almost same values for base moment and base shear for 183 m building when wind flow perpendicular to 46 m long side.

## 6. Maximum shell stress




(a)




(b)

Figure 5: Maximum shell stress in shear wall of the 183m building (a) wind flow perpendicular to 46 m side (b) wind flow perpendicular to 30 m side

The absolute maximum principal stresses in the shear walls, which are induced by the dead live and wind load were used for the comparison purpose and results are shown in Figures 5(a) and 5(b). Maximum shell stress can be observed for wind load derived by using AS 1170.2:1989 for the building. The maximum normalized value is 1.7 in 183 m high building. For 183 m high building wind loads derived from Euro code exert maximum shell stress in zone 2 and 3.

## 7. Drift index

Wind loading standards and design codes limit the allowable wind drift of the buildings in order to prevent damage to the cladding, partition and interior finishes, to reduce effect of motion perceptibility and to limit the P–Delta or secondary loading effects (Mendis et al, 2007). Therefore, drift limit is checked for 183 m tall building in order to determine whether the buildings would exceed the drift index limit or not. The maximum values of deflection in serviceability limit condition were obtained by wind loads applying to the finite element 3-D model for all three zones. According to the BS 8110-Part 2: 1985 the maximum allowable deflection is calculated as $h_s/500$, where $h_s$ is the storey height for single storey building. Therefore, maximum allowable deflection value calculated for 183 m height building is 366 mm. The average drift index is defined as a ratio between maximum deflections to total height of the building. The calculated drift index values are shown in Table 5.

Table 5: Drift index for 183 m height building in zone 1, 2 and 3

| Wind loading standard | Average drift Index | | |
|---|---|---|---|
| | Zone 1 | Zone 2 | Zone 3 |
| CP 3 Chapter V - Part 2:1972 | 1/961 | 1/1250 | 1/1785 |
| BS 6399.2:1997 | 1/935 | 1/1219 | 1/1754 |
| AS 1170.2:1989 | 1/425 | 1/862 | 1/1471 |
| AS/NZS 1170.2:2002 | 1/565 | 1/1020 | 1/1562 |
| BS EN 1991-1-4:2005 | 1/561 | 1/1010 | 1/1538 |

The generally acceptable average drift index limit for the high rise building is 1/500 (Mendis et al., 2001). By reference to Table 2, only the building model with wind loads derived from AS 1170.2:1989 in zone 1 exceeds the generally accepted drift limit because it uses both importance factor and the cyclonic terrain-height multiplier. However, rest of the cases satisfies the drift index requirement. In zone 3, all models have lower drift values, which are approximately half of the threshold value.

## 8. Along wind and cross wind acceleration

Only Australian and Euro codes facilitate the calculation of acceleration at top of the building. Euro code provides a method to calculate only the along wind acceleration, while both Australian standards provide methods to calculate both along wind and cross-wind accelerations. Australian standards use 5 years return period wind speeds for calculate serviceability limit state conditions, which is obtained by using probabilistic method proposed in BS 6399.2:1997 as shown in Table 6.

Table 6: Wind speeds used for acceleration calculations

| Return period | Wind speed ($ms^{-1}$) | | |
|---|---|---|---|
| | Zone 1 | Zone 2 | Zone 3 |
| 50 – years | 54 | 47 | 38 |
| 5 - years | 46 | 40 | 32 |

The acceleration values obtained from the calculations are shown in Table 7.

Table 7: Acceleration values at 183 m height in zone 1, 2 and 3

| Zone | Wind direction | Acceleration type | Acceleration (ms-2) | | |
|---|---|---|---|---|---|
| | | | AS 1170.2: 1989 | AS/NZS 1170 .2: 2002 | BS EN 1991-1-4: 2005 |
| Zone 1 | Normal to 46 m side | Along wind | 0.155 | 0.156 | 0.134 |
| | | Cross wind | 0.239 | 0.233 | - |
| | Normal to 30 m side | Along wind | 0.109 | 0.107 | 0.094 |
| | | Cross wind | 0.227 | 0.221 | - |
| Zone 2 | Normal to 46 m side | Along wind | 0.076 | 0.078 | 0.080 |
| | | Cross wind | 0.173 | 0.166 | - |
| | Normal to 30 m side | Along wind | 0.051 | 0.052 | 0.058 |
| | | Cross wind | 0.168 | 0.159 | - |
| Zone 3 | Normal to 46 m side | Along wind | 0.034 | 0.034 | 0.033 |
| | | Cross wind | 0.118 | 0.116 | - |
| | Normal to 30 m side | Along wind | 0.024 | 0.026 | 0.025 |
| | | Cross wind | 0.106 | 0.093 | - |

Euro code yields higher along wind acceleration values than Australian codes. However, these along wind acceleration values are much less than across wind acceleration values due to slenderness of 183 m height building. Most of the cases, these across-wind acceleration values could exceed the threshold value set for human comfort that is 0.15 ms$^{-2}$. Even for higher wind speed value in zone 1, along wind acceleration values do not reach the threshold value set for human comfort.

## 9. Conclusion

International wind loading standards have their own prefferences over choice of different basic wind speeds with averaging time and analysis methods and pressure coefficient values. Therefore, for the same building design, wind pressures obtained from different standards are not the same. The dynamic analysis methods give higher wind pressure values compared to the values obtained from the quassi- static method. However, wind pressure values obtained from the dynamic analysis are varying because of strategies adopted by standards such as pressure distribution according to the "Division –by- Parts" rule, higher pressure coefficient values, etc. Ultimately the building design should have sound safe and satisfactory behaviour in both serviceability and ultimate limit states. Therefore, the use of higher terrain height multiplier can be justified for the wind zone 1 in Sri Lanka for ultimate limit state design, which has higher probability of being hit by a cyclone. However, use of both terrain-height multiplier and an importance factor may lead to a more conservative design and thus, it is recommended not use both in one design. The cross-wind acceleration is more important than the along wind acceleration for tall slender buildings. According to the case study, for the wind loads derived from previous Australian standard exceeds the threshold value in zone 1, because it used both importance factor cum higher terrain height multiplier for its calculation. The same trend can be observed in drift index calculation that allowable drift index value was exceeded by the AS 1170.2:1989 in zone 1 but for other standards they are within the limit. Hence, an international standard used in any other country than its origin, it is recommended to carry out a detail analysis in order to determine the strategies adopted by those standards.

**Acknowledgement**
Authors of this paper like to express their gratitude towards National Disaster Management Center (NDMC), who provided financial support in throughout of this study. They also wish to thank the Civil Engineering Department of University of Moratuwa for facilitating their research.

**References**

1. Australian standard for wind loads ; AS 1170.2:1989, Standards Australia, New South Wales, Australia
2. Australian and New Zealand standards: Structural design actions Part 2: wind actions; AS/NZS 1170.2:2002, Standards Australia, New South Wales, Australia
3. British Standard: Eurocode 1: Actions on Structures – Part1- 4: General actions - wind actions; BS EN 1991-1-4:2005 , British Standard Institution, London
4. British Standard: Loading for buildings - Part 2: Code of Practice for wind loads; BS 6399- 2:1997, British Standard Institution, London
5. Clarke, A.G., Swane, R.A., Schneider, L.M, Shaw, P.J.R., Technical assistance to Sri Lanka on Cyclone Resistant Construction, Vol 1, Part 1 -4, 1979
6. Cook, N. J., Wind loading, A practical guide to BS 6399-2 Wind loads for buildings, Thomas Telford, 1999
7. CP 3 Chapter V: 1972, Code of Basic data for the design of buildings chapter V. Loading, Part 2 Wind Loads, British Standard Institution, London
8. Cyclone Events 1900 – 2000, Available Source, www.meteo.gov.lk, last acces on 03/09/2010
9. Dyrbye, C., Hansen, S., O., Wind Loads on Structures, John Wiley & Sons, 1999
10. Karunaratne, S.A., High rise buildings in Colombo (Structural Engineer's View), Proceedings of the international conference on "Advances in Continuum Mechanics, Materials Science, Nano science and Nano technology: Dedicated to Professor Munidasa P. Ranaweera", University of Peradeniya, Sri Lanka, 2008. pp 291- 302
11. Kijewski, T., Kareem. A., Dynamic wind effects, A comparative study of provision in codes and standards with wind tunnel data, 2001
12. Kolouske, V., Pirner, M., Fischer, O., Naprstek, J., Wind effects on civil engineering structures, Elsevier, 1984
13. Mendis, P., Ngo, T., Hariots, N., Hira, A., Samali, B., Cheung, J., Wind Loading on Tall buildings, EJSE special issue; Loading on structures, 2007. pp 41-54
14. Report on the Calibration of Euro code for wind loading (BS EN 1991 - 4) and its UK National Annex against the current UK wind code (BS 6399: Part 2:1997)

**About the Authors**

**A.U.WEERASURIYA,** B.Sc. Hons. Moratuwa, AMIE (SL) is a Research Assistant at the Department of Civil Engineering, University of Moratuwa. His research interests are in the areas of wind effects on structures, wind load codification and computational fluid dynamics.

**M. T. R. JAYASINGHE,** B.Sc. Hons. Moratuwa., Ph.D. Cambridge, MIE (SL), C.Eng. is a Professor at the Department of Civil Engineering, University of Moratuwa. His research interests are in the areas of tall buildings, energy efficiency, sustainable development and disaster resistant structures.

# ON THE VARIABILITY OF TREND TEST RESULTS

[1]A. Ramachandra Rao and M. Azli

[1]Department of Civil Engineering
Faculty of Engineering, University of Malaya, 50603 Kuala Lumpur, Malaysia.
[1]Tel: (603) 7967-5266, (6012) 314-5757 ; Fax: (603) 7967-5318
[1]E-mail: rao@um.edu.my, rao@ecn.purdue.edu

[1]Corresponding author: A. Ramachandra Rao

**Abstract**

Trend tests are used to investigate statistical significance of trends. The popular Mann–Kendall (MK) trend test was originally proposed for random data. It was later modified to handle correlated data. After the scaling hypothesis was introduced, the MK test was further modified to accommodate it. The results from these three versions of the MK test can be very different. The objective of the present paper is to illustrate these variations in the MK trend test results. Not considering these variations would lead to spurious conclusions about statistical significance of trends in data with associated erroneous deductions. Monthly temperature data from Malaysia are used for illustration.

**Keywords:** *Mann–Kendall trend test; Correlations; Scaling hypothesis; Monthly temperatures; Malaysia*

## 1.0 Introduction

Trend tests have been used to investigate the impacts of climate change and variability in hydrologic time series in different parts of the world. Trends in various series have been investigated: in Japanese precipitation series (Xu *et al.*, 2003); in Yangtze basin in China (Zhang *et al.*, 2006); in precipitation in Seoul, Korea (Wang *et al.*, 2006). Earlier studies include those by World Meteorological Organization (1988), Mitosek (1992), Chiew and McMahon (1993) and Burn (1994). In many of these studies tests based on assumption of randomness in data are used. With the exception of papers by Hamed (2008) and Kumar *et al.* (2009) the effect of scaling on trend detection is not considered.

A widely used non-parametric test for detecting trends in time series is the Mann–Kendall (MK) test (Mann, 1945; Kendall, 1975).The null hypothesis in the MK test is that data are random and independent, i.e. there is no trend or serial correlation among observations. However, observed hydrologic and climatic time series, especially monthly data, are generally autocorrelated. The autocorrelations in observed data will lead to misinterpretation of results of trend tests. This situation was recognized early by Cox and Stuart (1955) who stated that “positive serial correlation among the observations would increase the chance of significant answer even in the absence of a trend”. Problems in interpreting confusing trend test results explain in part the variety and even contradictory results reported from them.

Modifying the tests for trends to account for the effect of serial correlation in data and using the modified tests has been the approach used by several investigators. Lettenmaier (1976) and Hirsch and Slack (1984) were early investigators who considered the effect of serial correlation on the results from trend tests. Hamed and Rao (1998) introduced a modified MK trend test for autocorrelated data with arbitrary correlation structure.

The effect of scaling on trend detection was investigated by Hamed (2008). By using simulated fractional Gaussian series, Hamed (2008) demonstrated that the null hypothesis of no trend was rejected by the MK trend test by as small a percentage as ten percent for random data to as high as sixty percent for data with the Hurst parameter $H$ of 0.9. The number of rejections increases with increasing $H$ and decreases with lower significance levels. Because of the symmetry of the test statistic, which is not affected by scaling, both the false positive and negative trends occur in equal proportions. These results point out the importance of testing for scaling effects in trend tests. The objective of the research reported herein is to present the variation in results from trend tests depending on the assumptions on which the tests are based.

Monthly Malaysian temperature data from two stations are used in the study. Temperature data from the past three decades have been selected for study because global warming and its effects became prominent during this period (Fig. 1) Climate change and its effects started attracting attention and investigation during this period. Monthly temperature data from Alor Setar in Kedah and Senai in Johor are used in the study (Fig. 2). Alor Setar is located in the north of Peninsular Malaysia while Senai is located in the south. The duration of data is from 1979 to 2007.

*Fig. 1*
*Fig. 2*

Three tests, the MK test (Mann, 1945; Kendall, 1975), the modified MK test (Hamed and Rao, 1998), and the MK test under the scaling hypothesis (Hamed, 2008), are used in the study. Because the details of the test are available in these references, they are briefly discussed next.

## 2.0 Tests used in the study

### *2.1 MK test*

Consider a time series $X = [x_1, x_2, \ldots x_n]$. The test statistic $S$ is computed by Eq. (1).

$$S = \sum_{i<j} a_{ij} \tag{1}$$

$$a_{ij} = sgn(x_i - x_j) = sgn(R_i - R_j) = \begin{cases} 1 & x_i < x_j \\ 0 & x_i = x_j \\ -1 & x_i > x_j \end{cases} \tag{2}$$

where $R_i$ and $R_j$ in Eq. (1) are the ranks of observations $x_i$ and $x_j$ respectively of the time series. Assuming that the data are independent and identically distributed, Kendall (1975) showed that

$$\left.\begin{array}{l} E(S) = 0 \\ V_0(S) - n(n-1)(2n+5)/18 \end{array}\right\} \tag{3}$$

Kendall (1975) also showed that the significance of trends can be tested by comparing the standardised variable $u_1$ in Eq. (4) with the standard normal variate at a significance level α.

$$u_1 = \begin{cases} \frac{S-1}{\sqrt{V_0(S)}}, & S > 0 \\ 0 & S = 0 \\ \frac{S+1}{\sqrt{V_0(S)}} & S < 0 \end{cases} \tag{4}$$

The basic assumption in this test is that the data are random. If the data are correlated then the correlation may be removed by pre-whitening the data. Alternatively, the variance $V_0(S)$ may be modified to account for the correlation. Such a modification to the MK test proposed by Hamed and Rao (1998) is discussed below.

### *2.2 Modified MK test*

$V_0(S)$ in Eq. (4) is recalculated in this test as $V^*(S)$ by using Eq. (5).

$$V^*(S) = V_0(S) \times \frac{n}{n_s^*} = \frac{n(n-1)(2n+5)}{18} \times \frac{n}{n_s^*} \tag{5}$$

In Eq. (5), $(n/n_s^*)$ represents a correction to $V_0(S)$ because of the autocorrelations in the data. The approximation used for $(n/n_s^*)$ is the empirical expression in Eq. (6).

$$\frac{n}{n_s^*} = 1 + \frac{2}{n(n-1)(n-2)} \times \sum_{i=1}^{n-1}(n-i)(n-i-1)(n-i-2) \times \rho_s(i) \quad (6)$$

In Eq. (6), $\rho_s(i)$ are the autocorrelation coefficients of the ranks of the data.

As the ranks of the observations of observations are used in Eq. (6), $V^*(S)$ is computed without using either the data or their autocorrelation function. In the present study, significant correlation coefficients up to $N/10$ of $N$ ranks are used. The modified statistic $u_2$ is computed and tested for significance.

$$u_2 = \begin{cases} \frac{S-1}{\sqrt{V^*(S)}}, & S > 0 \\ 0 & S = 0 \\ \frac{S+1}{\sqrt{V^*(S)}} & S < 0 \end{cases} \quad (7)$$

### *2.3 MK test under the scaling hypothesis*

In this test, the data are detrended by using Sen's (1968) non-parametric trend estimator. The scaling coefficient $H$ is obtained by maximising log likelihood function in McLeod and Hipel (1978). This estimate of $H$ is approximately normally distributed for the uncorrelated case when true $H$ is 0.5 with the mean and variance given by Eqs. (8).

$$\left.\begin{aligned} \mu_H &= 0.5 - 2.874n^{-0.9067} \\ \sigma_H &= 0.77654n^{-0.5} - 0.0062 \end{aligned}\right\} \quad (8)$$

The significance of $H$ is tested by using $\mu_H$ and $\sigma_H$ in Eqs. (8). If H is significant, the trend test under the scaling hypothesis is conducted. The modified variance of the test statistic is computed by using Eq. (9).

$$V(S) = \sum_{i<j}\sum_{k<l} 2/\pi \times \sin^{-1}(r_{ijkl}) \quad (9)$$

where:

$$r_{ijkl} = \frac{\rho_{jl} - \rho_{il} - \rho_{jk} + \rho_{ik}}{\sqrt{(2-2\rho_{ij})(2-2\rho_{kl})}} \quad (10)$$

The variance $V(S)$ in Eq. (9) is corrected for bias by multiplying it with the factor $B$ in Eq. (11).

$$B = a_0 + a_1H + a_2H^2 + a_3H^3 + a_4H^4 \quad (11)$$

The coefficients $a_0$, $a_1$,... $a_4$ in Eq. (11) are functions of the sample size $n$ and are found in Hamed (2008). The modified test statistic $u_3$ is computed by using the modified variance and Eq. (4). If $u_3$ is significant, then the trend is significant; otherwise, it is not. The test under the scaling hypothesis is conducted only if the decisions from MK or modified MK tests are significant.

## 3.0 Data analysis and results

### *3.1 Results of the MK test*

The values of the statistic $S$ and the variance $V_0(S)$ for the data from Alor Setar are 10,457 and 4,702,775, respectively. The statistic $u_1$ is 4.822, and is significant at 10%, 5% and 2.5% levels. Therefore the conclusion is that the Alor Setar temperatures have a strong positive trend. The values of $S$ and $V_0(S)$ for data from Senai in Johor are, respectively, 11,650 and 4,702,775. The value of $u_1$ for Senai is 5.372 which is larger than $u_1$ for Alor Setar. Therefore the conclusion from this test may be that the positive trend in Senai data is stronger than that in Alor Setar. Depending only on these results one may conclude that there is a north-south gradient in the Malaysian temperature trend. But we will have to consider the strong correlation in monthly temperature data and perform the modified MK test.

### *3.2 Results of the modified MK test*

For the data from Alor Setar, the values of the modified variance $V^*(S)$, the variance inflation factor $V^*(S)/V_0(S)$, and the statistic $u_2$ are 10,452,906, 2.223, and 3.234, respectively. $u_2$ is smaller than $u_1$ which is 4.822 due to the effect of correlation in the data. $u_2$ is also significant at 10%, 5%, and 2.5% levels, and is positive which indicates an increasing trend in temperature. The values of $V^*(S)$, $V^*(S)/V_0(S)$, and $u_2$ for Senai are 28,410,496, 6.041, and 2.186, respectively. $u_2$ for Senai has decreased from 5.372 to 2.186, a reduction of 59.3%. $u_2$ is positive for both data sets indicating increasing trends in temperature. For both sets of data, $u_2$ is significant at 10%, 5%, and 2.5% significance levels. $u_2$ for Senai is smaller than that for Alor Setar which is opposite to the behaviour of $u_1$. As $u_2$ is statistically significant, the MK test under the scaling hypothesis is conducted to test the significance of the test statistic.

### *3.3 Results from the MK test under the scaling hypothesis*

Before performing the MK test under the scaling hypothesis, Hurst's parameter $H$, and mean and standard deviation of $H$ are estimated. The statistical significance of $H$ is tested and if $H$ is found significant, the MK test is performed under the scaling hypothesis. Otherwise, inferences from the previous tests are accepted. Accordingly, the $H$ value for Alor Setar, its mean and standard deviation are estimated to be 0.92, 0.486, and 0.035, respectively. The $H$ value for Senai is 0.90, and its mean and standard deviation are the same as for Alor Setar data. The $H$ estimates for both data sets are statistically significant. They are also close to unity which indicates that MK test should be run under the scaling hypothesis.

The bias-corrected variance $V(S)$, the variance inflation factor $V(S)/V_0(S)$, the bias correction factor $B$, and the statistic $u_3$ for the test under the scaling hypothesis are computed. For the Alor Setar data, the values of $V(S)$, $V(S)/V_0(S)$, $B$, and $u_3$ are 36,290,000, 7.717, 3.196, and 0.971, respectively. $u_3$ is statistically insignificant, and has decreased to 0.971 from $u_2$ of 3.234. Because of the high $H$ value, the variance inflation factor $V(S)/V_0(S)$ is quite large and so is the bias correction factor $B$. Consequently the trend is statistically insignificant. The values of $V(S)$, $V(S)/V_0(S)$, $B$, and $u_3$ for the Senai data are 54,190,000, 11.523, 2.255, and 1.054 respectively. In this case, $u_3$ is also insignificant indicating the statistical insignificance of the trend in the temperature data.

## 4.0 Summary and conclusions

As the example discussed above clearly illustrates, the MK test statistic is strongly affected by correlation in the data and by the scaling factor $H$. Conclusions drawn without considering these factors can be misleading or even wrong. Although the trend statistic $u_3$ is insignificant, plots of the temperature data in Malaysia during these years show an overall, general, gradual warming trend. But this trend is statistically insignificant in the data from all the stations. The situation is "mixed" in the sense that there are increasing but statistically insignificant trends in Malaysian monthly temperature data.

**Acknowledgements**

We would like to extend our gratitude Dr. Khaled H. Hamed of Cairo University for clarifying several points in his paper, to the Vice-Chancellor Professor Datuk Dr. Ghauth Jasmon and the Dean of the Faculty of Engineering Professor Dr. Mohd Hamdi Abdul Shukor of the University of Malaya for their support. The Department of Irrigation and Drainage, Ministry of Natural Resources and

Environment, Malaysia, provided the data used in this study. Their help is acknowledged. We would also like to thank the University of Malaya for partially supporting this work.

## References

Burn, D.H. (1994). Hydrologic effects of climatic change in West Central Canada. *J. Hydrol.*, 160, pp. 53–70.

Chiew, F.H.S., McMahon, T.A. (1993). Detection of trend or change in annual flow of Australian rivers. *Int. J. Climatol.*, 13, pp. 643–653.

Cox, D.R., Stuart, A. (1955). Some quick sign tests for trend in location and dispersion. *Biometrika*, 42, pp. 80–95.

Hamed, K.H. (2008). Trend detection in hydrologic data: the Mann-Kendall trend test under the scaling hypothesis. *J. Hydrol.*, 349, pp 350–363.

Hamed, K.H., Rao, A.R. (1998). A modified Mann-Kendall trend test for autocorrelated data. *J. Hydrol.*, 204, pp. 182–196.

Hansen, J., Ruedy, R., Sato, M., Lo, K. (2010). *Global surface temperature change*. Draft only (dated 1 June 2010). NASA Goddard Institute for Space Studies, New York, New York, USA. Available at: http://data.giss.nasa.gov/gistemp/ [Accessed 1 July 2010].

Hirsch, R.M., Slack, J.R. (1984). Non-parametric trend test for seasonal data with serial dependence. *Water Resour. Res.*, 20(6), pp. 727–732.

Kendall, M.G. (1975). *Rank correlation methods*. Griffin, London.

Kumar, S., Merwade, V., Kam, J., Thurner, K. (2009). Streamflow trends in Indiana: effects of long term persistence, precipitation and subsurface drains. *J. Hydrol.*, 374, pp. 171–183.

Lettenmaier, D.P. (1976). Detecting trends in water quality data from records with dependent observations. *Water Resour. Res.*, 12(5), pp. 1037–1046.

Mann, H.B. (1945). Nonparametric tests against trend. *Econometrica*, 13, pp. 245–259.

McLeod, A.I., Hipel, K.W. (1978). Preservation of the rescaled range: 1. a reassessment of the Hurst phenomenon. *Water Resour. Res.*, 14(3), pp. 491–508.

Mitosek, H.T. (1992). Occurrence of climate variability and change within the hydrologic time series: a statistical approach. CP-92-05, *International Institute for Applied Systems Analysis*, Laxenburg, Austria.

Sen, P.K. (1968). Estimates of the regression coefficient based on Kendall's tau, *J. Amer. Statistical Assoc.*, 63, pp. 1379–1389.

Wang, B., Ding, Q., Jhun, J.G. (2006). Trends in Seoul (1778–2004) summer precipitation. *Geophys. Res. Lett.*, 33, L15803

World Meteorological Organization. (1988). *Analyzing long time series of hydrological data with respect to climate variability.* WCAP-3 (WMO/TD-No.224).

Xu, Z.X., Takeuchi, K., Ishidaira, H. (2003). Monotonic trend and step changes in Japanese precipitation. *J. Hydrol.*, 279, pp. 144–150.

Zhang, Q., Liu, C., Xu, C.Y., Xu, Y., Jiang, T. (2006). Observed trends of annual maximum water level and stream flow during past 130 years in the Yangtze River basin, China. *J. Hydrol.*, 324, pp. 255–265.

## Figures

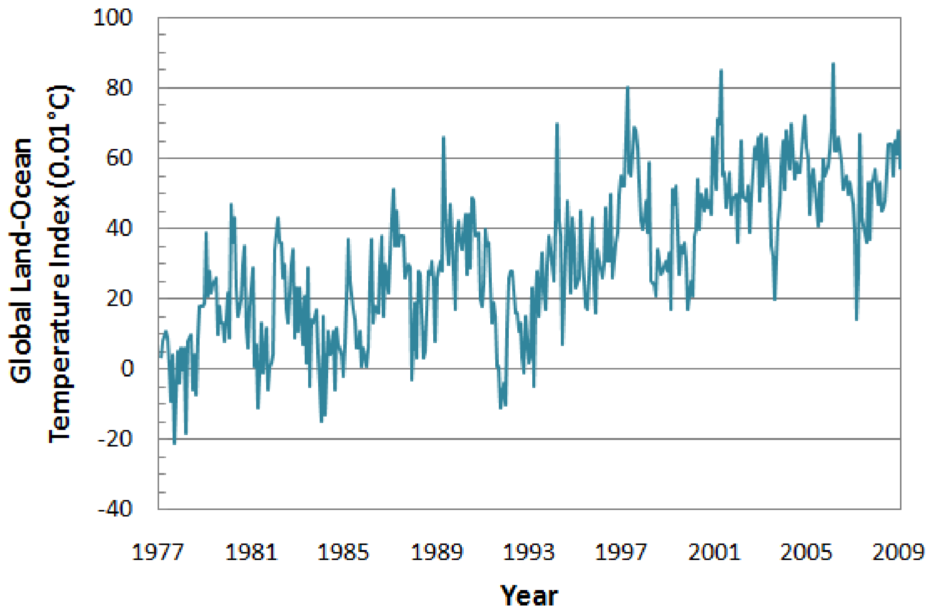



**Fig. 1.** *Global land-ocean temperature index for 1977–2009 (Hansen et al., 2010)*

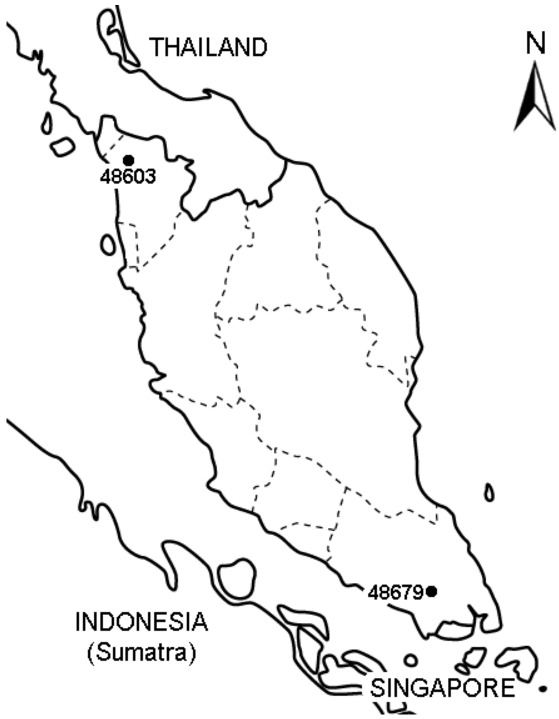



**Fig. 2.** *Locations of two meteorological stations in Peninsular Malaysia: Alor Setar in Kedah (48603) and Senai in Johor (48679)*

# SUSTAINABLE FARMING THROUGH MECHANIZATION: DEVELOPMENT OF A BUND MAKING MACHINE

Pathirana P. H. S[1], Bandara P. C[1], Ratnaweera A.C[1], Balasooriya C.P[2], Bandara M. A[2]

[1] Department of Mechanical Engineering, Faculty of Engineering, University of Peradeniya, Sri Lanka
[2] Department of Agriculture, Peradeniya, Sri Lanka

E-mail : prabpath@gmail.com

**Abstract**

As the decreasing rural population of Sri Lanka is becoming increasingly responsible for feeding the growing urban population, increase in productivity of agriculture has become an essential feature in stepping towards sustainability. As younger generation avoids the cultivation, the labour shortage has become very significant. To motivate the young people in to agriculture, mechanization plays a vital role. This paper describes about one such mechanization, which is done on replacing manual bund making process in low land paddy fields. Bund making is usually done in two steps using the mamotee; bund clearing and mud plastering. Hence two separate attachments have been developed for the two steps separately.

For bund clearing, a simple cutter with a height adjusting mechanism has been developed and tested. This attachment is designed for a two-wheel tractor as it was the most preferred tractor among Sri Lankan farmers.

A new concept was tested for bund plastering, as a direct attachment to the tractor. A closed rectangular channel which has a curve along its length has been suggested to convey mud towards the bund. The flowing ability of mud was supposed to be utilized in this new concept. Mechanizing the mud plastering process would be an entirely a new innovative concept and lot of empirical investigations are needed during the testing of the attachments. The two attachments were fabricated with a minimum cost and field-testing was conducted to check the proper functionality of the two attachments.

The bund clearing attachment worked successfully though several modifications were to be made. The mud plastering concept failed during testing. Approximately about 30-fold productivity gain in terms of time consumption is expected with the introduction of this mechanization compared to the conventional methods currently used by the farmers.

**Key Words**: Bund, Mechanization, Plastering, Clearing

## 1. Introduction

Sri Lanka has been an agricultural based country for centuries. The country's agriculture mainly depends on rice production and it was responsible for 30.5% of employment in the country in 2005, down from 36.8% in 1995[1]. At present the number has decreased much more as the younger generation refuses to perform hard work related to cultivation. As population in urban areas increases, the demand for food has been rapidly increased and the countries own food production has failed to cope with this growing demand within the past few years. As a result a sizable amount of rice is imported to the country, annually. To overcome this problem and to achieve sustainability in agriculture, development in the agricultural inputs was started to implement lately. One of the main areas of this is development of the agriculture related technologies. Among them, mechanization of agricultural activities plays a main role which could eventually give a solution to the growing labour shortage in agriculture. It also facilitates to increase the productivity of cultivation and create an efficient environment. Farm mechanization should be done in order to develop appropriate technologies to suit local conditions which are compatible with the socio-economic and field conditions available in Sri Lanka.

Paddy cultivation is mostly performed by the rural population of Sri Lanka. Around 879,000 farmer families are engaged in paddy cultivation each year and it represent 20% of country's population [2]. These people are rarely exposed to the modern advanced technologies. They are less adaptive to the expensive lifestyles due to their very low income. They mostly engage themselves within their villages and live a very simple life.

In this background, implementing new farm machinery could be a very challenging task. Even though there are lot of farm machineries introduced to farmers each year, only a very few of them become popular among the community [3]. Many of them failed due to the following reasons. Most of these machines are very expensive and due to scarcity of spare parts farmers are reluctant to buy such products. Difficulty in maintenance and handling can also be a major reason among them. It is also important to notice what facts the local farmers are mostly attracted when they are purchasing farm machinery. They often prefer engine driven mechanisms. Two wheel tractors is one of the most popular machinery among Sri Lankan local farmers. They also pay attention to the cost as well as the applicability of the machine to various activities. If the manual task takes lot of time and energy, they would prefer to go for an engine driven mechanized solution. That was the reason why the rotary tilling attachment and threshing machines have become very useful to the farmers. As successful machineries are implemented to cater the needs of farmers, the younger generation would be more encouraged to step in to cultivation. With successful mechanizations, people can be employed as operators, instead of labours. Hence it will eventually ensure the sustainability of the national food production.

Each task done during the paddy cultivation happens to be very hard and time consuming. Bund (or paddy dikes) preparation is one such difficult task which local farmers had been practicing it manually for centuries.
It is usually done in two steps at the beginning of the each crop season. First, the bund should be cleared from weeds and grass during initial plough. Then the bund should be plastered with a layer of mud after the second plough [4]. Good bunds help to limit water losses by seepage and under bund flows. Bunds should be well compacted and any rat holes should be plastered with mud. An average farmer would take approximately 30 mins for single side bund clearing and 45 mins for single side bund plastering of an 18 m long bund. Therefore, the main objective was to mechanize this bund making process, since it could be very helpful for farmers to save time, physical energy and make their work easy. The attachments for bund clearing and plastering should consist of the following features:

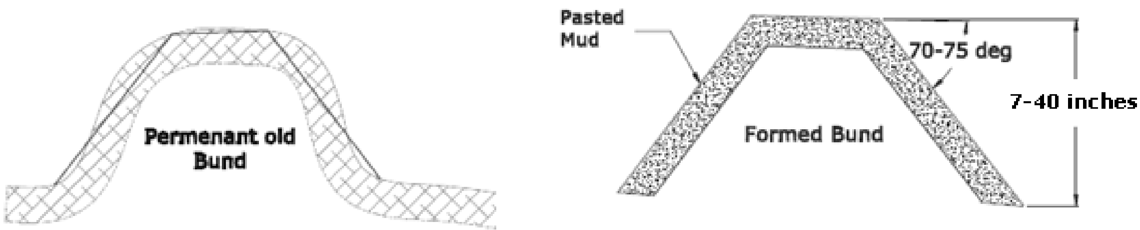
- Low cost
- A very simple mechanism which any farmer could understand
- Efficient than the manual bund preparation process
- Easy to operate for the local farmers
- Durable and which bear the rough conditions in the paddy field
- Easily maintainable
- Spare parts available

Mainly it should be developed to an affordable price so that any farmer could hire it during their cultivations.

## 2. Material and Methods

### *2.1 Literature Review*

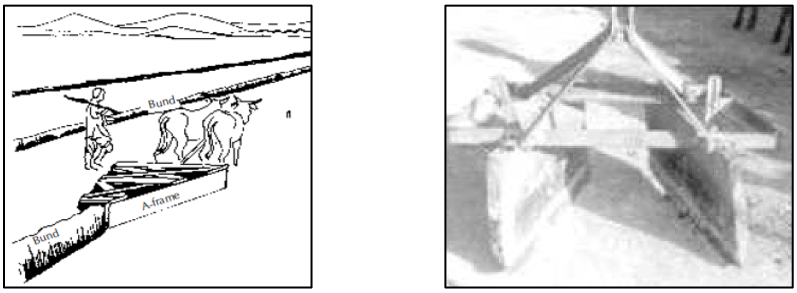
During bund clearing, usually the weeds are removed and the bunds are cut into an angle as shown in Fig 1(a). Then mud was plastered to create a uniform layer on the bund. This mud would moderately consist of water so that the mud can successfully retain on the slanted surfaces of the bund. The mamotee is used to lay a mud clod on the slanted surface of the bund and it was shaped up to form a uniform layer of mud as shown in Fig1 (b).




*(a)* *(b)*

Fig1: Bund clearing & bund plastering

During literature survey, several existing methods of bund making were found. Fig 2(a) shows one simple method for bund making, which is drawn by bullock [5]. Fig 2(b) shows the same method which was modified to drawn by the tractor [6]. But this method has been used to prepare new bunds in places where there are no bunds available.




*(a)* *(b)*

Fig 2: Bund forming using an A-frame
*(a) Bullock drawn (b) Tractor drawn*

However, there was a modern attachment for 4-wheel tractors for bund preparation, which is made in China as shown in Fig 3. But it costs more than Rs.600, 000. Therefore, it was not economically feasible for the Sri Lankan farming community since local farmers do not frequently use four wheel tractors for paddy cultivation.


Fig 3: Bund forming attachment for 4-wheel
Tractors, made in China

This machine uses a rotary blade arrangement for breaking the existing bund or the ground and collects the soil and compresses it to form a new bund with a rotating drum. This was suitable only for dry or slightly wet soil. When the moisture content increases, the bund formation was not successful. The drum should rotate at a high speed to form a well compacted bund.
There was another attachment only for bund clearing built by a Sri Lankan farmer for 2-wheel tractors. It is shown in Fig 4 and the problem with this design was the inability to vary the height of the cutter. Only small bunds were able to clear by it.

Fig 4: Bund clearing attachment for
2-wheel tractors

According to the information found during literature; no one has yet developed a method for mud plastering. Therefore, the major requirements for the bund clearing cutter is the ability to cut the bund to an inclined shape while allowing adjusting the height. The mud plastering attachment should be able to convey the mud towards the bund and shape it to form a uniform layer of mud. By mechanizing the bund making process, the speed of the process can be increased up to the tractor moving speed (around 0.45 m/s speed).

### *2.2 Bund Clearing Attachment*

To obtain a slanted cut in the bund, a cutter having straight blades, with an inclined cutter shaft was initially suggested, as shown in Fig 5.

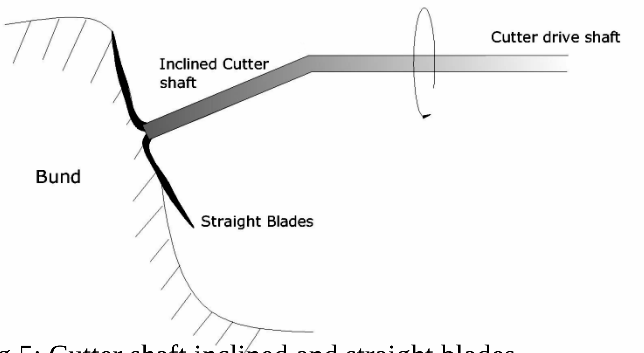



Fig 5: Cutter shaft inclined and straight blades

In order to clear the bund at different heights, there were three possible ways. These three possibilities were able to obtain by varying three parameters as shown in Fig 6.

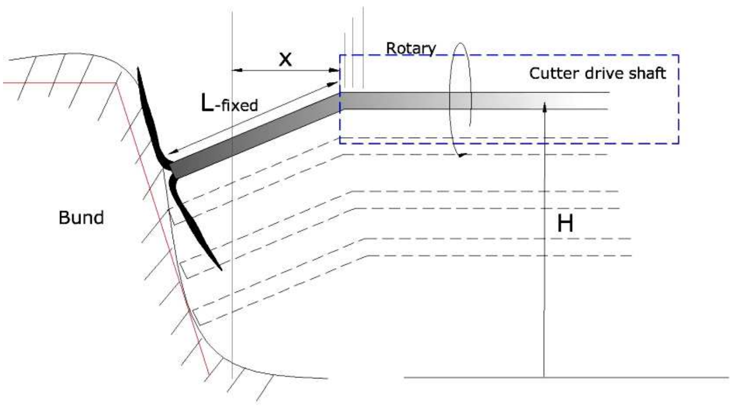



Fig 6: Fixed cutter shaft length (L) with variable height to the cutter drive shaft (H) and minimum distance from bund to the tractor wheel (x)

According to Fig 6, the three parameters were the height of the cutter drive shaft (*H*), the cutter shaft length (*L*) and the distance between the bund and the tractor mud wheel edge (*x*). One of the three parameters can made fixed and the other two can be varied and hence three possible ways can be obtained to adjust the height while cutting the bund to form slanted surfaces.

From these three possible ways, the configuration shown in the Fig 6 was selected. It is to have a fixed cutter shaft length (*L*) with a variable height (*H*) of the cutter allowing the operator to drive the tractor at various distances from the bund.

To make the cutter simpler and avoid the use of a universal joint to incline the shaft, a horizontal cutter shaft was suggested. Mean time, to achieve a slanted cut at the

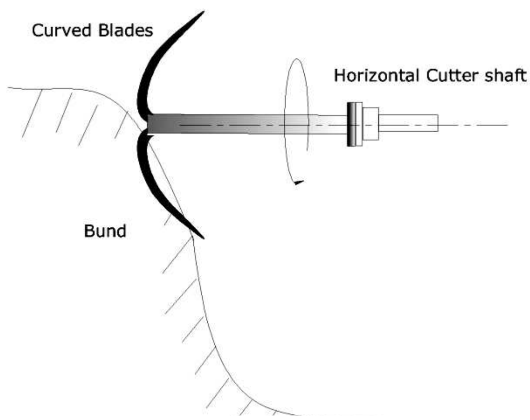



Fig 7: Horizontal Cutter shaft and curved blades

bund surface, the cutter blades were made inclined as shown in Fig 7.

To adjust the height of the cutter, a three gear wheel arrangement was suggested as shown in Fig 8. The dummy wheel is a free rotating wheel. To achieve various height levels, the driven wheel connected to the cutter shaft is rotated about the centre axis of the dummy gear wheel while the driver wheel is connected to the tiller blade shaft.

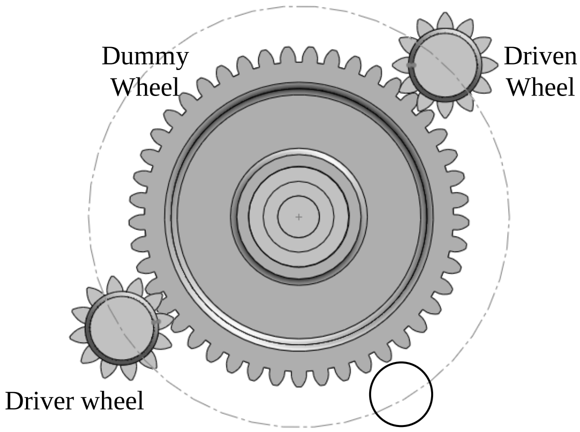



Fig 8: Height adjusting Mechanism

Fig 9 shows the fabricated bund clearing attachment. The fabrication was done with the commonly available materials (mild steel, sheet steel etc) and the total cost of fabrication was Rs. 24,000. The attachment was tested in a condition similar to the initial plough of the paddy field.

Fig 9: The fabricated Bund Clearing cutter

### *2.3 Bund Plastering Attachment*

The suggested mud plastering technique is shown in Fig 10. The main purpose of this channel was to convey mud when it moves forward with the tractor and direct mud towards the bund. This was fabricated using steel sheets and the total fabrication costs Rs.6000. The channel was fixed to the tractor at the hitch point and tested at a similar condition after the second plough in the field.

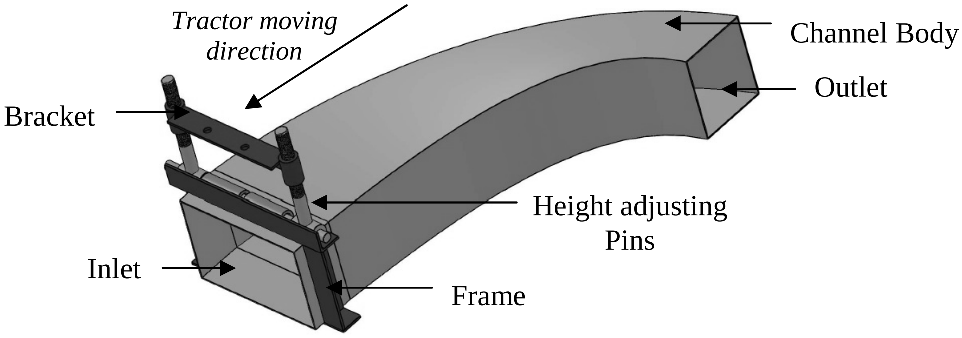



Fig 10: The Mud conveying Channel

## 3 Theory and Calculation

### *3.1 Maximum torque required for cutting action*

The following parameters were assumed for the worst possible case,

No of blades in the Cutter ($n$) = 4
Blade span ($s$) = 22.5 cm
Maximum blade thickness ($t_{max}$) = 6 mm
Shear stress of the soil ($\tau$) = 24 kPa

Cutter maximum operating speed ($N_{max}$) was considered as the tiller shaft speed at high rotary gear position and 1st main gear position [7].

$$N_{max} = 257 \text{ rev/min}$$

At any given moment, the cutter will only use half of the no of blades for bund clearing.
To calculate the torque generated due to cutting action, the blade profile was assumed to be on the periphery of a semi circle with a radii of half of the blade span ($s$). Fig 17 shows the approximate blade profile when viewed from the front.

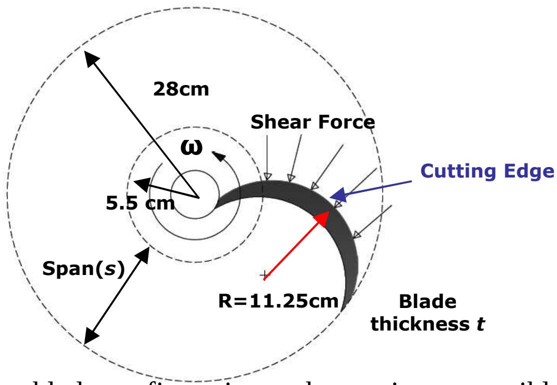



Fig 17: The approximate blade configuration at the maximum possible torque

$$\text{Total shear area } (A) = 2\,\pi\,R\,t$$
$$\text{Resultant shear force } (F) = \tau\,A$$
$$F = \underline{101.78 \text{ N}}$$

It was assumed that the resultant shear force acts along the centre point of the blade span. Then maximum torque required for cutting action would be,

$$\text{Maximum torque } (T_{max}) = F * R$$
$$= \underline{17.05 \text{ Nm}}$$

### *3.2 Maximum power required for cutting action*

Since the maximum rotational speed of the cutter is 27 rad/s (257 rev/min), the maximum power required for cutting action [8] would be,

$$P_{max} = T_{max} * \omega_{max}$$
$$= 17.05 \text{ Nm} * 27\text{rad/s}$$
$$= \underline{0.46 \text{ kW } (0.61 \text{ hp})}$$

The rated maximum power of the two wheel tractor is 12 hp. If a maximum of 20% transmission losses are taken, the minimum power supplied by the tractor transmission system would be ~ 9.6 hp. This would be more than enough for the cutting operation.

In addition a static FEM stress analysis was performed [9] using Solid work Simulation for the cutter blades to ensure safe operation with out failure.

## 4. Results

During testing, both attachments operated safely without any major component failure. The bund clearing cutter cleared the bund successfully and the height adjusting mechanism suit for the harsh conditions in the paddy field. But the inclination of the cutter blade was not sufficient to get a significant slanted cut in the bund. A mismatch in the cutter feed and speed was also observed which resulted in an improper cut along the bund, as expected.

The mud conveying channel concept failed during testing as the mud got stuck at the inlet of the channel.

## 5. Discussion

The above implementation was carried out under restricted condition due to financial and time limitations. As this is an ongoing work, to optimize the cutter blades to suit the purpose, several blade profiles and inclinations of the blade should be tested in the field as further study. The concept of the bund clearing cutter is entirely a new one and lot of trial and error tests should be carry out. In addition the ability of using the bund clearing cutter simultaneously with the rotary tiller should be investigated as it would be a significant method of increasing productivity in the land preparation during paddy cultivation.

One of the reasons for the failure in the mud conveying channel was the size of the channel at the inlet. So as further study, the inlet area can be increased and tested.

## 6. Conclusions

The mechanization of the bund making process is one of the important tasks to be done to achieve sustainability in paddy cultivation. As no similar method is developed to fulfil this task, a new conceptual design development and trial and error testings are required for improvement. The suggested design above is one such innovative concept and lot of further study is required to improve it. The successful mechanization of bund making process can provide an approximately about 30-fold productivity gain in terms of time consumption compared to the conventional methods currently used by the farmers.

### Acknowledgment

To all the staff members at Farm Mechanization Research Centre, Mahailluppallama and Mr. Aruna Jayawardane, Factory manager, Nikini Automation (Pvt.) Ltd, Ragama.

### References

[1] World Bank. "Sri Lanka: Priorities for Agriculture and Rural Development". Available at http://web.worldbank.org/WBSITE/EXTERNAL/COUNTRIES/SOUTHASIAEXT/EXTSAREGTOPAGRI/0,,contentMDK:202738

[2] Wikipedia. “Agriculture in Sri Lanaka”. Retreived from http://en.wikipedia.org/wiki/Agriculture_in_Sri_Lanka

[3] H.M Thilakaratna, I.G. Thilakaratna, “Farm Mechanization in Rice Cultivation”, Page 2-4

[4]Department of Agriculture, Sri Lanka, “Rice book (Sinhala)”, Page 3-5. Available at http://www.agridept.gov.lk/content/admin/pdf/Rice%20book%20(sinhala).pdf

[5] Seleshi, Bekele Awulachew, Philippe Lemperiere, Taffa Tulu (2009),” Irrigation methods: Surface irrigation-Options for mall holders”, Page 179.

[6] Department of Agriculture, India, “Directory of Agriculture Machinery and Manufacturers”, Page 42. Retrieved from http://agricoop.nic.in/dacdivision/Machinery1/chap1b.pdf

[7] Walking tractor MEGN-12,” The Operational Manual”, Page 3-4.

[8] R.S. Khurmi, J.K Gupta (2008), A Textbook of Machine Design, Eurasia Publishing house (Pvt.) Ltd, Ram Nagar, New Delhi, Page 470-557

[9] Daryl L. Logan (2002), A first course in Finite Element Method, Thomson learning Inc, CA 93950, USA. Page 421-440

# DESIGN OF POWER WEEDER FOR LOW LAND PADDY CULTIVATION

Ratnaweera A.C[1] , Rajapakse N. N.[2], Ranasinghe C. J.[1], Thennakoon T.M.S.[1], Kumara.R.S.[2], Balasooriya C. P[3], Bandara M. A[3]

[1]Senior Lecturer: Department of Mechanical Engineering, Faculty of Engineering, University of Peradeniya, Sri Lanka
[1]E-mail: asangar.ratnaweera@gmail.com

[2] Graduates, Department of Mechanical Engineering, Faculty of Engineering, University of Peradeniya, Sri Lanka
[2]E-mail: rajapaksenne@yahoo.com. [2]Telephone: +94-773979639

[3] Department of Agriculture, Peradeniya, Sri Lanka.

***ABSTRACT* —** Lack of man power has been identified as one of the major problems for the sustainability of the Sri Lankan paddy industry. Hence transplanters and seeders were well developed as a step for mechanization. However, weeding method is still not well developed up to mechanization. Therefore, our main objective is to design and fabrication of a power weeder. Weeding ability will be optimized by weeding three rows simultaneously. The machine is designed to use in the fields, cultivated by using mechanized seeder or mechanized transplanter introduced by Farm Machinery Research Centre (FMRC), Sri Lanka.
The double-action weeding drums will be driven by a small 1.3 kW gasoline engine, that can enable removal of weeds, while facilitating the forward motion of the machine. In addition, the conical shaped weeding drums will be designed to loose-up soil without harming the paddy. Totally six drums will be used, in such a way that rear three drums have high angular velocity with respect to the front drums.
A novel maneuvering method for row changing when the machine is in paddy fields also introduced in the design. More than 5-fold improvement of weeding efficiency in terms of weeding time is expected in this design. Further, a significant improvement of weeding quality is also expected in this design.

***Keywords –*** *Weeds, Paddy cultivation, Manual weeders, Mechanized weeders, Double action-weeding drums*

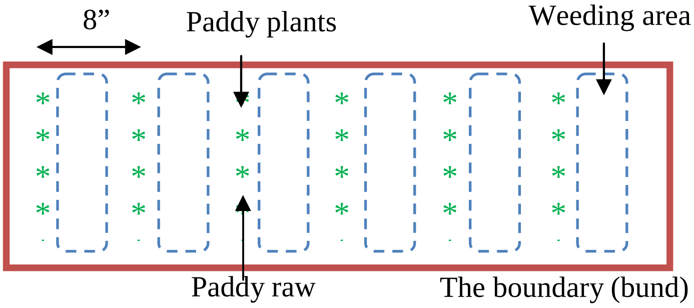
**ABBREVIATIONS**

Fos – Factor of safety

## 1. INTRODUCTION

Weeds are a major problem in paddy cultivation. Herbicides are usually used for weed controlling despite the fact that herbicides have many negative effects due to environmental contamination.
It has been understood that mechanized weeding significantly improves weeding efficiency as well as the quality of weeding. However, it may consume non-renewable petroleum for operations.
Cono-weeder, which is mechanized weeding (manual) method, capable of weeding about 0.18 ha/day. However, operational difficulties and slow weeding rate have been identified as major drawbacks of this weeder, particularly in large-scale cultivation.
The specifications of a paddy field, which is cultivated by using a mechanized seeder, are as follows.(ground requirement for power weeder)

- The space between each row of paddy is about 8 inches.
- The spaces between two paddy plants are 2-3 inches.




The machine is to be designed to remove weeds while travelling along the weeding area of the field as shown in Figure 1. Simultaneous removal of weeds in several weeding areas is a key requirement for efficient operation. Further, a mechanism is needed to move the machine between weeding areas without damaging paddy plants. The major mechanical engineering considerations in the design are: Driving mechanism, Weeding mechanism, Turning and row changing mechanism, Power transmission, Floating mechanism.

## 2. OBJECTIVES AND METHADOLOGY

The main objective is to design and fabrication of a power weeder, while minimum damages done to paddy plants, cost effectiveness, easy manuvelling, low weight, fabricationby using freely available components and easy maintenance are main features of this design. The following methodology was used:

- Development of the concept
    - No or minimum damage to the paddy
    - Easy maneuvering on wet fields
- Dynamic analysis and mechanical design
    - Kinematic analysis
    - Power transmission and drive systems design
    - Design of mechanical components
    - Assembly of components
- Fabrication of components
- Field testing and improvements
- The cost analysis

## 3. DEVELOPMENT OF THE CONCEPT

### *3.1. CHAIN DRIVEN SYSTEM*

The weeding drums are driven by a chain drive arrangement as shown in Figure 2. Accordingly specially designed double-acting weeding drums are proposed in the design to remove weeds while providing the necessary traction to move the machine forward. Rear group of weeding drums rotate faster than the front weeding drums to provide better shearing.

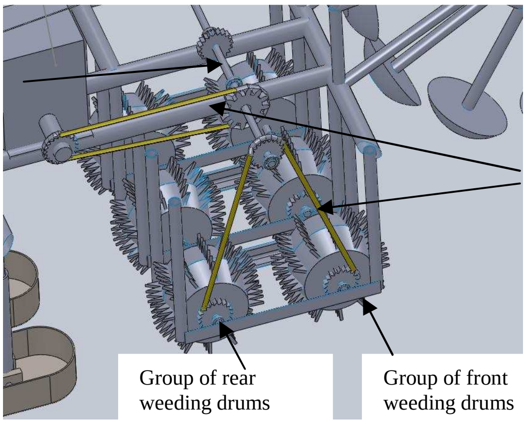



**Figure 2:** *Arrangement of the weeding drums*

### *3.2. DEFERENTIAL SPEED OF WEEDING DRUMS*

Two weeding drums which rotate at a deferential speed are suggested for each row. As shown in Figure 2, the drums at the rear are set to rotate faster than the drums at the front by using different sprockets. This design would enhance the shearing process due to the miss-match of linear speed of the two drums at the ground level. Further, it helps to push the weeds already removed under the mud to avoid any possible re-growth.

### *3.3. SHAPE OF THE WEEDING DRUMS*

The weeding drums are expected to perform two activities simultaneously: driving the machine forward and weeding. As shown in Figure 3, helical shaped teeth made on conical shaped drums are proposed for the weeding wheel. The conical shape helps to push mud to the roots of the paddy plants as it rotates which enhances the growth of the paddy plants. On the other hand, helical shaped teeth help to provide the shearing effect required for weeding and traction force required for the forward motion.

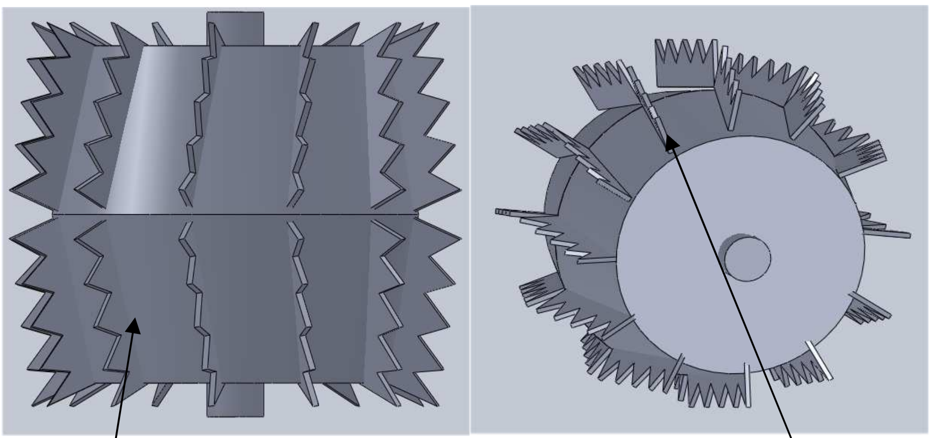



**Figure 3:** *The weeding draum*

### *3.4. NUMBER OF ROWS TO BE WEEDED SIMULTANEOUSLY*

It has been observed that if an odd number of rows are to be weeded simultaneously the maneuvering is easy as the operator cas easly walk in the middle raw. Therefore, it is decided to design the machine to weed three paddy rows simultaneously at a single operation. This is schematically shown in Figure 4.

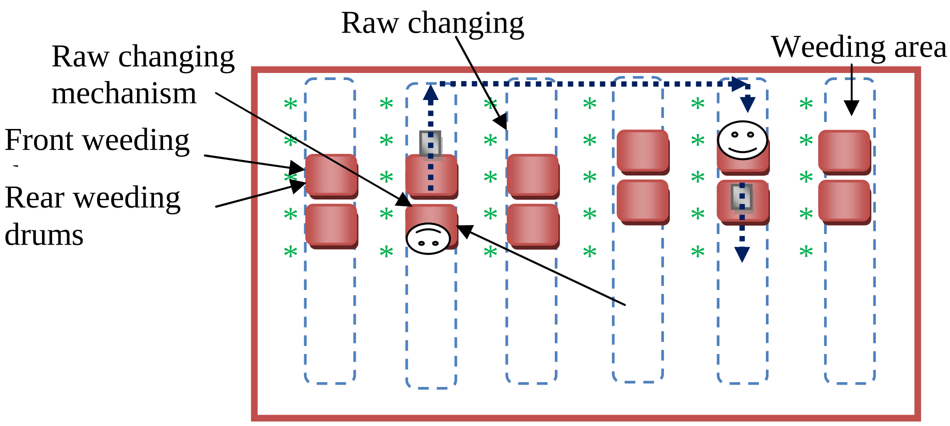



**Figure 4.** *Top view of the Operational behavior in the field.*

### *3.5. TURNING AND ROW CHANGING MECHANISM OF THE MACHINE*

In order to ensure continuous weeding, it is necessary to change the machine from one set of rows to another set without damaging the paddy plants. Thus an effective mechanism inspired by the human leg movement is invented. The initial configuration is shown in Figure 5.

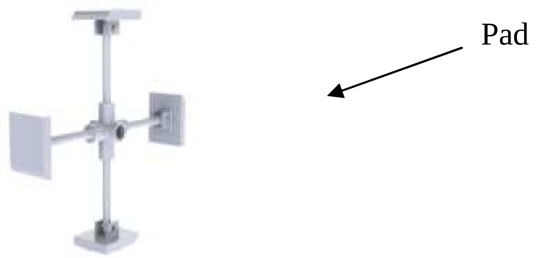



**Figure 5:** *Initial turning and raw changing mechanism* **Figure 6:** *Developed mechanism*

The initial design consists of four floater pads as shown in Figure 5. Each pad is pin jointed to a steel rod which is connected to the central hub using a bearing so that the rod is free to turn about its own axis. As a result each pad is free to rotate about two mutually perpendicular directions. This turning mechanism is fixed at the front of the machine.
The mechanism is fabricated and tested in paddy fields under different ground conditions. An acceptable operating characteristics were observed. However, several issues with regards to the durability and maintenance have been recognized as bearings and pin joints can easily get damaged in muddy conditions. Therefore, as shown in Figure 6, hemispherical floating pads rigidly connected to the central hub using steel rods are proposed in the final design.

### *3.6. FLOATING MECHANISM*

The floating mechanism is another important part of the machine, as it helps the machine to float in muddy conditions without sinking. The floaters reduce the ground reaction due to buoyancy effect. In the present study three adjustable mechanical floaters are considered to control the depth of shearing as required in different ground conditions.
The height adjustment has been achieved by using a simple arrangement with the aid of a screw as shown in Figure 7.

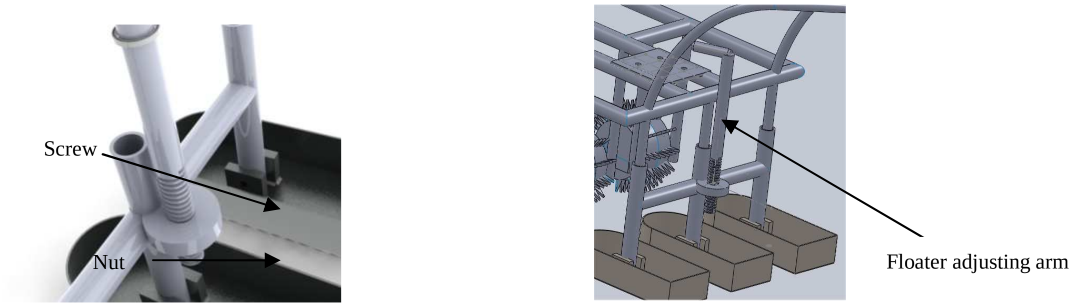



**Figure 7:** *The floating mechanism*

## 4. THEORY AND CALCULATIONS

### 4.1. POWER REQUIREMENT OF THE ENGINE.[3]

Maximum Shearing area for one blade = $4\text{x}10^{-3}$ $m^2$

(Assuming three blades are shear the soil at same time)

Effective shearing area for one weeding drum = $12*10^{-3}$ $m^2$

(Assume only three blades are done the effective shearing)

Shear stress of the soil (APPENDIX 2) = $5*10^3$ $Nm^{-2}$

Effective force on the weeding drum (area*shear stress) = 120 N. (FOS = 2)

The average speed of the drum (design value) =1.3 $ms^{-1}$(14.1rad /s)

Average radius of the weeding drum =$8*10^{-2}$ m

Average power requirement for one weeding drum (torque *speed) =$120*8*10^{-2}*14.1$

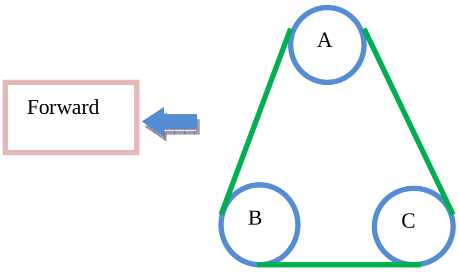
=135 W

Total power requirement for all weeding drums =810 W( 135*6)

#### 4.1.1. SPEED REDUCTION

The comfortable working speed in the paddy fields = 2.5 km/h(0.7 $ms^{-1}$ ) .[4]

The detailed schematic diagram is shown in Figure 4.




A– Driving wheel which connect to the intermediate shaft
B & C–Driven wheels which connect to the weeding drums

The speed of engine shaft / speed of intermediate shaft = 270 / 135
= 2 / 1

## 5. RESULTS.

**Table 2:** *The engine selection*

| Total power requirement/kW | safety factor | The estimated power/kW |
|---|---|---|
| 0.81 | 1.5 | 1.2 |

**Table 3:** *Design of chain drives*

| Rated power/kW | safety factor | Design power/kW |
|---|---|---|
| 1.3 | 1.5 | 1.95 |

**Table 4:** *Speeds of each drum*

| Speed of intermediate drum,$N_A$ / rev/min | Speed of front drum,$N_b$/ rev/min | Speed of rear drum,$N_c$/ rev/min |
|---|---|---|
| 135 | 121rev/min | 148 rev/min |

Note : $N_b < N_c$

### *APPENDIX 1*

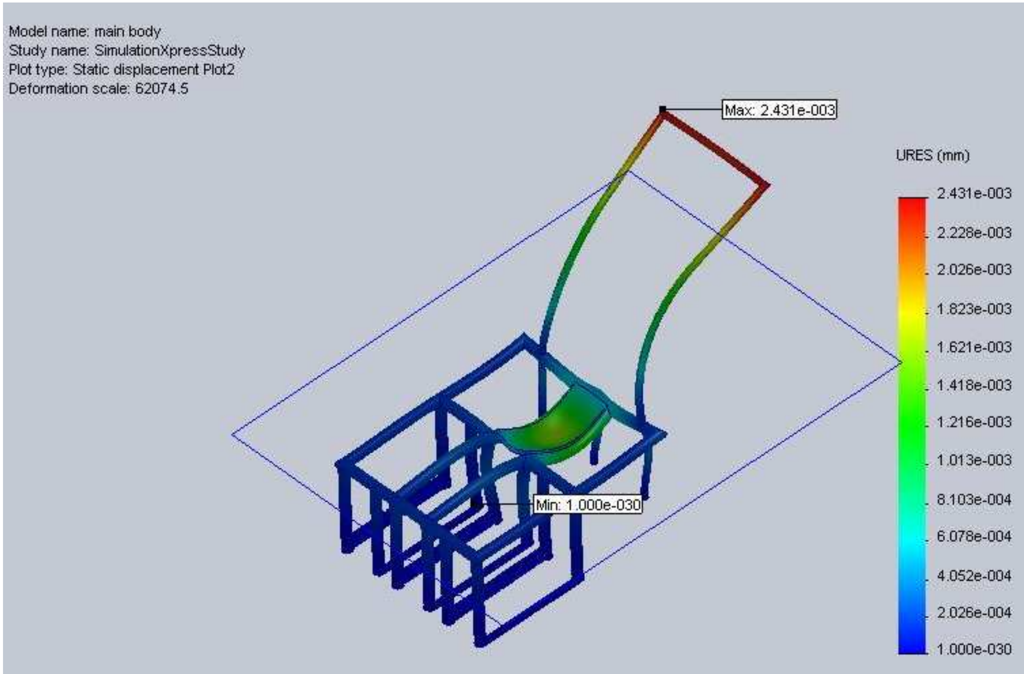



**Figure 8:** *Static deflection of the main body frame*

Hence main body frame was not statically deflected as the test shown in above figure 8. The test was done by using solid works.


**Figure 9:** *Static nodal stress of the main body frame*

Hence there was no excessive static nodal stresses developed in the main body as the test shown in above figure 9. The test was done by using solidworks

## *APPENDIX 2*

## *VANE SHEAR TESTER*

Vane shear test was done to investigate the shear stresses of the soil of paddy fields.

**Figure 10:** *Vane shear testing*

**RESULTS**

| | |
|---|---|
| Shear stress near bund | = 15 MPa |
| Shear in the middle of the paddy field | = 5 MPa |
| Average shear stress | = 10 MPa |

**DISCUSSIONS**

- The novel raw changing mechanism would be helpful for operating the machine by single person without destroying paddies.
- According to Figure 8 and 9 the structure would safe for static loads.

## 5. CONCLUSION

- Cost effective method for weed controlling
- Attracting the young generation for paddy industry because of mechanization.
- Well improved, unique raw changing mechanism would increase the effectiveness of the power weeder.
- A novel double action weeding drum is introduced in this design to facilitate the effective weeding while providing the necessary traction for the forward motion of the machine.
- Helical shaped teeth is formed in the weeding drums to enhance the shearing effect for weeding while loosing up the soil.
- The weeding drum is formed into conical shape to push the mud towards the paddy plants to ensure the proper growth.
- Two drums operate at differential speed is used to further improve the shearing process.
- Three sets of such drums were fixed in parallel to remove weeds in three paddy rows simultaneously

## REFERENCES

1. A Tajudding/ R Karunanithi/ K R Swaminathan./ Design, development and testing of and engine operated blade harrow for weeding/ Indian journal of agricultural engineering, vol 1, no 2, 1991
2. A Tajuddin./ Mechanising Indian Agriculture. /Kisan World, vol 31, no 6, 2004
3. R.S. khurmi/ J.K. gupta/ A Textbook of Machine Design/ Multicolor Edition
4. www.sri agricultural machineries.com

## NAMES AND ADRESSES OF AUTHERS

**1.** Corresponding author: Rajapakse N.N
Graduate, Department of Mechanical Engineering, Faculty of Engineering, University of Peradeniya, Sri Lanka.
Telephone: Mobile : +94773979639 Email: rajapaksenne@yahoo.com.
Residence : +94472254478

**2.** Authors: Thennakoon. T.M.S
Graduate, Department of Mechanical Engineering, Faculty of Engineering, University of Peradeniya, Sri Lanka.
Telephone: +94776115847. Email : thennakoonsuranjith@yahoo.com.

**3.** Ranasinghe S.N.B.M.C.J
Graduate, Department of Mechanical Engineering, Faculty of Engineering, University of Peradeniya, Sri Lanka.
Telephone: +94718159336. Email: chamilaranasinghe@gmail.com.

**4.** Kumara. G.R.R.S
Graduate, Department of Mechanical Engineering, Faculty of Engineering, University of Peradeniya, Sri Lanka.
Telephone: +94716670514. Email: ravindraruwan@gmail.com

**5.** Dr. A.C. Ratnaweera
Senior lecturer, Department of Mechanical Engineering, Faculty of Engineering, University of Peradeniya, Sri Lanka.
Telephone: +94812393627 E-mail: asangar.ratnaweera@gmail.com

**6.** Bandara M. A
Chief Engineer, Department of Agriculture, Peradeniya.
Telephone: +94 812388155, +94 714133403

# GLOBAL DROUGHT AND FLOODS IN 2009/10 EFFECTED BY ASTRONOMICAL CONDITIONS

[1]L. W. SENEVIRATNE

[1]Former Deputy Director of Irrigation, Irrigation Department, Colombo, Sri Lanka.
[1]E-mail: lwsenevi@sltnet.lk

**ABSTRACT**

Tropical countries are dependent on supply of meteoric water from wind masses. Geographical features contribute to evaporation, wind motion and precipitation in addition to infiltration and run off. Srilankan hydrological patterns are described and the identified extreme conditions are studied. Climatic conditions are studied. Precipitation pattern at Galle, Kurunegala and Puttalam locations are studied for last 60years. Global warming of 1deg C is recorded in the same period. Human actions are responsible for the release of green house gases in to the lower troposphere. With the help of recorded values of temperature, precipitation, pressure and wind directions other influence from the Moon, Sun, Jupiter and Saturn are observed with smaller effects from Venus, Mars, Mercury, Uranus and Neptune.

Precipitation is a process influenced by Earth's gravitational pull as well as by other heavenly bodies. Monsoons are formed by declination of Sun. Inter-monsoons are formed due to the motion of the Sun over the island. The Moon as a sub planet has a gravity control like the Sun. The declination of Moon and its rotation round the Earth has a great control over precipitation and tides. The Sun has little control over tides. The declination of heavy planets and their positions contribute little effects to precipitation. The Moon has a combined effect with Earth and is responsible for long term droughts. The author forecasted drought in year 2001 in 1993 due to the common plane episode of Earth and Moon. It came in Sri Lanka and Brazil and other tropical countries as predicted. As planets revolve in a definite time scale effects are cyclic and can be accurately predicted. The next drought has come in 2010.

Scientists who have no clue about this effect take that as El Nino.

*KEYWORDS*: ***Droughts, Floods, Astronomical Effects***

## INTRODUCTION

Tropical countries are periodically subjected to wind generated by annual Earth orbital motion. Monsoon is the wind carrying precipitation in South Asia and Southeast Asia. Recently due to global warming in climate, major changes took place in rainfall patterns. Earth orbital motion with fixed inclination of NS axis creates monsoon in Dec-Jan from NE and May-Jun from SW to precipitate water to Sri Lanka. In addition inter-monsoons caused by crossing the Sun over Sri Lanka come in September and April. Around the same time frontal type cyclones develop with heavy floods. Traditionally combined effect is used to compute the uncertainty of floods.

Global warming is a phenomenon in all tropical countries. South Asia has increased mean temperature by $1^0$C during the last 60years.

Astronomical features observed during the same time revealed many cyclic events leading to floods and droughts. The common plane episode of Moon and Earth leads to a prolonged drought in tropics as happened in 1974, 1983, 1992 and 2001. The short-term effects of heavy planets like Jupiter and Saturn are causing floods in half year when the earth is in between them and Sun. Full moon days are rainy in Sri Lanka. These astronomical aspects are excluded in calculating the design floods for spillways.

The shift in precipitation made Hambanthota dry and Colombo wet. Embankment designs carry greater impact from this shift. 200year storm selected for spillway design is revised. Greater impact has shown over earth slips in wet zone. Dry zone old major tanks need rehabilitation to suit the new conditions. Each basin needs reassessment of certainty in parameters using fresh data. Coastal hydraulics needs designs to face Tsunami in addition to Sea erosion.

Discovery in droughts and floods is common to tropics. Brazil had the same drought in 2001 as predicted by the author in 1993. Astronomical features appeared in cyclic form during the last 30years when the global warming increased in tropical countries. Variation in rainfall patterns is a classic example of sources to identify exceptions. Flood hydrology is disturbed when the normal rainfall is concentrated to less duration and increased droughts cause insufficient water in the reservoirs.

## 1. SRILANKA GEOGRAPHY

Sri Lanka is an island in the Indian Ocean closer to South India with a high seasonal precipitation. Most of the precipitation is drained through the catchment basins at a faster rate and creates flooding at narrow locations of the river system. The main land is formed with a lower plateau of less than 300m of elevation. A mid plateau of 100-300m is forming the central mountains. A high plateau of 1000-2000m is in the central steep hill region. The highest peaks are in the upper Mahaweli Region having more than 2000m high peaks. Pidurutalagala, Kirigal potta and Kikilimana are high peaks. These high peaks are not getting high rainfall due to its location in the central highlands. Next important high peaks are around Samanala Range. It receives high average annual rainfall. Watawala records 5500mm average annual rainfall. This precipitation is the main source of freshwater in the island. There is no snowfall and glacier formation and hence all the water is drained through rivers. Western slopes are receiving more rains and Kalu, Kelani, Walawe Rivers drain from Samanala Range. Mid plateau of southwest slopes receive about 3000mm annual precipitation. Gin and Nilwala rivers drain from this area. Smaller rivers also drain to western sea. Ben River, Panadura River and Mahaoya are those important basins. All these basins contribute to flooding after heavy rains. Mahaweli River takes water towards dry zone. Galle is a coastal city in the southwest wet zone of Sri Lanka. Puttalam is in the Northwestern Province.

Early settlements were recorded in the history for 500000 years but the cultivation age was noted in 4000 years ago. The Arian civilization was the result of agricultural activity along water sources. Agricultural settlers used the coastal areas, river valleys, flat basins for cultivation. High mountains were left as protected areas. Sedimentation of rivers was limited to low natural limits. In 19$^{th}$ century cash crops were introduced to central hills and sedimentation was doubled. The run off was increased and the villages near the rivers were subjected to floods. The specific yield of flow is increased in the wet zone. Rivers in dry zone originated from wet zone mountains, provided the necessary water for irrigation. In the drought period many streams dried and it was termed as Oya and dola basins. Ganga and ela are not drying in the year.

## 2. CLIMATE

The island is situated closer to Indian Peninsular and hence NE and SW monsoons are activated due to the revolution of the Earth round the Sun with an angle of inclination of $23.5^0$ between the normal to the plane of revolution and NS axis of the Earth. The movement of the earth in that inclination creates movement of the Sun from Tropic of Cancer to Tropic of Capricorn. Northward motion begins from December 22 and Sun reaches vernal equinoxes in March 21. Then it enters the Northern hemisphere and reaches Tropic of Cancer in June 22. Then it begins southward motion and reaches vernal equinoxes in September 22. Then it enters Southern Hemisphere and reaches Tropic of Capricorn in December 22. As the position of the Sun changes from equator up and down wind stream movements begin towards the Sun. Heated wind streams rise up to create further motion. Cooler winds are attracted with high potential when the Sun is at extremes. Hence SW monsoons are in full swing in June. It begins in May and reaches Sri Lanka with high precipitation. Then it enters India and for June- September period provides rains. It goes through monsoon trough in Assam and passes along Himalayas in a northwest direction. Sri Lanka gets rainfall in May June period. Early appearance and late disappearance brings more rain to the country. Major wind stream passes through Indian Ocean and it is rich with water vapor to satisfy about 1000 million people. But unfortunate situations arise as in 2001 due to the influence of moon to lead to a drought. In such situations the origin of winds was a landmass avoiding the sea.

The Southward Sun motion creates NE winds. Wind originates from Bay of Bengal and reaches Sri Lanka in the north and moves down to south. India is not getting rains from this NE wind as its origin is a landmass. Dry zone of Sri Lanka is fully benefited by these winds. November December months are usually rainy months. A change in the origin of winds may not carry full results. Sri Lanka is fortunate to have two inter monsoons created by the Sun passing across the island in April and September. March April period and October November period are rainy due to it. Due to effect of Moon these inter monsoon may be limited to very small rains as in 2001. In Sri Lanka cultivation season begins with inter monsoons in October. Coastal rains of convective type are coming to western and eastern coastal areas. Inter monsoons bring rains from many directions. The hilly regions are much benefited by this rainfall. But Hambanthota and Puttalam Districts are on the edge of the plate and hence get few rains. Badulla area receives rains from all four types of monsoons. Cyclonic wind patterns of frontal type occasionally develop in the Bay of Bengal and some times enter the island from the East. Early monsoons turn into cyclones in May and become dangerous. Cyclones brought heavy rains in Aug 1947, Dec 57, Dec 64, Nov 78 in the recent past.

## 3. CONDITIONS OF RAIN FALL

Dust form an essential role in rain formation. Presence of dust leads to nucleation of water drops and dynamic cooling of wind accelerates nucleation. Then it reaches point of saturation and releases drops of water. Insufficient dust storm is not able to form sufficient nuclei and the cloud passes the locality unsuccessfully. Cloud seeding is attempted in many occasions to spray cement dust or any other soft environment friendly material on to the cloud. The aircraft undergoes low pressure due to nucleation and needs quick escape. This operation is costly and rarely used. Dynamic heating of wind prevents further nucleation and absorbs water in to the wind mass and cloud disappears. Relative humidity drops in the downward motion of wind mass. This is the reason to precipitate rain along ascending path of the mountain and not on the other side. It can rain when it meets the ascending path of the next mountain range. Successive mountain ranges are responsible to account for heavy precipitation. Another drawback in this cause and effect is that, a change of wind direction results in low precipitation.

## 4. GLOBAL WARMING

Global warming is due to burning of fossil fuel in large quantities to produce high accumulation of green house gases. Carbon dioxide, ozone, ethane, propane, sulphur dioxide are heavy gases whose presence is catching more heat to lift up the mean temperature. Natural gases as by products of photo-synthesis, also collect in the lower troposphere. Solar energy is the source coming in short waves. Heat stores in the daytime using gases in the troposphere and water in the reservoirs and landmass in the continent. Emission in long waves slowly takes place in the afternoon and night. Cloud cover can reflect long waves back and form a green house to warm up the troposphere. This warming process is slowly building up during the last 60 years in the tropics. Temperature is measured twice daily at 9.00am and 5.00pm (CST) and a mean is recorded. Cloud cover in the wet zone reduces difference between maximum and minimum temperature. Highest temperature is recorded from Vavunia as $40.1^{0}$C.

## 5. CERTAINTY CHANGES

### *5.1. Tsunamis (Coastal waves)*

Earth as a planet contributes to variations of hydrosphere due to its volcanic eruptions located around plate boundaries. Indian Australian plate has the eastern boundary close to Andaman Islands. This zone is highly active in volcanic eruptions. Indonesia lies in the active zone of this plate with many active volcanoes. The dangerous tsunamis are caused by under water volcanic eruptions. In 1883 Krakato disaster was noted to form a Tsunami and killed 30 000 people in Jawa. A very same type disaster caused damage from Andaman and killed 120, 000 in Indonesia on December 26, 2004. Sri Lanka lost 36 000 people due to sudden waves propagated from Andaman in 2hours after the explosion. The eruption rated as 9.0 in the Richter scale and it lifted the bottom of the sea by 1km in a north south oriented long crack. The resulting waves propagated in the east and west directions. The dark water wave was 10m high and it carried water for about 2km in the Eastern coast. Also it attacked the Southern coast killing popular areas. Western coast is less affected but buildings were destroyed. Highly populated Western coast was not directly facing the Andaman Islands. This tsunami went to Maldives and then it attacked Somalia, Sey-shells. Coastal areas are rich with surface and under ground fresh water resources to attract high population. Tsunamis caused damage to them.

Dust emissions in 1883 reached atmosphere and visible for two years fading the sun light but not in 2004. History has recorded in 200BC a storm surge to destroy 500 villages in this country. It can be a tsunami. In 1615 another storm surge came. The economic damage is very high as the coastal population is 10% of the total. Tourism and fishing and industries dominate this coastal belt of 300m. Fresh water resources are more prominent in this coast. Damage to economy is very large due to complete eradication of villages and industries. Mixing of salinity with fresh water is long-term damage. Nobody expected tsunami as the memory runs back to 121 years. Storm surges occur in western coast with heavy sea erosion. Tidal waves combined with storms damage coastal beaches continuously. Groynes laid to protect eroded beaches are not attractive to tourists. Coral rocks are gradually destroyed by the acidity increase in the sea. Still water brings up concentration of acids around corals.

### *5.2. Floods*

Global warming has increased the intensity of floods. The flood occurred in May 2003 on a full moon day was more devastating. It attacked Ratnapura, Galle, Matara, Hambanthoa, Kalutara, Nuwara eliya Districts. A 330mm one day rainfall was showered on southwest hill country due to a frontal type cyclone. This is computed as 50year storm. It came after a long drought in the beginning of SW monsoons. Usually a long drought builds up a heavy dust cloud in the atmosphere and finally it causes a sudden downpour when water vapor enters in major bulk. This type of extremes is noted after 1970. Global warming has increased the mean temperature by $1^0$C in the last 60 years. This is noted in Sri Lanka, India and Nepal and commonly over the tropical areas. The main source of global warming is the burning of fossil fuels to produce mechanical energy. This is causing sea level rise by conduction and dissolution of ice caps. Hydrology cycle is increasingly affected as its operating temperature range is elongated. The result is a prolonged drought and a high precipitation. Tidal directions and flow are subjecting to change and the fish breeding grounds are disturbed. This is leading to alter fish catch and seasons. Flood control bunds are designed for 10year floods.

### *5.3. Droughts*

Global warming has created a cycle of droughts over tropical countries. Usually there are four seasons to bring rainfall in to the island. The SW and NE monsoons are major rainy seasons. Inter monsoons are benefiting to begin the cultivation. Rainfed cultivations are postponed if inter-monsoons are delayed or absent. If monsoons are in short duration or absent, then total cultivation fails. Reservoirs in dry zone can store water and feed the crops. But wet zone perennial crops die in the drought. Droughts in concentrated areas occur due to seasonal changes. In 2003/4 Maha season heavy drought affected Kurunegala District and vicinity. In late 2004, NE monsoons were high in central hills and Anuradhapura District. Floods caused several tank breaches. Potato farmers in Nuwaraeliya faced drought in Maha 2003/4. At the same time tsunami attacked coastal areas. Certainty changes in hydrology are heavily apparent in 2004. ICID had a theme topic as disasters on 2004 world water day. Farmers and dependents are facing heavy losses. Migration to wet zone is encouraged but tsunami discouraged coastal permanent occupation. Western Province bears 6M of population due to its sound water resources. Coastal districts carry 55% of population of Sri Lanka.

## 6. CHANGES IN HYDROLOGY

### *6.1. Kirindi Oya*

Observations in precipitation in each district of Sri Lanka are available and some data was from 19$^{th}$ century. Kurunegala is in the intermediate zone in between wet and dry zones. Ratnapura is in wet zone in upper Kalu basin. Kirindi Oya is in the Hambanthota District receive some water from Badulla area. Kirindi oya basin in 1930 was rich with water and the estimate for irrigation was 24000ha and this was dropped to 12000ha when the Kirindioya project was planned. However it was dropped to 8000ha in 1978. In 1983 water yield was dropped to lowest level. In 1987 the project cultivated 8000ha but soon in 1992 it was dropped to 4000ha for two seasons. Further in 2001 the drought reduced the capacity to 1000ha and in 2002 rice crop was confined to 4000ha for one season and banana cultivation was given 1000ha. 2000ha of old farmers were given benefit of two seasons from the Ellagala anicut. Farmers with land lots had to return to old villages due to low yield of water. Complete drying of reservoir in 2001 restricted drinking water supply completely. Two surviving king coconut species in Hambanthota area were now extinct due to this drought. Failure of inter-monsoons led to destruction in this year with the common plane episode of Earth and Moon. Brazil faced a severe drought in this year as it lies in tropics like Sri Lanka. Menik river basin is planned to dam at Weheragala and 40% of water is diverted to Kirindioya tank.

### *6.2. Kurunegala*

The average monthly precipitation was 150-200mm in the period 1950-1975 but it dropped to 120-200mm in the subsequent period 1975 to 2003. In addition it dropped to 80mm in 1987. It shows stabilization and gradual rise and fall in 30 years above 100mm (Figure 1). This area is far away to get high rainfall from Monsoons. A drop in patterns affect drinking water supply as well as irrigation.

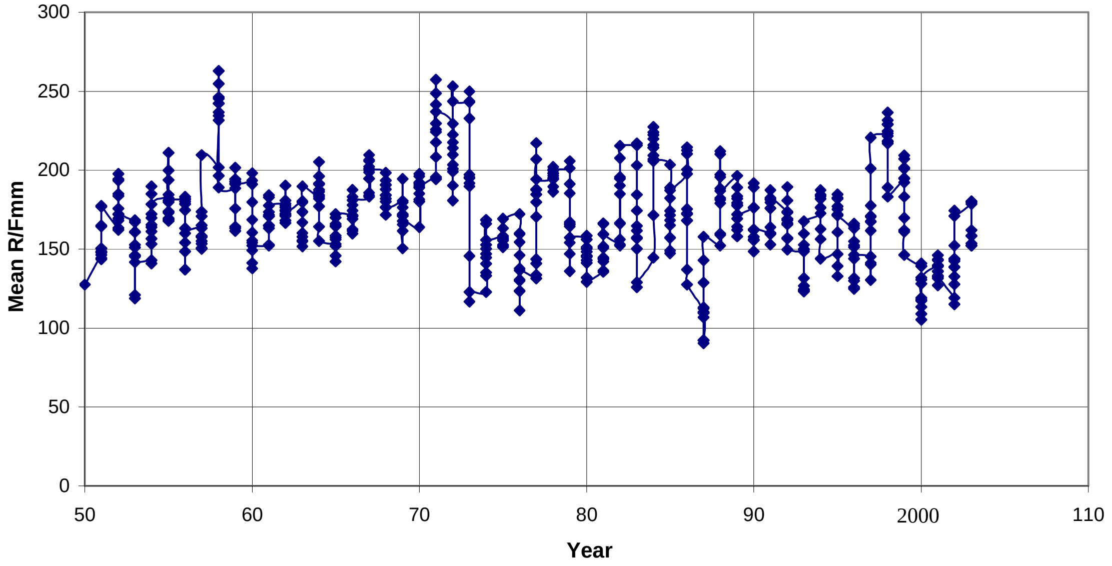



**Figure 1.** *12Month Running Average Precipitation in Kurunegala 1950-2003*

**Table 1. Puttalam Monthly Precipitation and Annual Total in mm**

| Year | Jan | Feb | Mar | Apr | May | Jun | Jul | Aug | Sep | Oct | Nov | Dec | Total |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **1973** | 0.0 | 33.0 | 56.6 | 128.8 | 79.2 | 126.7 | 15.2 | 5.3 | 1.3 | 323.1 | 91.2 | 411.0 | 1272 |
| **1974** | 0.0 | 62.2 | 21.8 | 281.7 | 86.1 | 11.2 | 19.1 | 47.8 | 33.3 | 3.0 | 108.0 | 84.6 | 759 |
| **1975** | 48.0 | 0.0 | 145.3 | 251.5 | 61.0 | 9.1 | 68.6 | 1.8 | 51.1 | 75.9 | 229.6 | 99.3 | 1041 |
| **1976** | 47.2 | 28.4 | 74.7 | 44.7 | 33.3 | 8.1 | 5.3 | 3.0 | 4.1 | 332.7 | 240.3 | 104.4 | 926 |
| **1977** | 36.1 | 0.5 | 181.6 | 183.6 | 281.7 | 11.9 | 15.0 | 14.7 | 27.7 | 424.7 | 186.9 | 85.6 | 1450 |
| **1978** | 0.4 | 0.4 | 176.3 | 170.8 | 178.2 | 3.6 | 4.4 | 10.4 | 8.0 | 228.8 | 547.1 | 170.7 | 1499 |
| **1979** | 1.6 | 53.6 | 0.0 | 48.6 | 0.1 | 5.2 | 10.4 | 22.1 | 159.1 | 187.8 | 352.5 | 220.6 | 1062 |
| **1980** | 0.0 | 0.0 | 1.5 | 174.9 | 16.9 | 83.9 | 5.5 | 0.5 | 196.7 | 194.1 | 292.3 | 56.5 | 1023 |
| **1981** | 16.9 | 34.1 | 60.8 | 124.5 | 123.1 | 59.1 | 2.5 | 23.9 | 87.6 | 126.9 | 360.0 | 32.6 | 1052 |
| **1982** | 0.0 | 0.0 | 24.9 | 88.3 | 100.8 | 120.6 | 9.8 | 3.3 | 20.1 | 289.9 | 239.5 | 67.1 | 964 |
| **1983** | 0.0 | 0.0 | 0.0 | 50.2 | 35.1 | 37.6 | 7.0 | 1.3 | 82.9 | 101.8 | 286.8 | 251.5 | 854 |
| **1984** | 279.7 | 226.2 | 257.4 | 652.6 | 33.2 | 0.1 | 77.1 | 0.0 | 62.7 | 78.7 | 438.4 | 62.8 | 2169 |
| **1985** | 73.2 | 219.5 | 60.5 | 116.1 | 220.9 | 15.5 | 0.5 | 0.0 | 107.9 | 192.6 | 435.7 | 94.6 | 1537 |
| **1986** | 69.5 | 51.7 | 28.1 | 144.0 | 136.4 | 10.8 | 0.0 | 8.0 | 17.3 | 145.4 | 145.1 | 63.6 | 820 |
| **1987** | 31.5 | 0.0 | 19.3 | 241.0 | 41.8 | 3.8 | 9.9 | 36.7 | 283.7 | 335.7 | 156.0 | 52.5 | 1212 |
| **1988** | 15.8 | 56.9 | 52.5 | 408.5 | 7.5 | 118.1 | 13.3 | 44.7 | 141.6 | 53.9 | 202.7 | 67.2 | 1183 |
| **1989** | 22.7 | 0.0 | 60.6 | 162.5 | 16.7 | 15.9 | 13.7 | 0.4 | 19.4 | 248.2 | 240.6 | 26.8 | 828 |
| **1990** | 217.0 | 0.0 | 35.2 | 33.5 | 291.7 | 0.5 | 3.4 | 0.4 | 2.4 | 380.3 | 225.1 | 180.7 | 1370 |
| **1991** | 54.0 | 8.3 | 79.3 | 106.6 | 250.8 | 101.5 | 7.6 | 1.2 | 12.9 | 342.3 | 228.4 | 88.4 | 1281 |
| **1992** | 5.7 | 0.0 | 0.0 | 93.8 | 174.9 | 38.2 | 82.3 | 9.6 | 38.5 | 287.1 | 288.7 | 59.0 | 1078 |
| **1993** | 4.9 | 10.1 | 78.7 | 118.6 | 105.9 | 0.7 | 9.2 | 6.4 | 45.8 | 244.4 | 239.1 | 317.2 | 1181 |
| **1994** | 109.0 | 100.9 | 72.9 | 60.6 | 76.4 | 0.2 | 35.2 | 6.4 | 169.0 | 250.8 | 186.6 | 36.5 | 1105 |
| **1995** | 66.6 | 80.0 | 30.2 | 312.5 | 194.0 | 7.7 | 19.1 | 1.1 | 0.4 | 192.5 | 463.2 | 24.4 | 1392 |
| **1996** | 28.6 | 70.0 | 0.4 | 172.9 | 56.3 | 103.9 | 2.0 | 43.5 | 145.5 | 279.0 | 96.3 | 114.1 | 1113 |
| **1997** | 0.2 | 4.4 | 7.4 | 64.7 | 149.1 | 58.9 | 16.7 | 0.0 | 72.2 | 253.6 | 400.2 | 73.4 | 1101 |
| **1998** | 36.1 | 6.3 | 35.3 | 47.2 | 186.4 | 12.4 | 151.9 | 25.9 | 51.6 | 104.1 | 323.5 | 202.1 | 1183 |
| **1999** | 90.7 | 110.8 | 0.0 | 154.7 | 45.2 | 11.6 | 1.3 | 3.3 | 61.4 | 246.4 | 171.1 | 69.0 | 966 |
| **2000** | 111.3 | 79.3 | 41.2 | 154.2 | 7.4 | 5.8 | 0.0 | 80.1 | 100.1 | 45.8 | 161.1 | 161.5 | 948 |
| **2001** | 114.5 | 44.7 | 0.0 | 327.6 | 13.2 | 51.9 | 87.3 | 0.2 | 24.1 | 46.4 | 199.3 | 115.1 | 1024 |
| **2002** | 20.9 | 36.1 | 138.6 | 410.5 | 100.9 | 11.5 | 0.0 | 0.8 | 14.0 | 334.1 | 237.8 | 279.9 | 1585 |
| **2003** | 106.5 | 7.7 | 142.1 | 190.4 | 99.3 | 93.3 | 133.9 | 12.2 | 3.4 | | | | |

Effectivr rainfall = 0.67(R-25) mm

### *6.3. Puttalam*

Annual precipitation is very low in Puttalam (Table 1) District. It is around 1000mm. But in drought periods it drops to 820mm. SW monsoons shift to India in June with the Kerala mountains covering Puttalam, Mannar and Jaffna Districts. Despite coastal rains in the coastal area interior Mi Oya basin is very dry and expects storage in Tabbowa and Inginimitiya tanks.

Puttalam has gained exceptional rains in 2002 and 1984. This was in contrast to the pattern shown by other locations. These two years were common drought years and heavy precipitation has recorded after the drought. The change in rainfall patterns in Sri Lanka is accommodated with a shift in NW direction in these exceptional years.

### *6.4. Dry Zone*

Dry zone area has commonly dropped precipitation but shows stability. The 1850mm isohyet demarcates the wet zone boundary. About one third of Sri Lanka falls in to wet zone. This wet area is shrinking. Matara city now falls out of wet zone. The Sinharaja natural forest lost many trees and reduced to scrubs from SE direction. Matara and Hambanthota Districts need diversion from Nilwala and Gin rivers for town expansion. Cooler areas of Nuwaraeliya, Bandarawela lost morning mist and coolness in morning. Only Colombo and Matara districts had slight increase from the mean of 1960-90 and dry zone had the worst hit. NE monsoons lost 19% and next Inter monsoons lost 10%. Malale district reduced rains by 410mm during 50 years as the worst. Districtwise increases are Colombo 60mm Matara 30mm and decreases are Vavunia 25mm, Hambanthota 40mm, Kalutara 50mm, Puttalam 60mm, Galle 80mm, Mannar 90mm, Trinco 120mm, Jaffna 120mm, Kilino 130mm, Anurada 130mm, Polonna 135mm, Batti 160mm, Kuru 160mm, Monera 160mm, Mulaitiv 170mm, Kegalle 200mm, Nuwaraeli 200mm, Badulla 220mm, Ampara 250mm, Kandy 315mm and Matale 410mm.

### *6.5. Ratnapura*

Ratnapura District in the boundary of wet zone is gradually reducing the level of precipitation. Storage tanks are not planned due to its low head but floods create difficulties in civil life. Earth slips are increasing due to increased sudden one-day storms. Increased population lives on dangerous sites in the hills, also plant tea bushes in place of big trees. This increases the risk of earth slips. Increased uncertainty demands more drainage facility. Also flood control and relief measures need more attention. Water is a resource, which can be diverted to drought areas. But still 60% of precipitation goes to the sea. Gem mining is leading to soil erosion. Mini hydropower projects are operated during wet season.

## 7. WEATHER PATTERNS

Sri Lanka has a very good record of hydrology data for the last 200 years. Historical records show tsunami, storm surges, droughts, floods and epidemics. Beminitiya was a drought existed for 12 years from 45BC. This led the need to write down hither to by hearted Buddhist texts on ola leaves at Alu vihara Temple. Dry zone major tanks had very low storages due to droughts. Agricultural development in tea, rubber and coconut performed by British maintained rain gauges in estates.

### *7.1. Observed pattern*

Rainfall patterns are usually changing but mean of Galle rainfall data (Table 2) is maintaining close to 2300mm. Annual rainfall in 1974, 1983, 1992, 2001 showed a drop when compared with rest of the years. Galle is facing the sea and SW monsoons are directly reaching without any obstruction. **The research conducted by the author discovered this phenomenon in 1993 and forecasted the drought in 2001 and 2010.** This pattern is no doubt attributed to astronomical features. The drought becomes so serious due to global warming. There was no serious report on drought before 1973. Stable atmosphere causes high-pressure situation. High pressure is causing drought in the island.

### *7.2. Destabilizing forces*

Tides are caused by gravity pull of Sun and Moon. When Earth and Moon taken together the resultant center of gravity lies over the earth surface. Rotation of Earth in the eastward direction rotates the center of gravity westward with same speed of 1600km/hour. This center drags water towards it and in doing so reach a peak in 6 hours and ends in 12hours. Direction changes in 12 hours and low tides begin. It reaches a peak in 6 hours and ends the cycle in another 6 hours. Westward rivers show this phenomenon to take high tides a long distance. Ben River takes water up to Hattaka. Boats took that advantage to travel up stream.

Sun is the other object to alter the tides. Its effect is about 10% of that of the Moon. On the full moon days the two forces are canceling each other. On new moon days the two forces are added together. We can see rainfall differences on these different days. Full moons are usually rainy days. When Earth comes to the middle water has free movement and rain is more.

Tides are changing according to the declination of Moon. Direction of tides is changing accordingly. Declination envelope of Moon is changing from $18,5^0$ to $28,5^0$ in 18.55year cycle. In this cycle it reaches the coplanar position in 9.25years. This episode came in drought years in Sri Lanka as noted above. The coplanar activity of Earth and Moon taking place for two-year period is very calm and clouds are not seen in the sky. Usual showers are not matured to give sufficient rains. All the tropical areas are affected to this drought. Brazilian drought came in 2001 in the same period as in Sri Lanka, [Seneviratne, (1993)].

### *7.3. Short-term effects*

Droughts lasting for 3month period and excessive floods appearing in tropical countries can have astronomical reasons. Declination and position of Jupiter and Saturn is important. Jupiter and Saturn are heavy objects in the solar system. Jupiter balances the system by pushing Sun back and has some influence on water. Its period of revolution is 12 years. Every year half the period it is visible in the outer orbit in the night. Rest of the year it moves to the other side of the Sun and is not visible in the night. When it is visible in the night we are in between Jupiter and Sun. Hence it is a chance for free movement of water and rainfall continues. Same case is repeated for Saturn, which has a period of 30 years. These two planets come near in 20 years. The impact of combined planets is greater in years around 1961, 1981, 2001 than the years around 1971, 1991and 2011 due to positioning. Precipitation in 1980, 1981and 1982 September months was more than January February months of same years (Table 2).

**Table 2.** *Monthly Precipitation at Galle, Sri Lanka in mm*

| **YEAR** | **Jan mm** | **Feb Mm** | **Mar Mm** | **Apr Mm** | **May Mm** | **Jun Mm** | **Jul mm** | **Aug Mm** | **Sep Mm** | **Oct mm** | **Nov Mm** | **Dec mm** | **Total Mm** |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 73 | **36.3** | 65.8 | 213.9 | **109.7** | 263.7 | 364.2 | 239.0 | 128.5 | 157.5 | 422.4 | 196.6 | 113.8 | 2311.4 |
| 74 | **0.5** | **73.2** | **54.4** | 408.2 | 393.4 | 193.8 | 365.5 | 140.7 | 394.2 | **45.2** | 208.0 | 121.4 | 2398.5 |
| 75 | 88.9 | 85.3 | 156.7 | 342.1 | 180.6 | 286.0 | 144.8 | 162.1 | 155.4 | 352.3 | 324.1 | 106.4 | 2384.7 |
| 76 | 95.8 | **18.3** | 87.1 | 397.8 | 163.6 | 99.6 | 170.7 | 111.0 | 50.8 | 350.0 | 750.3 | 312.7 | 2607.7 |
| 77 | **8.9** | 264.2 | 118.1 | **100.8** | 312.4 | 139.4 | **78.5** | 157.2 | 168.4 | 469.1 | 410.5 | 114.8 | 2342.3 |
| 78 | 84.7 | 90.1 | 143.3 | 178.8 | 390.1 | 200.0 | **70.7** | **28.5** | 217.6 | **141.7** | 425.6 | 119.1 | 2090.2 |
| 79 | 53.9 | 102.0 | 69.3 | 219.1 | 233.6 | 286.9 | 198.1 | **36.6** | 468.5 | 299.0 | 657.6 | 181.2 | 2805.8 |
| 80 | 125. | **46.0** | 63.4 | 171.3 | **102.3** | 126.5 | **61.3** | 148.8 | 416.4 | 244.5 | 286.0 | 476.4 | 2267.9 |
| 81 | 102. | 83.2 | 88.7 | 273.0 | 394.0 | **52.8** | **44.8** | 147.9 | 148.0 | 239.9 | 305.9 | 205.8 | 2086.0 |
| 82 | **17.3** | **1.8** | 151.3 | 421.3 | 322.0 | 235.4 | 191.9 | 158.6 | 174.6 | 218.6 | 577.1 | **87.3** | 2557.2 |
| 83 | 76.4 | **0.6** | **54.5** | **18.7** | **110.1** | 122.6 | 113.5 | 150.7 | 338.0 | 180.2 | **144.6** | 240.0 | 1549.9 |
| 84 | 212. | 102.1 | 60.5 | 304.2 | 466.4 | **28.0** | 149.6 | **18.8** | **28.1** | **98.8** | 310.3 | 106.6 | 1885.4 |
| 85 | 139. | 114.0 | 91.7 | **82.1** | 212.5 | 402.2 | **71.6** | 229.3 | 159.2 | 525.9 | 224.1 | 240.7 | 2492.7 |
| 86 | 63.2 | 60.1 | 151.8 | 148.8 | 224.1 | **79.1** | **31.1** | 90.3 | 122.0 | 223.3 | 131.0 | 186.4 | 1693.2 |
| 87 | 87.7 | **3.7** | **5.1** | 242.6 | **140.6** | 237.2 | **8.1** | 476.9 | 334.1 | 523.5 | 335.3 | **71.7** | 2466.5 |
| 88 | 92.0 | 114.0 | 225.4 | 222.7 | 228.1 | 328.4 | 224.7 | 301.9 | 249.3 | **60.1** | 260.7 | **83.6** | 2390.9 |
| 89 | 86.7 | **23.0** | 66.6 | 322.1 | 386.1 | 228.7 | 141.1 | 118.9 | 179.2 | 428.9 | 284.0 | **58.0** | 2323.3 |
| 90 | **37.3** | **44.3** | **44.7** | 432.5 | 179.2 | 147.6 | 170.0 | 187.0 | 320.0 | 348.4 | 214.5 | 125.3 | 2250.8 |
| 91 | 140. | **42.1** | **20.7** | 159.8 | 545.1 | 138.4 | 105.8 | 116.1 | 100.7 | 391.9 | 169.5 | 178.3 | 2153.9 |
| 92 | **47.6** | **0.2** | **4.7** | **64.1** | **5.5** | 203.7 | **5.9** | 88.7 | **5.3** | 201.7 | 378.7 | 88.2 | **1094.3** |
| 93 | **15.9** | **11.7** | **51.8** | 209.2 | 738.7 | 283.4 | 151.3 | **50.6** | 201.8 | 180.2 | 294.4 | 283.5 | 2472.5 |
| 94 | 89.9 | 78.8 | 58.3 | 262.9 | 317.1 | **59.3** | 283.4 | 156.6 | 461.1 | 621.9 | 84.7 | 124.5 | 2598.9 |
| 95 | 89.1 | 67.5 | **17.3** | 377.4 | 327.1 | 472.1 | 125.8 | 196.5 | **69.2** | 261.2 | 309.4 | **8.7** | 2321.3 |
| 96 | 98.8 | 105.5 | 103.5 | 203.7 | **69.8** | 256.3 | 163.5 | **77.8** | 470.1 | 174.2 | **145.0** | 102.1 | 1970.3 |
| 97 | **15.2** | 54.6 | **23.3** | **40.5** | **129.2** | 128.2 | 256.4 | 150.2 | 425.4 | 407.7 | 319.3 | 144.3 | 2150.3 |
| 98 | 50.2 | **33.4** | **15.1** | **77.2** | **117.3** | 234.0 | 405.0 | 176.1 | 203.2 | 270.0 | **22.1** | 490.0 | 2093.6 |
| 99 | 115. | 137.3 | 14.2 | 523.0 | 385.2 | 191.1 | 109.0 | 120.2 | 329.0 | 795.0 | **127.2** | 119.0 | 2864.2 |
| 00 | 236. | 112.2 | 281.1 | 210.3 | 288.3 | 192.1 | **59.2** | 215.0 | 323.0 | **157.3** | 298.5 | 145.7 | 2519.0 |
| 01 | 97.7 | 103.4 | 163.5 | 186.0 | **99.4** | **89.7** | **85.0** | **46.4** | 150.0 | 387.3 | 157.9 | 205.3 | 1771.6 |
| 02 | **40.0** | **49.7** | **56.8** | **57.4** | 234.2 | **54.6** | 118.1 | **52.7** | **98.5** | 312.8 | 322.7 | 139.2 | 1533.7 |
| Av | 99.0 | 103.0 | 115.0 | 225.0 | 298.0 | 191.0 | 196.0 | 166.0 | 243.0 | 342.0 | 307.0 | 187.0 | 2470 |

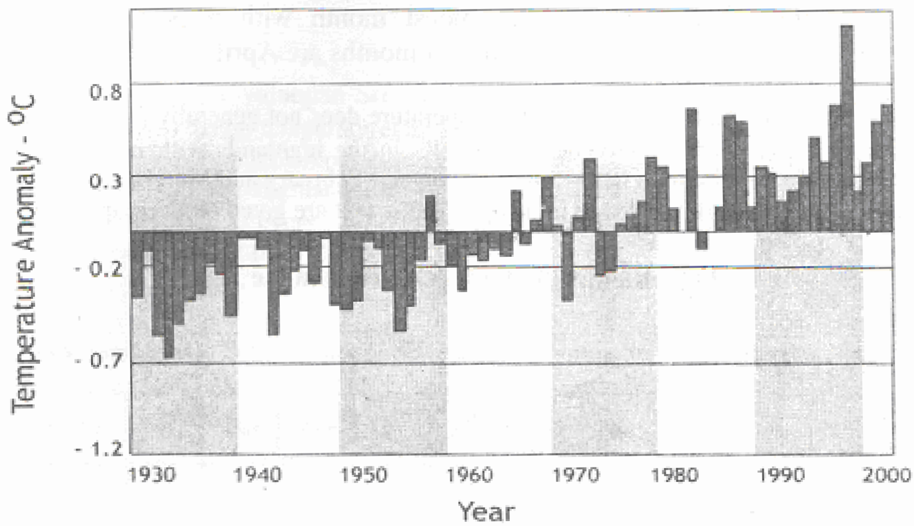
Av=Mean Precipitation of 1951-1980 period

Monthly precipitation below half of the mean is given in bold face. Source: Hydrology Annual

### *7.4. Long Term Effects*

The effect of Moon creating a drought when the two planes are coinciding is a long-term effect. Some times the drought extends to 18 months. It has shown a gradual extension from 1974 to 2001. This can be attributed to global warming. Tropical countries are subjected to this event. Astronomical features are similar to those lands with same geographic conditions and subjected to atmospheric pressure changes due to wind. Brazil, Indonesia and Sub Saharan Sahel countries face a drought with Sri Lanka in 2010. Tsunami is a rare occasion created by sudden lift of ocean floor for about 1km. High floods are occurring due to various changes in the atmosphere. Consistent changes need proper designing of spillways, embankments and drainage canals to suit the new changes. Early detection of a disaster minimizes the losses due to it. Augmentation is necessary to divert water from available basin to a depleted basin. Planning of new projects needs serious consideration of water resources management.

**Deviation of Sri Lanka's Mean Annual Air Temperature from 1961-90 Mean since 1930**




It is noteworthy to mention that the global mean surface temperature has increased by between 0.6 ± 0.2 °C since the late 19th century which has been attributed to increasing concentrations of greenhouse gases due to rapid industrial growth (IPCC, 2001).

## 8. CONCLUSION

Sri Lanka has 103 river basins including 14 perennial rivers. Many river basins discharge high floodwater to sea in the event of floods. At the same time global warming is continuing to show adverse impacts on hydrology. Increasing droughts and high intensity rainfall are developing during the last 30 years. Certainty changes are applicable to alter design parameters of old projects and take precaution for new projects to minimize the cost and maximize benefits.

Identification of astronomical impacts leads to proper planning of agricultural projects. Drought prediction has saved many losses by taking suitable actions without investing. Irrigation projects need design of embankment with suitable storage justified by projected hydrology. Canal system needs high efficiency to distribute water to command area. Hydrology changes abandon most of the projects and it is useful for all tropical countries for experience. The next drought was predicted in 2010 for all tropical countries. Agriculturists need to take care for rain-fed (hena) shifting farming in the wet zone of Sri Lanka. Young tea bushes dry during drought. As the Earth in January 4$^{th}$ is farthest from Sun and slows down the crust of Earth become weaker. On the full moon day Earth is pulled from both sides and becomes highly weak. Eruptions are likely in this period as happened in December full moon day. Tropical areas show more freedom for raining during full moon days. Tropical countries include Sri Lanka, India, Indonesia, Malaysia, Singapore, Australia, Ethiopia, Somalia, Sudan, Sahelian Countries, Brazil, Peru, Equador, Colombia.

Tidal changes in the cycle affect fish catch and regular fish breeding. Drought mitigation activities are needed in the island. Previous records show that all common plane episodes of Earth and Moon had droughts except in 1928.

Verification of common plane episode in the early period is commensurate with the 1965(66), 55(55), 47(47), 37(38), 18(18), 1909(08/09). However in the middle a seasonal drought occurs as in 1979, 1976, 1968, 1958, 1950, 1945, 1934, 1923, 1914/15 which resulted for other reasons other than astronomical. Little drought came in 1928. Hydrology records in the other tropical countries need a comparison for this episode.

## REFERENCES

[1] SENEVIRATNE, L. W., Drought Forecasting, *Irrigation Department, Colombo, Sri Lanka* 1993.

# A SIMPLE FRACTURE CRITERION TO PREDICT FAILURE OF STEEL STRUCTURES IN EXTREMELY-LOW CYCLE FATIGUE REGION

S.A.S.C.Siriwardane[1], P.B.R. Dissanayake[2], M. Ohga[3], K.S. Wanniarachchi[4]

[1]Senior Lecturer, Department of Civil Engineering, University of Ruhuna, Galle, Sri Lanka.
[1]E-mail: siriwardane@cee.ruh.ac.lk
[1]Telephone: + 94-71-8461275; Fax: + 94-91-2245762

[2]Senior Lecturer, Department of Civil Engineering, University of Peradeniya, Peradeniya, Sri Lanka.
[2]E-mail: ranjith@civil.pdn.ac.lk
[2]Telephone: +94-89-2393581; Fax: + 94-89-4476797

[3]Professor, Department of Civil & Environmental Engineering, Ehime University, Matsuyama, Japan.
[3]E-mail: ohga@cee.ehime-u.ac.jp
[3]Telephone: +81-89-9279816; Fax: +81-89-9279816

[4]Senior Lecturer, Department of Civil Engineering, University of Ruhuna, Galle, Sri Lanka.
[4]E-mail: wanniarachchi@cee.ruh.ac.lk
[4]Telephone: + 94-91-2245765; Fax: + 94-91-2245762

**Abstract:** This paper presents a simple criterion to predict the failure of steel structures due to the interaction effect of fracture and fatigue which is termed as extremely-low cycle fatigue (ELCF) failure. The criterion has been obtained from further simplification of available cyclic void growth model (CVGM). Initially the simplified ELCF fracture criterion is clearly presented and associated ELCF fracture prediction methodology is also indicated. The simplified criterion is then employed to determine ELCF fracture of some structural models. Hence verification of the simplified criterion is confirmed by comparing the results with previous criterion-based estimations. Then the simplified criterion is applied to predict the ELCF fracture of a reduced beam section specimen. Finally, study tends to conclude that the simplified criterion produces reasonable accurate prediction to ELCF fracture of steel structures where magnitude of triaxiality remains relatively constant.

**Keywords:** Low cycle fatigue, Earthquake loading, Cyclic plasticity, Finite element method

## 1. Introduction

The mechanism of extremely-low cycle fatigue (ELCF) was recently recognized with some of sudden failures of existing structures, which were characterized by large scale cyclic yielding due to occasional loadings such as earthquakes, typhoons. Generally, experimental approaches are popular for ELCF failure prediction. As for the authors view, only one theoretical study has recently been published [1], and the observed failure mechanism is based on void growth process. The fracture is calculated to occur when cyclic void growth index ($VGI_{cyclic}$) exceeds its critical value. The $VGI_{cyclic}$ demand is calculated based on complex integrations of a function, which depends on triaxiality and incremental plastic strain. However, it is required to modify commonly available finite element method (FEM) employed programs to cater this integration and finally it hindered the usage of general propose FEM packages as it is to estimate fracture in ELCF region. As a result, found applications of this criterion are very less.

Therefore, this study tends to simplify the above criterion to provide a new criterion to assess the real ELCF fracture of steel structures using available general-purpose FEM packages. However, application of this criterion is limited to the situation where triaxiality remains relatively constant during its loading history. As highlighted from previous studies [2, 3], in many realistic situations, this statement can be applicable. The failure mechanism of this criterion is also similar as previous criterion. But the fracture criterion is totally different from the previous one such that the ELCF fracture is calculated to occur when accumulated equivalent plastic strain at cyclic loading exceeds its critical value. Initially, the details of simplified criterion are briefly indicated. Then the fractures of several models in ELCF region are estimated. Hence verification of the simplified criterion is confirmed by comparing the results with previous criterion-based estimations. Finally, study tends to conclude that the proposed criterion gives reasonably accurate prediction to fracture in ELCF region of steel structures where triaxiality remains relatively constant during its loading.

## 2. Simplified ELCF fracture criterion

The simplified fracture criterion and associated failure mechanism are briefly summarized in this section. For further understanding of the details about simplification of this criterion from the previous CVGM model and there distinguish features, refer authors' previous publications [4, 5].

The failure mechanism of this criterion mainly depend on two main aspects such as level of void growth (demand), including the effects of void growth and shrinkage during reversed cyclic loading and critical level of the void growth (capacity), related to cyclic strain concentrations of the inter void ligament material. Once this void growth demand is reach to its capacity the failure is determined. The situation where magnitude of triaxiality remains relatively constant during the loading history, level of void growth is properly described by accumulated equivalent plastic strain [4, 5]. Hence the simplified ELCF fracture criterion is defined as,

$$\bar{\varepsilon}_p^{cyclic} > (\bar{\varepsilon}_p^{cyclic})_{critical} \tag{1}$$

where $\bar{\varepsilon}_p^{cyclic}$ is the accumulated equivalent plastic strain at cyclic loading and it can be determined by subtracting equivalent plastic strain that has accumulated during every tensile excursion of cyclic loading ($\bar{\varepsilon}_p^t$) from accumulated equivalent plastic strain for every compressive excursion of cyclic loading ($\bar{\varepsilon}_p^c$) as follows.

$$\bar{\varepsilon}_p^{cyclic} = (\bar{\varepsilon}_p^t - \bar{\varepsilon}_p^c) \tag{2}$$

The critical value (capacity) of accumulated equivalent plastic strain at cyclic loading $(\bar{\varepsilon}_p^{cyclic})_{critical}$ can be obtained as a degraded function of critical value (capacity) of accumulated equivalent plastic strain at monotonic loading $(\bar{\varepsilon}_p^{monotonic})_{critical}$ as bellow.

$$(\bar{\varepsilon}_p^{cyclic})_{critical} = (\bar{\varepsilon}_p^{monotonic})_{critical}\ \exp(-\lambda\varepsilon_p^{accumulated}) \tag{3}$$

where the $(\bar{\varepsilon}_p^{monotonic})_{critical}$ is critical value (capacity) of accumulated equivalent plastic strain at monotonic loading. Also this parameter represents the critical level of void growth (critical void size) under monotonic loading and it is determined by following expression.

$$(\bar{\varepsilon}_p^{monotonic})_{critical} = \alpha \quad \exp(-1.5T) \tag{4}$$

The $\alpha$ is termed as toughness index which is an experimentally determined material constant. The $\varepsilon_p^{accumulated}$ in Eq. (3) is defined as the equivalent plastic strain that has accumulated up to the beginning of each tensile excursion of loading. As described by Eq. (3), the critical accumulated equivalent plastic strain at cyclic loading $(\bar{\varepsilon}_p^{cyclic})_{critical}$ reduces to critical monotonic limit for monotonic loading situation. Because the accumulated equivalent plastic strain $\varepsilon_p^{accumulated}$ is zero for monotonic loading situations since it is calculated at the beginning of each tensile excursion. Hence the increment of cumulative equivalent plastic strain during current tensile cycle is not contributing to the damage that occurs within that loading increment. As a result the damage level is at a constant value within each tensile excursion. However, the accumulated equivalent plastic strain during current tensile cycles contributes only to void growth process, such that for each tensile cycle $\bar{\varepsilon}_p^{cyclic}$ is compared to a constant value of $(\bar{\varepsilon}_p^{cyclic})_{critical}$ which is calculated at the beginning of that cycle. This explanation confirms that ELCF fracture can only be initiated during tensile loading excursions.

The prediction of ELCF crack initiation is made when the $\bar{\varepsilon}_p^{cyclic}$ exceeds its critical value $(\bar{\varepsilon}_p^{cyclic})_{critical}$ over a characteristic length measure ($l^*$) in the region of high stresses and plastic strains. However, places where triaxiality varies significantly during loading histories, the mentioned simplified criterion might produce a less accurate result than the CVGM criterion.

The described ELCF fracture criterion involve three parameters such as toughness index ($\alpha$), damageability parameter ($\lambda$) and the characteristic length ($l^*$). The $\alpha$ and the $\lambda$ are determined through testing and finite element analysis of circumferentially smooth-notched tensile specimen. The characteristic length ($l^*$) can be determined through the micro-structural measurements and observation of the fracture surface [1].

To utilize this fracture criterion, it is needed to determine the mentioned plastic variables during the multiaxial cyclic loading. Here, it is compulsory to perform a proper elasto-plastic analysis using a proper cyclic hardening model, which is compatible to complex structures with a reasonable accuracy.

## 3. Verification of simplified ELCF fracture criterion

Simplified ELCF fracture criterion is verified by comparing the simplified criterion predicted fracture displacements with CVGM [1] predicted results of three different structural models. In this comparison, a single hardening model is utilized with both fracture criteria [6, 7]. The A572-grade 50 steel is considered as the constructed material of all four models. The ELCF material constants, toughness index ($\alpha$), damageability coefficient ($\lambda$) and characteristic length ($l^*$) are 1.18, 0.49 and 0.18 mm respectively [1]. This material exhibits nearly non-linear kinematic hardening behavior.

### *3.1 Fracture prediction of a plate with a hole (Model 1)*

The geometry of the considered structural model is shown in Figure 1. Considering symmetry of the geometry, loading and boundary conditions, the one-fourth of the geometry was subjected to FE analysis. The nine-node isoperimetric shell element was used for FE mesh as shown in Figure 3. Initially considered geometry is subjected to the monotonic load analysis. By observing the stress

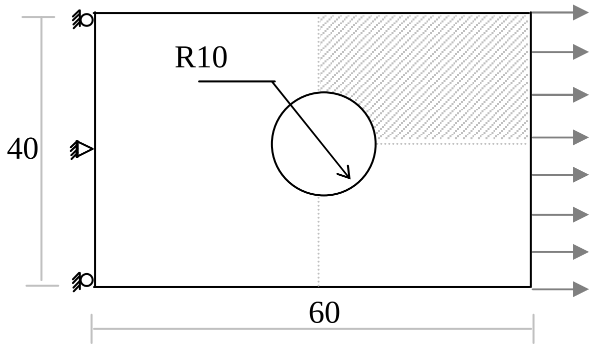



Figure 1: *Geometrical details of Model 1*

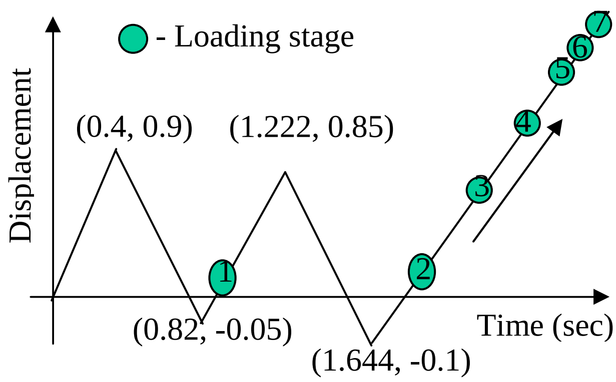



Figure 2: *Displacement history of Model 1*

distribution at ductile fracture (stress contour is shown in Figure 4), it is able to conclude that the critical zone lies along the transverse centerline of the specimen as shown in Figure 4. From the monotonic load analysis, variation of triaxility ($T$) versus effective plastic strain was plotted for sampling Gauss points at critical zone (Figure 5). These variations reveal that the triaxiality $T$ does not illustrate significant variation with increment of plastic loading and the average value was considered for future calculations. Hence, the critical values of accumulated equivalent plastic strain

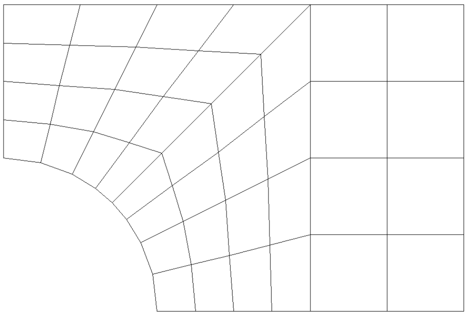


Figure 3: *FE mesh for Model 1*

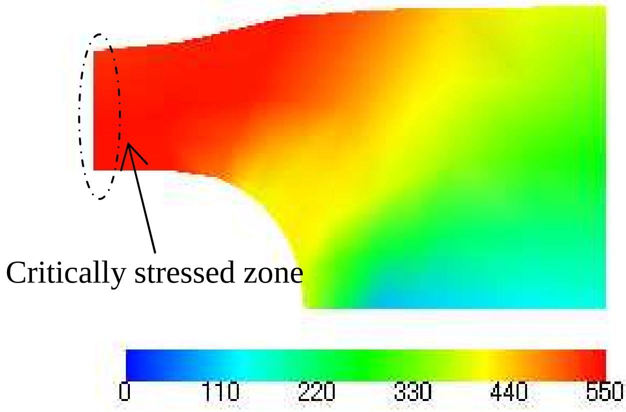



Figure 4: *von Mises stress contour of Model 1*

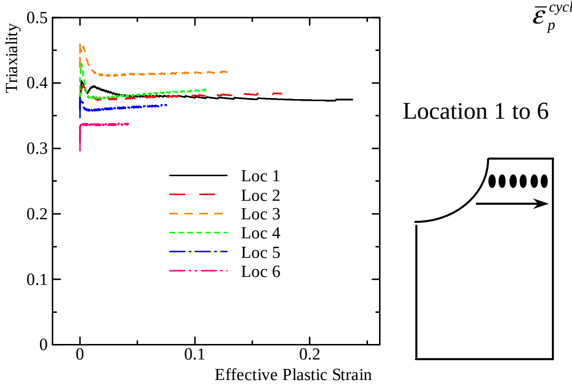



Figure 5: *Triaxiality variation versus plastic strain at different location for Model 1*

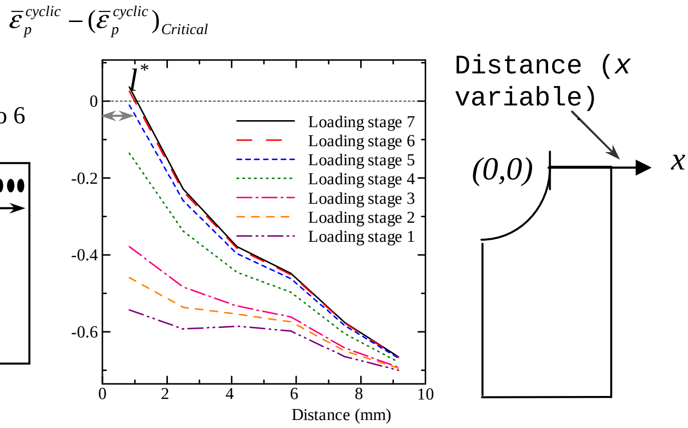



Figure 6: *The* $\bar{\varepsilon}_p^{cyclic} - (\bar{\varepsilon}_p^{cyclic})_{Critical}$ *variation along the length of critical zone for Model 1*

$((\bar{\varepsilon}_p^{monotonic})_{critical})$ at monotonic loading are calculated for each sampling Gauss points along the transverse centerline of the model. Then FE elasto-plastic analysis was conducted for cyclic loading based on considered hardening model. The applied displacement history is indicated in Figure 2. Finally, cyclically degraded values of the critical accumulated equivalent plastic strain, $(\bar{\varepsilon}_p^{cyclic})_{critical}$ variations are plotted at different loading stages. Simultaneously, the demands of accumulated equivalent plastic strain are also calculated for sampling Gauss points. Hence $\bar{\varepsilon}_p^{cyclic} - (\bar{\varepsilon}_p^{cyclic})_{critical}$ variations along the transverse centerline were determined as shown in Figure 6. Finally, ELCF macro crack initiation was made when $\bar{\varepsilon}_p^{cyclic} - (\bar{\varepsilon}_p^{cyclic})_{critical}$ exceeds zero over the characteristic length at loading stage seven. The applied displacement corresponding to this loading stage is recorded as the fracture displacement in Table 1.

### *3.2 Fracture prediction of a hollow cylindrical pier (Model 2)*

The corresponding geometric details are shown in Figure 7. The horizontally applied uni-directional ground displacement is considered for this model as shown in Figure 8. The whole geometry was subjected to FE analysis using nine-node isoperimetric shell element. The followed procedures for ELCF fracture prediction are similar to the case of Model 1. The displacements at fracture are recorded in Table 1.

### *3.2 Fracture prediction of a hollow squared pier (Model 3)*

The geometric details are shown in Figure 9. In this case also the horizontally applied uni-directional ground displacement is considered for this model as shown in Figure 10. The FE analysis was conducted using nine-node isoperimetric shell element. The followed procedures for ELCF prediction are also as same as Model 1. The Table 1 shows the applied displacements of ELCF fracture.

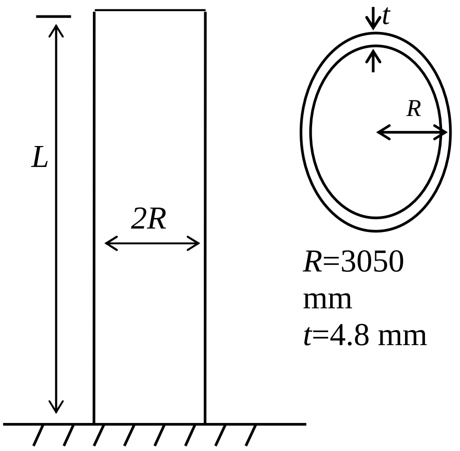



Figure 7: *Geometrical details of Model 2*

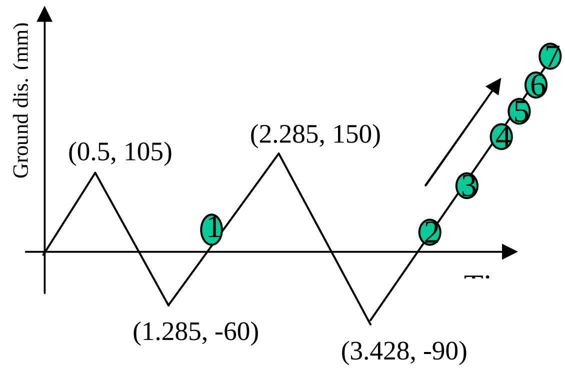



Figure 8: *Displacement history of Model 2*

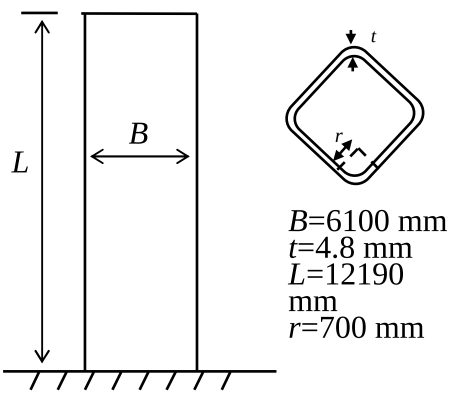



Figure 9: *Geometrical details of Model 3*

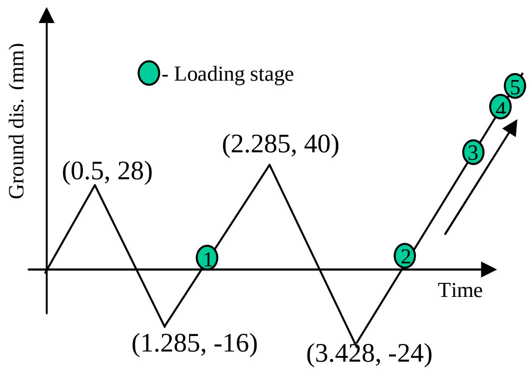



Figure 10: *Displacement history of Model 3*

Table 1: *Comparison of ELCF fracture displacement of structural models*

| Model | Description | ELCF fracture displacement (mm) | | Difference (%) |
|---|---|---|---|---|
| | | CVGM Criterion | Simplified Criterion | |
| 1 | Plate with a hole | 2.43 | 2.35 | 3.4 |
| 2 | Hollow cylindrical pier | 190.20 | 213.30 | 10.8 |
| 3 | Hollow squared pier | 65.62 | 62.05 | 5.8 |

Comparison of fracture displacements of both criterions in Table 1 shows that there is a similarity between fracture criterions. However, the percentage wise difference of fracture displacement of Model 2 is slightly higher than other models. Assumed reason is that, even though the Triaxiality (*T*) of the Model 2 is relatively constant, it shows slight rate of increment during plastic loading. However, this comparison reveals that simplified ELCF fracture criterion produces reasonable accurate predictions to structures where the Triaxiality is relatively constant.

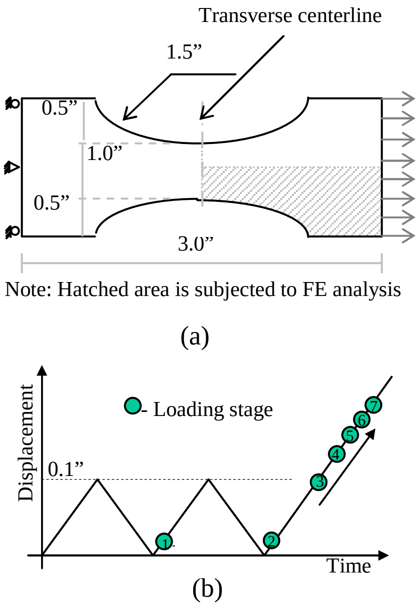
## 4. Case Study: Fracture prediction of a reduced beam section specimen

During the Northridge and Kobe earthquake, welded connections in steel moment frames were found to be a weak link in structural systems [1, 2]. After these disasters, reduced beam section specimen (RBS) or dog-bone type connection detail (Figure 11) was developed to concentrate the plastic hinge a certain distance away from the connection within the beam. Though such connections have the potential to prevent sudden and brittle failures such as those in welded connections, there is always the possibility of ductile fracture under large plastic strains. This section initially describes the experimental determination procedure of ELCF fracture of RBS specimen. Then theoretical ELCF fracture was also determined for same RBS specimen. Finally the accuracy and the applicability of simplified fracture criterion are verified by performing comparisons to obtained results.

Figure 11: *RBS type connection*

Considering symmetricity of RBS specimen (Figure 12-(a)) geometry, loading and boundary conditions, the one-fourth of the geometry was subjected to FE analysis. The nine-node shell element was used for FE mesh as shown in Figure 13-(a).

Initially considered geometry is subjected to the monotonic load analysis. By observing the stress distribution at ductile failure (stress contour is shown in Figure 13-(b)) it is able to conclude that



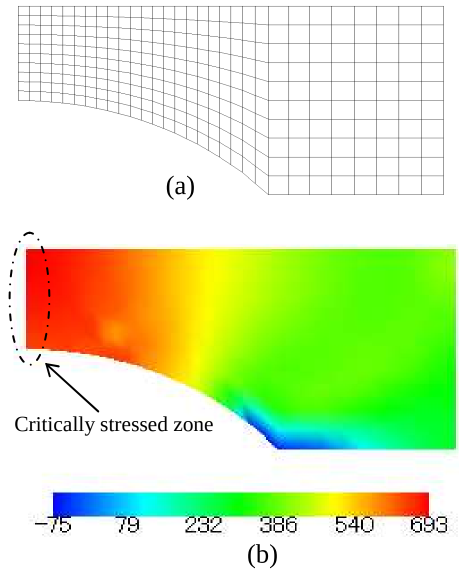
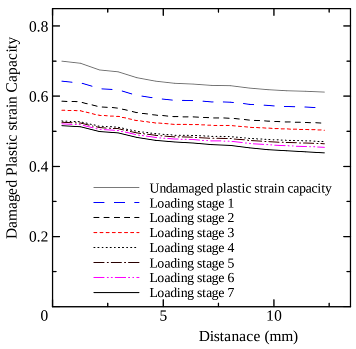
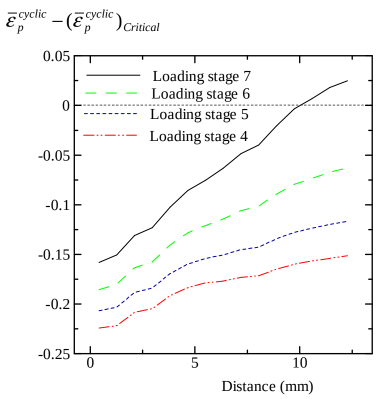




Figure 12: *(a). Geometry of RBS specimen (b). Loading history (quasi-static)*

Figure 13: *(a) FEM mesh (b) von Mises stress contours for monotonic loading*



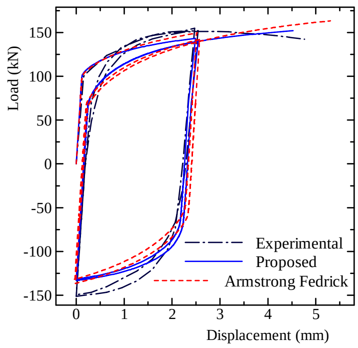




Figure 14: *Damaged* $(\bar{\varepsilon}_p^{cyclic})_{Critical}$ *capacity*

Figure 15: $\bar{\varepsilon}_p^{cyclic} - (\bar{\varepsilon}_p^{cyclic})_{Critical}$ *Variation*

Figure 16: *Load/displacement comparison*

critical zone lies along the transverse centerline of the specimen as shown in Figure 13-(b). The considered material is, A572-grade 50 steel and toughness index ($\alpha$) was taken as 1.18 [3]. Hence critical values of accumulated equivalent plastic strain ($(\bar{\varepsilon}_p^{monotonic})_{critical}$) at monotonic loading is calculated for sampling Gauss points along the transverse centerline of the specimen.

Table 2: *Comparison of ELCF fracture displacement of RBS specimens*

| Description | Fracture displacement (mm) | Error with experimental displacement (%) |
|---|---|---|
| Experiment | 4.76 | - |
| ELCF Simplified criterion | 4.50 | 5.46 |
| ELCF CVGM criterion | 4.72 | 0.84 |
| Ductile fracture criterion | 6.23 | 30.88 |

Then cyclic load FE analysis was conducted for one-fourth part of the specimen. The applied load (displacement) versus time variation is indicated in Figure 12-(b) and it has been predicted to simulate same effect of the experimental cyclic load test. Cyclically degraded values of the critical accumulated equivalent plastic strain, $(\bar{\varepsilon}_p^{cyclic})_{critical}$ variations are plotted at different loading stages as shown in Figure 14. Simultaneously the demands of accumulated equivalent plastic strain are also calculated for sampling Gauss points. Hence the $\bar{\varepsilon}_p^{cyclic} - (\bar{\varepsilon}_p^{cyclic})_{Critical}$ variations along the transverse centerline were determined and plotted as Figure 15.

Finally the prediction of ELCF macro crack initiation is made when $\bar{\varepsilon}_p^{cyclic} - (\bar{\varepsilon}_p^{cyclic})_{Critical}$ exceeds zero over the characteristic length ($l^*$) at loading stage seven. The corresponding displacement to this instant is recorded as the fracture displacement, and the theoretical prediction of ELCF failure is compared with experimental results as shown in Table 2. The corresponding load displacement relations are drawn in Figure 16.

## 5. Conclusions

The general CVGM (cyclic void growth model) based extremely low cycle fatigue (ELCF) fracture criterion was simplified for steel structures where magnitude of the triaxiality remains relatively constant during their loading history. Then the ELCF fractures of few structural components were separately predicted by both simplified and CVGM fracture criterion and results were compared.

The comparisons of ELCF fracture initiation life of simplified model with previous CVGM model of some structural components reveal that simplified criterion produces reasonable accurate prediction to steel structures where magnitude of the triaxiality remains relatively constant. The case study exhibits that the simplified ELCF fracture criterion work well to obtain the more realistic predictions to ELCF fracture. Further, it determines the location of fracture accurately. Considering all these reasons, it is advisable to use simplified ELCF criterion to describe ultimate limit state of steel structures in seismic design practice. The main advantage behind this simplified criterion is that it can be easily utilized with commonly available elasto-plastic finite element (FE) packages. Because most of elasto-plastic FE programs produce outputs of simplified criterion dependent plastic variables such as accumulated equivalent plastic strain, stress components and effective stress. Finally, these reasons conclude that this study provide a new theoretical platform for structural design and maintenance communities by contributing convenient, precise and reliable ELCF criterion to describe real life of structures.

If indeed it is feasible to apply these ELCF proposed models at a larger scale, e.g. in large-scale beam column connections under cyclic loading. Large scale modeling of specimens often requires transition from smaller to larger elements to capture local (microstructure-level) as well as global stress effects. As a result, large-scale modeling, especially with cyclic loading, can be computationally very demanding. Advances in computational technology suggest that it would be worthwhile to explore the feasibility of applying such models at the larger scale. Even though some options are available, sub-structuring method becomes as more convenient and such option based ELCF fracture predictions of large-scale structures are more appropriate for future studies.

### References


81. Kanvinde, A.M., and Deierlein, G.G., “Cyclic void growth model to assess ductile farcture initiation in structural steels due to ultra-low cycle fatigue”, *Journal of Engineering Mechanics*, ASCE, Vol: 133, No: 6, 2007, pp.701-712.
82. Chi, W.M., Kanvinde, A.M., and Deierlein, G.G., “Prediction of ductile fracture in steel connection using SMCS criterion”, *Journal of Structural Engineering*, ASCE, Vol:132, No: 2, 2006, pp.171-181.
83. Kanvinde, A.M., and Deierlein, G.G., “Void growth model and stress modified critical strain model to predict ductile fracture in structural steels”, *Journal of Structural Engineering*, ASCE, Vol: 132, No: 12, 2006, pp.1907-1918.
84. Adasooriya, A.M.N.D., Siriwardane, S.A.S.C., Ohga, M., Kaita, T. and Dissanayake, P.B.R., “A simplified fracture initiation criterion for ultra-low cycle fatigue”, *International conference on continuum mechanics, nanoscience and nanotechnology*, Peradeniya, Sri Lanka, 26-27 September 2008, pp. 87-94.
85. Siriwardane, S.C., Ohga, M., Dissanayake, R. and Taniwaki, K., “Comparative study for remaining fatigue life estimation of existing railway bridges”, *Proceeding of 4th International Symposium on Steel Structures ISSS06*, Seoul, Korea, 16-18 November 2006, pp. 912-921.
86. Lemaitre J. and Chaboche J.L., “Mechanics of solid materials”, UK, Cambridge University Press, 1990.
87. ABAQUS, Theory manual, version 6.5, Hibbitt, Karlsson, and Sorensen, Inc., Providence, R.I, 1998.



### Acknowledgements

The authors wish to express their sincere gratitude to Emeritus Professor M.P Ranaweera, University of Peradeniya, Sri Lanka and other team of experts who worked in the Kelani Railway bridge project for their great advice, which laid the strong foundation of this research.

# CRACKING DUE TO TEMPERATURE GRADIENT IN CONCRETE

[1]W.G.J. Prasanna and [2]A.P. Subhashini

[1]Eng. W.G.J. Prasanna, AMIE(SL), SSE(SL) B.Sc. Eng. (Moratuwa)
[1]E-mail: janaka81lk@yahoo.co.uk

[2]Eng. A.P. Subhashini, AMIE(SL), B.Sc. Eng. (Moratuwa)
[2]E-mail:subha81lk@yahoo.co.uk

**Abstract:**Mass concrete is used in many projects related to the massive construction such as raft foundations, pile caps, thick beams, walls and dams. Since cement hydration is an exothermic reaction, the temperature rise within a large concrete mass can be quit high. As a result, significant tensile stresses and strains may be developed from the volume change associated with the increase and decrease of temperature within the mass concrete which will lead to crack the concrete. Cracks caused by thermal gradient may cause loss of structural integrity and monolithic action or shortening of service life of the structures. The objective of this research is to determine the thermal strain variation from arisentemperature datawhich in turn can be used to predict, whether the relevant concrete section is going to be cracked or not by comparing with tensile strain capacity values.

**Keywords:** Surface Gradient Analysis, Balanced Temperature, Tensile Strain Capacity

## 1 Introduction

According to ACI 207[1], “Mass Concrete (MC)is any large volume of concrete with dimensions large enough to require that measures be taken to cope with the generation of heat and attendant volume change to minimize cracking.”

The most important characteristic of mass concrete is thermal behavior. When Portland cement combines with water, the resulting exothermic (heat-releasing) chemical reaction causes a temperature rise in the concrete mass. The actual temperaturerise inmass concrete structures depends upon the heat generating characteristics of the concrete mixture, its thermal properties, environmental conditions, geometry of the MCstructure, and construction conditions. Since concrete has a low conductivity, a great portion of generated heat is trapped in the center of mass concrete element and escapes very slowly. This situation leads to a temperature gradient between center and outer part of the mass concrete element. Temperature gradient is a cause for tensile stresses, and when stress exceeds the tensile strain capacity of concrete,“Thermal Cracks”are formed in the concrete structure.

Usually the peak temperature is reached in a few days to weeks after placement, followed by a slow reduction in temperature. A change in volume occurs in the MC structure proportional to the temperature change and the coefficient of thermal expansion of the concrete. If volume change is restrained during cooling of the mass, by the foundation, the previously placed concrete, or the exterior surfaces, sufficient tensile strain can develop to cause cracking. Cracking generally occurs in the main body or at the surface of the MC structure. These two principal cracking phenomena are termed Mass Gradient and Surface Gradient cracking, respectively [1].

In this study, Tensile Strain Capacity (TSC)of concrete is used with the results of temperature analysis to determine the risk of forming cracks in MC.

## 2 Literature Survey

ACI207.1R contains detailed information on heat generation, volume change, restraint, and cracking in concrete. The analysis procedure for Surface Gradient was carried out according to the method given in technical report by U.S. Army Corps of Engineers [1]. Surrounding data was taken from “CIRIA Report 91 – Early age thermal crack control in concrete” [3].

### *2.1 Surface Gradient Analysis*

Surface gradient cracking occurs due to the "Internal Restraint",in which changes in thetemperature profile across the element can cause one part (exterior) of the section to restrain the movement of another part (interior) of same section. Strain was used as a basis for the surface gradient cracking analysis, sinceit is not suitable to rely upon the constant Modulus ofElasticity as it varies with age & temperature of the concrete.

The strain due to thermal gradient in concretecan be determined by equation (1) given inACI 207.2R,

(1) $$\varepsilon = (C_{th})(dT)(K_R)$$

Where,

$\epsilon$= induced tensile strain (x $10^{-6}$)

$C_{th}$= coefficient of thermal expansion - x $10^{-6}/^{0}$ C

*dT*= temperature difference with respect to interior temperature - deg C

$K_R$ = internal restraint factor

Typical values for the coefficient of thermal expansion for mass concrete are in the range 5-14 x $10^{-6}/^{0}$C. Fora constant value of 10.5 x $10^{-6}/^{0}$C coefficient of thermal expansion was considered for this study.

### *2.1.1 Surface gradient restraint factor ($K_R$)*

The degree of restraint cannot bedetermined exactly but can be estimated based on the thickness of the exterior surface layer being restrained. The restraint factor, $K_R$, is computed from following equations depending upon the value of *L/H*, where *L* is the monolith width (between joints or between ends of the monolith) and H is the distance from the interior strain and stress-free surface (Thermal neutral surface) to the exterior surface, called "Tension Block Width" as shown in Figure 2.1

For *L/H* greater or equal to 2.5 — For *L/H*lessthan 2.5

(2) $$K_R = \left[\frac{L/H - 2}{L/H + 1}\right]^{h/H} \qquad K_R = \left[\frac{L/H - 1}{L/H + 10}\right]^{h/H}$$

The $K_R$ can be determined from equation (2)

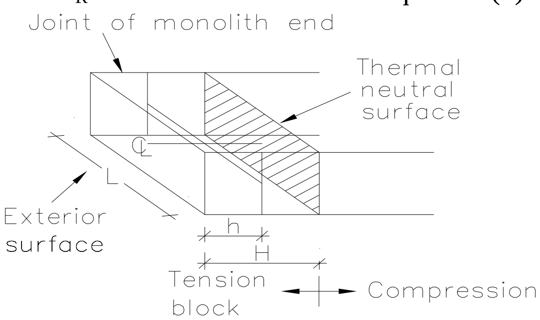



Figure 2.1: *Surface Gradient Restraint Model*

### *2.1.2 Determination ofdT andH*

The temperature distribution can induce tension near the surface and compression within the interior of concrete. ACI 207.2R states that for sectional stability, the summation of tensile stresses (and strains) induced by a temperature gradient in a gross section must be balanced by equal compressive stresses (and strains).

Therefore,thetemperature differences (dT) are arranged to provide equal tension and compression in the section, providing a graphical representation of the surface gradient restraint model (Figure 2.1).

While Negative temperature differences are producing tensile stresses, positive temperature differences produce compressive stresses (Figure 2.2). The location of dT=0 determines the location of the tension block relative to the exterior surface (H). By equating the shaded positive and negative areas in Figure 2.2, H can be calculated.

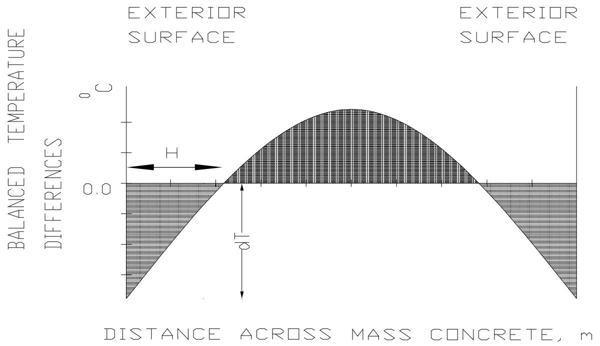



Figure 2.2: *Shape of the Balanced TemperatureDistribution*

The main steps in calculating tensile strain due to temperature distribution are given in Figure 2.3 and the detailed description is included in “Case Study”.

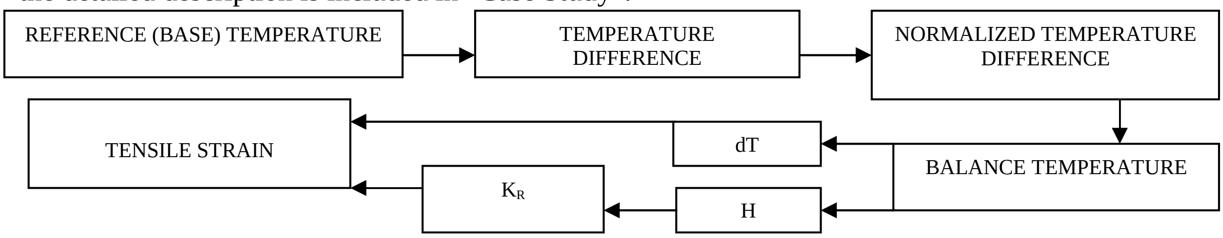



Figure 2.3: *Method of Finding Tensile Strain*

Reference (Base) temperatures for a surface gradient analysis are defined as the temperatures in the structure at the time when the concrete begins to harden and material properties begin to develop. It wasassumed that concrete begins to gain elastic form at 2 hrs after mixing.

## 3 Experimental Investigation

### *3.1 Temperature Rise in Concrete Cube*

A 1.5m concrete cube was cast on ground with plywood formwork on sides and bottom. Top surface was insulated with a polystyrene sheetand a sand layer.

In order to obtain temperature distribution fixed thermo couples (TC1 to TC6) were embedded at the centre of the block as shown in Figure 3.1.

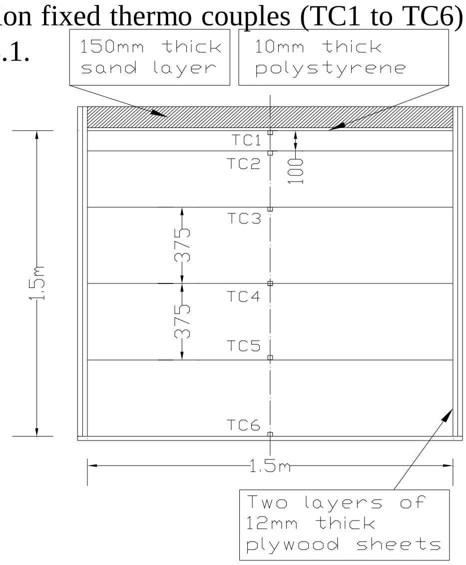



Figure 3.1: *The Typical Concrete Mass*

Temperature data obtained from thermo couples are shown in Table 3.1. Temperature changes or differences have been determined by taking the base temperatures as temperatures at the concrete age of 2 hr.

| | | Temperature distribution – C | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Location | distance(h) | 2 hr | 12 hr | 18 hr | 24 hr | 48 hr | 72 hr | 96 hr | 120 hr |
| TC1 | 0.000 | 30.39 | 49.28 | 57.84 | 60.00 | 57.53 | 49.20 | 43.12 | 37.76 |
| TC2 | 0.100 | 31.12 | 52.8 | 62.16 | 64.32 | 61.74 | 52.56 | 45.28 | 39.46 |
| TC3 | 0.375 | 31.20 | 59.44 | 68.56 | 71.84 | 68.96 | 58.72 | 49.84 | 42.72 |
| TC4 | 0.750 | 31.15 | 60.64 | 70.72 | 74.32 | 71.92 | 60.80 | 51.36 | 43.92 |
| TC5 | 1.125 | 31.17 | 59.6 | 68.96 | 71.52 | 67.70 | 57.68 | 49.28 | 42.64 |
| TC6 | 1.500 | 30.46 | 50.16 | 56.4 | 57.28 | 54.86 | 48.08 | 42.48 | 37.96 |

Table 3.1: *Temperature Data*

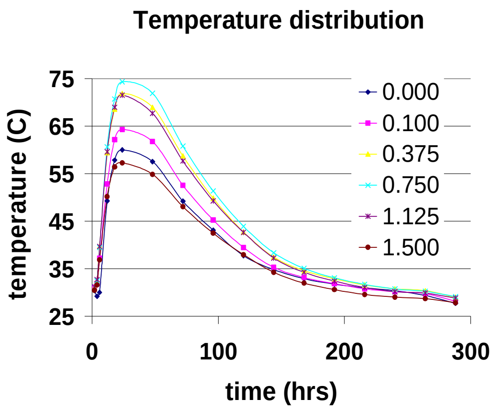



Figure 3.2: *Temperature Variation of the Concrete Section with Time*

Temperature differences relative to the base temperatures are shown in Table 3.2.

Table 3.2: *Temperature Difference*

| h | 2.0 | 12.0 | 18.0 | 24.0 | 48.0 | 72.0 | 96.0 | 120.0 |
|---|---|---|---|---|---|---|---|---|
| 0 | 0.00 | 18.89 | 27.45 | 29.61 | 27.14 | 18.81 | 12.73 | 7.37 |
| 0.1 | 0.00 | 21.68 | 31.04 | 33.20 | 30.62 | 21.44 | 14.16 | 8.34 |
| 0.375 | 0.00 | 28.24 | 37.36 | 40.64 | 37.76 | 27.52 | 18.64 | 11.52 |
| 0.75 | 0.00 | 29.49 | 39.57 | 43.17 | 40.77 | 29.65 | 20.21 | 12.77 |
| 1.125 | 0.00 | 28.43 | 37.79 | 40.35 | 36.53 | 26.51 | 18.11 | 11.47 |
| 1.5 | 0.00 | 19.70 | 25.94 | 26.82 | 24.40 | 17.62 | 12.02 | 7.50 |

Firstly, Normalized temperature differences are obtained by subtracting the surface temperature differences from the corresponding interior temperature differences at the same time intervals (seeTable 3.3).

Table 3.3: *Normalized Temperature Difference*

| h | 2.0 | 12.0 | 18.0 | 24.0 | 48.0 | 72.0 | 96.0 | 120.0 |
|---|---|---|---|---|---|---|---|---|
| 0 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 0.1 | 0.00 | 2.79 | 3.59 | 3.59 | 3.48 | 2.63 | 1.43 | 0.97 |
| 0.375 | 0.00 | 9.35 | 9.91 | 11.03 | 10.62 | 8.71 | 5.91 | 4.15 |
| 0.75 | 0.00 | 10.60 | 12.12 | 13.56 | 13.63 | 10.84 | 7.48 | 5.40 |
| 1.125 | 0.00 | 9.54 | 10.34 | 10.74 | 9.39 | 7.70 | 5.38 | 4.10 |
| 1.5 | 0.00 | 0.81 | -1.51 | -2.79 | -2.74 | -1.19 | -0.71 | 0.13 |

ThenBalanced Temperature (dT)can be calculated by equating positive and negative areas (by integrating) of normalized temperature difference distribution. Those Balanced temperature values are shown inFigure 3.3.Consequently, Tension Block Widths (H)that underwent tensile stresses at each age of the concrete,were calculated (table 3.4).

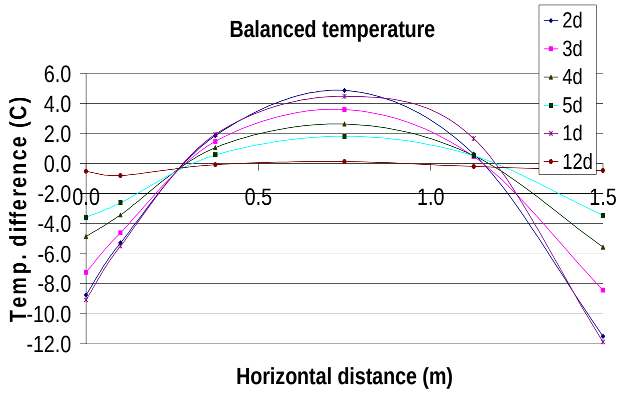



Figure 3.3: *Balanced Temperatures*

Table 3.4: *Tension Block Widths*

| Time(hr) | 12.0 | 18.0 | 24.0 | 48.0 | 72.0 | 96.0 | 120.0 |
|---|---|---|---|---|---|---|---|
| Top(m) | 0.318 | 0.280 | 0.284 | 0.287 | 0.295 | 0.305 | 0.327 |
| Bot. (m) | 0.298 | 0.310 | 0.332 | 0.354 | 0.331 | 0.340 | 0.327 |

$K_R$(Restraint factors) were calculated according to the equation (2) and given in Table 3.5.

Table 3.5: *Restraint Factors*

| | 12.000 | 18.0 | 24.0 | 48.0 | 72.0 | 96.0 | 120.0 |
|---|---|---|---|---|---|---|---|
| Top surf. | 1.000 | 1.000 | 1.000 | 1.000 | 1.000 | 1.000 | 1.000 |
| 0.1m from top | 0.791 | 0.796 | 0.796 | 0.795 | 0.794 | 0.793 | 0.790 |
| Bot. surf. | 1.000 | 1.000 | 1.000 | 1.000 | 1.000 | 1.000 | 1.000 |

Out of interior nodes, only a node which was located 0.1m below from the top should be considered, because all other nodes are located in compression zone. Assuming the Coefficient of thermal expansion as 10.5 x $10^{-6}/^{0}$C, induced tensile strainswere found by using equation (1) and are shown in Table 3.6.

Table 3.6: *Induced Tensile Strain values in millionths*

| time | 12.0 | 18.0 | 24.0 | 48.0 | 72.0 | 96.0 | 120.0 |
|---|---|---|---|---|---|---|---|
| top surface | 86 | 87 | 95 | 92 | 76 | 51 | 38 |
| 0.1m from top | 45 | 40 | 46 | 44 | 38 | 29 | 22 |
| bottom surface | 77 | 103 | 125 | 121 | 89 | 58 | 36 |

Figure 3.4 shows thetensile strain variation with time for top, bottom and 0.1m below top surface.

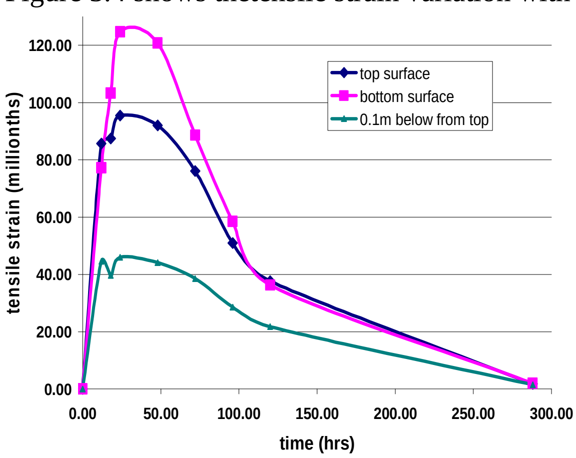



Figure 3.4: *Tensile Strain Variation with Time*

It can be seen that maximum tensile strains were develop at the age of 24 hrs.

### *3.2 Tensile Strain Capacity of Concrete*

Tests were carried out to obtain the tensile strain capacity of Gr 40 concrete.Three types of tests were conducted,*Rapid load flexural beam test, Compressive strength test andSplitting tensile test*.Since the strains due to surface gradients develop more rapidly, the Rapid-load beam test was conducted.

#### *3.2.1 Rapid Load Flexural Beam Test*

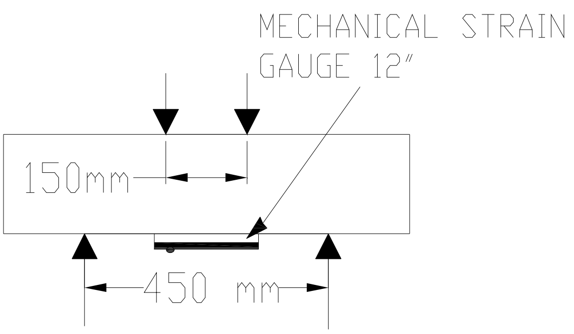



Figure 3.5: *Loading Arrangement of Rapid Load BeamTest*

*Test Specimens and mix design properties*

Table 3.7 gives No. of beams tested at each age whereas Table 3.8 gives the mix proportions of Gr 40 concrete (Slump – 180 mm)used in the test series.

Table 3.7: *No of Test Beams*Table 3.8: *Mix Proportions of Gr 40 concrete*

| 17.5 hrs | 24 hrs | 42 hrs | 144 hrs |
|---|---|---|---|
| 3 nos of beams | 2 nos of beams | 2 nos of beams | 2 nos of beams |

| Material | Quantity |
|---|---|
| Cement | 485 kg |
| Sand + Quarry dust (1:1) | 762 kg |
| Machine crushed Aggregate | 1009 kg |
| Admixtures (Super Crete) | 4800 ml |
| Water | 160 kg |

***Test Method***

Loading of the beams was done in accordance with the procedure given in CRD-C 16 [2]. For each loading age, beams were loaded at a rapid loading rate of 0.28 MPa/min.The loading arrangement is shown in Figure 3.5.

A continuous record of load and strain was obtained throughout the test by a "Mechanical strain gauge"untilthe beam failure.

***Results***

Ultimate tensile strain capacities at each age are given in Table 3.9.

Table 3.9: *Tensile Strain Capacities*

| Time | 17.5 hr | 24 hr | 42 hr | 144 hr |
|---|---|---|---|---|
| Failure Load (kN) | 21.7 | 24.7 | 30.0 | 32.9 |
| Strain Capacity (millionths) | 156 | 168 | 213 | 228 |

Compressive strength andsplitting tensile strength of concrete at critical ages are given in Table 3.10.

Table 3.10: *Strength Results*

| | | | |
|---|---|---|---|
| Age of the concrete (hrs) | 17.5 | 24.0 | 42.0 |
| Compressive Strength (N/mm2) | 18.1 | 21.2 | 43.5 |
| Tensile Strength (N/mm2) | 1.647 | 2.147 | 2.563 |

## 4 Analysis of Results and Discussion

### *4.1 Backward Analysis*

Based on equation (1), for a given tensile strain capacity, maximum temperature difference (dT) can be calculated at maximum restraint of $K_R$=1 (See Table 4.1)

Table 4.1: *Temperature Differences for $K_R$=1*

| Strain capacity Є(millionths) | $C_{th}$, millionths/deg C | *Kr* | *dT* |
|---|---|---|---|
| 156 | 10.5 | 1 | 14.86 |
| 168 | 10.5 | 1 | 16 |
| 213 | 10.5 | 1 | 20.29 |
| 228 | 10.5 | 1 | 21.71 |

The determination of the variation betweennormalized temperatures vs. distance is a trial and error process. The variation should be a polynomial function of fourth degree. Tension block width was assumed and therefore, the known coordinates are as follows (Table 4.2). According to those coordinates, a relevant graph could be drawn.

Table 4.2: *Coordinates of graph for $K_R$=1*

| | | | | |
|---|---|---|---|---|
| X | 0 | H | 1.5-H | 1.5 |
| Y | 0 | ***dT*** | ***dT*** | 0 |

The area of the graph should be equated by the Y = dTas to satisfy tension and compression stresses equal. The final graphs obtained from the above process are given inFigure4.1.

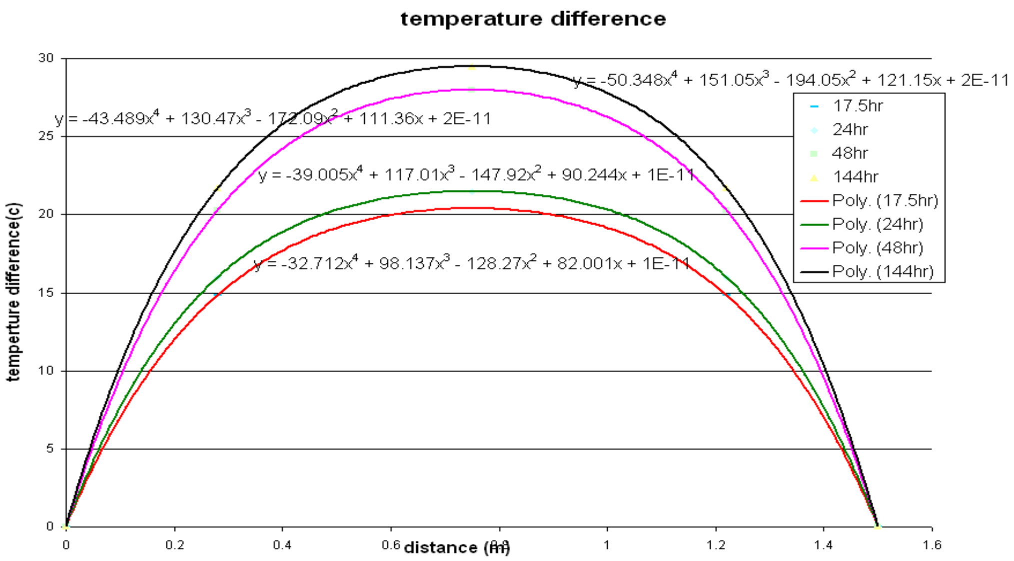



Figure 4.1: *Balanced Temperature curves for $K_R$=1*

Therefore maximum temperature differences are as given in Table 4.3,

Table 4.3-*Maximum Temperature differences*

| | | | | |
|---|---|---|---|---|
| Age | 17.5 | 24 | 42 | 144 |
| Maximum normalized temperature | 20.4 | 21.5 | 28 | 29.5 |
| Max. Norm. temp. for slow load test | 28 | 30 | 38 | 41 |

Since the tensile strain capacity values should be multiplied by 1.4 [1], to determine the tensile capacity values under the slow loading, the temperature differences are also varying as given in Table 4.3.

These normalized temperature differences are developed by subtracting the surface temperature differences from the corresponding interior temperature differences at the same time intervals.

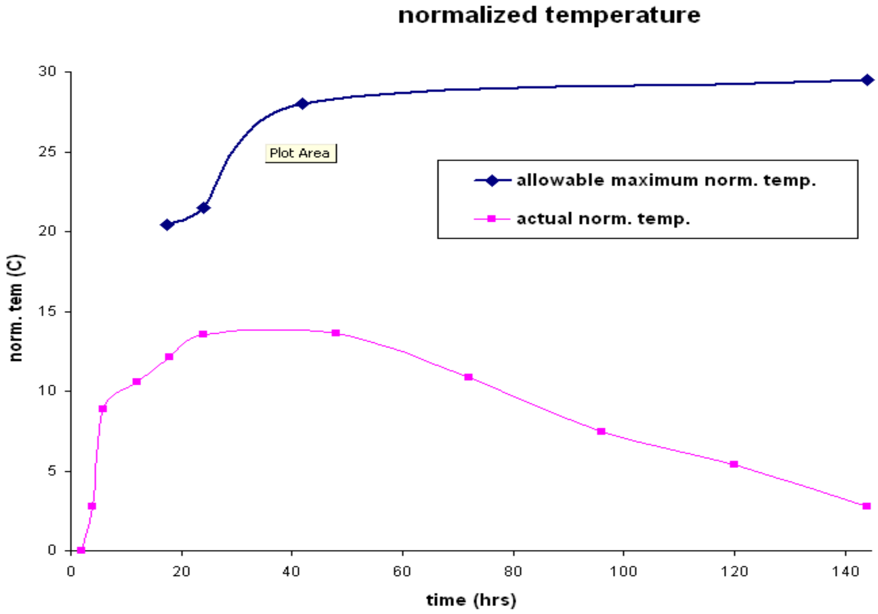



Figure 4.2: *Comparision of Temperature values*

Figure 4.2 shows that allowable maximum tensile capacities are clearly much higher than the actual ones. The allowable maximum temperature differences cannot be determined before 17.5 hrs accurately, because actual strain capacities cannot be obtain through an experiment due to practical difficulties.

***4.2 Discussion***

In this case, to evaluate induced tensile strain values in surface gradient analysis it has used strain values but not stress values. This is because critical induced strain values would not in the region of elasticity but may be in the region of plasticity. Therefore it is impossible to find Modulus of Elasticity value to convert these strain values to stress values.

Tensile strain capacity values found from this experimental part were lower values than the actual values due to following two errors.

*Strain values were not due to the "Pure Bending"*

In four point flexural loading test, the length at which the pure bending occurs is 150mm. But strain meter used here had 300mm length. Therefore strain meter reading is not due to the constant pure bending. Hence, actual strain capacity values were greater than the values obtained from the test.

*Strain meter could not measure curve length*
Since the strain meter was a mechanical one, it is not possible to measure curved lengths

On the other hand, to get more accurate tensile strain capacity values, it is necessary to take measurements through a "Data Logger" by using a calibrated 150mm strain gauge.

When dealing with thermal effect of mass concrete structures, normally maximum temperature difference within the concreteis limited to 20 C. Through a back calculation, it was found that the minimum allowable temperature difference is around 20.5 C at the age of 17.5 hr for rapid load tension test &28.0 C for slow load tension test (modified by 1.4) for this particular concrete block. Furthermore this difference got higher with the increase of the age of concrete. Therefore, from this approach, one can predict the maximum temperature differences exactly at each age for a given concrete section. Placement temperature of the concrete 31 C did not make any effect on thermal stresses according to this analysis. Finally, it can be concluded that maximum lift height of 1.5m can be safely applicable for concreting in the particular project.

## References


1. U.S. Army Corps of Engineers, "Thermal studies of mass concrete structures", Department of the Army, CECW-EG Washington, 1997.
2. U.S. Army Corps of Engineers, "Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)", CRD C 16-94 (ASTM C 78-94), Washington, 1997.
3. Harrison, T. A., "Early age thermal crack control on concrete", CIRIA report 91,Construction Industry Research and Information Association, London, 1981.


## Acknowledgement

We would like to express our sincere gratitude to Prof. S.M.A. Nanayakkara who gave us his kind guidance and supervision to complete this research.

## About the Authors

**W.G.J. PRASANNA,** B.Sc. Hons. Moratuwa.,currently doing his PhD at Victoria University, Australia.

**A.P. SUBHASHINI,** B.Sc. Hons. Moratuwa.,currently doing her MSc at Saitama University, Japan.

# COMBINED HIGH AND LOW CYCLE FATIGUE MODEL FOR PREDICTION OF STEEL BRIDGE LIVES

M. OHGA[1*], K. KARUNANANDA[1], P.B.R. DISSANAYAKE[2] and S.A.S.C. SIRIWARDANE[3]

[1]Department of Civil and Environmental Engineering, Ehime University, Matsuyama, Japan

[2]Department of Civil Engineering, University of Peradeniya, Peradeniya, Sri Lanka

[3]Department of Civil and Environment Engineering, University of Ruhuna, Galle, Sri Lanka

**ABSTRACT**

A new fatigue model is presented to predict life of steel bridges for combined high and low cycle fatigue. It consists of a modified strain-life curve and a new strain based damage index. The damage variable is based on a modified von Mises equivalent strain to account for effects of loading non-proportionality and strain path orientation in multiaxial stress state. The proposed model was verified with experimental test results of two materials, available in the literature. Then, the proposed model was applied to a wrought iron railway bridge to estimate the fatigue life due to usual traffic and earthquake loadings. The obtained results confirm the importance and effectiveness of the proposed model over commonly used Miner's rule based life prediction of steel bridges.

**Keywords:** High cycle fatigue, Low cycle fatigue, Steel bridges, Life prediction, Earthquake loading.

## 1. Introduction

Bridges are generally subjected to high cycle fatigue (HCF) due to low amplitude loading by usual traffic during their service life. However, they may be subjected to low cycle fatigue (LCF) due to high amplitude loadings such as earthquake loadings. The combined damage of HCF and LCF may be a reason for a much reduced life (Kondo and Okuya 2007).

Most of fatigue life estimation of bridges is concentrated on multiaxial high cycle fatigue. There is almost no literature considering the combined damage of HCF and LCF of bridges. In other fields such as aircraft engineering, von Mises equivalent strain and Coffin-Manson strain-life curve are used with the Miner's rule as the general method to estimate the life for combined damage of HCF and LCF (Suresh 1998). However, von Mises equivalent strain cannot capture the effects due to non-proportional loading and orientation of strain path (Borodii and Strizhalo 2000). The Miner's rule is the simplest and the most widely used fatigue life prediction technique. However under many variable amplitude loading conditions, Miner's rule based life predictions have been found to be unreliable since it cannot capture loading sequence effect (Siriwardane et al. 2008).

These reasons raise the question about accuracy of the Miner's rule based life estimation for combined damage of HCF and LCF in bridges which are generally subjected variable amplitude

*Corresponding author, Tel/Fax: +81-89-927-9816, E-mail: ohga@cee.ehime-u.ac.jp

loading and multiaxial stress state. Therefore, it is necessary to have a different model, which is based on commonly available material properties, to estimate more accurately the life for combined damage of HCF and LCF due to variable amplitude loading.

The objective of this paper is to propose a new model to accurately estimate the fatigue life (crack initiation life) when a bridge is subjected to combined damage of HCF and LCF. Initially, the proposed combined HCF and LCF is presented. Then, verification of the model is discussed. Finally, the proposed model is applied to an existing railway bridge to estimate fatigue life.

## 2. Proposed fatigue model

This section proposes the new fatigue model to estimate life of steel structures. Initially, the details relevant to proposed damage variable, modified strain-life fatigue curve are discussed. Then, the proposed damage indicator is explained.

### 2.1. Damage variable

The damage variable for combined HCF and LCF is given as (Borodii and Strizhalo 2000),

$$\varepsilon_{eq} = (1+\alpha\phi)(1+kSin\varphi)\varepsilon_{VM} \tag{1}$$

where $\varepsilon_{eq}$ is the equivalent strain amplitude, $\alpha$ is the material parameter for loading non-proportionality, $\phi$ is the cycle non-proportionality parameter, $k$ is the material parameter for strain path orientation, $\varphi$ is the angle from the principal direction to the applied strain path and $\varepsilon_{VM}$ is the von Mises strain as given,

$$\varepsilon_{VM} = \frac{1}{(\sqrt{2}\times(1+\nu))}\left[(\varepsilon_{xx}-\varepsilon_{yy})^2+(\varepsilon_{yy}-\varepsilon_{zz})^2+(\varepsilon_{zz}-\varepsilon_{xx})^2+\frac{3}{2}\times(\gamma_{xy}^2+\gamma_{yz}^2+\gamma_{zx}^2)\right]^{1/2} \tag{2}$$

where $\nu$ is the Poisson's ratio. $\varepsilon$ and $\gamma$ are the axial and shear strain amplitudes in respective planes.

### 2.2. Strain-life curve

It is necessary to modify the strain-life fatigue curve in HCF regime in order to consider the combined damage of HCF and LCF. The proposed curve consists of two parts as shown in Figure 1. The first part of the curve describes fatigue life of plastic strain cycles which usually affect LCF. To describe this part, Coffin-Manson strain-life curve is utilized as shown below.

$$\varepsilon_{eq} = \frac{\sigma_f^{'}}{E}(2N)^b + \varepsilon_f^{'}(2N)^c \tag{3}$$

where $\varepsilon_{eq}$ is the equivalent strain amplitude, $N$ is the number of cycles to failure, $\sigma_f^{'}$ is the fatigue strength coefficient, $b$ is the fatigue strength exponent, $\varepsilon_f^{'}$ is the fatigue ductility coefficient, $c$ is the fatigue ductility exponent and $E$ is the elastic modulus of the material.

*Corresponding author, Tel/Fax: +81-89-927-9816, E-mail: ohga@cee.ehime-u.ac.jp

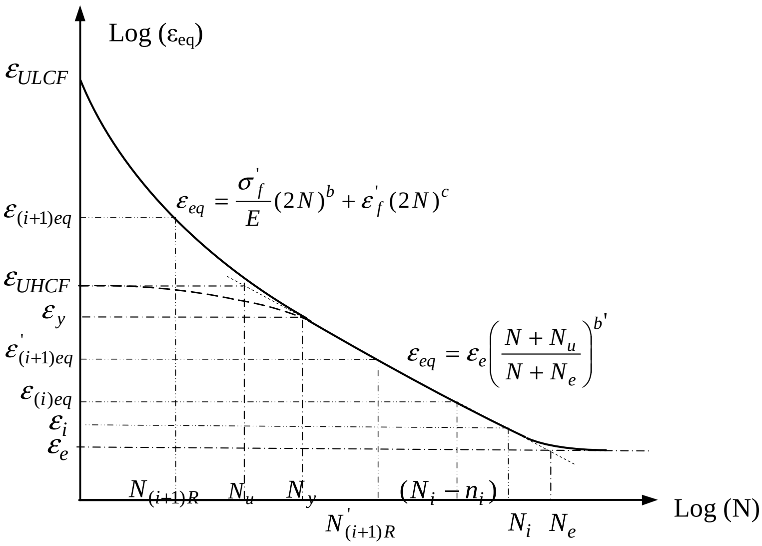


Figure 1: Schematic representation of the proposed strain-life curve

The ultimate strain of low cycle fatigue $(\varepsilon)_{ULCF}$ which is the equivalent strain amplitude corresponding to failure in half reversal (a quarter of a cycle) is obtained from Eq. (3) as,

$$(\varepsilon)_{ULCF} = \varepsilon_f^{'} \tag{4}$$

The second part of the curve describes the fatigue life of elastic strain cycles which usually affects HCF. This part of curve represents hypothetical fully known curve. The shape of the curve is obtained by directly transforming the previous fully known stress-life curve (Siriwardane et al. 2008) to elastic strain-life curve as shown below.

$$\varepsilon_{eq} = \varepsilon_e \left( \frac{N + N_u}{N + N_e} \right)^{b'} \tag{5}$$

where $\varepsilon_e$ is the strain amplitude of the fatigue limit, $N_e$ is the corresponding number of cycles to failure. The $\varepsilon_y$ and $N_y$ are the yield strain and the corresponding number of cycles to failure. The $b'$ is the slope of the finite life region of the curve. The $(\varepsilon)_{UHCF}$ is the ultimate strain of HCF which is the elastic strain amplitude corresponding to half reversal (a quarter of a cycle) is expressed as,

$$(\varepsilon)_{UHCF} = \left( \frac{\sigma_u}{E} \right) \tag{6}$$

where $\sigma_u$ is the ultimate tensile strength of the material. The $N_u$ is the number cycles corresponding to the intersection of the tangent line of the finite life region and the horizontal asymptote of the ultimate elastic strain amplitude $(\varepsilon)_{UHCF}$ as shown in Figure 1.

*Corresponding author, Tel/Fax: +81-89-927-9816, E-mail: ohga@cee.ehime-u.ac.jp

### 2.3. Damage indicator

The proposed damage indicator considers combined damage of HCF and LCF due to variable amplitude loading. Suppose a component is subjected to a certain equivalent strain amplitude $(\varepsilon)_i$ of $n_i$ number of cycles at load level *i*. $N_i$ is the fatigue life (number of cycles to failure) corresponding to $(\varepsilon)_i$ (Figure 1). Therefore, the reduced life at the load level *i* is obtained as $(N_i{-}n_i)$. The damage equivalent strain $(\varepsilon)_{(i)eq}$ (Figure 1), corresponding to the failure life $(N_i{-}n_i)$ is defined as $i^{th}$ level damage equivalent strain. Then, the new damage indicator, $D_i$ is stated as,

$$D_i = \frac{(\varepsilon)_{(i)eq} - (\varepsilon)_i}{(\varepsilon)_u - (\varepsilon)_i} \tag{7}$$

where the $(\varepsilon)_u$ is

$$(\varepsilon)_u = \begin{cases} \varepsilon_{ULCF} & (\varepsilon)_i \geq \varepsilon_y \\ \varepsilon_{UHCF} & (\varepsilon)_i < \varepsilon_y \end{cases} \tag{8}$$

At the end of $i^{th}$ loading level $(\varepsilon)_{i+1}$, damage $D_i$ has been accumulated (occurred) due to the effect of loading cycles, the damage is transformed to load level *i+1* as below.

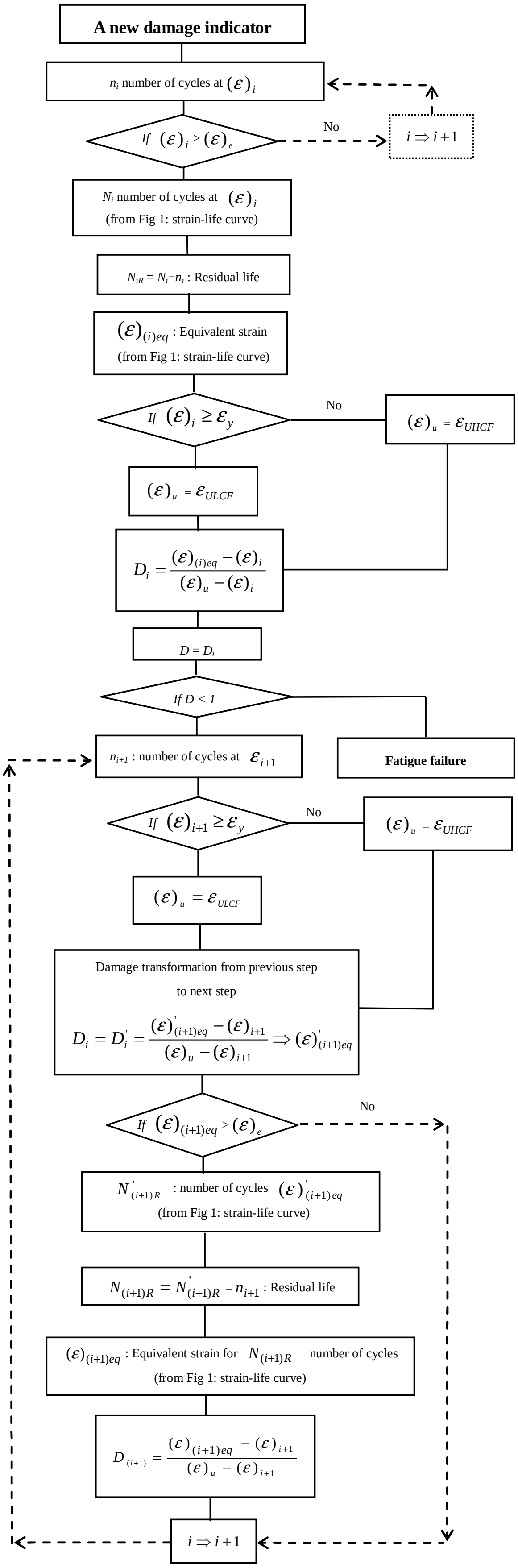
$$D_i = \frac{(\varepsilon)'_{(i+1)eq} - (\varepsilon)_{i+1}}{(\varepsilon)_u - (\varepsilon)_{i+1}} \tag{9}$$

and $(\varepsilon)_u$ is expressed,

$$(\varepsilon)_u = \begin{cases} \varepsilon_{ULCF} & (\varepsilon)_{i+1} \geq \varepsilon_y \\ \varepsilon_{UHCF} & (\varepsilon)_{i+1} < \varepsilon_y \end{cases} \tag{10}$$

Then, $(\varepsilon)'_{(i+1)eq}$ is the damage equivalent strain at loading level *i+1* and it is calculated as,

$$(\varepsilon)'_{(i+1)eq} = D_i[(\varepsilon)_u - (\varepsilon)_{i+1}] + (\varepsilon)_{i+1} \tag{11}$$




The corresponding equivalent number of cycles to failure $N'_{(i+1)R}$ is obtained from the strain-life curve as

*Corresponding author, Tel/Fax: +81-89-927-9816, E-mail: ohga@cee.ehime-u.ac.jp

shown in Figure 1. The $(\varepsilon)_{i+1}$ is the strain at the level *i+1* and supposing that it is subjected to $n_{(i+1)}$ number of cycles, then the corresponding residual life at load level *i+1*, $N_{(i+1)R}$ is calculated as,

$$N_{(i+1)R} = N'_{(i+1)R} - n_{(i+1)}$$
(12)

Therefore, strain, $(\varepsilon)_{(i+1)eq}$ which corresponds to $N_{(I+1)R}$ at load level *i+1*, is obtained from the strain-life curve as shown in Figure 1. Then the cumulative damage at the end of load level *i+1* is defined as,

$$D_{(i+1)} = \frac{(\varepsilon)_{(i+1)eq} - (\varepsilon)_{i+1}}{(\varepsilon)_u - (\varepsilon)_{i+1}}$$
(13)

This is carried out until $D_i$ is equal to 1. Flow chart of the damage indicator is given Figure 2.

## 3. Verification of the proposed model

This section explains the verification of the proposed fatigue model by comparing fatigue test results available in the literature. Two experimental test results were used: S45C steel and Haynes 188.

### 3.1. Verification for S45C Steel

Fatigue tests performed by Chen et al. 2006 were used to verify the proposed fatigue model. Axial (A) and torsional (T) testing were performed in HCF and LCF regimes. The parameter, *k*, is estimated as 0.15 from the constant amplitude fatigue tests (Kim et al 1999). Then, fatigue lives of the proposed model and Miner's rule based previous model are estimated as given Table 1.

Table 1 Experimental summary and predicted fatigue lives of S45C steel

| Test | First load level | | Second load level | | Experimental life ($n_1$+$n_2$) | Predicted life | |
|---|---|---|---|---|---|---|---|
| | Strain amplitude | No of cycles ($n_1$) | Strain amplitude | No of cycles ($n_2$) | | Previous model | Proposed model |
| AT4 | 0.0047 | 1250 | 0.0035 | 119638 | 120888 | 202891 | 117808 |
| AT5 | 0.0047 | 2500 | 0.0035 | 116826 | 119326 | 184654 | 75357 |
| AT6 | 0.0019 | 25000 | 0.0097 | 4547 | 29547 | 35112 | 36241 |
| AT7 | 0.0019 | 50000 | 0.0097 | 6411 | 56411 | 58280 | 60547 |
| AT8 | 0.0019 | 75000 | 0.0097 | 6019 | 81019 | 81418 | 82633 |
| TA1 | 0.0105 | 1250 | 0.0022 | 44879 | 46129 | 97492 | 57322 |
| TA2 | 0.0105 | 2500 | 0.0022 | 35756 | 38256 | 84697 | 47253 |
| TA3 | 0.0105 | 3750 | 0.0022 | 21598 | 25348 | 71901 | 34153 |
| TA4 | 0.00495 | 25000 | 0.00464 | 4091 | 29091 | 33598 | 34346 |
| TA5 | 0.00495 | 50000 | 0.00464 | 3281 | 53281 | 56284 | 61352 |
| TA6 | 0.00495 | 75000 | 0.00464 | 2327 | 77327 | 75142 | 77950 |

*Corresponding author, Tel/Fax: +81-89-927-9816, E-mail: ohga@cee.ehime-u.ac.jp

The percentage variations of the predictions are determined with the experimental results. The previous model gives a percentage variation of 23.9 % while the proposed model gives a value of 6.2 %. Therefore, the proposed model based fatigue lives are more accurate than previous model predictions.

### 3.2. Verification for Haynes 188

Fatigue test performed by Kalluri and Bonacuse 2002 were used verify the proposed fatigue model. Axial (A) and torsional (T) testings have been performed in different sequences (AA, AT, TT and TA). The parameter, $k$ was estimated as 0.17 from constant amplitude tests given (Kalluri and Bonacuse 1999). Experimental results were compared with the predicted lives of the proposed fatigue model. In addition, the previous model used with the Miner's rule was also used in this case. The obtained comparisons are given in Table 2.

Table 2 Experimental summary and predicted fatigue lives of Haynes 188

| Test | First load level | | Second load level | | Experimental life ($n_1$+$n_2$) | Predicted life | |
|---|---|---|---|---|---|---|---|
| | Strain amplitude | No of cycles ($n_1$) | Strain amplitude | No of cycles ($n_2$) | | Previous model | Proposed method |
| AA1 | 0.0033 | 3926 | 0.0101 | 789 | 4715 | 4365 | 4413 |
| AA2 | 0.0033 | 7851 | 0.0101 | 758 | 8609 | 8249 | 8337 |
| AA3 | 0.0033 | 15702 | 0.0101 | 659 | 16361 | 15977 | 16147 |
| AA4 | 0.0033 | 23553 | 0.0102 | 815 | 24368 | 23709 | 23931 |
| TT1 | 0.0060 | 5857 | 0.0173 | 1250 | 7107 | 7276 | 7414 |
| TT2 | 0.0060 | 11714 | 0.0175 | 1100 | 12814 | 12923 | 13189 |
| TT3 | 0.0060 | 23427 | 0.0173 | 1343 | 24270 | 24316 | 24832 |
| TT4 | 0.0059 | 35141 | 0.0173 | 1467 | 36608 | 35677 | 36219 |
| TT5 | 0.0060 | 40998 | 0.0175 | 1294 | 42292 | 41348 | 41812 |
| AT1 | 0.0035 | 3926 | 0.0174 | 1189 | 5115 | 5084 | 5345 |
| AT2 | 0.0035 | 7851 | 0.0173 | 1218 | 9069 | 8660 | 9093 |
| AT3 | 0.0033 | 15702 | 0.0172 | 930 | 16632 | 16058 | 16600 |
| AT4 | 0.0033 | 23553 | 0.0173 | 1253 | 24806 | 23885 | 24185 |
| TA1 | 0.0061 | 5857 | 0.0101 | 560 | 6417 | 6316 | 6367 |
| TA2 | 0.0060 | 11714 | 0.0101 | 494 | 12208 | 12133 | 12216 |
| TA3 | 0.0059 | 23427 | 0.0100 | 459 | 23886 | 23740 | 23907 |
| TA4 | 0.0059 | 35141 | 0.0102 | 427 | 35568 | 35322 | 35588 |

The percentage variations of previous model predictions with experimental results were estimated as 0.74 % while the proposed model has a percentage variation of 0.62 %. Therefore, the predicted fatigue lives by the proposed fatigue model are more accurate than previous model predictions.

*Corresponding author, Tel/Fax: +81-89-927-9816, E-mail: ohga@cee.ehime-u.ac.jp

## 4. Case study: fatigue life estimation of a bridge member

The proposed model was applied to a wrought iron railway bridge member to estimate the fatigue life due to traffic and earthquake loadings. The selected bridge is situated near Colombo in Sri Lanka and one of its members was selected for life estimation. The evaluations are especially based on secondary stresses and strains, which are generated around the riveted connection of the member due to stress concentration effect of primary stresses caused by usual traffic and earthquake loadings. The selected member is shown in Figure 3 (a) and (b).

Considered member

(a) (b)

Figure 3: Views of (a) the bridge; (b) considered member

The combined damage of HCF and LCF is evaluated considering all six rivets are active while all the riveted locations have no clamping force. The clamping force is generally defined as the compressive force in the plates which is induced by the residual tensile force in the rivet. Since this study assumes that the riveted locations have no clamping force (value of clamping force is zero), the connected members are considered to subject to the biaxial stress state. Therefore, a critical member without rivets can be considered to analyze the biaxial state of stress of a 2D finite element analysis. The nine node isoperimetric shell elements were used for the FE analysis.

Earthquake was considered to occur at different times in the bridge life as shown in Table 3. It is assumed that usual traffic load is followed after the earthquake. The fatigue life of the member was estimated using approaches: (1) proposed model; (2) previous model (Coffin-Manson curve with the Miner’s rule). The obtained results are given in Table 3. The results indicate that combined damage of HCF and LCF causes an appreciable reduction of bridge life. For the proposed model, percentage reduction of life is the highest when the earthquake occurs at 50 years. If the earthquake amplitude is increased, the maximum percentage reduction occurs before 50 years. For the previous model, the reduction of service life is constant irrespective of time of earthquake occurrence since Miner’s rule cannot capture the loading sequence effect. Comparison of fatigue life reveals that the proposed model predictions differ from the previous model predictions. This verifies that the proposed strain-life curve with new damage indicator better represent the combined HCF and LCF behaviour than Coffin-Manson relationship with Miner’s rule.

Table 3 Fatigue life of the member for different earthquake occurrences

*Corresponding author, Tel/Fax: +81-89-927-9816, E-mail: ohga@cee.ehime-u.ac.jp

| Time of earthquake* (years) | Previous model (Miner's rule) | | Proposed model | |
|---|---|---|---|---|
| | Fatigue life (years) | Percentage reduction of life (%) | Fatigue life (years) | Percentage reduction of life (%) |
| 10 | 127.7 | 5.0 | 130.9 | 19.6 |
| 50 | 127.7 | 5.0 | 109.6 | 32.7 |
| 75 | 127.7 | 5.0 | 116.3 | 28.6 |
| 100 | 127.7 | 5.0 | 130.5 | 19.9 |
| No earthquake | 134.5 | | 162.8 | |

*After construction

The differences of case study results confirm the importance of accurate combined HCF and LCF model to estimate the fatigue life of existing steel bridges.

## 5. Conclusions

A new model for combined damage of HCF and LCF was proposed to estimate life of steel bridge. A verification of the proposed model was conducted by comparing the predicted lives with experimental lives of two materials. It was shown that the proposed fatigue model gives an accurate fatigue life for combined damage of HCF and LCF where detailed stress histories are known. The proposed fatigue model was applied to estimate the fatigue life of a wrought iron railway bridge Case study realized the importance of consideration of the earthquake induced LCF damage in addition to HCF damage due to usual traffic loading in steel bridges. The importance and effectiveness of accurate prediction of combined damage of HCF and LCF was also confirmed.

### References

Bonacuse P and Kalluri S (2002). Sequenced axial and torsional cumulative fatigue: low amplitude followed by high amplitude loading, Biaxial/Multiaxial Fatigue and Fracture, Elsevier.

Borodii MV and Strizhalo VA (2000). Analysis of experimental data on a low cycle fatigue under nonproportional strain hardening, International Journal of Fatigue, 22(4), pp. 275-282.

Chen X, Jin D and Kim KS (2006). Fatigue damage of medium carbon steel under sequential application of axial and torsional loading, Fatigue and Fracture of Engineering Materials and Technology, 29(3), pp. 191-199.

Kalluri S and Bonacuse PJ (1999). Cumulative axial and torsional fatigue: an investigation of load-type sequencing effects, Symposium of multiaxial fatigue and deformation: testing and prediction, Seattle, USA.

Kim KS, Park JC and Lee JW (1999). Multiaxial fatigue under variable amplitude loads, Journal of Engineering Materials and Technology, 121(3), pp. 286-293.

Kondo Y and Okuya K (2007). The effect of seismic loading on the fatigue strength of welded joints. Material Science and Engineering A. 468-470, pp. 223-229.

Siriwardane S, Ohga M, Dissanayake R and Taniwaki K (2008). Application of new damage indicator-based sequential law for remaining fatigue life estimation of railway bridges. Journal of Constructional Steel Research. 64(2), pp. 228-237.

Suresh S (1998). Fatigue of Materials, UK: Cambridge University Press, Cambridge.

*Corresponding author, Tel/Fax: +81-89-927-9816, E-mail: ohga@cee.ehime-u.ac.jp

# FATIGUE LIFE PREDICTION OF BRIDGES CONSIDERING THE EFFECT OF MULTIAXIAL STRESSES

M. OHGA[1*], K. KARUNANANDA[1], P.B.R. DISSANAYAKE[2] and A.M.N.D. ADASOORIYA[1]

[1]Department of Civil and Environmental Engineering, Ehime University, Japan

[2]Department of Civil Engineering, University of Peradeniya, Sri Lanka

**ABSTRACT**

This paper presents a new low cycle fatigue model to predict life of steel bridges. It consists of Coffin-Manson strain-life curve with a new strain based damage index. The damage variable is based on a modified von Mises equivalent strain to account for effects of loading non-proportionality and strain path orientation in low cycle multiaxial stress state. The proposed model was verified by comparing with experimental test results of two materials. Then, it was applied an existing riveted wrought iron railway bridge to estimate fatigue life due to usual traffic and earthquake loadings. The obtained results verify the importance and effectiveness of the proposed model over commonly used Miner's rule model in fatigue life estimation of steel bridges.

**Keywords:** High cycle fatigue, Low cycle fatigue, Steel bridges, Life prediction, Earthquake loading.

## 5. Introduction

High cycle fatigue (HCF) caused by low amplitude traffic loading is one of the main safety considerations of steel bridges. In addition, there are certain situations that a bridge may be subjected to high amplitude loading such as earthquake or unexpected stress concentrations during its service life. When such an event occurs, some members may undergo inelastic stresses. These inelastic stresses may cause low cycle fatigue (LCF) damage during the high amplitude loading while subjecting to HCF in service conditions. This combined damage of HCF and LCF may be a reason for a much reduced life (Kondo and Okuya 2007).

The von Mises equivalent strain and Coffin-Manson strain-life curve are used with Miner's rule as the general method to estimate the life for LCF conditions (Suresh 1998). The Miner's rule is the simplest and the most widely used fatigue life prediction technique. One of its interesting features is that life calculation is simple and reliable when the detailed loading history is unknown. However under many variable amplitude loading conditions, Miner's rule based life predictions have been found to be unreliable since it cannot capture loading sequence effect (Siriwardane et al. 2008). Further, von Mises equivalent strain cannot capture the effects due to non-proportional loading and orientation of strain path which are the key features of multiaxial LCF stress state (Borodii and Strizhalo 2000). von Mises strain generally predicts a lesser strain value than the actual strain of the material that undergoes. Due to these reasons, LCF life estimation by Miner's rule based model may be inaccurate in multiaxial variable amplitude loading. Therefore, it is

[*]Corresponding author, Tel/Fax: +81-89-927-9816, E-mail: ohga@cee.ehime-u.ac.jp

necessary to have a different model, which is based on commonly available material properties, to estimate more accurately the life for LCF due to variable amplitude loading.

The objective of this paper is to propose a new model to accurately estimate the LCF life (crack initiation life) due to a high amplitude loading. Initially, the proposed model is presented and then the verification of the proposed model is discussed. Finally, the proposed model is applied to an existing wrought iron railway bridge to estimate fatigue life.

## 6. Proposed fatigue model

This section proposes the new low cycle fatigue model to estimate life of steel structures. Initially, the details relevant to proposed damage variable, Coffin-Manson strain-life fatigue curve are discussed. Finally, it clearly describes the proposed damage indicator.

### 6.1. Damage variable

The proposed damage variable for low cycle multiaxial stress state is given as (Borodii and Strizhalo 2000),

$$\varepsilon_{eq} = (1+\alpha\phi)(1+kSin\varphi)\varepsilon_{VM} \tag{1}$$

where $\varepsilon_{eq}$ is the equivalent strain amplitude in multiaxial stress state, $\alpha$ is the material parameter for loading non-proportionality, $\phi$ is the cycle non-proportionality parameter, $k$ is the material parameter for strain path orientation, $\varphi$ is the angle measured from the principal direction to the applied strain path and $\varepsilon_{VM}$ is the von Mises equivalent strain as given,

$$\varepsilon_{VM} = \frac{1}{(\sqrt{2}\times(1+\nu))}\left[(\varepsilon_{xx}-\varepsilon_{yy})^2+(\varepsilon_{yy}-\varepsilon_{zz})^2+(\varepsilon_{zz}-\varepsilon_{xx})^2+\frac{3}{2}\times(\gamma_{xy}^2+\gamma_{yz}^2+\gamma_{zx}^2)\right]^{1/2} \tag{2}$$

where $\nu$ is the Poisson's ratio. $\varepsilon$ and $\gamma$ are the axial and shear strain amplitudes in respective planes.

The first expression in parentheses of Eq. (1) is the degree of additional strain hardening depending on the cycle geometry (to account for non-proportional loading). The second expression in parentheses is strain hardening depending on the orientation of the cyclic strain path (proportional loading). The material parameters ($\alpha$ and $k$) have to be estimated by additional testing of the material. $\phi$ and $\varphi$ can be estimated for given strain path considering cycle geometry and its orientation, respectively (Borodii and Strizhalo 2000).

The parameter, $\varphi$, is estimated by the orientation of the applied strain path (measured angle) with respect to the principal direction. The principal direction of a material is the direction that gives the highest live and usually it is the torsion axis for most of materials. However, this parameter does not represent the characteristics of material and presented by the parameter, *k*. The parameter, *k*, is estimated by at least three fatigue tests. In fact, the parameters, *k* and $\varphi$, collectively represent the effect of proportional loading.

The parameter, $\phi$, is estimated from the ratio of areas of a given non-proportional cycle path to a circular cycle path. As this parameter is related to cycle geometry, a different parameter is necessary to represent material characteristics. It is represented by the parameter, $\alpha$**,** and three fatigue tests are necessary to estimate the parameter, $\alpha$. These two parameters $(\alpha,\phi)$ collectively represent the effect of non-proportional loading.

### 6.2. Strain-life curve

The strain-life curve used in this study is the Coffin-Manson relationship as given,

$$\varepsilon_{eq} = \frac{\sigma_f^{'}}{E}(2N)^b + \varepsilon_f^{'}(2N)^c \tag{3}$$

where $\varepsilon_{eq}$ is the equivalent strain amplitude in multiaxial stress state, $N$ is the number of cycles to failure, $\sigma_f^{'}$ is the fatigue strength coefficient, $b$ is the fatigue strength exponent, $\varepsilon_f^{'}$ is the fatigue ductility coefficient, $c$ is the fatigue ductility exponent and $E$ is the elastic modulus of the material.

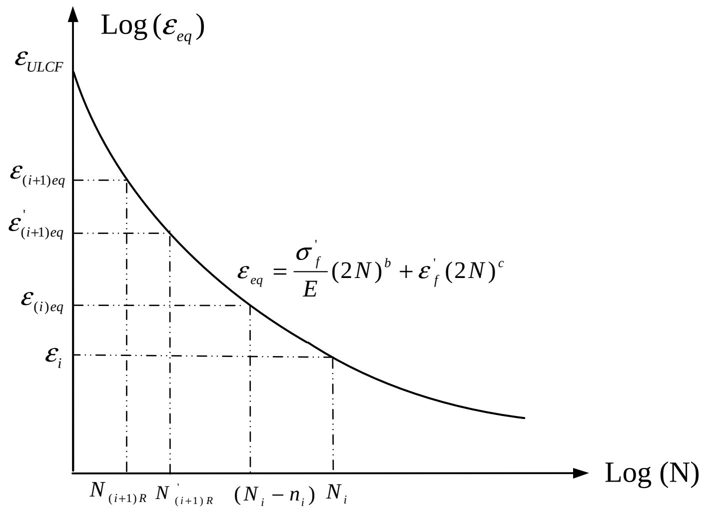



Figure 1: Schematic representation of the Coffin-Manson strain-life curve

The ultimate strain of low cycle fatigue $(\varepsilon)_{ULCF}$ which is the strain amplitude corresponding to failure in half reversal (a quarter of a cycle) is obtained from Eq. (3) as,

$$(\varepsilon)_{ULCF} = \varepsilon_f^{'} \tag{4}$$

Most of pure metals and alloys, fatigue properties are available in the literature and therefore corresponding Coffin-Manson strain-life curve can be obtained easily.

### 6.3. Damage indicator

The proposed damage indicator considers damage of LCF due to variable amplitude loading. Consider, a component is subjected to a certain equivalent strain amplitude of $(\varepsilon)_i$, $n_i$ number of cycles at load level $i$, $N_i$ is the fatigue life (number of cycles to failure) corresponding to $(\varepsilon)_i$ (Figure 1). Therefore, the reduced life at the load level $i$ is obtained as $(N_i-n_i)$. The damage equivalent strain $(\varepsilon)_{(i)eq}$ (Figure 1), corresponding to the failure life $(N_i-n_i)$ is defined as $i^{th}$ level damage equivalent strain. Then, the new damage indicator, $D_i$ is stated as,

$$D_i = \frac{(\varepsilon)_{(i)eq} - (\varepsilon)_i}{(\varepsilon)_u - (\varepsilon)_i}$$
(5)

where the $(\varepsilon)_u$ is given in Eq. (4)

At the end of $i^{th}$ loading level, damage $D_i$ has been accumulated (occurred) due to the effect of $(\varepsilon)_{i+1}$ loading cycles, the damage (same damage given in Eq. 5) is transformed to load level *i+1* as below.

$$D_i = \frac{(\varepsilon)'_{(i+1)eq} - (\varepsilon)_{i+1}}{(\varepsilon)_u - (\varepsilon)_{i+1}} \quad (6)$$

Then, $(\varepsilon)'_{(i+1)eq}$ is the damage equivalent strain at loading level *i+1* and it is calculated from Eq. (6) as,

$$(\varepsilon)'_{(i+1)eq} = D_i[(\varepsilon)_u - (\varepsilon)_{i+1}] + (\varepsilon)_{i+1} \quad (7)$$

The corresponding equivalent number of cycles to failure $N'_{(i+1)R}$ is obtained from the strain-life curve as shown in Figure 1. The $(\varepsilon)_{i+1}$ is the strain at the level *i+1* and supposing that it is subjected to $n_{(i+1)}$ number of cycles, then the corresponding residual life at load level *i+1*, $N_{(i+1)R}$ is calculated as,

$$N_{(i+1)R} = N'_{(i+1)R} - n_{(i+1)} \quad (8)$$

Therefore, strain, $(\varepsilon)_{(i+1)eq}$ which corresponds to $N_{(i+1)R}$ at load level *i+1*, is obtained from the strain-life curve as shown in Figure 1. Then the cumulative damage at the end of load level *i+1* is defined as,

$$D_{(i+1)} = \frac{(\varepsilon)_{(i+1)eq} - (\varepsilon)_{i+1}}{(\varepsilon)_u - (\varepsilon)_{i+1}}$$
(9)

This procedure is carried out until $D_i$ is equal to 1. The proposed damage indicator calculation is shown in the flow chart given in Figure 2.

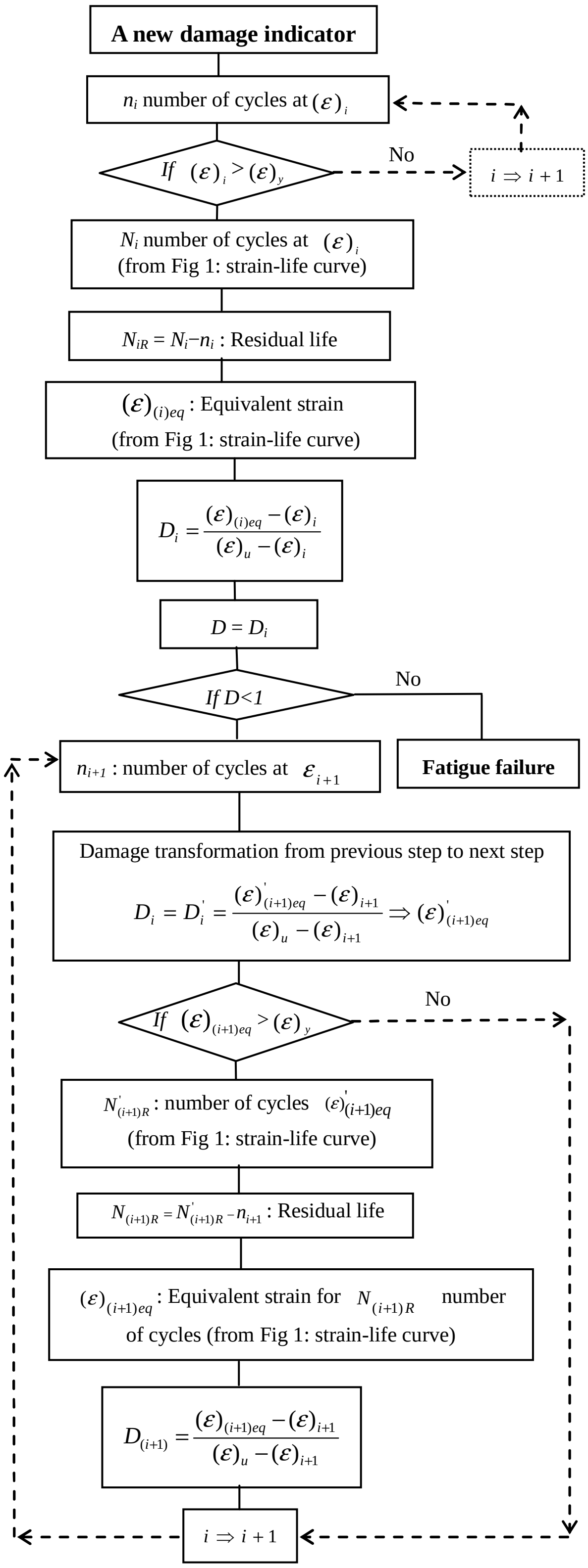



Figure 2: Flow chart of the proposed damage indicator

## 7. Verification of the proposed model

This section explains the verification of the proposed LCF model by comparing experimental fatigue test results of two materials which were obtained from the literature. Two materials are pure titanium and S304 stainless steel. During these tests, axial (A), torsional (T), in-phase (I) and 90$^o$ -out- of-phase (O) loadings were used in different sequences. Strain variations of strain-controlled fully reversed axial, torsional, in- phase and out-of-phase loadings are shown in Figure 3.

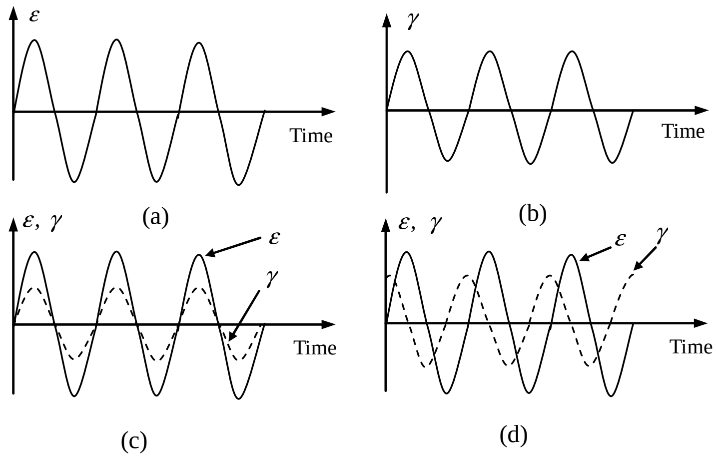



Figure 3: Strain variations for (a) axial loading; (b) torsional loading; (c) in-phase loading; (d) 90$^o$ -out-of-phase loading

### 7.1. Verification for Pure Titanium

Block loading fatigue tests performed by Shamsaei et al. 2010 were used verify the proposed fatigue model. In the block loading test, axial (A), torsional (T), 90$^o$ -out-of-phase (O) loadings were applied in different combinations as shown in Table 1. Applied wave forms were sinusoidal as shown in Figure 3.

Table 1 Experimental summary and predicted fatigue lives of pure Titanium

| Test | First load level | | Second load level | | Experimental life (cycles) | Predicted life (cycles) | |
|---|---|---|---|---|---|---|---|
| | von Mises Strain amplitude | No of cycles ($n_1$) | von Mises Strain amplitude | No of cycles ($n_2$) | | Previous model | Proposed model |
| AA1 | 0.0070 | 491 | 0.0110 | 214 | 705 | 808 | 873 |
| AA2 | 0.0110 | 104 | 0.0070 | 302 | 406 | 1399 | 1073 |
| AA3 | 0.0110 | 200 | 0.0070 | 186 | 386 | 1133 | 743 |
| TT1 | 0.0073 | 1115 | 0.0113 | 242 | 1357 | 1270 | 1373 |
| TT2 | 0.0113 | 198 | 0.0073 | 805 | 1003 | 1155 | 770 |
| AT | 0.0090 | 228 | 0.0093 | 397 | 625 | 766 | 761 |
| TA | 0.0093 | 434 | 0.0090 | 375 | 809 | 763 | 767 |
| AO | 0.0090 | 228 | 0.0112 | 235 | 463 | 497 | 530 |
| OA | 0.0112 | 138 | 0.0090 | 155 | 293 | 620 | 536 |
| OT | 0.0112 | 138 | 0.0093 | 467 | 605 | 635 | 544 |
| TO | 0.0093 | 428 | 0.0112 | 520 | 683 | 600 | 648 |
| TAOTOA | Strain amplitudes = 0.0073, 0.0070, 0.0088, 0.0073, 0.0088, 0.0070 | | Each loading mode with number of cycles = 50 | | 1050 (3.5 blocks) | 1160 | 1158 |

Further, authors (Shamsaei et al. 2010) have published constant amplitude fatigue test results. From that, parameters, $k$ and $\alpha$ are estimated as 0.04 and 0.08, respectively. Fatigue lives were predicted using the proposed and previous models as given in Table 1.

Percentage variations of predictions from experimental results were estimated for previous and proposed models. The previous model has a percentage variation of 27.7 % while the proposed model has a value of 17.7 %. Therefore, the proposed model based fatigue lives are more accurate than previous model predictions for the pure titanium.

### 7.2. Verification for S304 steel

Fatigue tests performed by Chen et al. 2006 were used verify the proposed fatigue model. Axial (A) torsional (T), in-phase (I) and $90^{o}$-out-of-phase (O) loadings have been applied in different sequences. Applied wave forms of axial and torsional loadings were triangular and in-phase and $90^{o}$ out-of phase loadings were sinusoidal as shown in Figure 3. Parameters, $k$ and $\alpha$, were obtained as 0.20 and 0.80, respectively (Borodii 2007). Fatigue lives were predicted using the proposed and previous models as given in Table 2.

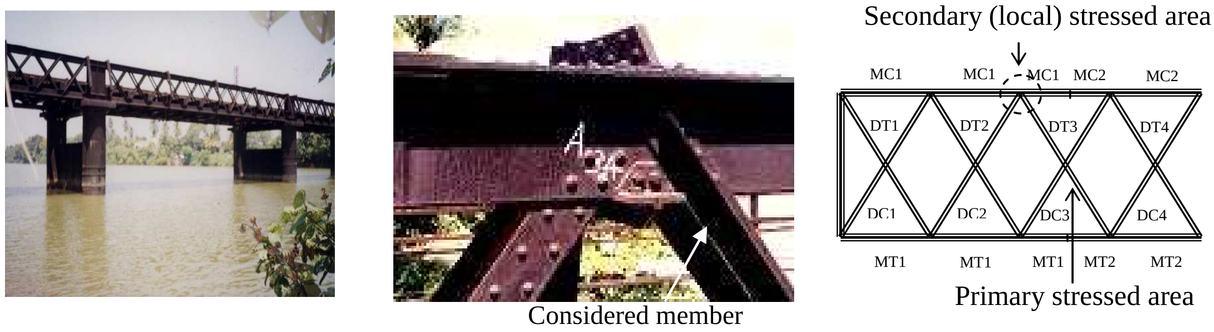
Table 2 Experimental summary and predicted fatigue lives of S304 stainless steel

| Test | First load level | | Second load level | | Experimental life (cycles) | Predicted life (cycles) | |
|---|---|---|---|---|---|---|---|
| | von Mises strain amplitude | No of cycles ($n_1$) | von Mises strain amplitude | No of cycles ($n_2$) | | Previous model | Proposed method |
| AT1 | 0.006 | 973 | 0.006 | 2994 | 3967 | 5321 | 4891 |
| AT2 | 0.006 | 1946 | 0.006 | 981 | 2927 | 4474 | 3998 |
| IO1 | 0.0057 | 1228 | 0.0057 | 1053 | 2281 | 2518 | 2578 |
| IO2 | 0.0057 | 1965 | 0.0057 | 1225 | 3190 | 3122 | 3209 |
| IO3 | 0.0057 | 2456 | 0.0057 | 687 | 3143 | 3525 | 3629 |
| IO4 | 0.0057 | 3685 | 0.0057 | 549 | 4234 | 4532 | 4671 |
| OI1 | 0.0057 | 364 | 0.0057 | 3572 | 3936 | 6726 | 5345 |
| OI2 | 0.0057 | 583 | 0.0057 | 2574 | 3157 | 5732 | 4139 |
| OI3 | 0.0057 | 728 | 0.0057 | 2481 | 3209 | 5073 | 3448 |
| OI4 | 0.0057 | 1093 | 0.0057 | 2165 | 3258 | 3416 | 2157 |
| TA1 | 0.006 | 1559 | 0.006 | 1310 | 2869 | 4022 | 4052 |
| TA2 | 0.006 | 3117 | 0.006 | 825 | 3942 | 4748 | 4821 |
| TA3 | 0.006 | 4676 | 0.006 | 368 | 5044 | 5474 | 5505 |

The percentage variations of predictions from the experimental results were estimated for the previous and proposed models as 11.3 and 6.9 %, respectively. Therefore, the proposed model based predicted fatigue lives are more accurate than previous model predictions for S304.

## 8. Case study: fatigue life estimation of a bridge member

The proposed method was applied to find the fatigue life of a wrought iron railway bridge member. The selected bridge (Figure 4a) is one of the longest railway bridges in Sri Lanka located near Colombo and the considered member is shown in Figure 4b (Siriwardane et al. 2008). The evaluations are especially based on secondary stresses and strains, which are generated around the riveted connection of the member due to stress concentration effect of primary stresses caused by usual traffic and earthquake loadings. Schematic representation of primary and secondary stress areas of the considered member is given in Figure 4(c).




(a) (b) (c)

Fig. 4. Views of (a) the bridge; (b) considered member; (c) schematic representation of the critical member and related areas for primary and local stresses

The damage due to LCF is evaluated based on the state of strain when all rivets are active (tight rivets) while they have no clamping force. The clamping force is generally defined as the compressive force in the plates which is induced by the residual tensile force in the rivet. Since this study assumes that the riveted locations have no clamping force (value of clamping force is zero), the connected members are considered to subject to the biaxial stress state. Therefore, a critical member without rivets can be considered to analyze the biaxial state of stress of a 2D finite element analysis. The nine node isoperimetric shell elements were used for the FE analysis.

Earthquake is considered to occur at different times (10, 50, 75 and 100 years) of the bridge life. It is assumed that usual traffic load is followed after the earthquake. The fatigue damages due to earthquake and usual traffic loadings were estimated using the proposed model and the HCF model given in Siriwardane et al. (2008), respectively. Obtained fatigue lives are given in Table 3 (column 4). In addition, the previous model (Coffin-Manson curve with the Miner's rule) was also used in life estimation and the corresponding results are given in Table 3 (column 2).

Table 3 Fatigue life of the member for different earthquake occurrences

| Time of earthquake* (years) | Previous model (Miner's rule) | | Proposed model | |
|---|---|---|---|---|
| | Fatigue life (years) | Percentage reduction of life (%) | Fatigue life (years) | Percentage reduction of life (%) |
| 10 | 127.7 | 5.0 | 148.5 | 8.8 |
| 50 | 127.7 | 5.0 | 120.5 | 26.0 |
| 75 | 127.7 | 5.0 | 145.1 | 10.9 |
| 100 | 127.7 | 5.0 | 159.9 | 1.8 |
| No earthquake | 134.5 | - | 162.8 | - |

*After construction

The results indicate that LCF damage by earthquake loading causes a considerable reduction of bridge life. For the proposed model, percentage reduction of life is higher when the earthquake occurs at the 50 years compared to those occurring in other times. The relative amplitude difference between traffic and earthquake loadings determines the year at which maximum fatigue life is reduced. For the previous model, the reduction of service life is constant irrespective of time of earthquake occurrence. Comparison of fatigue life reveals that the proposed model predictions differ from the previous model predictions.

The obtained results verifies that the Coffin-Manson strain-life curve with new damage indicator better represents LCF damage than the Coffin-Manson relationship with Miner's rule. The differences of case study results confirm the importance of accurate LCF model to estimate the fatigue life of existing steel bridges.

## 9. Conclusions

A LCF model was proposed to predict the fatigue life of bridges due to high amplitude loading. A verification of the model was conducted by comparing the predicted lives with experimental lives of two materials. It was shown that the proposed fatigue model gives a more accurate fatigue life for damage of LCF situations where detailed stress histories are known. The proposed fatigue model was utilized to estimate the fatigue life of a bridge member. Case study realized the importance and effectiveness of considering the earthquake induced LCF damage in addition to HCF damage due to usual traffic loading in steel bridges.

## 6. Acknowledgement

The authors wish to express their sincere gratitude to Professor M.P Ranaweera and the team of experts who worked in the Sri Lankan Railway Bridge project, for their great advice in carrying out this research.

## References

Borodii MV (2007). Life calculation for materials under irregular nonproportional loading, Strength of Materials, 39(5), pp. 560-565.

Borodii MV and Strizhalo VA (2000). Analysis of experimental data on a low cycle fatigue under nonproportional strain hardening, International Journal of Fatigue, 22(4), pp. 275-282.

Chen X, Jin D and Kim KS (2006). Fatigue life prediction of type 304 stainless steel under sequential biaxial loading, International Journal of Fatigue, 28(3), pp. 289-299.

Kondo Y and Okuya K (2007). The effect of seismic loading on the fatigue strength of welded joints. Material Science and Engineering A. 468-470. pp. 223-229.

Shamsaei, N, Galdskyi M, Panasovskyi K, Shukaev S and Fatemi A (2010). Multiaxial fatigue of Titanium including step loading and load path alteration and sequence effects, International Journal of Fatigue, 32(11), pp. 1862-1874.

Siriwardane S, Ohga M, Dissanayake R and Taniwaki K (2008). Application of new damage indicator-based sequential law for remaining fatigue life estimation of railway bridges. Journal of Constructional Steel Research. 64(2). pp. 228-237.

Suresh S (1998). Fatigue of Materials, UK: Cambridge University Press, Cambridge.

# EVALUATION OF TENSILE STRENGTH DETERIORATION OF STEEL BRIDGE PLATES DUE TO CORROSION

Ohga M[1*], Appuhamy J.M.R.S[1], Kaita T[2], Fujii K[3] and Dissanayake P.B.R[4]

[1]Department of Civil & Environmental Engineering, Ehime University, Japan

[2]Department of Civil Engineering & Architecture, Tokuyama College of Technology, Japan

[3]Department of Social & Environmental Engineering, Hiroshima University, Japan

[4]Department of Civil Engineering, University of Peradeniya, Sri Lanka

**Abstract**

Over the past decades there have been many damage examples of older steel bridge structures due to corrosion around the world. Even though there are some published methods to assess the strength reduction due to corrosion of bridges, it is hard to find any with lesser number of measuring variables which eventually accounts for the accuracy and the convenience of the investigation for adequate bridge management. So, in this study, a simple method of calculating the remaining yield and tensile strength by using a concept of representative effective thickness ($t_{eff}$) with correlation of initial thickness ($t_0$) and standard deviation of thickness ($\sigma_{st}$) is proposed, based on the results of many tensile coupon tests of corroded plates obtained from a steel plate girder used for about 100 years with severe corrosion.

**Keywords:** Corrosion, Effective thickness, Remaining strength, Standard deviation of thickness

## 1. Introduction

Corrosion is a serious threat which affects the long term function and the integrity of a steel bridge. Exposure of a steel structure to the natural environment without or inadequate protection will cause corrosion of the structure, leading to impairment of its operation and weakening of the structure. Since corrosion will deteriorate the performance of steel structures with time, careful evaluation of the feasibility for current usage and strengthening the existing structure by retrofitting some selected corroded members are essential. Therefore, understanding of the influence of damage due to corrosion on the remaining load-carrying capacities is of high concern among the bridge maintenance engineers at present.

The results of this deterioration generally range from progressive weakening of a steel structure over a long time, to rapid structural failure. Though it's a maintenance issue, it can be addressed appropriately by specification of a proper corrosion system in the design phase. It has been proved that the corrosion played a significant role in the catastrophic collapse of both the Silver Bridge (Point Pleasant, WV) in 1967 and the Mianus River Bridge (Connecticut) in 1983, USA (Steel Bridge Design Handbook). Those collapses indicated the paramount importance of attention to the condition of older bridges, leading to intensified inspection protocols and numerous eventual retrofits or replacements. Therefore corrosion is not an issue to be taken lightly either in design phase or in maintenance stage.

To assure adequate safety and determine the ongoing maintenance requirements, thorough regular inspections are required. These inspections should form the essential source of information for carrying out a

[*] Corresponding author, Tel/Fax: +81-89-927-9816, E-mail: ohga@cee.ehime-u.ac.jp

comprehensive evaluation of its current capacity. The accurate estimation of remaining strength of steel members will give the necessary information on establishing the performance recovery methods and necessary retrofitting techniques or replacements of severe corroded members. Therefore, establishment of more accurate remaining strength estimation method will be the core part in all maintenance tasks.

Several experimental studies were carried out during the past few years, to investigate the remaining strength of corroded tensile plates. Namely, Matsumoto *et al.* (1989) investigated the tensile strength, using tensile coupons with corrosion. They predict the remaining tensile strength of corroded plates, using minimum value of average thickness ($t_{sa}$) of the cross section perpendicular to the loading axis as a representative thickness. Further, Muranaka *et al.* (1998) and Kariya *et al.* (2003) proposed different representative thickness parameters with a correlation of average thickness ($t_{avg}$) and standard deviation of thickness ($\sigma_{st}$), to estimate the tensile strength of corroded members based on many tensile tests. Thus, it is very clear that, many researchers usually use representative thickness based on several statistical parameters to estimate the remaining strength.

It is known that the corrosion wastage and stress concentration caused by the surface irregularity of the corroded steel plates influence the remaining strength of corroded steel plates (Kariya, 2005). Therefore, the evaluation of the effect of different forms of corrosion to the remaining strength capacities of existing structures is a vital task for maintenance management of steel highway and railway infrastructures.

## 2. Objective

It was noticed that many corrosion pits of more than 30mm diameters exist in actual severe corroded members. But, the widths of above mentioned test specimens are very small (less than 30mm). So, the influence of such corroded conditions could have been derelict and hence their actual remaining strengths might be different than those were obtained from those experimental studies. Therefore, in order to clarify the effect of corrosion conditions on remaining strength, it is an essential task to conduct some experimental studies with steel members close to the actual size of the steel members. For this purpose, tensile tests were conducted on 26 specimens with 70-180 mm width and different corrosion conditions in this research study. A simple and more accurate method to predict their remaining yield and tensile strength capacities with the correlation of initial thickness ($t_0$) and standard deviation of thickness ($\sigma_{st}$) is presented and compared with the other available remaining strength estimation methods in this paper.

## 3. Corroded Test Specimens

### 3.1. Test Specimen Configuration

The test specimens were cut out from a steel girder of Ananai River in Kochi Prefecture on the shoreline of the Pacific Ocean, which had been used for about hundred years. This bridge was constructed as a railway bridge in 1900, and in 1975 changed to a pedestrian bridge, when the reinforced concrete slab was cast on main girders. The bridge was dismantled due to serious corrosion damage in year 2001. Many severe corrosion damages distributed all over the girder, especially, large corrosion pits or locally-corroded portions were observed on upper flanges and its cover plates. Then, 21 (F1~F21) and 5 (W1~W5) test specimens were cut out from the cover plate on upper flange and web plate respectively.

Before conducting the thickness measurements, all rusts over both surfaces were removed carefully by using the electric wire brushes and punches. Then, two new SM490A plates (t=16mm) were jointed to both sides of specimen by the butt full penetration welding for grip parts to loading machine, as shown in Figure 1. Here, the flange and web specimens have the widths ranged from 70-80mm and 170-180mm respectively. The test specimen configuration is shown in Figure 1.

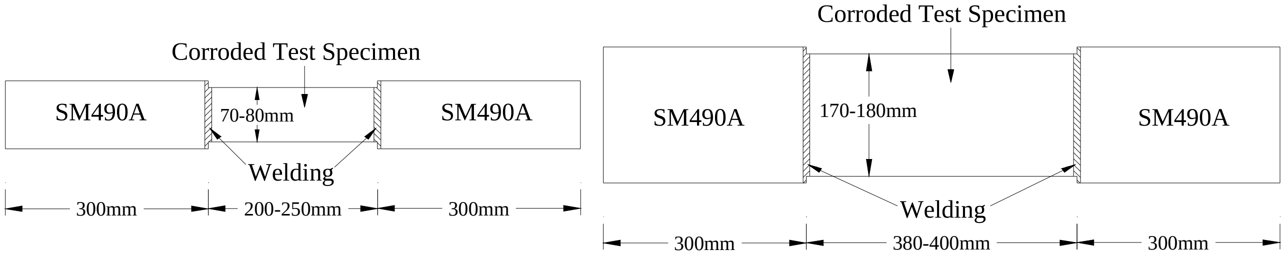



Figure 1: Dimensions of test specimens

### 3.2. Corrosion Surface Measurement

Accuracy and easiness are highly demanded in the measurement of corrosion surface irregularities. Furthermore, portability, good operability and lightness would be also imperative for choosing of a measurement device for on-site measurements. Therefore, the portable 3-dimentional scanning system, which can measure the 3-dimentional coordinate values at any arbitrary point on the corrosion surface directly and continuously, was used for the measurement of surface irregularities of the test specimens. Here, the thickness of the corroded surface can be calculated easily from those measured coordinates.

The measuring device has three arms and six rotational joints, and can measure the coordinates of a point on steel surface by using the non-contact scanning probe (laser line probe). So, the thicknesses of all scratched specimens were measured by using this 3D laser scanning device and the coordinate data was obtained in a grid of 0.5mm intervals in both X and Y directions. Then, the remaining thicknesses of all grid points were calculated by using the difference of the coordinate values of both sides of those corroded specimens. Then, the statistical thickness parameters such as average thickness ($t_{avg}$), minimum thickness ($t_{min}$), standard deviation of thickness ($\sigma_{st}$) and coefficient of variability (CV) were calculated from the measurement results.

Figure 2: Specimen prepaired for the tensile test

## 4. Tensile Test of Corroded Specimens

### 4.1. Experimental Setup and Loading Conditions

All the flange and web specimens were prepared for the tensile loading tests. There, different numbers of strain gauges were attached to each specimen considering their corrosion conditions. One example of prepared corroded specimen with strain gauges is shown in Figure 2. There, more attention was paid on both

the minimum section and local portions with serious corrosion damage for attaching the strain gagues. And the intervals of strain gauges were decided by considering the surface condition.
Tensile loading tests were carried out at constant velocity under loading control by using a hydraulic loading test machine (maximum load: 2940KN) for all 26 specimens with different corrosion conditions. The loading velocity was set to 200N/sec for minor corroded specimens and 150N/sec for moderate and severe corroded specimens.

### 4.2. Classification of Corrosion Sates

It is necessary to categorize the different corrosion conditions which can be seen in actual steel structures, into few general types for better understanding of their remaining strength capacities considering their visual distinctiveness, amount of corrosion and their expected mechanical and ultimate behaviors. The Figure 3 shows the relationship between the nominal ultimate stress ratio ($\sigma_{bn}/\sigma_b$) and the minimum thickness ratio ($\mu$), where $\sigma_{bn}$ is the nominal ultimate stress and $\sigma_b$ is the ultimate stress of corrosion-free plate. Here, the minimum thickness ratio ($\mu$) is defined as:

$$\mu = \frac{t_{min}}{t_0} \tag{1}$$

There, the initial thickness ($t_0$) of the flange specimens and web specimens are 10.5mm and 10.0 mm respectively. Therefore, three different types of corrosion levels were identified according to their severity of corrosion and they are classified accordingly as follows:

$\mu > 0.75$ ; Minor Corrosion
$0.75 \geq \mu \geq 0.5$ ; Moderate Corrosion
$\mu < 0.5$ ; Severe Corrosion

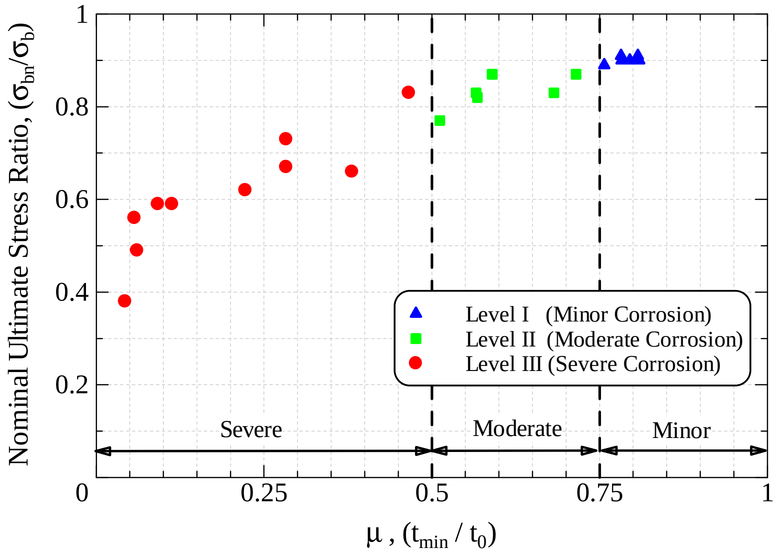



Figure 3 Relationship of ultimate stress ratio & minimum thickness ratio (μ)

Further, the Figure 4 shows three tensile test specimens with above three classified corrosion types. In minor corrosion type, it can be seen that many small corrosion pits were spread on all over the plate surface and an example of this corrosion type (F-14) is shown in Figure 4(a). When the corrosion is more progressed, the moderate corrosion type can be seen where few considerable corroded pits exist in some places. An example of this corosion type (F-13) is shown in Figure 4(b). Further, as the corrosion is more progressed than the moderate corrosion condition, severe corrosion type can be seen with several extensive corroded regions

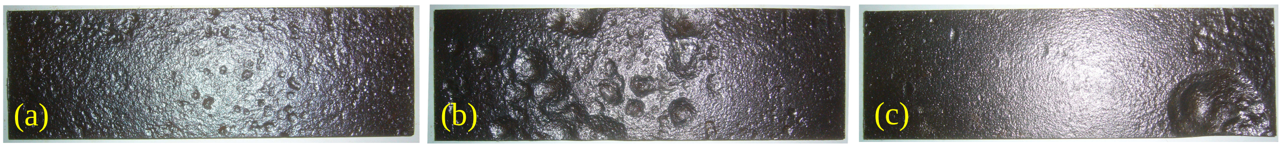



Figure 4: Plates with (a) minor corrosion (F-14), (b) moderate corrorion (F-13) and (c) severe corrosion (F-19)

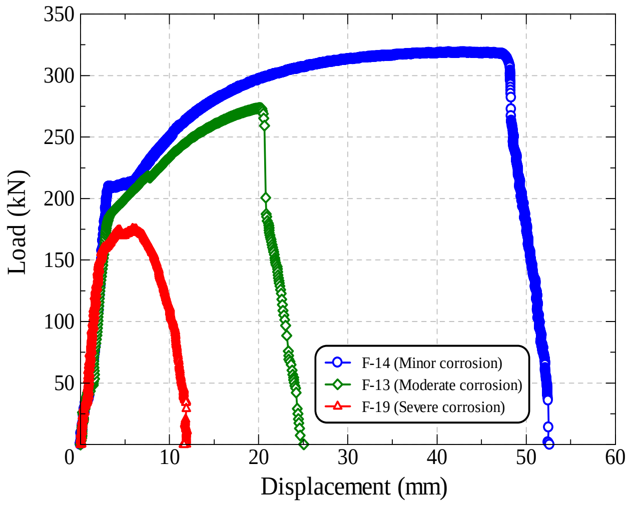



Figure 5: Load-displacement curves

(maximum corrosion depth over 5mm and the diameter of the corroded pits are exceeding 25mm) on the member. One example of severe corrosion type (F-19) is shown in Figure 4(c).

### 4.3. Experimental Results and Discussion

Figure 5 shows the Load-elongation curves for three different corroded specimens (F-14, F-13 and F-19) with 3 corrosion types. Herein, the specimen (F-14) with minor corrosion has almost same mechanical properties (such as apparent yield strength and load-elongation behavior etc.) as the corrosion-free specimen. On the other hand, the moderate corroded specimen (F-13) and the severe corroded specimen (F-19) show obscure yield strength and the elongation of the specimen F-19 decreases notably. The reason for this is believed to be that the local section with a small cross-sectional area yields at an early load stage because of the stress concentration due to irregularity of corroded steel plate. And this will lead moderate and severe corroded members to elongate locally and reach to the breaking point.

## 5. Remaining Strength Estimation

The two basic definitions can be expressed for the experimentally predicted parameters for the yield effective thickness ($t_{e\_y}$) and the tensile effective thickness ($t_{e\_b}$) as follows:

$$t_{e\_y} = \left( \frac{P_y}{B \cdot \sigma_y} \right) \tag{2}$$

$$t_{e\_b} = \left( \frac{P_b}{B \cdot \sigma_b} \right) \tag{3}$$

Where, $P_y$: yield load, $P_b$: tensile load, B: width of the specimen for the corroded state and $\sigma_y$ and $\sigma_b$ are yield and tensile stress of corrosion-free plate respectively. But the above defined effective thickness parameters cannot be obtained for the in-service structures. So, a measurable statistical parameter with a high correlation with the effective thickness parameter will be essential for remaining strength estimation of those structures. Therefore, the correlations between the effective thickness ($t_{eff}$) and many measureable statistical parameters were examined (such as minimum thickness $t_{min}$, average thickness $t_{avg}$, minimum average thickness $t_{avg\_min}$ and standard deviation of thickness $\sigma_{st}$ etc.) and two relationships were defined for remaining yield and ultimate strength estimations of corroded steel plates.

### 5.1. Estimation of Yield and Tensile Strengths

The correlation between the yield and tensile effective thickness and measureble statistical thickness parameters were examined and the best relationships were found with the standard deviation of thickness. The Figure 6(a) and Figure 6(b) show the two linear relationships obtained for yiled and tensile stress conditions.

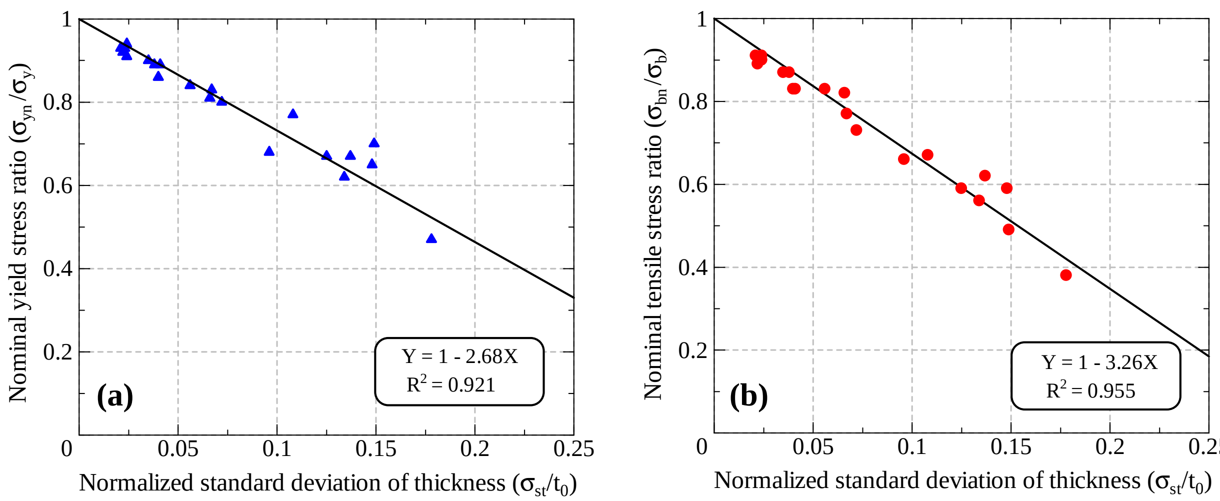



Figure 6: Relationship of (a) yield stress ratio, (b) tensile stress ratio and normalized standard deviation of thickness ($\sigma_{st}/t_0$)

So, considering the relationships shown in Figure 6, two equtionss for representative effective thickness ($t_{eff}$) for yield and tensile states can be obtained as described below.

From Figure 6(a),

$$\left(\frac{\sigma_{yn}}{\sigma_y}\right) = 1 - 2.68\left(\frac{\sigma_{st}}{t_0}\right)$$

$$t_{eff} = t_0 - 2.7\sigma_{st} \tag{4}$$

In same way from Figure 6(b),

$$t_{eff} = t_0 - 3.3\sigma_{st} \tag{5}$$

Further, the Figure 7 shows the relationship between different statistical thickness parameters with tensile effective thickness. It was found that the tensile strength estimation using minimum thickness ($t_{min}$) will provide considerably underestimated results as shown in Figure 7(a). On the other hand, Figure 7(b) shows that the average thickness ($t_{avg}$) tends to become larger than effective thickness, as the influence of stress concentration due to corrosion will not be able to consider carefully. Figure 7(c) shows that the minimum average thickness ($t_{avg\_min}$) also gives larger values than the effective thickness and hense, the strength estimation using only $t_{avg}$ or $t_{avg\_min}$ will overestimate the remaining tensile strength. This will lead the structure in danger on decissision taken regading its maintenace management plan. But, as it can be seen from Figure 7(d), the proposed effective thikness gives more accurate and better remaining strengths estimation.

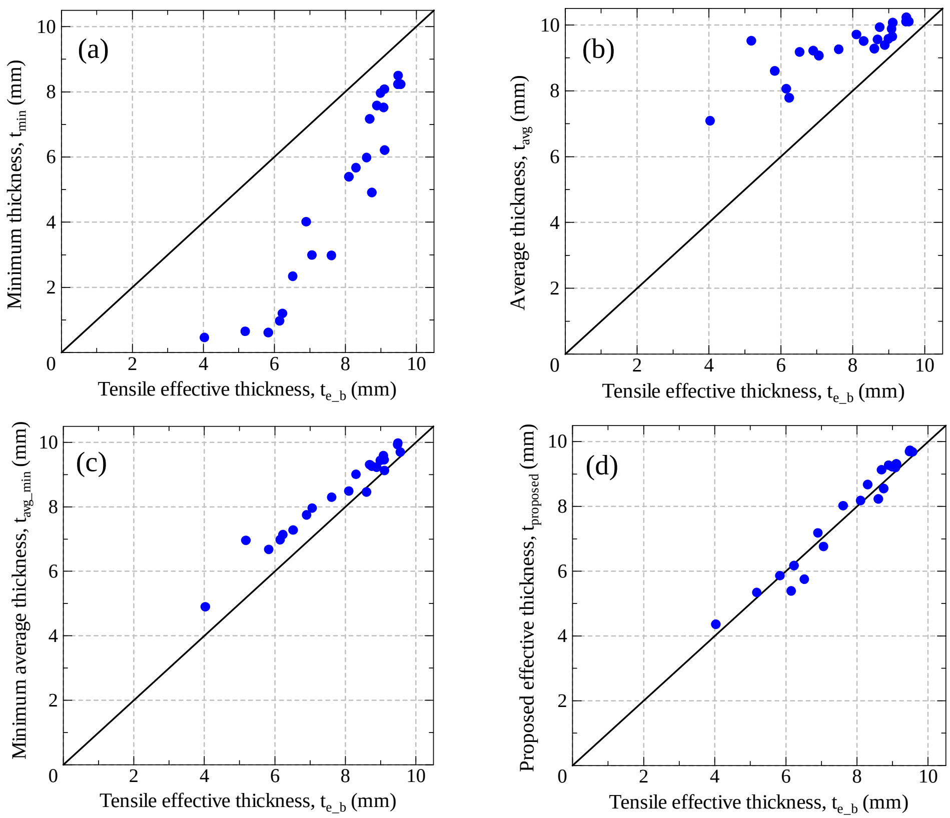



Figure 7: Relationship of (a) $t_{min}$ vs $t_{e\_b}$, (b) $t_{avg}$ vs $t_{e\_b}$, (c) $t_{avg\_min}$ vs $t_{e\_b}$ and (d) proposed effective thickness ($t_{eff}$) vs $t_{e\_b}$

### 5.2. Comparison of Proposed Effective Thickness

Table 1: Comparison of correlation coefficients of different effective thickness prediction methods

| Method | | Matsumoto *et al.* 1989 | Muranaka *et al.* 1998 | Kariya *et al.* 2003 | Proposed, $t_{eff}$ |
|---|---|---|---|---|---|
| **Equation of thickness** | | $t_{sa}$ | $t_{avg} - 0.7\sigma_{st}$ | $t_{avg} - 1.3\sigma_{st}$ | Yield: $t_0 - 2.7\sigma_{st}$<br>Tensile: $t_0 - 3.3\sigma_{st}$ |
| **Correlation Coefficient** | Yield | – | – | – | 0.92 |
| | Tensile | 0.70 | 0.14 | 0.75 | 0.96 |

The Table 1 shows that the proposed effective thickness parameter gives more reliable and better prediction than other available methods for estimating remaining yield and tensile strengths.

## 6. Conclusions

The steel surface measurements and tensile tests were conducted on many wide specimens with different corrosion conditions, which are obtained from a plate girder which had been used for about 100 years with severe corrosion. The main conclusions of this study can be summarized as:

(1) The corrosion causes strength reduction of steel plates and minimum thickness ratio ($\mu$) can be used as a measure of the level of corrosion and their strength degradation. Three basic corrosion categories can be defined according to their severity of corrosion as, minor corrosion ($\mu > 0.75$), moderate corrosion ($0.75 \geq \mu \geq 0.5$) and severe corrosion ($\mu < 0.5$).

(2) Remaining yield strength of corroded steel plates can be estimated by using the representative effective thickness defined as: $t_{eff} = t_0 - 2.7\sigma_{st}$ with high accuracy.

(3) The remaining tensile strength estimation can be done by using the representative effective thickness defined as: $t_{eff} = t_0 - 3.3\sigma_{st}$ with high accuracy.

## 7. Acknowledgements

The authors would like to thank the technical staffs of Mitutoyo Corporation (Japan) for their assistance in surface measurement of corroded specimens.

## References

'Corrosion Protection of Steel Bridges', Steel Bridge Design Handbook, Chapter 23, National Steel Bridge Alliance.

Kaita, T., Fuji, K., Miyashita, M., Uenoya, M., Okumura, M. and Nakamura, H. (2005), 'An Experimental Study on the residual bending strength of corroded plates', *Proceedings of Structural Engineering,* Vol.51A, pp.139-148.

Kariya, A., Tagaya, K., Kaita, T. and Fujii, K. (2003), 'Basic study on effective thickness of corroded steel plate and material property', *Annual conference of JSCE*, pp 967-968. (In Japanese)

Kariya, A., Tagaya, K., Kaita, T. and Fujii, K. (2005), 'Mechanical properties of corroded steel plate under tensile force', *Proceedings of the 3rd International Structural Engineering and Construction Conference (ISEC-03),* Japan, pp 105-110.

Matsumoto, M., Shirai, Y., Nakamura, I. and Shiraishi, N. (1989), 'A Proposal of effective Thickness Estimation Method of Corroded Steel Member', *Bridge Foundation Engineering,* vol. 23, No. 12, pp 19-25. (In Japanese)

Muranaka, A., Minata, O. and Fujii, K. (1998), 'Estimation of residual strength and surface irregularity of the corroded steel plates', *Journal of Structural Engineering,* vol. 44A, pp 1063-1071 (In Japanese).

# NUMERICAL STUDY ON REMAINING STRENGTH PREDICTION OF CORRODED STEEL BRIDGE PLATES

Ohga M[1†], Appuhamy J.M.R.S[1], Kaita T[2], Fujii K[3] and Dissanayake P.B.R[4]

[1]Department of Civil & Environmental Engineering, Ehime University, Japan

[2]Department of Civil Engineering & Architecture, Tokuyama College of Technology, Japan

[3]Department of Social & Environmental Engineering, Hiroshima University, Japan

[4]Department of Civil Engineering, University of Peradeniya, Sri Lanka

**Abstract**

Corrosion causes strength deterioration of aged steel infrastructures and hence careful evaluation of their remaining load-carrying capacities are of high importance in maintenance engineering. To develop a more reliable strength estimation technique, only experimental approach is not enough as actual corroded surfaces are different from each other. However in modern practices, numerical simulation is being used to replace the time-consuming and expensive experimental work and to comprehend on the lack of knowledge of mechanical behavior, stress distribution, ultimate behavior and so on. Therefore, using of numerical analysis method will give important knowledge not only for strength estimation but also for subsequent repair and retrofitting plan. The results of non-linear FEM analysis of many actual corroded plates with different corrosion conditions and comparison of them with the respective tensile coupon tests results are presented in this paper. Further, the feasibility of establishing of an analytical methodology to predict the residual strength capacities of a corroded steel member with fewer number of measuring points are also discussed.

**Keywords:** Corrosion, Numerical analysis, Strength estimation, Tensile test

## 1. Introduction

Many steel bridge infrastructures of the world are getting old and subjected to age-related deterioration such as corrosion wastage, fatigue cracking, or mechanical damage during their service life. These forms of damage can give rise to significant issues in terms of safety, health, environment, and life cycle costs. Therefore it is important to develop advanced technologies which can be used to assist proper management and control of such age-related deterioration as many of these structures are currently in need of maintenance, rehabilitation or replacement.

Detailed regular inspections are necessary in order to assure adequate safety and determine maintenance requirements, in bridge infrastructure management. These inspections should form the essential source of information for carrying out a comprehensive evaluation of its current capacity. But the number of steel bridge infrastructures in the world is steadily increasing as a result of building new steel structures and extending the life of older structures. Most of these structures are subjected to corrosion due to environmental exposure which can reduce their carrying capacities. So, there is a need of more brisk and accurate assessment method which can be used to make reliable decisions affecting the cost and safety.

---

† Corresponding author, Tel/Fax: +81-89-927-9816, E-mail: ohga@cee.ehime-u.ac.jp

Usually, the accurate predictions are based on how accurately statistical parameters are estimated and therefore mainly depends on experimental and field data. In the past few decades, some researchers have done some experimental studies and detailed investigations of the corroded surfaces to introduce methods for estimating the remaining strength capacities of corroded steel plates. But, to develop a more reliable strength estimation technique, only experimental approach is not enough as actual corroded surfaces are different from each other. Further, due to economic constraints, it is not possible to conduct tests for each and every aged bridge structure within their bridge budgets. Therefore, bridge engineers are faced with the lack of experimental and field data. Therefore, nowadays, use of numerical analysis method could be considered to have a reliable estimation in bridge maintenance industry.

Further, it is not easy to measure several thousands of points, to accurately reproduce the corroded surface by numerical methods and to predict the behavior of that corroded member with more precisely. Therefore, study the effect of corroded surface data measurement intensity on their present load carrying capacities and investigation of the possibility of establishing a simple and accurate procedure to predict the remaining strength capacities of a corroded steel member by measuring lesser number of points with an acceptable accuracy level would be a vital task for the maintenance management of steel highway and railway infrastructures.

## 2. Preliminary Investigation of Corroded Specimens

In this study, 42 specimens (21 each from flange and web; denoted as FT and WT respectively) cut out from a steel bridge girder of Ananai River in Kochi Prefecture on the shoreline of the Pacific Ocean, which had been used for about hundred years. Before conducting the thickness measurements, the rust and paint on the surface were removed by using a steel wire brush and then applying high pressure water in order to care not to change the condition of the corrosion irregularity. Then the thicknesses of all scratched specimens were measured by using a laser displacement gauge and the tensile tests were performed in order to clarify their remaining strength capacities. The JIS No.5 test specimen is shown in Figure 1.

As the corrosion conditions in actual steel structures are different from each other, it is necessary to

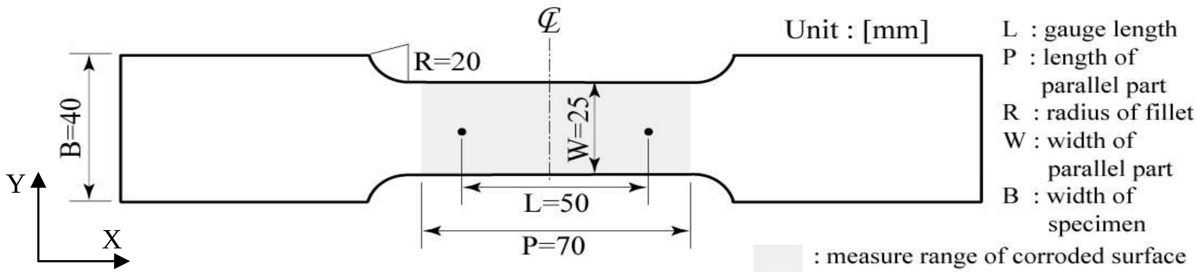



Figure 1: JIS No.5 Specimen for tensile test

categorize them into few general types for better understanding of their remaining strength capacities considering their visual distinctiveness, amount of corrosion and their expected mechanical and ultimate behaviors. Therefore, in this study, all specimens were categorized into typical 3 corrosion types concerning their corrosion conditions and minimum thickness ratio, $\mu$ (minimum thickness/ initial thickness). The corrosion conditions with the minimum thickness ratio, $\mu > 0.75$ are defined as 'minor corrosion'. And the 'moderate corrosion' type is defined when the minimum thickness ratio, $0.75 \geq \mu \geq 0.5$. Further, the 3rd


Figure 2: Plates with (a) minor corrosion [FT-22], (b) moderate corrosion [FT-18] and (c) severe corrosion [FT-15]

corrosion type with the minimum thickness ratio, $\mu < 0.5$ is defined as 'severe corrosion'. There, the initial thickness ($t_0$) of flange specimens and web specimens are 10.5mm and 10.0 mm. Three specimens; FT-22, FT-18 and FT-15 with minor, moderate and severe corrosion conditions respectively are shown in Figure 2.

The significance of three proposed corrosion conditions can be identified even with the visual examination of those members, as members with minor corrosion have tiny corrosion pits (less than 3mm depth) throughout the member, members with moderate corrosion have few considerable corroded pits (depth of 3-5 mm) in some places while many non-corroded portions also remain widely and members with severe corrosion have several extensive corroded regions (maximum corrosion depth over 5mm).

## 3. Numerical Analysis

### 3.1. Analytical Model

The 3D isoparametric hexahedral solid element with eight nodal points (HX8M) and updated Lagrangian method based on incremental theory were adopted in these analyses. Non linear elastic-plastic material, Newton-Raphson flow rule and Von Mises yield criterion were assumed for material properties. Further, an automatic incremental-iterative solution procedure was performed until they reached to the pre-defined termination limit.

The analytical models with length and width dimensions of 70mm x 25mm (X and Y directions) were modeled with their respective corrosion conditions. One edge of the member's translation in X, Y and Z directions were fixed and only the Y and Z direction translations of the other edge (loading edge) were fixed to simulate with the actual experimental condition. Then the uniform incremental displacements were applied to the loading edge. Yield stress $\sigma_y$ = 299.9 [MPa], Elastic modulus E = 195.8 [GPa], Poisson's ratio $\nu$ = 0.278 were applied to all analytical models, respectively.

### 3.2. Ductile Fracture Criterion

The "Stress Modified Critical Strain Model (SMCS)" was proposed by Kavinde *et al.* (2006), to evaluate the iniation of ductile fracture as a fuction of multiaxial plastic strains and stresses. This method was adopted in this analytical study. In SMCS criteion, the critical plastic starin ($\varepsilon_p^{Critical}$) is determined by the following expression:

$$\varepsilon_p^{Critical} = \alpha \cdot \mathrm{Exp}\left(-1.5\frac{\sigma_m}{\sigma_e}\right) \tag{1}$$

Where, $\alpha$ is toughness index and the stress triaxiality T = ($\sigma_m / \sigma_e$), a ratio of the mean or hydrostatic stress ($\sigma_m$) and the effective or von Mises stress ($\sigma_e$). The toughness index $\alpha$ is a fundamental material property and hence obtained from the tensile test conducted for the non corroded specimen. The ultimate strength of each corroded specimen was calculated accordingly by using the SMCS criterion and compared with their experimental ultimate capacities to understand the feasibility of the numerical modeling approach for remaining strength estimation of corroded steel plates.

### 3.3. Analytical Results

Non corroded specimen was modeled at first, with the above described modeling and analytical features to understand the accuracy of the adopted procedure. It was found that the analytical model results were almost same as the experimental results with having a negligible percentage error of 0.03% and 0.02% in yield and tensile strength respectively.Then, all other experimentally successful specimens were modeled accordingly and their yield and ultimate strengths were compared with the experimentally obtained values.

The Figures 3(a), 3(b) and 3(c) show a very good comparison of experimental and analytical load-elongation behaviors for all three classified corrosion types. Here, the percentage errors in yield and tensile strength predictions of the analytical models are 2.11% and 0.56% in FT-22, 0.84% and 0.49% in FT-18 and 0.19% and 4.48% in FT-15 respectively. Further, the Figure 3(d) shows the comparison of ultimate load capacities of all 32 specimens in experimental and numerical analyses. Having a coefficient of correlation of $R^2$= 0.963 indicate the accuracy and the possibility of numerical investigation method to predict the tensile strength of actual corroded specimens.

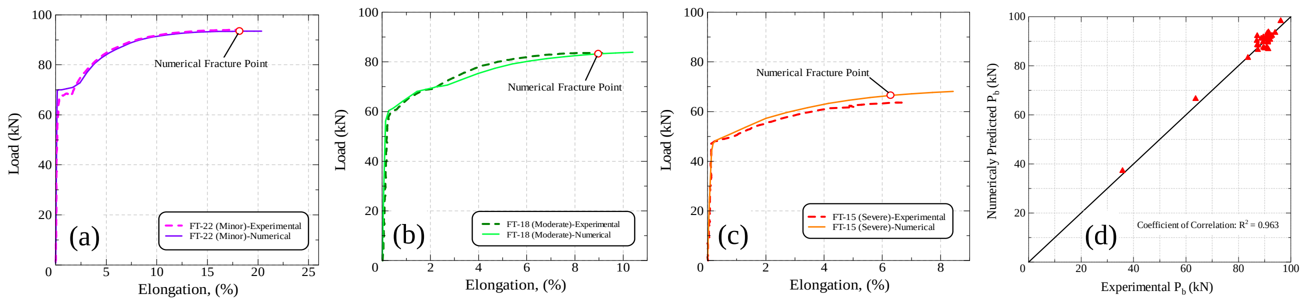



Figure 3: Comparison of experimental and analytical load-elongation curves of specimens with; (a) minor corrosion [FT-22], (b) moderate corrosion [FT-18], (c) severe corrosion [FT-15] and (d) comparison of experimental and analytical ultimate load capacities

## 4. Effect of Measuring Points

It is necessary to conduct regular assessments of each and every structure to ensure their safety and determine maintenance needs. But, it wouldn't be an easy task as the number of highway and railway steel structures are steadily increasing in the world. So, developing a rapid and accurate methodology to estimate the remaining strength capacities of steel infrastructures is a vital task in maintenance engineering.

## 4.1. Numerical Modeling with Regular Corroded Surface Measurements (RCSM) Methods

### 4.1.1. Analytical Models

Table 1: Data interval and total no. of measuring points

| Model No. | Maximum Data Interval /(mm) | Total measuring points |
|---|---|---|
| 1 | 1 | 1846 |
| 2 | 2 | 504 |
| 3 | 5 | 90 |
| 4 | 10 | 32 |
| 5 | 15 | 18 |
| 6 | 25 | 8 |

Six different finite element models with different corroded surface measurement intervals in X and Y directions were modeled and analyzed for each corroded specimen and compared them with the results of Model 1 with 1mm mesh data to understand the effect of corroded surface data intensity with their remaining yield and tensile strength capacities. The Table 1 shows the maximum data measurement interval and total number of measuring points in each model. The same modeling features and analytical procedure as described in chapter 3 were adopted for all analyses.

### 4.1.2. Analytical Results and Discussion

The Figure 4 shows the percentage errors in yield and tensile strength estimations of different RCSM models for three members FT-22, FT-18 and FT-15 with minor, moderate and severe corrosion conditions respectively. It can be seen that the data intensity for minor corrosion members is not very significant for their remaining strength estimation. This fact can be comprehended as the overall amount of corrosion or the corrosion attack for a particular location is very small in minor corrosion members. But, it can be noted that

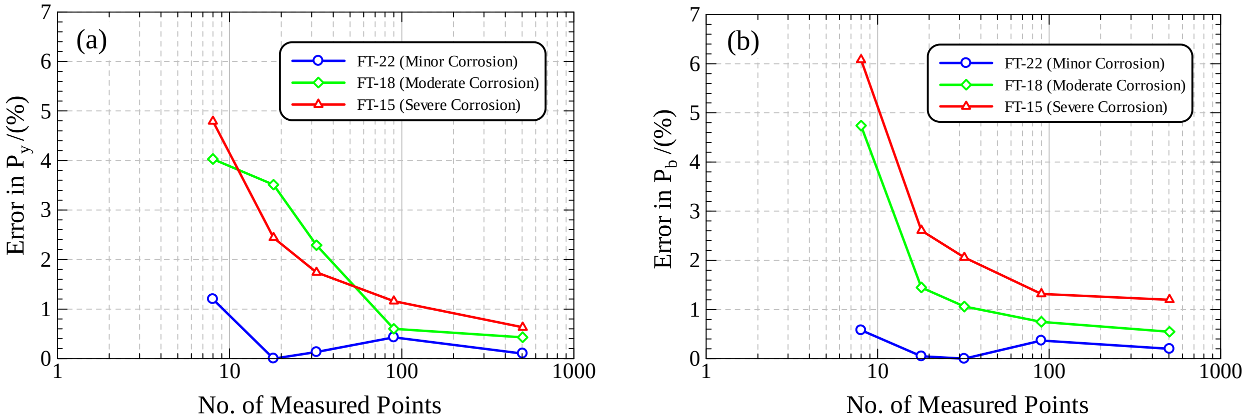



Figure 4: Comparison of %errors in (a) yield and (b) tensile strength estimation

the percentage errors in both yield and tensile strength estimations are increased with the reduction of the intensity of corroded surface measurement points in moderate and severe corrosion members. The reason for this could be the missing of the maximum corroded location or some severe corroded portions during this kind of regular data measurement. So the effect of stress concentration will diminish in some of the models considered in this study, which are having smaller number of measuring points. So the remaining strengths are over estimated with the increase of coarseness of the data measurement, and this could lead the infrastructure in danger with decision taken regarding its maintenance management plan. So, a special surface measurement method with few data points, concerning the severity of corrosion and stress concentration is required for moderate and severe corrosion members.

### 4.2. Numerical Modeling with Special Corroded Surface Measurements (SCSM) Methods

#### 4.2.1. Analytical Models

Five different models were created by considering the irregularity of the corroded surface and stress concentration effect and their analytical results were compared with the model with 1mm corroded surface data (Model-1). The Figure 5 shows different SCSM analytical models used in this study. The outer edges were taken as the initial thickness ($t_0$) in all models except the Model-1. The Model-2 consists only the measurement of minimum thickness point ($t_{min}$) and linear variation was considered between $t_{min}$ point and edges of the plate. The Model-3 and Model 4 were created with $t_{min}$ point and two other thickness measurements taken at the edges of the corroded pit in longitudinal direction and width direction respectively. These models also were created by considering the linear variation among measured points and plate boundaries. The Models 5a and 5b consists five corroded surface measurements of the corroded pit including its $t_{min}$ and two points each in longitudinal and width directions. Then, linear and Spline variations were used to model the corroded portion in Model-5a and Model-5b respectively. Further, linear variation between the corroded pit and plate edges were used in both models.

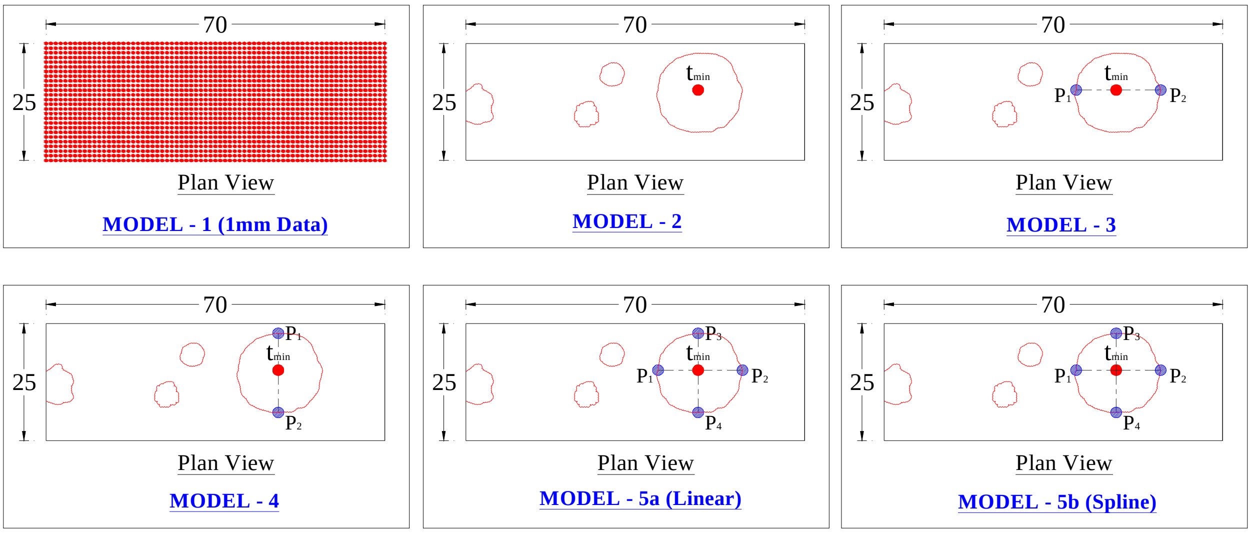
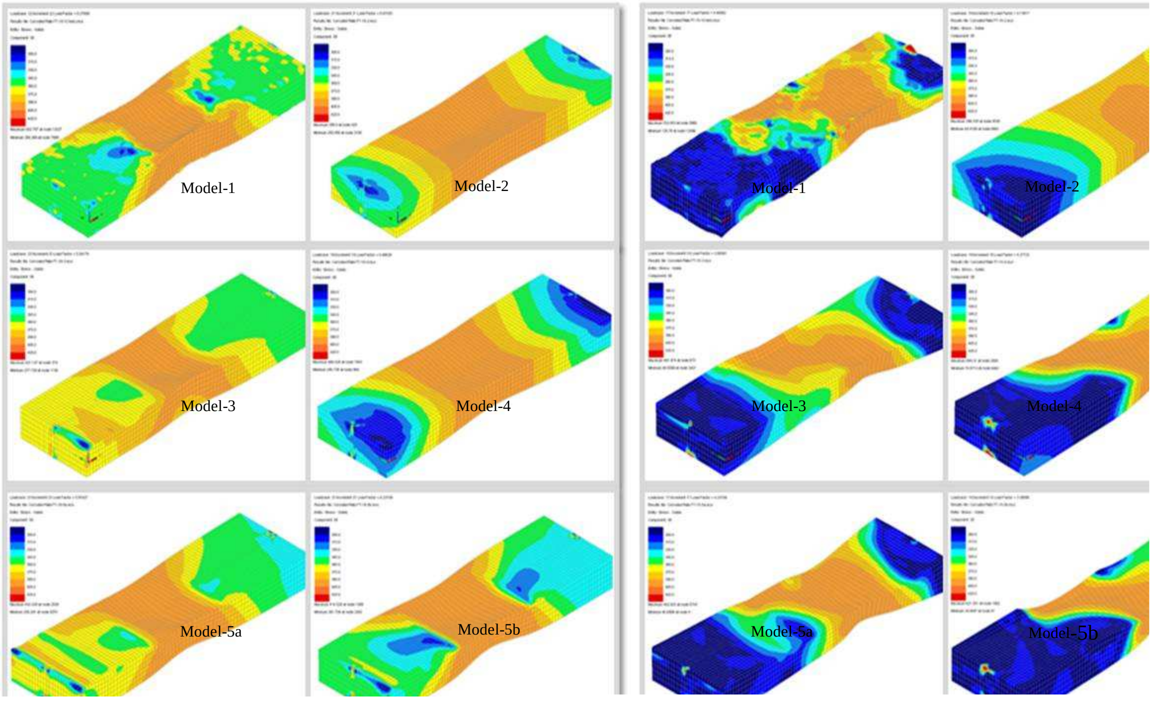



Figure 5: Analytical models with special corroded surface measurement points

Same modeling features, non-linear elastic-plastic material properties and analytical proceedure were adopted for all above models and the analysis was continuied untill they reach to their predefined termination limits. Then, their load-elongation behaviours, yield and tensile strengths and ultimate behaviors were compared with the Model-1 with 1mm corroded surface data, to understand the effect of special corroded surface data measurement methods having fewer number of measuring points.

#### 4.2.2. Analytical Results and Discussion

The Figure 6(a) and Figure 6(b) show the ultimate stress distributions of different SCSM models of moderate corrosion member (FT-18) and severe corrosion member (FT-15) respectively. The stress concentration effect can be seen in all these SCSM models, as all of them were developed including the minimum thickness point ($t_{min}$). But, the differences in stress distribution can be seen in different models as the irregularity of each




(b) Moderate corrosion member, FT-18 (a) Severe corrosion member, FT-15

Figure 6: Stress distributions of different special analytical models at ultimate load

model is different. Further, more similar stress distribution can be seen in Model-5a and Model-5b with the results of their 1mm data model. The percentage errors in yield and tensile strength estimations of different SCSM models for members FT-18 and FT-15 are shown in Figure 7. There, it can be seen that the %errors of moderate corrosion specimen (FT-18) are very small in Models 5a and 5b. Further, it can be seen that even though the %errors of severe corroded specimen (FT-15) in Models 5a and 5b are comparatively smaller than the other models, those errors are quite significant too. Therefore, more SCSM models to be developed to represent the corroded surface irregularity more accurately for severe corroded members in future studies.

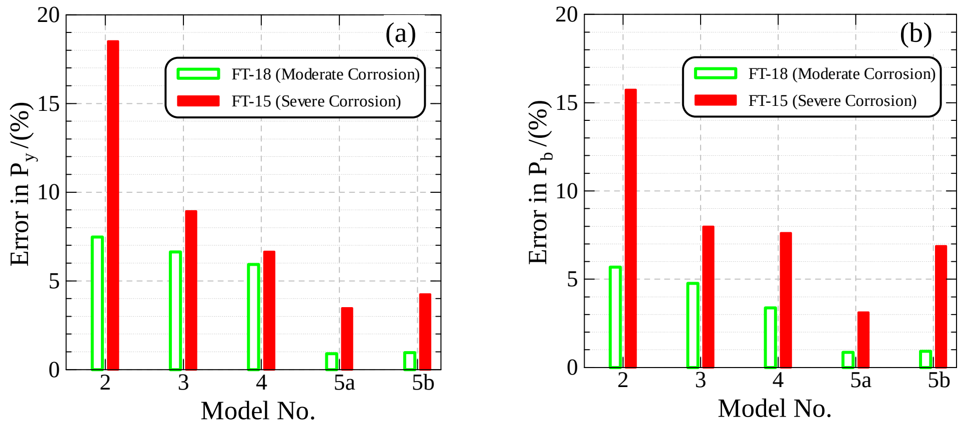



Figure 7: Comparison of %errors in (a) yield and (b) tensile strength estimation of different special analytical models

## 5. Conclusions

The surface irregularity measurement, tensile testing and non linear FEM analyses were conducted for corroded steel specimens and the following conclusions can be made from this study.

(4) The corrosion causes strength reduction of steel plates and minimum thickness ratio ($\mu$) can be used as a measure of the level of corrosion and their strength degradation.

(5) A very good agreement between the experimental and analytical results can be seen for all three classified corrosion types. So, the adopted numerical modeling technique can be used to predict the remaining strength capacities of actual corroded members accurately.

(6) Though the intensity of corroded surface measurement is not very significant for minor corrosion members, it was found that it affects for moderate and severe corrosion members considerably in prediction of their remaining strength capacities.

(7) Therefore, a regular coarse surface data measurement is sufficient for minor corroded members and a special analytical model with fewer corroded surface measuring points, concerning the severity of corrosion and stress concentration effect is necessary for moderate and severe corrosion members to estimate their remaining strength capacities.

### References

Fujii, K., Kaita, T., Nakamura, H. and Okumura, M. (2003), 'A Model Generating Surface Irregularities of Corroded Steel Plate for Analysis of Remaining Strength in Bridge Maintenance', *The 9th East Asia-Pacific Conference on Structural Engineering and Construction (EASEC-9),* Vol. 9, pp 32-38.

Kaita, T., Kawasaki, Y., Isami, H., Ohga, M. and Fujii, K. (2008), 'Analytical Study on Remaining Compressive Strength and Ultimate Behaviors for Locally-corroded Flanges', *EASEC-11*, Taiwan, CDROM.

Kavinde, A.M. and Deierlein, G.G. (2006), 'Void Growth Model and Stress Modified Critical Strain Model to Predict Ductile Fracture in Structural Steels', *Journal of Structural Engineering*, Vol.132, No.12, pp 1907-1918.

Matsumoto, M., Shirai, Y., Nakamura, I. and Shiraishi, N. (1989), 'A Proposal of effective Thickness Estimation Method of Corroded Steel Member', *Bridge Foundation Engineering*, vol. 23, No. 12, pp 19-25. (In Japanese)

Muranaka, A., Minata, O. and Fujii, K. (1998), 'Estimation of residual strength and surface irregularity of the corroded steel plates', *Journal of Structural Engineering*, vol. 44A, pp 1063-1071 (In Japanese).

# NUMERICAL INVESTIGATION OF FUTURE TSUNAMI HAZARD ON SRI LANKA FROM THE EARTHQUAKES OF SUMATRA-ANDAMAN REGION

Ohga M[1‡], Appuhamy J.M.R.S[1], Tinti S[2], Lai C[3] and Dissanayake P.B.R[4]

[1]Department of Civil & Environmental Engineering, Ehime University, Japan

[2]Dipartimento di Fisica, Settore di Geofisica, Università di Bologna, Italy

[3]European School for Advanced Studies in Reduction of Seismic Risk, University of Pavia, Italy

[4]Department of Civil Engineering, University of Peradeniya, Sri Lanka

**Abstract**

The mega event of Indian Ocean Tsunami 26th December 2004, stressed the need for assessing tsunami hazards in vulnerable coastal areas in Sri Lanka. Two major areas of the management of disaster prevention are to evacuate people in the coastal area to safer areas as soon as possible and pre-modification of coastal structures to resist the tsunami waves effectively. Often the only way to determine the potential run-ups and inundation from a local or distant tsunami is to use numerical modeling, since data from past tsunamis is usually insufficient. It then might be possible to use such simulations to predict tsunami behavior immediately after an earthquake is detected. This paper consists of results of the numerical models of 26th December 2004 Tsunami and three other possible Tsunamis in this region which eventually can be used to create inundation and evacuation maps to minimize future damages.

**Keywords:** Disaster Prevention, Earthquake, Evacuation, Numerical modeling, Tsunami

## 6. Introduction

The 26th of December 2004 was an unforgettable day for all Sri Lankans as well as for the whole world. On that fateful day, tsunami waves struck the Eastern and Southern coasts of Sri Lanka as well as parts of Northern and Western coasts sweeping people away, causing flooding and destruction of infrastructures. When the huge waves surged up the coasts of Sri Lanka, the devastation of a tsunami brought forth a surge of generosity the likes of which the world has rarely seen. The tsunami waves were caused by an earthquake, measuring 9.1 on the Richter scale, occurred in the sea near Sumatra, Indonesia. Since many Sri Lankans did not have any previous experience of this nature, the damage caused to their lives was incredible. Thousands of people were displaced and disappeared or killed within a very short time.

The tsunami is the most formidable of all natural hazards. It is usually generated as a result of seismotectonic motions of the ocean bottom in the seismic source zone. Tsunami waves propagate far from the source and can cause damage even in regions where the earthquake was not manifested. A good definition of tsunami may be the following one: the tsunami is a series of ocean waves of extremely long wave length and long period generated in a body of water by an impulsive disturbance that displaces the water.

Large vertical movements of the Earth's crust can occur at plate boundaries. Plates interact along these boundaries called faults. Around the margins of the Pacific Ocean, for example, denser oceanic plates slip under continental plates in a process known as subduction. Subduction earthquakes are particularly effective in generating tsunamis.

‡ Corresponding author, Tel/Fax: +81-89-927-9816, E-mail: ohga@cee.ehime-u.ac.jp

Compared with wind-driven waves, tsunamis have periods, wavelengths, and velocities tens or a hundred times larger. So they have different propagation characteristics and shoreline consequences. As a result of their long wavelengths, tsunamis behave as shallow-water waves. Shallow-water waves are different from wind-generated waves, the waves many of us have observed on a beach. Wind-generated waves usually have period of 0.5 to 20 seconds and a wavelength up to about 200 meters. A tsunami can have a period in the range of ten minutes to two hours and a wavelength in excess of 500 km (Prager, 1999).

A wave is characterized as a shallow water wave when the ratio between the water depth and its wavelength gets very small. The rate at which a wave loses its energy is inversely related to its wave length. Since a tsunami has a very large wavelength, it will lose little energy as it propagates. Hence in very deep water, a tsunami will travel at high speeds and travel great transoceanic distances with limited energy loss. For example, when the ocean is 6100 m deep, unnoticed tsunami travel about 890 km/hr, the speed of a jet airplane. And they can move from one side of the Pacific Ocean to the other side in less than one day.

## 7. Basic Equations of Wave Motion

### 7.1. The Velocity Potential

The simplest and general most useful theory is the small amplitude wave theory first presented by Airy *et al.* 1845. Solving the Laplace equation develops the small amplitude wave theory for two-dimensional periodic waves, where x and y are the horizontal and vertical co-ordinates respectively:

$$\frac{\partial^2\phi}{\partial x^2}+\frac{\partial^2\phi}{\partial y^2}=0 \tag{1}$$

With the bottom and surface conditions, the following velocity potential is obtained in an ocean of constant depth d,

$$\phi=\frac{a\sigma}{k}\frac{\cosh\ k(y+d)}{\cosh\ kd}\cos(kx-\sigma t) \tag{2}$$

for a progressive wave traveling in positive x direction. The corresponding wave profile:

$\eta = a\sin(kx-\sigma t)$ is given by,

$$\phi=\frac{a\sigma}{k}\frac{\cosh k(y+d)}{\cosh kd}\sin(kx-\sigma t) \tag{3}$$

### 7.2. Wavelength and Wave Celerity

The relation between wavelength, wave period and water depth is written as:

$$L=\frac{gT^2}{2\pi}\tanh(2\pi d/L) \tag{4}$$

Eqn. (4) is an implicit equation, since the unknown variable L appears both in the left and right hand sides of the equation. For given T and d values, to obtain L it may require to carry out several trial calculations. However, for convince, solutions are all ready given in graphical form, or in tables.

Wave celerity is equal to the ratio of wavelength to wave period as:

$$C=L/T \tag{5}$$

Thus using Eqns. (4) and (5) we get,

$$C = \frac{gT}{2\pi}\tanh(2\pi d / L) \tag{6}$$

$$C = \left(\frac{gL}{2\pi}\tanh(2\pi d / L)\right)^{1/2} \tag{7}$$

### 7.3. Constancy of Wave Period

For a simple harmonic wave train, the wave period is independent of depth. This can be proven by the following argument. Let us suppose that the wave period can depend on the depth. Let us then take a region where wave enters from one side and exit from the opposite side. Let us further suppose that at these two sides the ocean depth is different, and therefore the wave entering waves have period $T_1$ and the outgoing waves have period $T_2$. In a given time interval $\Delta t$, the number of waves which enter into the region is $n_1$ while, while the number of waves leaving the region is $n_2$ with $n_1 = \Delta t / T_1$ and $n_2 = \Delta t / T_2$.

Then, the number of waves which accumulate within the region is $n_1 - n_2 = \Delta t\ (1/T_1 - 1/T_2)$. When the time interval $\Delta t \rightarrow \infty$, the number of waves accumulated within the region will be $\pm$ depending on $T_1 >/< T_2$. This is physically unrealistic. Then the only realistic possibility is $T_1 = T_2 = T$, this result holds for any depth d.

### 7.4. Tsunami Wave Velocity, Wavelength and Period

Classical theory assumes a rigid seafloor overlain by an incompressible, homogeneous, and non-viscous ocean subjected to a constant gravitational field. Linear wave theory presumes that the ratio of wave amplitude to wavelength is much less than one. By and large, linearity is violated only during the final stage of wave breaking and perhaps, under extreme nucleation conditions. In classical theory, the phase velocity $c(\omega)$, and group velocity $u(\omega)$ of surface gravity waves on a flat ocean of uniform depth d are:

$$c(\omega) = \sqrt{\frac{gd}{k(\omega)d}\tanh[k(\omega)d]} \tag{8}$$

and,

$$u(\omega) = c(\omega)\left[\frac{1}{2} + \frac{[k(\omega)d]}{\sinh[k(\omega)d]}\right] \tag{9}$$

Here $k(\omega)$ is the wave number associated with a sea wave of frequency $\omega$. Wave number connects to wavelength $\lambda(\omega)$ as $\lambda(\omega)=2\pi/k(\omega)$. Wave number also satisfies the relation:

$$\omega^2 = gk(\omega)\tanh[k(\omega)d] \tag{10}$$

$c(\omega)$, $u(\omega)$, and $\lambda(\omega)$ vary widely, both as a function of ocean depth and wave period. Waves whose velocity or wavelength varies with frequency are called ‘dispersive’.

## 8. Tsunami Simulation of 26th December, 2004 Event

The tsunami generation which includes four major processes; initiation, split, amplification and run-up can be numerically modeled and compared with the available actual tsunami event data to understand the accuracy of the numerical modeling procedures and subsequently use these numerical methods to develop a tsunami evacuation maps for future disaster mitigation purposes. Here the simulations were carried out by using the AVI-NAMI (computer program developed by C++ programming language and developed/distributed under the support of UNESCO) tsunami modeling program. This chapter consists of the simulation results of the Indian ocean Tsunami 2004 ($M_w$= 9.1) concerning the effects around Sri Lankan island.

### 8.1. Analytical Model Data

The 3rd largest tsunamigenic earthquake of $M_w$ = 9.1 occurred off the west coast of Northern Sumatra on 26th December 2004. There, in this mega tsunami event, the tsunami generation occurred by two major fault segments. But due to the limitation of the program (only one fault segment is permitted), the following data, as shown in Figure 1(a) have been used as the seismic fault data to initiate the seismic event and to compute the best results in the water level elevations along the coastal belt of Sri Lankan island. The Figure 1(b) shows the initial vertical sea floor offset due to the initiation of this seismic event. The Figure 1(c) shows the 21 gauge points, which were used to obtain the water level elevations arround the Sri Lankan island during this simaulation process and three gauge points have been selected for each location. The locations are denoted as: J– Jafna, T – Trincomalee, K – Kalmunai, Y – Yala, H – Hambantota, G – Galle and C – Colombo respectively.

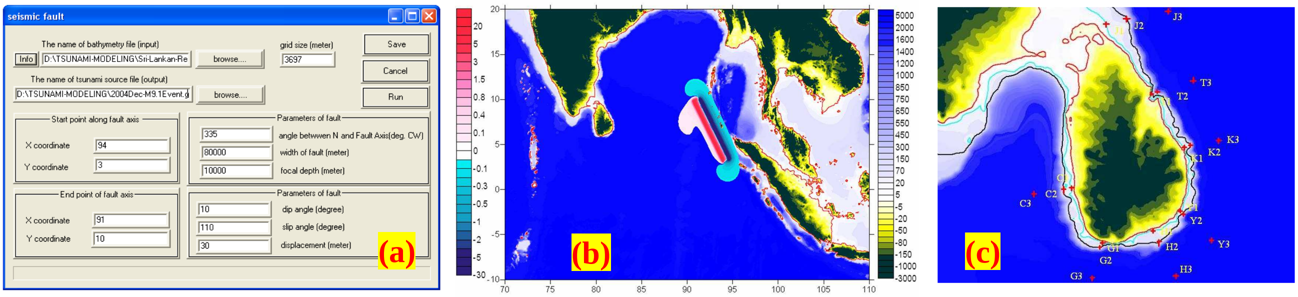



Figure 1: (a) AVI-NAMI data input file, (b) initial vertical sea floor offset for the 26th December 2004 event and (c) gauge point locations around Sri Lanka used for the simulation

### 8.2. Numerical Simulation Results

The Figure 2 shows the maximum and minimum water level elevations due to the simulation process of the above mentioned event of $M_w$ = 9.1 Indian Ocean tsunami, 26th December 2004. Further, the Figure 3 shows the propagation of the tsunami wave and the sea states at different instants. Those results show that the first wave reach to the Sri Lankan island took about 105 min

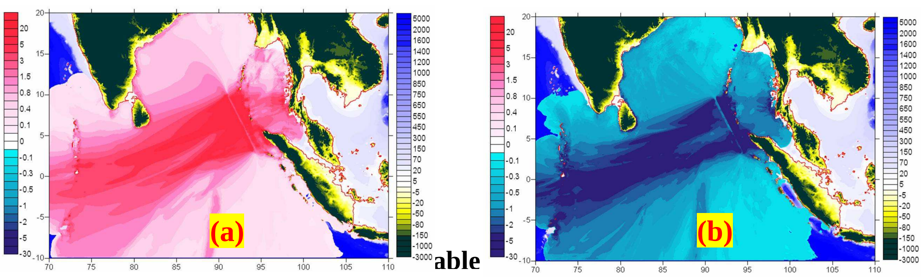



Figure 2: Maximum (a) & minimum (b) water level elevations for the 26th December 2004 event

from the time of the fault rupturing near Sumatra, which is confirmed by the actual available data as well. From observations we can clearly see that the waves reach to Yala, Hambantota and Galle took about 110-120 minutes and that the western coasts was affected after about 150 minutes when we see the waves in Colombo. Also, it can be seen that there are two significant waves attacking the Sri Lankan north, eastern and south coasts and that a third wave reached the western coasts which reflected from the Maldives islands, and this was confirmed by many eye witnesses in those areas as well. So, these factors show that the predicted results are accurate enough and acceptable and hence they can be used for tsunami inundation modeling in which tsunami propagation results are continued on to shore using detailed local bathymetry and topography.

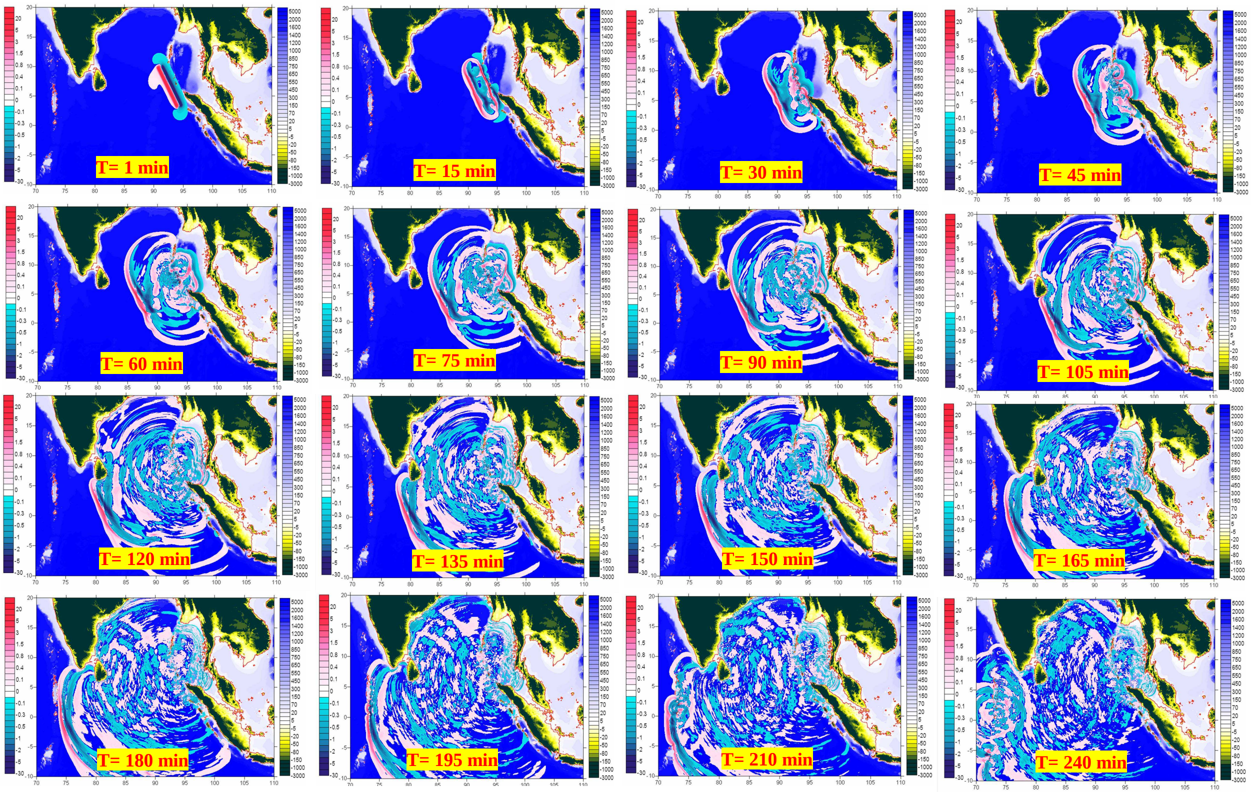



Figure 3: Sea states at different instants for 26th December, 2004 event– ($M_w$=9.1)

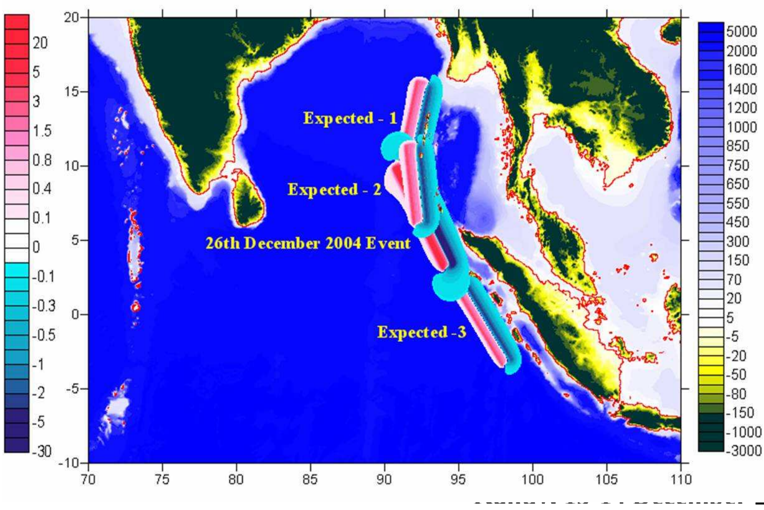
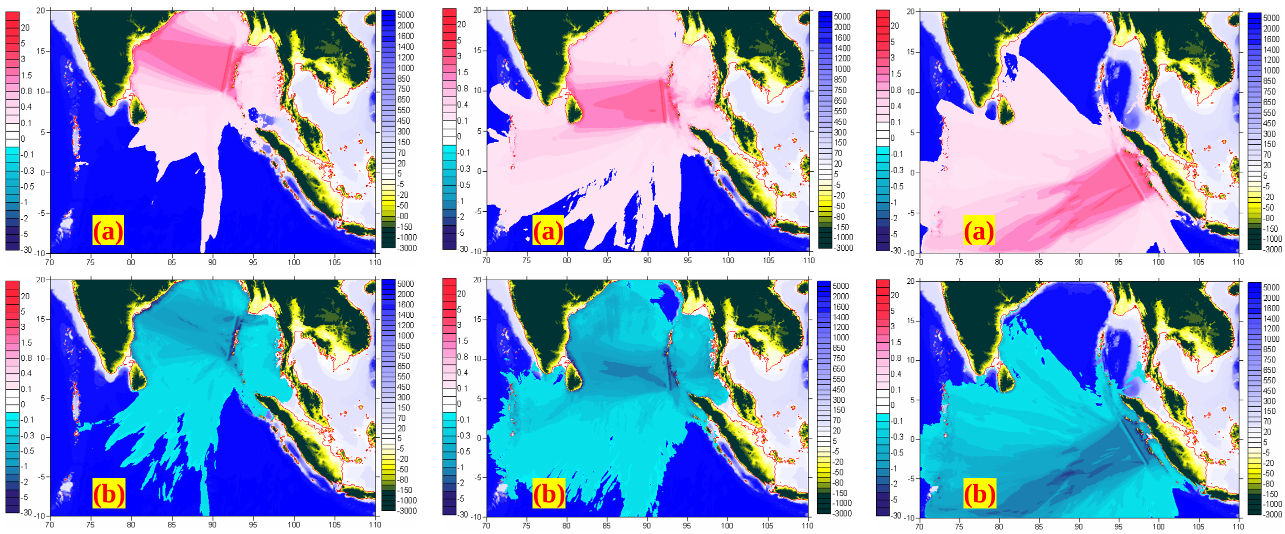



Figure 4: Four major different tsunamigenic scenarios

## 9. Simulation of Probable Tsunami Events

The seismic activities and historical records of great Sumatra-Andaman fault area were studied to identify the probable tsunamigenic scenarios in this region. The 1st possible scenario was identified close to the top edge of the Sumatran fault as there is a risk for the occurrence of an earthquake in that region (where energy is still to be released). And the 2nd probable tsunamigenic scenario was identified at the middle part of the Sumatran fault. But, the probability of occurrence this expected event-2 seems very low since most of the energy in this region was released due to the 2004 December and 2005 March events. According to historical records, there is a high risk of occurring of the 3rd probable tsunamigenic scenario at the most south part of the Sumatran fault, where the 1833 earthquake can repeat once again in near future. Hence, the simulation process for the above mentioned three probable tsunami events also were carried out to clarify their effects and the possible damages to Sri Lankan island and its surrounding nations. Considering the seismic records of this area, all these three probable events were modeled with an earthquake of $M_w = 8.8$ for each event. The Figure 4 shows the initial vertical sea floor offset due to those 3 probable tsunamigenic events and the 26th December,2004 event. The Figure 5 shows the maximum and minimum water level elevations 3 probable events. Further, the expected wave height and the wave arrival times of 1st leading wave and peak wave for all 3 probable events are listed in Tables 1 and 2 respectively.




Probable event-1 Probable event-2 Probable event-3

Figure 5: Maximum (a) and minimum (b) water level elevations for 3 probable tsunami events

Table 1: Expected wave height and wave arrival time of 1st leading elevation wave

| City | Expected 1st wave height (m) [At 25 m deep water] | | | Expected 1st leading elevation wave arrival time | | |
|---|---|---|---|---|---|---|
| | Event-1 | Event-2 | Event-3 | Event-1 | Event-2 | Event-3 |
| Jafna | 0.676 | 2.137 | - | 1 hr 51 min | 1 hr 56 min | - |
| Trincomalee | 0.240 | 2.796 | 0.295 | 1 hr 32 min | 1 hr 46 min | 2 hrs 30 min |
| Kalmunei | 0.431 | 2.486 | - | 1 hr 40 min | 1 hr 47 min | - |
| Yala | 0.494 | 2.381 | 0.323 | 1 hr 46 min | 1 hr 42 min | 2 hrs 20 min |
| Hambantota | 0.318 | 1.709 | 0.329 | 2 hrs 08 min | 2 hrs 06 min | 2 hrs 24 min |
| Galle | 0.161 | 1.013 | 0.319 | 2 hrs 10 min | 2 hrs 08 min | 2 hrs 26 min |
| Colombo | 0.044 | 0.225 | 0.133 | 2 hrs 34 min | 2 hrs 30 min | 2 hrs 50 min |

Table 2: Expected maximum water level elevation and the peak wave arrival time

| City | Maximum expected water level elevation (m) [At 25 m deep water] | | | Expected peak wave arrival time | | |
|---|---|---|---|---|---|---|
| | Event-1 | Event-2 | Event-3 | Event-1 | Event-2 | Event-3 |
| Jafna | 0.676 | 2.137 | - | 1 hr 51 min | 1 hr 56 min | - |
| Trincomalee | 0.240 | 2.796 | 0.295 | 1 hr 32 min | 1 hr 46 min | 2 hrs 30 min |
| Kalmunei | 0.516 | 4.001 | - | 2 hrs 42 min | 2 hrs 20 min | - |
| Yala | 0.505 | 2.381 | 0.529 | 2 hrs 58 min | 1 hr 42 min | 3 hrs 49 min |
| Hambantota | 0.402 | 1.709 | 0.464 | 3 hrs 23 min | 2 hrs 06 min | 3 hrs 13 min |
| Galle | 0.235 | 1.112 | 0.505 | 3 hrs 22 min | 3 hrs 18 min | 3 hrs 34 min |
| Colombo | 0.063 | 0.225 | 0.133 | 2 hrs 57 min | 2 hrs 30 min | 2 hrs 50 min |

Out of the three possible tsunamigenic scenario simulations, results of the first and second probable scenarios which are lying in northern Sumatra segment show that there is a huge risk of receiving devastating tsunami waves again to Sri Lankan north, eastern and south coasts if there will be an earthquake in that region. But the third probable tsunamigenic scenario (which lies on the southern Sumatra segment) simulation results show that the risk is low to receive big tsunami waves to Sri Lankan island and its surrounding nations.

We know from the historical records that some great earthquakes have occurred repeatedly in the same region: $M_w$=8.5 earthquake of 2005 occurred at the rupture zone of the $M_w$=8.7 earthquake of 1861, and the rupture zone of the 1833 $M_w$=8.7 earthquake encompassed the 1797 $M_w$=8.2 earthquake rupture zone. Though smaller tsunamigenic earthquakes of magnitude 7.5 to 8.0 have occurred more frequently, at intervals of over a few decades, like 1907 and 1935, major earthquakes occurred near the 1861 source zone. From these considerations the probability of a severe tsunami hitting Sri Lanka within a couple of decades from Andaman–northern Sumatra region appears to be low, since this area has already produced the 2004 and 2005 great earthquakes. The southern Sumatra segment is a potential zone for a great earthquake. However, Sri Lanka does not lie perpendicular to the fault in this part of the trench. Hence, damage due to tsunami from such event may not be substantial in Sri Lanka.

## 10. Conclusions

The numerical simulation of the Indian ocean Tsunami 2004 event of $M_w$ = 9.1 and the other three expected tsunamigenic scenarios were simulated with a magnitude of $M_w$ = 8.8. The findings from this study can be summarized as:

(1) The simulation results of the 26th December, 2004 mega tsunami event and the available data have very good agreeability and hence this analytical procedure can be used to obtain realistic results that can be reliably used to develop evacuation maps used to ensure public safety from tsunami.

(2) The simulation results of the other three probable tsunami events and the historical earthquake records of the great Sumatran fault area and its orientation fathomed that damage due to such tsunami event, triggered from that region may not be substantial to Sri Lankan island for couple of decades.

But, it is necessary to state that further investigation on the current seismic events on this region and other possible tsunamigenic scenarios around Sri Lankan island to be studied in detail in future to establish a more accurate decision. In any case as Sri Lankan island is located far enough from the destructive tsunamigenic plate boundaries, accurate and well timing warning can avoid that Sri Lankan people will experience another agony as we had on 26th December 2004.

## 11. Acknowledgements

The authors would like to express gratitude to Prof. Laura Kong (Director, UNESCO-IOC),Prof. Emile Okal, Prof. Costas Synolakis, Prof. Ahmet Yalciner and Dr. Andrey Zaytsev for giving this great opportunity, sponsoring and excellent guidance during the International Training workshop conducted by UNESCO-IOC at Oostende, Belgium and providing the AVI-NAMI tsunami simulation program for this research work. Further, our heartiest compliments goto Prof. Charles L. Mader (Director, Tsunami society, Holonolu, Hawaii), Dr. Nimal Wijerathne (Senior Lecturer, University of Ruhuna, Sri Lanka), Prof. Charitha Pattiaratchi (Senior Professor, University of Western Australia), Dr. K. Arulanathan and Mr. S.U.P. Jinadasa (NARA, Sri Lanka) for their kind contribution for this research.

## References

Airy, G. B., (1845), "Tides and Waves", *Encyclopedia Metropolitan*, London, pp. 241-396.

Borrero, J., (2005), "Learning from Earthquakes, the Great Sumatra Earthquake and Indian Ocean Tsunami December 26, 2005, Field Survey on Northern Sumatra", Report 1 EERI, Special Earthquake Report.

Costas, E. Synolakis, Earthquake Engineering Handbook, Chapter 9, pp 1-90

Imamura, F., Yalciner, A. C. and Ozyurt, G., (2006), "Tsunami Modeling Manual", pp 1-71.

Kulikov, E. A., Medvedev, P. P. and Lappo, S. S., (2005), "Satellite Recording of the Indian Ocean Tsunami on Dec. 26, 2004", Doklady Earth Sciences, Vol. 401A, No.3, pp 444–448.

Prager, E.J., (1999), "Furious Earth", McGraw-Hill, New York.

Tinti, S., Armigliato, A., Zaniboni, F., Manucci, A., Tonini, R., Pagnoni, G., Gallazzi, S. and Beatriz, B., "December 26, 2004 Sumatra Tsunami", Tsunami Research Team: University of Bologna. http://labtinti4.df.unibo.it/site/simulation_sumatra.php

UNESCO IOC-IGC/ITSU Website: http://ioc.unesco.org/itsu/

# COMPARATIVE PERFORMANCE OF PERMEABLE AND POROUS PAVEMENTS

Niranjali Jayasuriya[1] and Nilmini. Kadurupokune[2]

[1]Senior Lecturer, School of Civil Environmental and Chemical Engineering, RMIT University, Melbourne,
nira.jayasuriya@rmit.edu.au
[1]Telephone: +61-3-99253795; Fax: + 61 3 96390138

[2]Civil Engineer, Waterways and Coastal, GHD Pvt Ltd, Brisbane
nilmini.kadurupokune@rmit.edu.au
[2]Telephone: +61-7-33163000; Fax: + 61-7- 33163333

**Abstract:** The traditional approach to stormwater management is based on the development of urban drainage networks to convey stormwater away from developed areas quickly. With the increase in impermeable areas due to urban development, the quantity of stormwater runoff is significantly increased, overloading existing infrastructure. Pollutants carried by stormwater to receiving waters are also a major concern. Pervious pavements in car parks and driveways have potential to reduce peak discharge and the volume of runoff flowing into urban drains and improve runoff water quality by trapping the sediments in the infiltrated water. The paper focuses on presenting results from field tests carried out in Melbourne, Australia to evaluate reductions in peak discharge and the volume of stormwater after infiltrating through pervious pavement surfaces. The current study examines two types of pervious pavement surfaces: namely C&M Ecotrihex pavers and Atlantis turf cells and compares their performance against a conventional asphalt paved car park. Considerable reduction in peak discharges, runoff volumes, pollutant concentrations and loads were obtained from field tests. These reductions reduce the stresses on hydraulic infrastructures and on the ecosystem of the receiving waters. Furthermore, both C&M Ecotrihex and Atlantis turf cell surfaces reduces the lag-time by at least one hour to the peak discharge compared to the asphalt surface. This would further reduce the pressure on the infrastructure during a big storm event.

**Keywords:** Pervious pavements, Stormwater management, Stormwater pollutants, Water Sensitive Urban Design, Stormwater quality improvement, Stormwater peak reductions

## 1 Introduction

Urbanization has had a substantial impact on stormwater quantity and quality. The increase in impermeable area causes the quantity of runoff to significantly increase, stretching the hydraulic capabilities of stormwater infrastructure. Pollutants carried by stormwater to receiving waters are also a major concern. Water Sensitive Urban Design (WSUD) is a practice that is emerging throughout the world as a cutting edge initiative adopted to deliver improved receiving water health and where possible, contribute to producing fit-for-purpose water for use in urban communities. WSUD is a structural initiative that is used for a given set of conditions to reduce the quantity and improve the quality of stormwater runoff in the most cost-effective manner. Pervious pavements have been identified in Australia as a successful element of the WSUD concept.

Pervious pavements in car parks and driveways have potential to reduce peak discharge and the volume of runoff flowing into urban drains and improve runoff water quality by trapping the sediments in the infiltrated water. As a result, the risk to the health of rivers and natural water bodies from pollutants such as suspended solids, phosphorous, nitrogen, heavy metals, oils and hydrocarbons will be reduced. Researchers (Booth et al., 2003; Thomson and James, 1995; Lerget et al., 1996) noted a significant reduction in runoff volume and improvements to water quality parameters when using pervious pavements instead of conventional impervious asphalt type pavements. However, it has been shown that the design and application of pervious pavements is site specific and hence, the need for research on pervious pavements still remain as an urgent need to accelerate the practical adoption of them in the field.

The key objectives of the study reported herein are to quantify the amount of stormwater captured through pervious pavement infiltration and quantify improvements to the quality of stormwater infiltrated through the pervious pavement. The paper focuses on presenting results from field tests carried out in Melbourne, Australia to evaluate reductions in peak discharge and volume of stormwater after infiltrating through pervious pavement surfaces. The current study examines two pervious pavement types: namely C&M Ecotrihex pavers and Atlantis turf cells and compares the performance against a conventional asphalt paved car park.

## 2 Pervious Pavement Types

Pervious pavements can be classified as either porous pavements or permeable pavements. Although both types of pavements strive to achieve the same benefits, they differ considerably in the way they operate and in their appearance Jayasuriya et al. (2005).

The classification of pervious pavements depends on the surface layout and the surface layer materials. There is a significant difference between porous and permeable pavements. According to Zhang et al. (2006), Argue and Pezzaniti (2005) defined porous pavements and permeable pavements as follows. A porous pavement is a thick porous layer with a strong infiltration capacity. A porous pavement contains a grass or gravel surface with a well compacted graded sand and gravel base. On the other hand, a permeable pavement surface is normally constructed by impervious paver concrete blocks with infiltration voids between the blocks. Infiltration capacities of permeable pavements are high due to the coarse aggregate between concrete blocks.

A number of issues need to be resolved before adopting pervious pavements for managing stormwater. Durability of the pavement material, stormwater permeability of the pavement, traffic load capacity, and maintenance of the pavement are some of the issues that need resolving prior to use of pervious pavements. More laboratory and field tests are needed before pervious pavements are accepted by practicing engineers, regulators and councils.

A typical sub-base of a conventional pavement consists of Class 1 sub-base material with large fines content (VicRoads, 1997). This gives the pavement its strength and stiffness but is adversely affected when the sub-base is in contact with water. Pervious pavements require a single size grading (or open graded) to give open voids. Although it will have a lower stiffness than Class 1 material, stiffness will not be significantly reduced by the presence of water within it provided there is sufficient friction between particles when saturated. The choice of materials for use in capping and sub-base layers below pervious pavements is therefore a compromise between stiffness, permeability and storage capacity (Jayasuriya et al., 2005).

## 3 Experimental Site

A car park was built with two pervious surfaces namely C&M Ecotrihex pavers (permeable) and Atlantis Turf cells (porous) and an impervious asphalt surface as a control at the Centre for Education and Research in Environmental Strategies (CERES), in Melbourne, Australia. Jayasuriya and Kandurupokune (2008) describe the experimental setup at CERES in detail. The size of the study area was 229m$^2$ (18m*13m). Agricultural (aggi) pipes were placed around the catchment to prevent stormwater from the surrounding areas entering the experimental site and the three surfaces subject to experimentation (Figure 1). A subsurface geo-membrane structure was introduced to mitigate lateral water flow between experimental sites. Each surface type will consist of 2 car park/entry spaces of 50m$^2$ (5m*10m). Stormwater will flow through the pervious surface and subsurface media and be drained to the outlet via a geo-sock protected perforated header pipe. Details of the pavement design and structure of the pavements were reported in Jayasuriya et al. (2006a) and Kadurupokune and Jayasuriya (2007).

Three on-line flow meters were installed to measure the surface flow from the control surface (asphalt) and infiltrated water from the two pervious pavement surfaces. The flow meters were calibrated to activate when the depth of water in the channel was 10 mm. Three water quality auto samplers were also installed in special pits in the field to collect event based water quality data. One sampler will collect surface runoff from the impervious surface (control surface). Stormwater from the control pavement surface will flow to the 300 mm*400 mm channel. Another two samplers will be collecting the water infiltrated through the Ecotrihex and the turf cells respectively. Samples will be collected from the drainage pipes laid on the base layer. The stormwater infiltrated through the pervious concrete blocks and the turf cells are captured via a properly designed drainage system and the water is diverted to a small onsite dam for productive recycling use in the future. The quality of the infiltrated water in the car park through the pervious material will be monitored and benchmarked against the surface water quality flowing through the conventional car park. The water quality parameters monitored included Total Suspended Solids (TSS), Total Nitrogen (TN), Total Phosphorus (TP), Oil and Grease, Lead (Pb), Copper (Cu), Zinc (Zn) and Cadmium (Cd).

Rainfall data were collected from a Tipping Bucket rain gauge installed at the site. These rainfall values were compared with the nearby continuous rain gauges operated by Melbourne Water (within a 4 km radius) to determine the rainfall pattern and the intensity of the rain event at the experimental site.

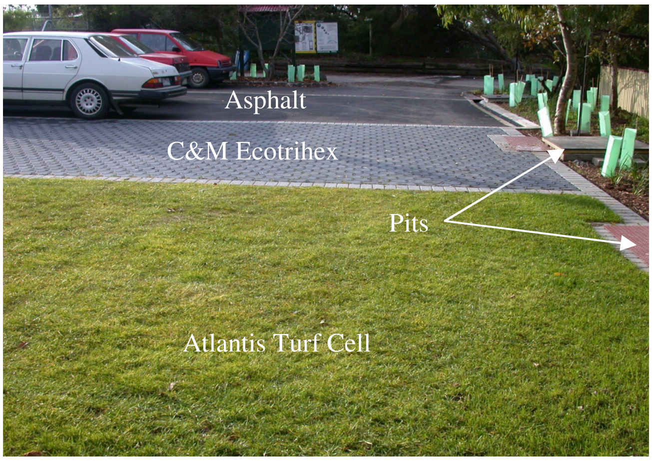



Figure 1 Experimental car park at CERES, Melbourne, Australia; (Jayasuriya & Kandurupokune, 2008)

## 4 Water Quantity Analysis

The City of Melbourne received below average rainfall for the last fourteen years (1996 – 2009). It was possible to collect stormwater only from 7 storms during the experimental period in 2006 and 2007. Natural storms that produced runoff are presented in Tables 1 and 2. The storms that occurred on 02 November 2006, 24 March 2007 and 18 May 2007 did not produce any runoff from the turf cells. The minimum rainfall required to produce runoff from Ecotrihex and turf cells surfaces are 13mm and 18mm respectively. As expected, asphalt pavement produced runoff with a minimum rainfall of 3mm. The runoff hydrograph produced on 12$^{th}$ July 2007 is given in Figure 2. The area under the hydrograph gives the total runoff produced from each storm.

The detailed hydrograph information obtained for a number of rain events are shown in Table 1. This Table gives the % reduction in runoff from C&M Ecotrihex surface and Atlantis turf cells when compared to runoff from the asphalt surface. Table 1 also present the % reduction of peak and total runoff obtained from the two pervious pavements.

From the hydrographs, it is clear that the runoff volumes and peak discharges from the pervious pavements are much lower than from the asphalt pavement. As presented in Table 1 and Figure 2, the average percentage reduction in peak discharge varied between 40% to 55% for C&M Ecotrihex pavement and 45% to 60% for the Atlantis turf cell pavement. C&M Ecotrihex pavement reduced the runoff volume by 43% to 53% whilst Atlantis turf cell pavement reduced the total runoff between 52% to 62%. From the values presented in the above table and the figure, it is concluded that pervious pavements are effective in managing stormwater flow. The water that is retained within the pavement structure will evaporate back to the atmosphere. Hogland et al. (1987; 1990), Larson (1990), Mantle (1993) and Pratt et al., (1989; 1990; 1995) reported that the percentage reduction in the runoff volume through pervious pavements is between 34% to 47%. It is clear that the results obtained from C&M Ecotrihex are within the range or better than the values observed or reported by previous researchers.

Table 1 Comparison of the two hydrographs from Asphalt and pervious surfaces

| Date | Total rainfall (mm) | Difference in time in commencement of runoff (min) | | Difference in time to peak (min) | | Reduction in peak discharge (%) | | Reduction in runoff volume (%) | |
|---|---|---|---|---|---|---|---|---|---|
| | | C&M | Turf cell | C&M | Turf cell | C&M | Turf cell | C&M | Turf cell |
| 2/11/2006 | 13 | 300 | N/A | 50 | N/A | 50 | N/A | 51.0 | N/A |
| 24/3/2007 | 14 | 39 | N/A | 48 | N/A | 53 | N/A | 42.0 | N/A |
| 18/5/2007 | 15 | 330 | N/A | 25 | N/A | 44 | N/A | 43.9 | N/A |
| 27/6/2007 | 18 | 90 | | 35 | | 55 | | 46.8 | |
| 13/7/2007 | 20 | 150 | | 85 | | 37 | | 44.5 | |
| 4/11/2007 | 24 | 150 | | 85 | | 48 | | 53.4 | |
| 21/12/2007 | 25 | 210 | | 165 | | 42 | | 46.3 | |

## 5 Water Quality Data Analysis

The reduction in TN, TP, Oil and greases, TSS and Heavy metals (Pb, Cd, Cu, and Zn) concentrations were investigated analyzing the stormwater infiltrated through both pervious surfaces. In order to obtain the pollutographs, water samples were collected as soon as the flow commenced from the drainage pipes installed within the pavements. The first three samples were collected at 15 minute intervals followed by samples every 30 minutes until the outflow ceases. The collected samples were preserved and analyzed at the laboratory. The pollutograph was also drawn for all pollutants for each storm event. Event Mean Concentration (EMC) was calculated using Equation 1.

$$EMC = \frac{\sum_{t=0}^{T} Q_t C_t}{\sum_{t=0}^{T} Q_t} \quad (1)$$

where,

EMC = Event mean concentration of a particular water quality parameter (mg/L)

$Q_t$= Discharge at a given time t (L/s)

$C_t$= Concentration of the water quality parameter at time t (mg/L)

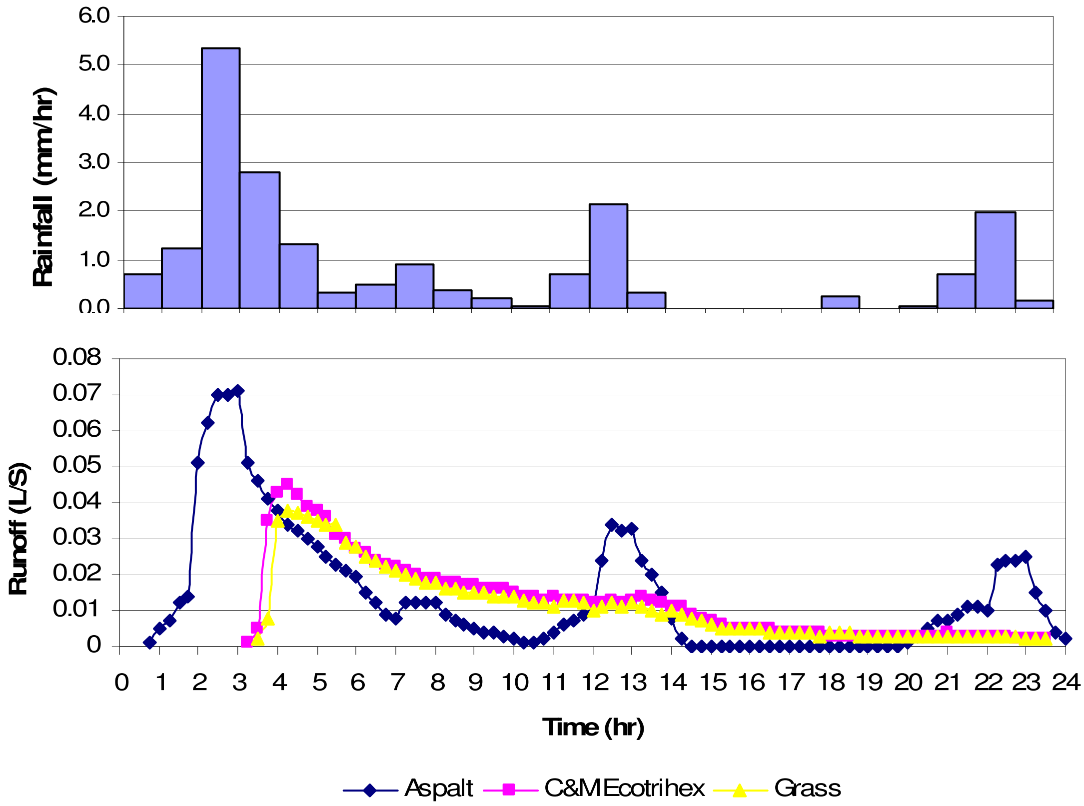



Figure 4.25 Runoff hydrographs produced by the Asphalt, C&M Ecotrihex and Atlantis turf cell pavements from 12th July 2007 storm event

T = Time base of the hydrographs

Tables 2 depicts the EMC of the water quality parameters from the runoff water collected from the asphalt, C&M Ecotrihex and Atlantis turf cell surfaces for the actual storm occurred on 12 July 2007. Tables 3 and 4 depict the removal efficiencies of pollutant concentrations and loads for all the pollutants from the actual storms. The removal efficiencies for Cd and Pb are not reported in Tables 3 and 4 as the concentrations and loads for the above two variables were below the detectable levels.

The removal efficiencies of water quality parameters from C&M Ecotrihex and Atlantis turf cell pavements were calculated by comparing concentrations and loads from the asphalt pavement. On average TSS, TP, Zn and oil concentrations reduced between 87% to 92%, 55% to 62%, 76% to 93% and 89% to 92% respectively from the C&M pavement and 90% to 92%, 50% to 56%, 91 to 92% and 93% to 94% from the Atlantis turf cell pavement. The removal efficiency of Cu is around 40 % and 60% from C&M Ecotrihex and Atlantis turf cell pavements respectively. However, the initial TN concentration levels from the water infiltrated through both pervious pavement types were very high. This was discovered to be due to reducing flow values and leaching of TN from the subbase of the pavement.

Table 2 Event Mean Concentrations (EMC) of the water quality parameters from the storm occurred on 12 July 2007

| Parameters | Asphalt (mg/L) | C&M (mg/L) | Atlantis (mg/L) | Removal Efficiency (%) | |
|---|---|---|---|---|---|
| | | | | C&M | Atlantis |
| TP | 0.936 | 0.356 | 0.41 | 62.0 | 56.2 |
| TN | 7.1 | 1.943 | 2.83 | 72.6 | 60.1 |
| Zn | 0.076 | 0.0051 | 0.006 | 93.4 | 91.2 |
| Cu | 0.0589 | 0.0351 | 0.0238 | 40.4 | 59.6 |
| TSS | 134.3 | 10.3 | 13.2 | 92.3 | 90.2 |
| Oil | 624 | 61.6 | 42.4 | 90.1 | 93.2 |
| Cd | BDL | BDL | BDL | BDL | BDL |
| Pb | BDL | BDL | BDL | BDL | BDL |

BDL – Below Detectable Level

Table 3 Removal Efficiencies of pollutant concentrations

| Date | % Removal Efficiency in Concentrations | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Zn | | TN | | Cu | | TP | | TSS | | Oil | |
| | C&M | Turf | C&M | Turf | C&M | Turf | C&M | Turf | C&M | Turf | C&M | Turf |
| 02/11/06 | N/A | N/A | N/A | N/A | N/A | N/A | 50 | N/A | 88 | N/A | 80 | N/A |
| 24/03/07 | 76 | N/A | 37 | N/A | -21 | N/A | 55 | N/A | 93 | N/A | 88 | N/A |
| 18/05/07 | 91 | N/A | 40 | N/A | 54 | N/A | 56 | N/A | 89 | N/A | 89 | N/A |
| 27/06/07 | 89 | 91 | 38 | 55 | 73 | 52 | 62 | 50 | 90 | 94 | 89 | 92 |
| 13/07/07 | 93 | 92 | 40 | 60 | 73 | 60 | 62 | 56 | 90 | 93 | 92 | 90 |
| 04/11/07 | 90 | 91 | 43 | 52 | 74 | 66 | 57 | 63 | 94 | 93 | 91 | 90 |
| 21/12/07 | 92 | 92 | 43 | 47 | 63 | 69 | 56 | 60 | 94 | 95 | 91 | 91 |

Table 4 Removal Efficiencies of pollutant loads

| Date | % Removal Efficiency in Load | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Zn | | TN | | Cu | | TP | | TSS | | Oil | |
| | C&M | Turf | C&M | Turf | C&M | Turf | C&M | Turf | C&M | Turf | C&M | Turf |
| 02/11/06 | N/A | N/A | N/A | N/A | N/A | N/A | 75 | N/A | 94 | N/A | 90 | N/A |
| 24/03/07 | 86 | N/A | 64 | N/A | 30 | N/A | 74 | N/A | 96 | N/A | 93 | N/A |
| 18/05/07 | 95 | N/A | 66 | N/A | 74 | N/A | 75 | N/A | 94 | N/A | 94 | N/A |
| 27/06/07 | 94 | 96 | 67 | 80 | 86 | 78 | 79 | 78 | 95 | 97 | 94 | 96 |
| 13/07/07 | 96 | 96 | 67 | 81 | 85 | 81 | 79 | 79 | 95 | 97 | 96 | 95 |
| 04/11/07 | 95 | 96 | 74 | 80 | 88 | 86 | 80 | 85 | 97 | 97 | 96 | 96 |
| 21/12/07 | 96 | 97 | 69 | 78 | 80 | 87 | 76 | 83 | 97 | 98 | 95 | 96 |

## 6 Conclusions

A car park with three different surfaces (two pervious surfaces and one impervious surface as a control) was newly constructed at the Centre for Education and Research in Environmental Strategies (CERES) at Brunswick in Melbourne. The experimental site was fully automated with flow meters and autosamplers to collect flow and stormwater quality data.

Reduction in peak flow from the two pervious surfaces compared to the asphalt surface was comparable with previous studies. The percentage reduction in peak discharge and runoff volume from pervious pavements varied between 45% to 55% and 50% to 60% respectively when compared to the conventional asphalt pavement. From above results, it could be stated that both pervious pavements are effective in managing stormwater flow, although the Atlantis turf cell pavement facilitated more infiltrated water than the C&M Ecotrihex type pavement. The water that is retained within the pavement structure will evaporate back to the atmosphere. The reduction of peak discharge and volume reduces the surcharging stresses on hydraulic infrastructures. Furthermore, both C&M Ecotrihex and Atlantis turf cell surfaces effected at least one hour lag time on the peak discharge compared to the asphalt surface. This would further reduce the pressure on the infrastructure during large storm events.

The results obtained from the field experiments clearly prove that pervious pavements have an ability to reduce peak discharges while filtering the pollutants generated by urban stormwater. Consistent water quality improvements were observed from this pervious paver study, with reductions in TSS, TP and TN of around 70% to 100%, 40% to 80% and 60% to 80% respectively confirming findings by previous researchers. Clogging of pervious pavements and the maintenance required to facilitate continual efficient performance are two areas that require further study.

## 7 References


1. Argue R.J. and Pezzaniti D. (2005); Porous and permeable pavng: background and design issues. 29th Hydro and Water Resources Sym. Engineers Australia. Electronic Copy
2. Booth, D.B., J. Leavitt, and K. Peterson, 2003, *The University of Washington permeable pavement demonstration project*, Centre for Urban Water Resources Management, University of Washington, Seattle.
3. Hogland W., Niemczynowicz J. and Wahlman T., 1987, The Unit Superstructure during the construction period. *The Science of the Total Environment,* 59, pp.411-424.
4. Hogland W., Larson M. and Berndtsson R., 1990, The pollutant build-up in pervious road construction. *Proc.* 5th Int. Conf. on Urban Storm Drainage, Osaka, Japan, pp.845-852.
5. Jayasuriya N., Zhang J., Setunge S. and Furniss J, 2005, Improved Stormwater Management by Pervious Pavements; *29th Hydrology and Water Resources Symposium , Engineers Australia,* Electronic Copy
6. Jayasuriya N, Zhang J., Jarrar A.and Setunge S, 2006a, Sustainable Stormwater Management Using Pervious Pavements; 3rd APHW Conference On Wise Water Resources Management Towards Sustainable Growth And Poverty Reduction 16th – 18th October 2006 Bangkok, Thailand (Electronic Copy)
7. Jayasruiya N. and Kandurupokune N., (2008); Impact of pervious pavements on drainage infrastructure; Proceedings of 11th ICUD, Melbourne, 31st of August to the 5th of September 2008; CD copy
8. Kadurupokune N., Jayasuriya N., (2007), Field testing the effectiveness of pervious pavements as a water sensitive urban design initiative, Proceedings of Rainwater and Urban Design 2007, Sydney, Australia 21-23 August 2007
9. Larson R., 1990, Swedish experience with design and construction of pervious asphalt constructions. *Proc.* Standing Conf. on Stormwater Source Control, Volume I, Ed. C.J. Pratt, Coventry University, ISBN 0 905949 14 5.
10. Legret M., Colandini V. and Le Marc C., 1996, Effects of a porous pavement with reservoir structure on the quality of runoff water and soil. *The Science of the Total Environment*, 189-190, pp.335-340.
11. Mantle J.D.G., 1993, *Onsite Reduction and Attenuation of Urban Stormwater Runoff.* Unpublished Thesis, Nottingham Trent University.

12. Pratt C.J., Mantle J.D.G. and Schofield P.A., 1989, Urban stormwater reduction and quality improvement through the use of permeable pavements. *Water Science and Technology*, 21,pp 769-778.
13. Pratt C.J., 1990, Permeable pavements for stormwater quality enhancement. In: *Urban Stormwater Quality Enhancement – Source Control, Retrofitting and Combined sewer Technology,* Ed. H.C. Torno, ASCE, ISBN 0 87262 759 4, pp.131-155.
14. Pratt C.J., Mantle J.D.G. and Schofield P.A., 1995, UK research into the performance of permeable pavement, reservoir structures in controlling stormwater discharge quantity and quality. *Proc.*2nd Int. Conf. on Innovative technologies in Urban Storm Drainage, Lyon, France, pp.337-344.
15. Thompson, M.K. and James, W. (1995): Provision of parking lot pavements for surface water pollution control studies. Ch. 24 in: Modern Methods for Modeling the Management of Stormwater Impacts. Proc of the Stormwater and Water Quality Modeling Conf., Toronto, Mar. 3-4 1994. Pub by CHI, Guelph. ISBN 0-9697422-4-X. pp. 381- 398.
16. VicRoads,1997, *Investigation into The Use of Recycled Crushed Concrete For Road Base Use*. Vic Roads, Melbourne Australia
17. Zhang J., Jayasuriya N. and Setunge S., 2006, Application of pervious pavements a laboratory scale study, *7th Urban Drainage Modelling and the 4th Water Sensitive Urban Design, Melbourne, Australia*, 2nd - 7th April, pp 227-234.

## 8 Acknowledgements

This project has been assisted by funding from the Victorian Government through the Department and Environment as part of the Stormwater and Urban Water Conservation Fund. Assistance provided by Keith Jesse from the Centre for Education and Research in Environmental Strategies (CERES) for conducting this study is greatly appreciated. The authors also wish to acknowledge senior management at C&M *Ecotrihex* and Atlantis pavement manufactures for providing the paving material and Melbourne Water for providing the auto-samplers for the study.

**About the Authors**

**Niranjali Jayasuriya,** B.Sc. Hons. Sri Lanka, M,Sc (Eng) AIT, Ph.D. Melboure., Member (EA), C.Eng. is a Senior Lecturer at the School of Civil, Environmental and Chemical Engineering, RMIT University, Australia. Her research interests are in the areas of Stormwater Management, Water Conservation. Rainfall Runoff modeling and Water Recourses.

**Nilmini Kandurupokune,** B.Sc. Sri Lanka, M.Eng. RMIT University, Melbourne. She is currently working in Waterways and Coastal service group as a Civil Engineer at GHD Pvt Ltd, Brisbane, Australia. Her expertise are in WSUD, urban and rural drainage design, hydraulic and hydrology modeling.

# Insight into the structural and optoelectronic properties of e-beam evaporated nanostructured $TiO_2$ thin films annealed in air

**Abdul Faheem Khan[1], Mazhar Mehmood[1], A. M. Rana[2], M. Ashraf[3], A. Mahmood[4], S.K. Durrani[5]**

[1]National Centre for Nanotechnology & Department of Chemical and Materials Engineering, Pakistan Institute of Engineering and Applied Sciences (PIEAS), Islamabad 45650 , Pakistan.

[2]Department of Physics, Bahauddin Zakariya University, Multan 60800, Pakistan

[3]Optics Laboratories, P. O. Nilore, Islamabad, Pakistan.

[4]NILOP, Islamabad 45650, Pakistan

[5]P.O. Nilore, Materials Division, PINSTECH

# corresponding author (M. Mehmood): mazhar@pieas.edu.pk, Phone: +92-51-2207813, Fax: +92-51-2208070; (A.F. Khan) faheem_khan_1977@yahoo.com

**Abstract**

About 480 nm thick titanium oxide ($TiO_2$) thin films have been deposited by electron beam evaporation followed by annealing in air at 300—600$^oC$ with a step of 100$^oC$ for a period of two hours. Optical, electrical and structural properties are studied as a function of annealing temperature. All the films are crystalline (having tetragonal anatase structure) with small amount of amorphous phase. Crystallinity of the films improves with annealing at elevated temperatures. XRD and FESEM results suggest that the films are composed of nanoparticles of 25-35 nm. Raman analysis and optical measurements suggest quantum confinement effects since Raman peaks of the as-deposited films are blue-shifted as compared to those for bulk $TiO_2$. Optical band gap energy of the as-deposited $TiO_2$ film is 3.21 eV, which decreases to about 3.06 eV after annealing at 600 $^oC$. Refractive index of the as-deposited $TiO_2$ film is 2.26, which increases to about 2.32 after annealing at 600 $^oC$. However the films annealed at 500 $^oC$ represent peculiar

behavior as its band gap increases to highest value of 3.24 eV whereas refractive index, RMS roughness and dc-resistance illustrate a drop as compared to all other films. Illumination to sunlight decreases the dc-resistance of the as-deposited and annealed films as compared to dark measurements possibly due to charge carrier enhancement by photon absorption.

***Keywords:*** *Nanostructured $TiO_2$ thin films, quantum confinement, Raman spectroscopy, band gap energy, impedance spectroscopy, and atomic force microscopy.*

## 1. Introduction

Among the transition-metal oxides, $TiO_2$ is one of the most extensively studied materials. The rising interest in its applications and research in the last few years is due to its unique and outstanding structural, optical and electronic properties [1-4]. $TiO_2$ is known to exist in three polymorphic forms: anatase, rutile and brookite. The most widely used crystallographic structures (tetragonal) are anatase and rutile. Depending on the structure, the properties of $TiO_2$ vary greatly, which make them useful for many applications. The rutile phase has a direct band gap at 3.06 eV and indirect band gap at 3.10 eV [5] with high refractive index and dielectric constant (~ 80) [6,7], better thermal and chemical stability etc. Owning to such properties, the rutile phase has found applications in optical coatings, solar energy converters, storage capacitors in dynamic random access memories (DRAM), as protective layers and as dielectric layer in microelectronic applications [8-10]. On the other hand, the anatase phase exhibits direct band gap ranging from 3.77 to 3.85 eV [11] and the indirect band gap at 3.23 eV with relatively low refractive index and dielectric constant (~ 12-30) [12]. Due to its large band gap, $TiO_2$ covers only the UV range of the solar spectrum. The unique properties of anatase phase make it a candidate material for gas sensors, solar cells, dielectrics in semiconducting FETs, self-cleaning windows, anti-fogging glasses, self-sterilizing, and photo-catalytic applications [13, 14]. Contrary to crystalline phase, amorphous $TiO_2$ with high refractive index demonstrate optical isotropy and high packing density making them suitable for optical applications [8, 15]. Because of successful applications among various fields, nanostructured $TiO_2$ thin films have attracted tremendous attention due to their quantum confinement effects. Such nanostructured films are considered to be promising host materials as they possess high surface area, chemical stability, good semiconducting properties and low cost [16].

Electrical resistance of $TiO_2$ is known to alter substantially by introducing oxygen defects leading to its semiconducting behavior [11]. $TiO_2$ can be used as an electrode for high electron injection as well as multi-layered structures in photovoltaic devices to provide direct electrical pathway for photogenerated electrons to increase the rate of electron transport, which leads to a higher efficiency [17, 18].

$TiO_2$ thin films have been prepared by many techniques such as chemical vapor deposition [19], sputtering [1, 20], sol-gel [3, 21], pulsed laser deposition [22], plasma oxidation [23] etc. Electron beam evaporation provides large area, high quality films with high volume deposition rates and good adhesion to substrate and purity. In addition, the process parameters like deposition rate and thickness can be easily controlled.

In this paper structural, optical and electrical properties of e-beam evaporated nanostructured $TiO_2$ thin films have been investigated in pre- and post-thermal annealed (in air over the temperature range of 300–600 $^{o}$C) conditions using X-ray diffraction

(XRD), Raman spectroscopy, transmission spectroscopy, impedance spectroscopy (IS), field emission scanning electron microscopy (FESEM), and atomic force microscopy (AFM).

## 2. Experimental

$TiO_2$ thin films were deposited using electron beam evaporation of $TiO_2$ powder (99.99%) as a starting material onto BK7 glass substrates using tungsten crucible. The system was pumped to a base pressure of less than $10^{-5}$ mbar before deposition and $O_2$ was injected into the chamber during evaporation at a partial pressure below 2 x $10^{-4}$ mbar. The substrate was set at a temperature of $300^{o}$C and at a distance of 35 cm from the source and rotated at 30 RPM during deposition to obtain uniform and homogeneous films. The deposition parameters were optimized to reduce the film roughness. Thickness of film and rate of its deposition were controlled with the help of an *in situ* quartz crystal thickness monitor. Intended thickness of the film was about 500 nm and the deposition rate was set at 0.45 nm $s^{-1}$. However, the thickness estimated by ellipsometery is found to be 480 ± 10 nm, and is considered to be more reliable.

These e-beam deposited films were then annealed in air at various temperatures ranging from $300-600^{o}$C with a step of $100^{o}$C for a fixed time of 2 h. Structure of these films was determined by recording X-ray diffraction (XRD) patterns at room temperature before and after thermal annealing using Bruker D8 Discover diffractometer equipped with Cu $K_{\alpha}$ radiations. The Raman spectra were obtained at room temperature using confocal mode of Micro-Raman-Spectrometer (MST-1000A, DongWoo Optron Co. LTD, South Korea) under excitation with HeCd laser beam at 442 nm. Optical transmittance and reflectance of the as-deposited and annealed films were recorded at room temperature by a Perkin Elmer UV/VIS/NIR Lambda 19 spectrophotometer in the wavelength range 200–2500 nm. Impedance spectroscopic (IS) measurements were made at room temperature using Alpha-A High Performance Frequency Analyzer, Novocontrol Technologies, Germany in the frequency range of 0.1 Hz to 1MHz. Silver paint electrodes were used at a separation of about 15 mm on 25 mm wide films. The contacts were allowed to dry for 24 h in open air before making measurements. Surface morphology of the films was investigated by FESEM (JSM7500F, JEOL, Japan), and atomic force microscopy (Quesant Universal SPM, Ambios Technology) in non-contact mode. An AFM tip of silicon nitride was used having an approximate radius of curvature of 10 nm.

**3. Results and Discussion**

$TiO_2$ thin films deposited on BK7 glass substrates are almost free from pinholes and physically stable. Cracks and blisters are not found even after annealing up to 600$^o$C. Fig. 1 shows XRD patterns of the as-deposited and annealed $TiO_2$ films. As-deposited and annealed films are polycrystalline having anatase tetragonal structure (I41/amd (141)) [24]. Weak intensity and broadness of diffraction peaks for the as-deposited film suggest the presence of some amorphous phase. Crystallinity of the films seems to improve with the rise of annealing temperature as confirmed by the sharpness and intensity of the diffraction peaks. X-ray diffraction peaks are relatively broad signifying that the present films are composed of small nanoparticles [25-28]. The temperature dependence of XRD patterns can be explained primarily by the mobility of atomic species in thin films at various annealing temperatures. At low annealing temperature, the evaporated species possess small energy and hence a low surface mobility causing a less ordered surface structure. The low mobility of atoms prevents full crystallization of the films. However on annealing at high temperature, atoms acquire high enough mobility to organize themselves in a more crystalline arrangement [29]. The average particle size as estimated from planes with orientations (101) and (004) using Scherrer formula [26] is in the range of 20 nm to 28 nm and demonstrates an improvement with annealing temperature as clear from Fig. 2. Also included in Fig. 2 is the change in unit cell volume with annealing temperature. Furthermore, the unit cell volume of as-deposited and annealed films is found to be slightly greater than that for bulk $TiO_2$ (0.1363 $nm^3$).

Fig. 3 shows FESEM images of the as-deposited $TiO_2$ films. Images of the as-deposited film depict crystalline nature of $TiO_2$ with small amount of amorphous phase. It has been noticed that the as-deposited film shows the presence of $TiO_2$ nanocrystals, which appear as spherical clusters and their agglomerates. The average particle size lies in the range of 25 – 35 nm, which is slightly larger than that obtained from the XRD results. The agreement in the XRD and FESEM data can be made by keeping in view the fact that smaller primary particles have a large surface free energy and would, therefore, tend to agglomerate faster and grow into larger grains [1].

Two and three-dimensional AFM images of the as-deposited, 300$^{o}$C and 600$^{o}$C annealed $TiO_2$ films are shown in Fig. 4. It can be seen that all the films are crystalline with some amount of amorphous phase. The crystallinity of the films improves with the rise of annealing temperature as observed from Fig. 4. The surface asperities and depressions are of the order of 25-35 nm wide. RMS roughness of the as-deposited film is about 26 nm, which increases to about 36 nm on annealing except at 500$^{o}$C where it decreases to about 28 nm as clear from the inset of Fig. 4, representing non-uniform growth. This is also confirmed from the height scales of 2D AFM images.

Another very useful tool to distinguish between various crystalline phases of $TiO_2$ thin films is the Raman spectroscopy. The anatase phase of $TiO_2$ is known to comprise of six atoms per unit cell (with space group $\mathbf{D}_{4h}^{19}$) giving 15 vibrations modes. Among these optical modes $A_{2u}$, $B_{2u}$, $E_u$ are IR active while $A_{1g}$, $B_{1g}$, $E_g$ modes at 144, 197, 399, 515, 519 and 639 $cm^{-1}$ are Raman active [14, 17, 30]. The intensity and shape of IR and/or Raman peaks show variations with microstructure, particle shape, size and defects. Fig. 5 shows Raman spectra of the as-deposited and annealed $TiO_2$ films. The frequencies for various vibration modes were recognized by applying a Gaussian peak fit analysis. Three Raman peaks are visible in the spectrum of as-deposited film at about 394, 512 and 630 $cm^{-1}$ and are assigned to $B_{1g}$, $A_{1g} + B_{1g}$ and $E_g$ optical phonons respectively [14, 17] and no Raman peak for the rutile phase has been noted. These Raman peaks are blue-shifted as compared to those for bulk $TiO_2$ (399, 515 and 639 $cm^{-1}$) [30], suggesting quantum confinement effects. These Raman peaks remain unaffected up to annealing temperature of 400°C. But the peak at 394 $cm^{-1}$ corresponding to $B_{1g}$ mode show more blue-shift on annealing at higher temperatures. While peaks at 512 and 630 $cm^{-1}$ corresponding to $A_{1g}+B_{1g}$ and $E_g$ modes show a red-shift on annealing at higher temperatures, possibly caused by the grain growth and orderedness. The deviation of Raman peaks may also be due to non-stoichiometry of the $TiO_2$ [25]. Intensity of the Raman peaks shows slight increase with annealing temperature due to improvement in the structural order. The asymmetry of the Raman peaks may be attributed to small size of nanoparticles and structural defects [25, 26].

Fig. 6 shows plot of transmittance of the as-deposited and annealed $TiO_2$ films at different temperatures as a function of wavelength. An almost similar behavior of transmittance is observed for all the films in the wavelength range of 300-3000 nm with a slight shift in the peak position towards higher wavelengths. Ripples in the transmittance spectra are a consequence of the light interference [11]. The blunt and slow decrease below 500 nm probably due to the absorption edge manifests the presence of amorphous phase in the as-deposited film as observed by XRD and FESEM. Factors like non-stoichiometry, orderedness, residual stresses and defects formed during film deposition may cause observed variations in the transmittance.

Fig. 7 is a plot of absorption coefficient $\alpha$ as a function of photon energy, $h\nu$, for the determination of band gap. Band gap energy, $E_{opt}$, values estimated by extrapolating $\alpha - h\nu$ plots of Fig. 7 are portrayed in Fig. 8 as a function of annealing temperature. Optical band gap values of the $TiO_2$ films show almost a decreasing trend with a substantial fall from 3.21 eV (for the as-deposited film) to 3.06 eV except for the film annealed at 500°C which indicates a rise of $E_{opt}$ value to 3.24 eV. These $E_{opt}$ values are almost in agreement with the indirect band gap value (3.23 eV [12]) of $TiO_2$ films. The lowest value of $E_{opt}$ for the film annealed at 600°C matches with the direct forbidden band gap value of 3.03 eV [14]. This forbidden gap seems to be generated with an indirect allowed transition [31], but as the direct forbidden transitions are weaker in strength, the indirect allowed transitions play a dominant role in the optical absorption [14]. The decreasing behavior of $E_{opt}$ values with annealing has also been observed by Karunagaran *et al*. [14] and Radecka *et al*. [32]. Such variations in the $E_{opt}$ values could be associated with the structural modifications as seen in Raman studies (Fig. 5), and/or rise in crystallite size as observed in XRD (Figs. 1,2).

Fig. 9 shows reflectance, *R*, of the as-deposited and annealed films as a function of wavelength. An almost similar trend is observed for all films with a slight shift of peaks towards lower wavelengths up to an annealing temperature of 400ºC but above this temperature peaks shift slightly towards higher wavelengths. It might be caused by structural modifications as observed in Raman and AFM studies. Transmittance data can be employed to determine refractive index, *n,* using the relations described elsewhere [25]. Fig. 10 depicts refractive index (calculated at $\lambda$=550 nm) as a function of annealing temperature ($T_A$). The refractive index is found to increase from 2.26 to 2.32 except for the film annealed at 500°C which shows the lowest value of 2.1. Refractive index reported for crystalline anatase $TiO_2$ thin film is 2.40 and for the anatase single crystal is 2.50 [33]. Present values of refractive index are slightly lower than the reported anatase values. The low value of refractive index could seem to be due to partial crystallinity and low adatom mobility of the films at room temperature. It has been noticed that the refractive index and band gap energy show opposite trend with annealing. Such type of behavior has also been noticed by Ali *et al* [34].

Impedance spectroscopy is a useful tool for the characterization of intrinsic electrical properties of a material. Fig. 11 represents a typical complex impedance spectrum (Z'' plotted against Z') for the as-deposited $TiO_2$ film recorded at room temperature while illuminating the film to the sunlight as well as keeping it in the dark. Such impedance spectra for all the annealed films show single depressed semicircles (not shown here) analogous to that of the as-deposited film for both types of measurements.

The depressed semicircles are caused by the distribution of relaxation time showing the non-Debye nature of the films [35]. Measured data is well fitted with the simulated curve using an equivalent RC parallel circuit (shown as inset of Fig. 11). The dc resistance can be determined from the intercept on Z'-axis in the fitted Nyquist plot (Fig. 11). Optimum (best-fitted) value of capacitance, $C_1$, is found to be approximately same for all the films and is about 50 pF. In contrast, optimum value of dc-resistance, $R_1$, indicates an increasing performance (2.8-10.3 x $10^9$ Ω) with rise of annealing temperature except for the film annealed at 500°C which shows a decrease to 4.25 x $10^9$ Ω for the measurements made in the dark as obvious from Fig. 12. This behavior of $R_1$ is in-line with those of refractive index and RMS roughness but opposite with that of optical band gap (Figs. 4, 8, 10, 12). Measured resistances and capacitances may be a characteristic of grain boundaries between nanocrystals [28,36]. The fall of dc resistance (Fig. 12) seems to be a consequence of the structural modifications of the film, whereas the rise of resistance may be related with the reduction of carrier density through trapping of electrons at grain boundaries by adsorbed oxygens [36] during annealing in air. It is also notable from Fig. 12 that the dc-resistance is low when the measurements were made in sunlight as compared to those measured in the dark. It is an indicative for the rise of charge carrier density due to the absorption of photons.

## 4. Conclusions

The e-beam evaporated $TiO_2$ thin films are crystalline in nature possessing the anatase structure along with some amorphous phase. The amorphous phase reduces and improves crystalline character of the films with annealing at high temperatures. The post thermal annealing demonstrates greater affinity to alter the structural and optoelectronic properties of $TiO_2$ thin films which contain nanoparticles of size 25-35 nm. Raman spectrum of as-deposited $TiO_2$ film is blue-shifted indicating nanostructure and quantum confinement effects. Due to such quantum confinement effects and structural adjustments band gap energy of $TiO_2$ decreases from 3.21 eV to 3.06 eV, whereas an increase in refractive index, RMS roughness and dc-resistance has been noticed. The film annealed at 500 $^{o}$C represents peculiar behavior as band gap increases to highest value of 3.24 eV whereas refractive index decreases to lowest value of 2.1 as well as RMS roughness and dc-resistance show a drop as compared to other films. The films reveal lower resistance if illuminated to sunlight in comparison with measurements in the dark caused by increase in charge carriers by the absorption of photons.

**Acknowledgement**

Authors are thankful to Higher Education Commission (HEC), Government of Pakistan, for financial support.

**References**

[1] P. Singh, A. Kumar, D. Kaur, Physica B 403 (2008) 3769.

[2] G.W. Lee, J.H. Byeon, Mater. Char. 60 (2009) 1476.

[3] D. Crisan, N. Dragan, M. Crisan, M. Raileanu, A. Braileanu, M. Anastasescu, A. Lanculescu, D. Mardare, D. Luca, V. Marinescu, A. Moldovan, J. Phys. Chem. Solids 69 (2008) 2548.

[4] S. Murugesam, P. Kuppusami, E. Mohandas, Mater. Res. Bull. 45 (2010) 6.

[5] A. Welte, C. Waldauf, C. Brabec, P.J. Wellmann, Thin Solid Films 516 (2008) 7256.

[6] P. Chowdhury, H.C. Barshilia, N Selvakumar, B. Deepthi, K.S. Rajam, A.R. Chaudhuri, S.B. Krupanidhi, Physica B, 403 (2008) 3718.

[7] G.D. Wilk, R.M. Wallace, J.M. Anthony, J. Appl. Phys. 89 (2001) 5243.

[8] A. Bendavid, P.J. Martin, E.W. Preston, Thin Solid Films 517 (2008) 494.

[9] R. Sikora, J. Phys. Chem. Solids 66 (2005) 1069.

[10] J.Y. Gan, Y.C. Chang, T.B. Wu, Appl. Phys. Lett. 72 (1998) 332.

[11] C. Yang, H. Fan, Y. Xi, J. Chen, Z. Li, Appl. Surf. Sci. 254 (2008) 2685.

[12] M. Kadoshima, M. Hiratani, Y. Shimamoto, K. Torii, H. Miki, S. Kimura, T. Nabatame, Thin Solid Films 424 (2003) 224.

[13] M. Maeda, T. Watanabe, Surf. Coat. Technol. 201 (2007) 9309.

[14] B. Karunagaran, K. Kim, D. Mangalaraj, J. Yi, S. Velumani, Sol. Energy Mater. Sol. Cells 88 (2005) 199.

[15] B.K. Tay, Z.W. Zhao, D.H.C. Chua, Mater. Sci. Engg. R52 (2006) 1.

[16] G.H Du, Q. Chen, R.C. Che, Z.Y. Yuan, L.M. Peng, Appl. Phys. Lett. 79 (2001) 22.

[17] X. Dong, J. Tao, Y. Li, H. Zhu, Appl. Surf. Sci. doi: 10.1016/j.apsusc.2009.10.100.

[18] L.H. Chong, K. Mallik, C.H. de Groot, R. Kersting, J. Phys: Condens. Matter. 18 (2006) 645.

[19] V.G. Besserguenev, R.J.F. Pereira, M.C. Mateus, I.V. Khmelinskii, R.C.Nicula, E. Burkel, Int. J. Photoenergy 5 (2) (2003) 99.

[20] L.J. Meng, V. Teixeira, H.N. Cui, F. Placido, Z. Xu, M.P. dos Santo, Appl. Surf. Sci. 252 (2006) 7970.

[21] G. Facchin, G. Carturan, R. Campostrini, S. Gialanella, L. Lutterotti, L. Armeleo, G. Marci, L. Palmisano, A. Sclafani, J. Sol-Gel Sci. Technol. 18 (2000) 29.

[22] A. De Giacomo, O. De Pascale, Appl. Phys. A 79 (4–6) (2004) 1405.

[23] J.C. Tinco, M. Estrada,G. Romero, Microelectron. Reliab. 43 (2005) 895.

[24] The Joint Committee on Powder Diffraction Standards (JCPDS), Card No. 89-4921, International Centre for Diffraction Data (ICDD), Swarthmore, PA, USA.

[25] A.F. Khan, M. Mehmood, A.M. Rana, M.T. Bhatti, A. Mahmood, Chin. Phys. Lett. 7 (2009) 077803.

[26] A.F. Khan, M. Mehmood, A.M. Rana, M.T. Bhatti, Appl. Surf. Sci. **255** (2009) 8562.

[27] A.F. Khan, M. Mehmood, A.M. Rana, T. Muhammad, Appl. Surf. Sci. (2009), doi:10.1016/j.apsusc.2009.09.043.

[28] A.F. Khan, M. Mehmood, M. Aslam, M. Ashraf, Appl. Surf. Sci. (2009), doi:10.1016/j.apsusc.2009.10.047.

[29] H.R. Fallah, M. Ghasemi, A. Hassanzadeh, Physica E 39 (2007) 69.

[30] J. Zhang, M.J. Li, Z.C. Feng, J. Chen, C. Li, J. Phys. Chem. B 110 (2006), 927.

[31] K.M. Glassford, J.R. Chelikowsky, Phys. Rev. B 46 (1992) 1284.

[32] M. Radecka, K.Z. Akrzewska, H. Czternastek, T. Stapinski, S. Debrus, Appl. Surf. Sci. 65/66 (1993) 227.

[33] M. Lazzeri, A. Vittabine, A. Selloni, Phys. Rev. B 65 (2002) 119901.

[34] H.M. Ali, M.M. Abou-Mesalan, M.M. El-Shorbagy, J. Phys. Chem. Solids 71 (2010) 51.

[35] A.F. Khan, M. Mehmood, M. Aslam, S.I. Shah, J. Colloid Interface Sci. (2009), doi:10.1016/j.jcis.2009.11.045.

[36] W. Göpel, K.D. Schierbaum, Sens. Actuators B 26–27 (1995) 1.

## Captions

Fig.1. X-ray diffraction patterns of $TiO_2$ thin films, as-deposited (a) and annealed for 2hrs at 300 $^oC$ (b), 400 $^oC$ (c), 500 $^oC$ (d), and 600 $^oC$ (e).

Fig.2. Plots of volume of the unit cell (*curve 1*) and particle size (*curve 2*) as a function of annealing temperature ($T_A$) for $TiO_2$ thin films.

Fig.3. FESEM images of the as-deposited $TiO_2$ thin films.

Fig.4. AFM images (1000 x 1000 $nm^2$) showing the surface morphology of $TiO_2$ thin film annealed at 350$^oC$ for 2 hrs. The inset represents plot of RMS roughness vs. annealing temperature ($T_A$).

Fig.5. Raman Spectra of $TiO_2$ thin films at various annealing temperatures.

Fig.6. Optical transmittance of $TiO_2$ thin films, as-deposited (a) and annealed for 2hrs at 300 $^oC$ (b), 400 $^oC$ (c), 500 $^oC$ (d), and 600 $^oC$ (e).

Fig.7. Plots of $\alpha$ vs. photon energy, $h\nu$, for $TiO_2$ thin films at various annealing temperatures.

Fig.8. Plot of optical band gap, $E_{opt}$, vs. annealing temperature, $T_A$, for $TiO_2$ thin films.

Fig.9. Optical reflectance of $TiO_2$ films as a function of wavelength at various annealing temperatures.

Fig.10. Plots of refractive index, *n,* vs. annealing temperature, $T_A$, for $TiO_2$ thin films at wavelength $\lambda$ = 550 nm.

Fig.11. Nyquist plots of as-deposited $TiO_2$ thin film along with fitted data using equivalent circuit as shown.

Fig.12. Plot of dc-resistance vs. annealing temperature, $T_A$, for $TiO_2$ thin films.

**Fig. 1**

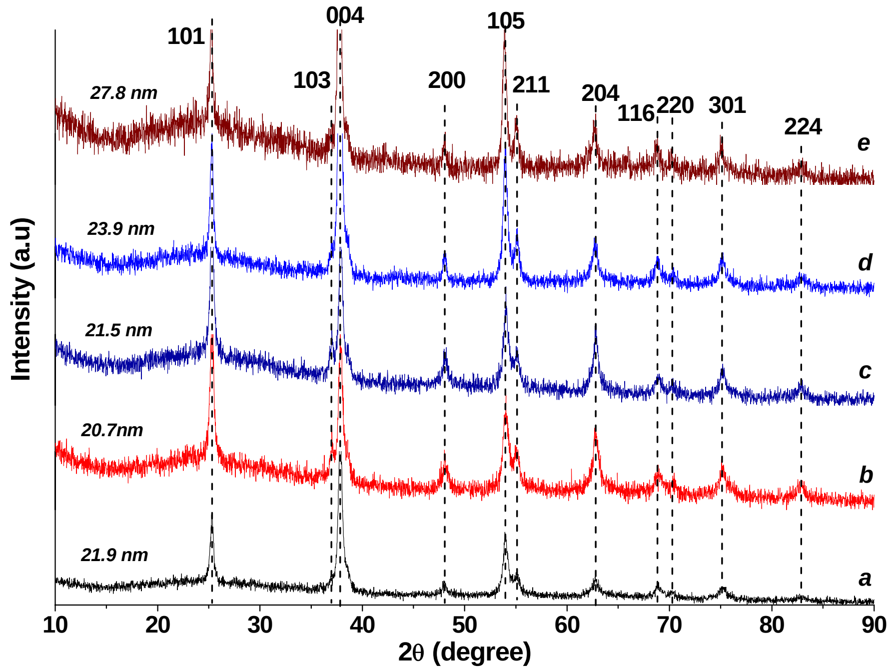

101
004
105
103
200
211
204
116
220
301
224
27.8 nm
23.9 nm
21.5 nm
20.7nm
21.9 nm
e
d
c
b
a
Intensity (a.u)
10
20
30
40
50
60
70
80
90
2θ (degree)

**Fig. 2**

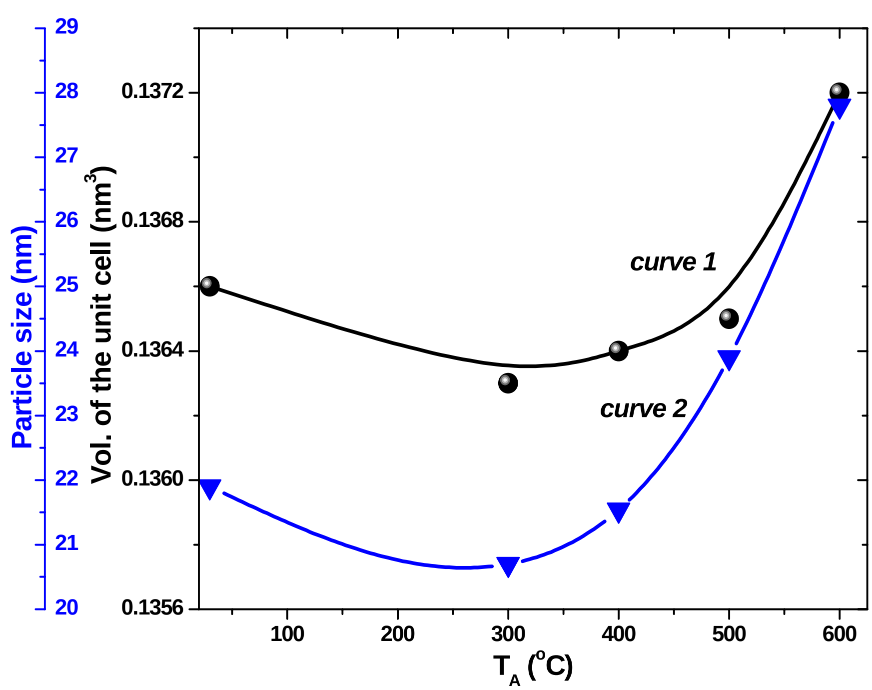

Particle size (nm)
Vol. of the unit cell (nm3)
20
21
22
23
24
25
26
27
28
29
0.1356
0.1360
0.1364
0.1368
0.1372
curve 1
curve 2
100
200
300
400
500
600
TA (oC)

**Fig. 3**

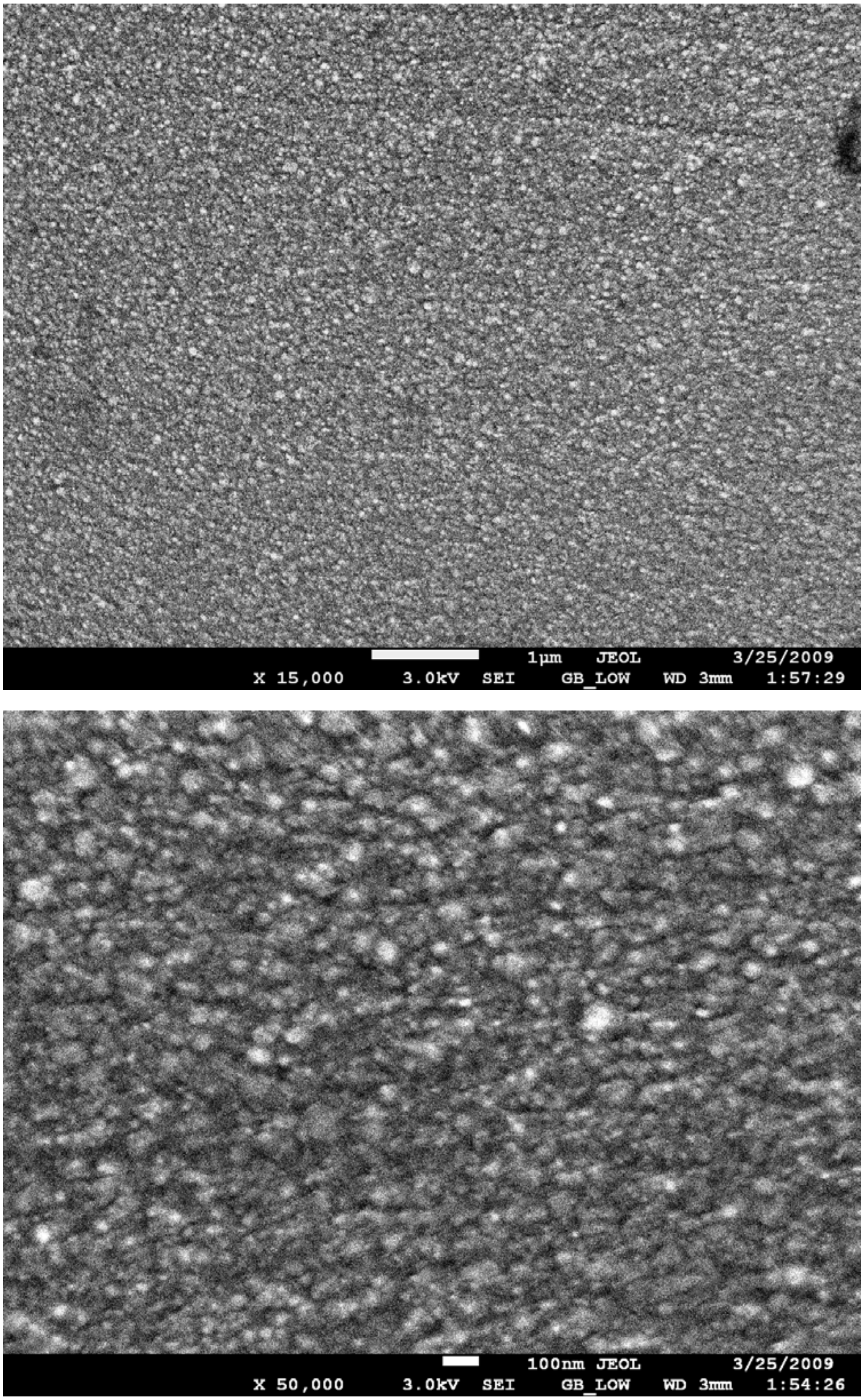

1µm JEOL 3/25/2009
X 15,000 3.0kV SEI GB_LOW WD 3mm 1:57:29
100nm JEOL 3/25/2009
X 50,000 3.0kV SEI GB_LOW WD 3mm 1:54:26

**Fig. 4**

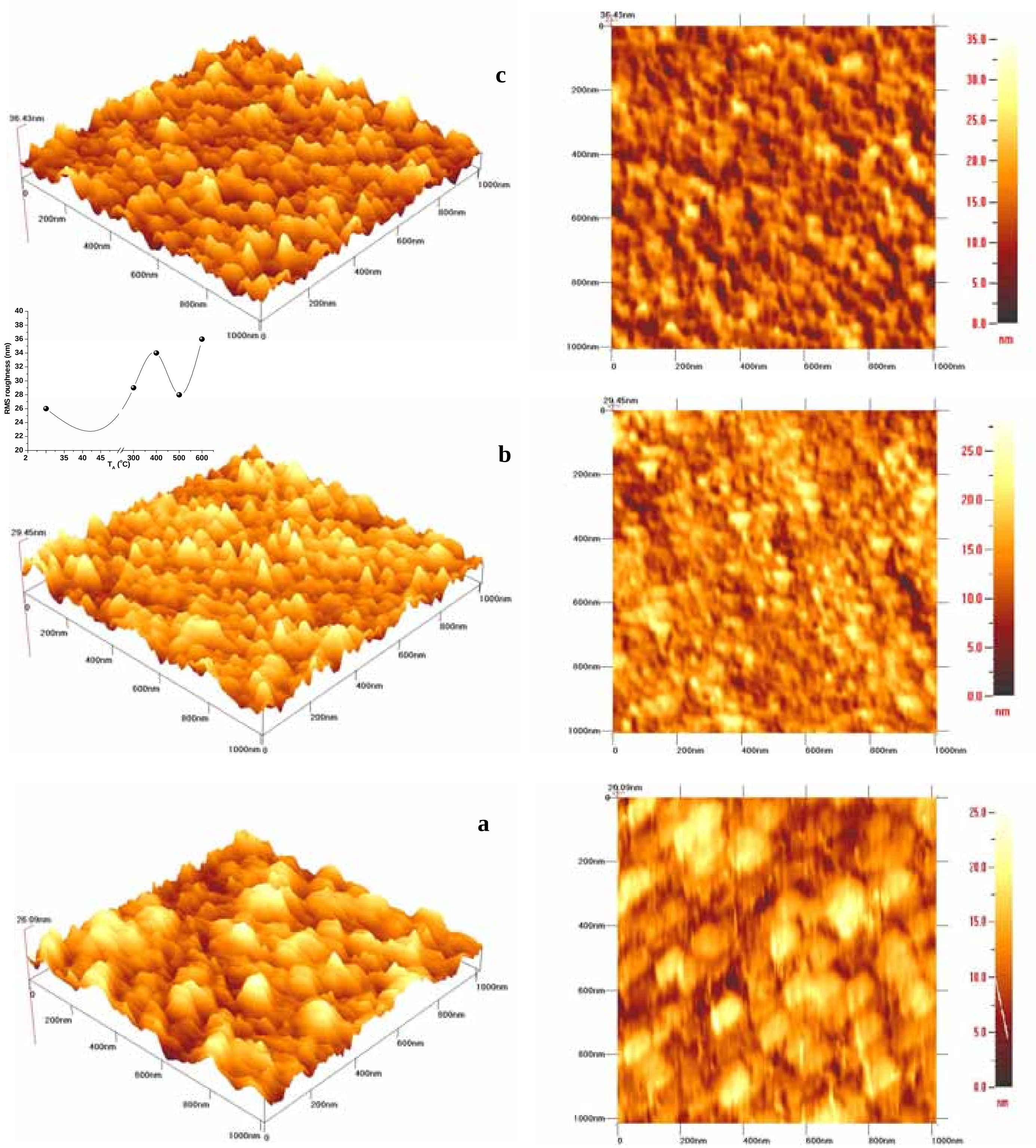


**Fig. 5**

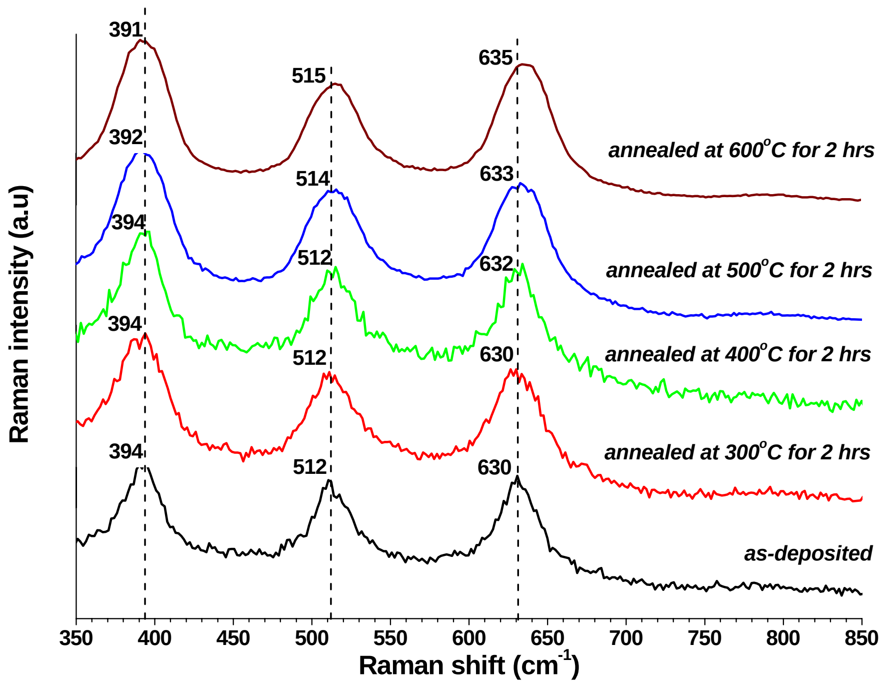

391
515
635
392
514
633
394
512
632
394
512
630
394
512
630
annealed at 600°C for 2 hrs
annealed at 500°C for 2 hrs
annealed at 400°C for 2 hrs
annealed at 300°C for 2 hrs
as-deposited
Raman intensity (a.u)
350
400
450
500
550
600
650
700
750
800
850
Raman shift (cm-1)

**Fig. 6**

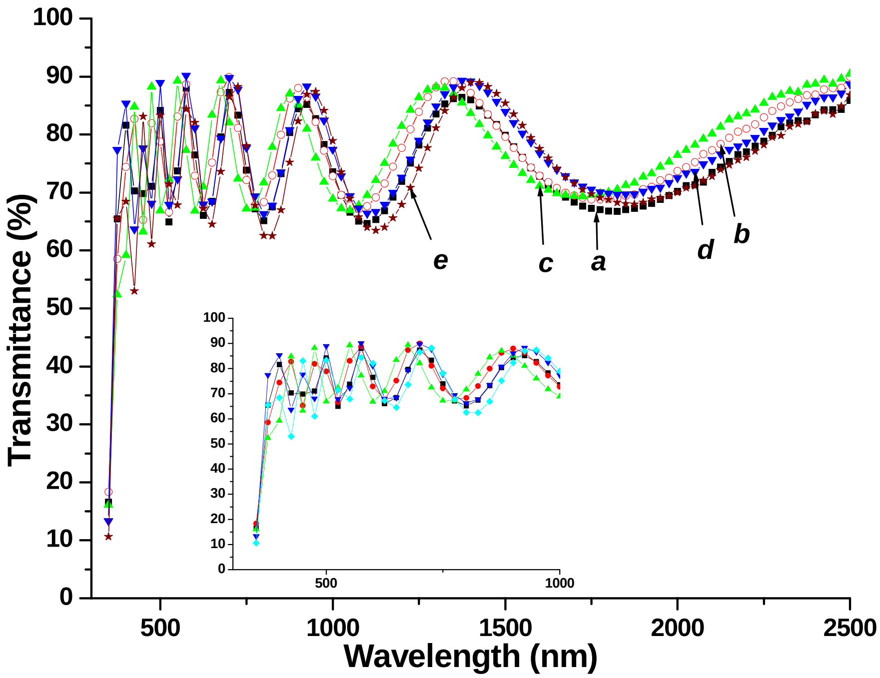


**Fig. 7**

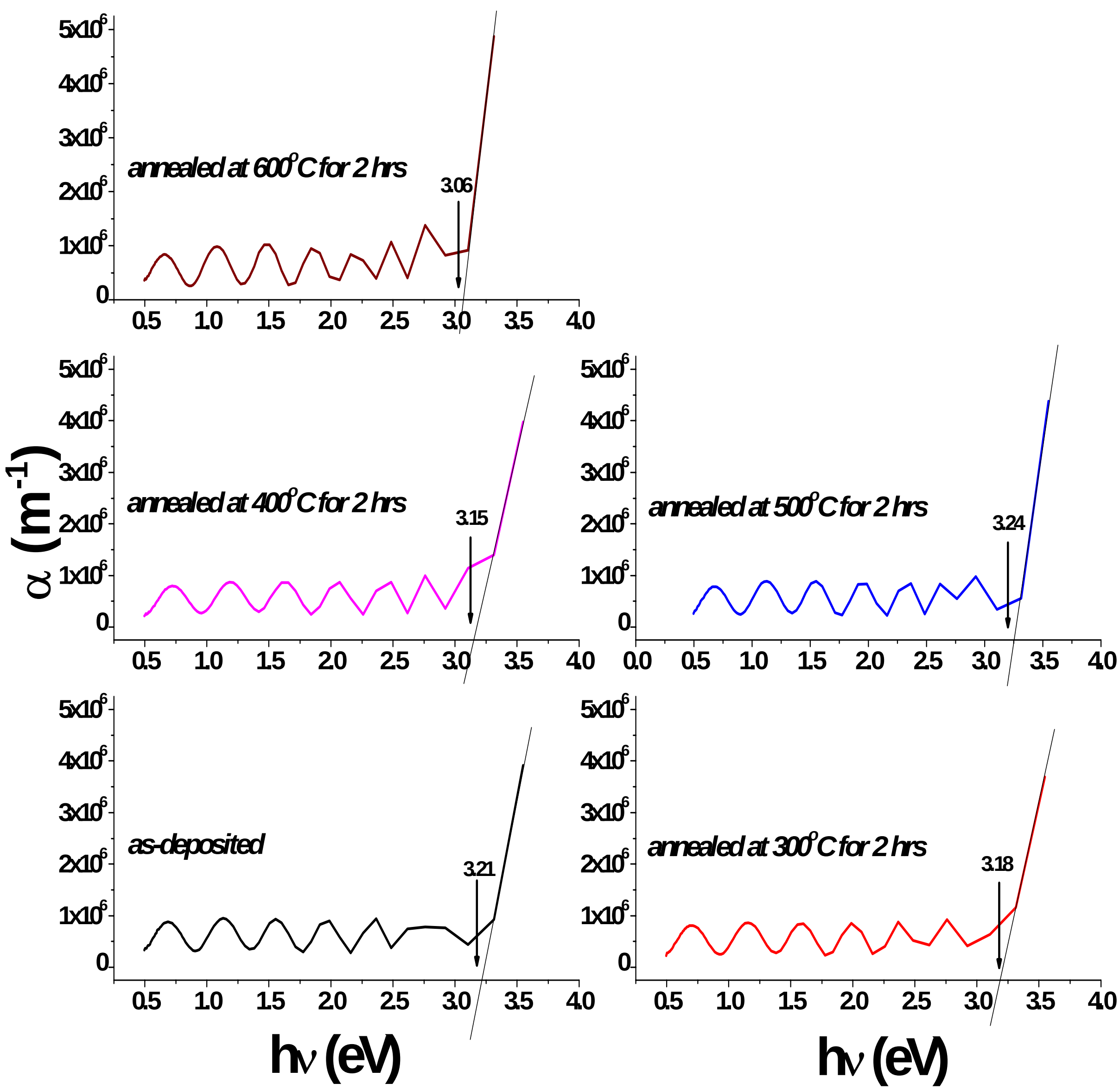

annealed at 600°C for 2 hrs
3.06
annealed at 400°C for 2 hrs
3.15
annealed at 500°C for 2 hrs
3.24
as-deposited
3.21
annealed at 300°C for 2 hrs
3.18
α (m-1)
hν (eV)
hν (eV)

**Fig. 8**

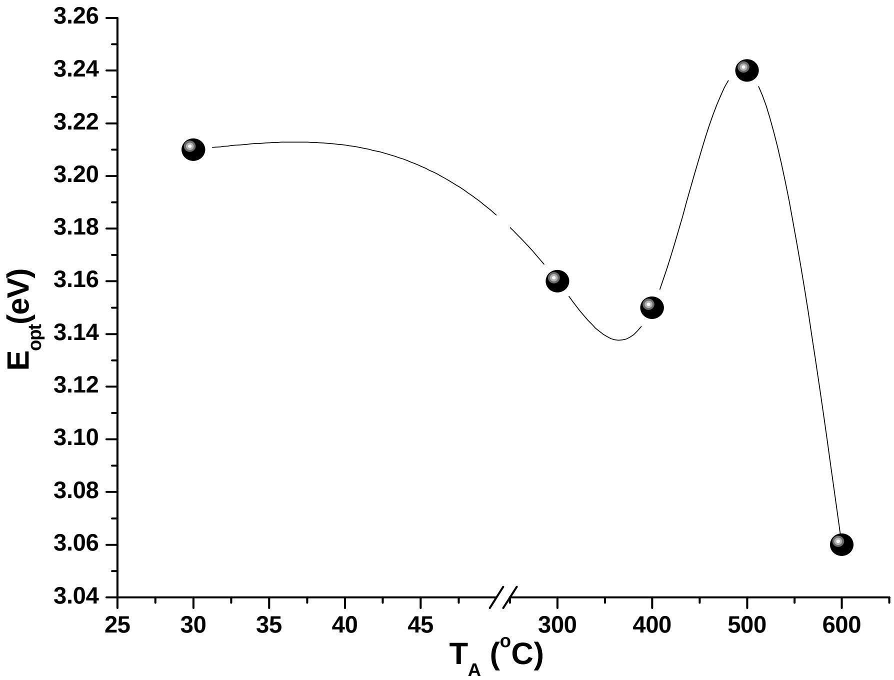

3.26
3.24
3.22
3.20
3.18
3.16
3.14
3.12
3.10
3.08
3.06
3.04
E_opt(eV)
25
30
35
40
45
300
400
500
600
T_A (°C)

**Fig. 9**

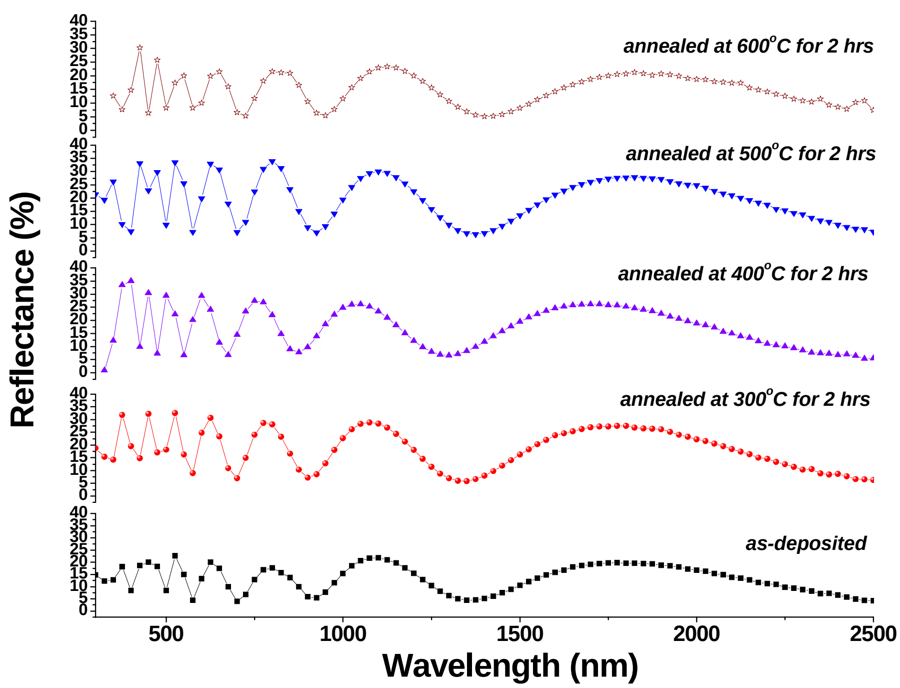
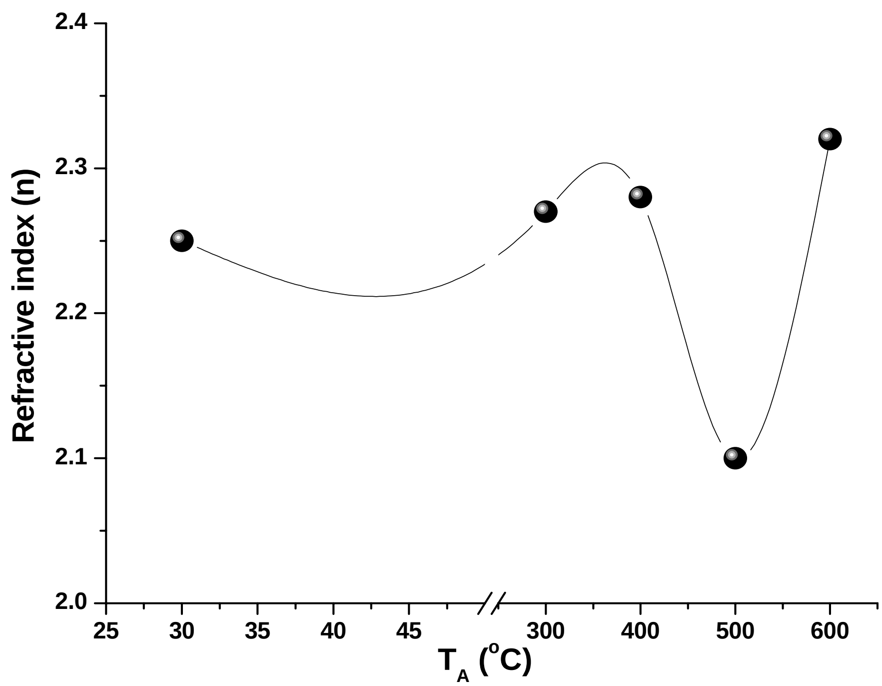
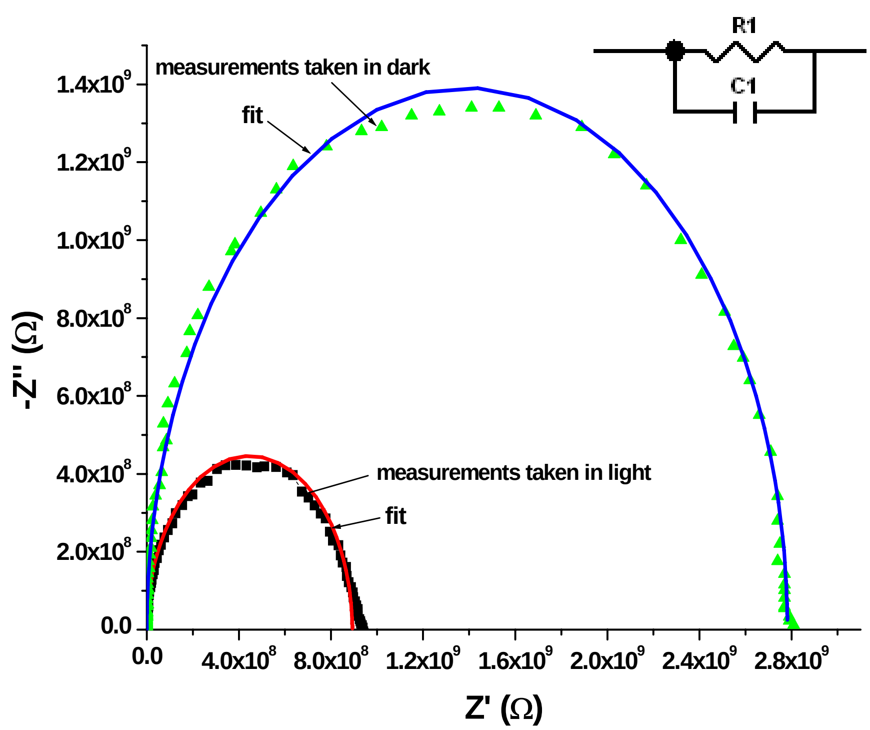

annealed at 600°C for 2 hrs
annealed at 500°C for 2 hrs
annealed at 400°C for 2 hrs
annealed at 300°C for 2 hrs
as-deposited
Reflectance (%)
0
5
10
15
20
25
30
35
40
500
1000
1500
2000
2500
Wavelength (nm)

**Fig. 10**



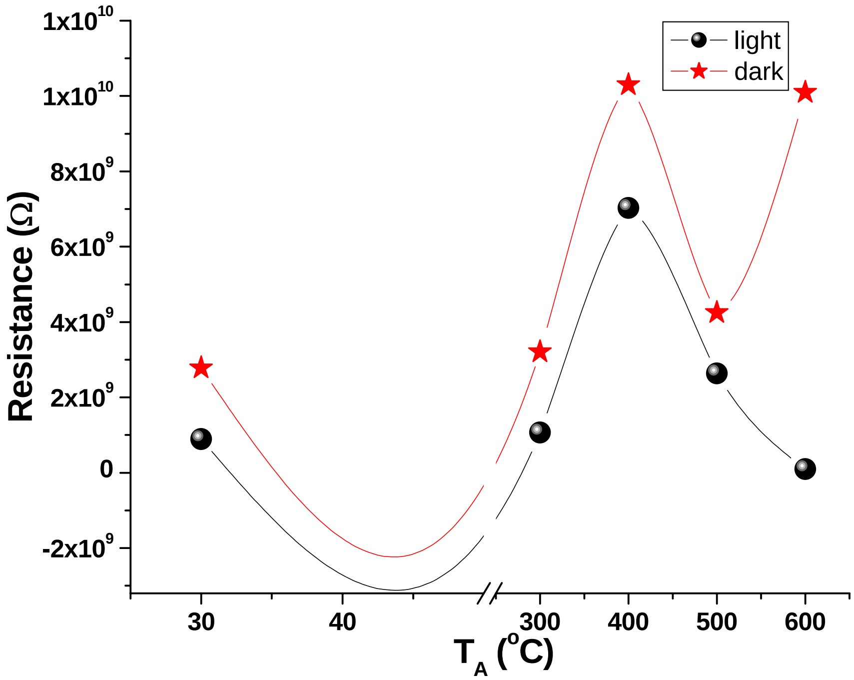
Refractive index (n)
2.4
2.3
2.2
2.1
2.0
25
30
35
40
45
300
400
500
600
$T_A$ ($^oC$)


**Fig. 11**



R1
C1
measurements taken in dark
fit
measurements taken in light
fit
$1.4x10^9$
$1.2x10^9$
$1.0x10^9$
$8.0x10^8$
$6.0x10^8$
$4.0x10^8$
$2.0x10^8$
0.0
-Z'' (Ω)
0.0
$4.0x10^8$
$8.0x10^8$
$1.2x10^9$
$1.6x10^9$
$2.0x10^9$
$2.4x10^9$
$2.8x10^9$
Z' (Ω)

**Fig. 12**



1x10^10
1x10^10
8x10^9
6x10^9
4x10^9
2x10^9
0
-2x10^9
Resistance (Ω)
light
dark
30
40
300
400
500
600
T_A (°C)

# EXPOSURE RISK ASSESSMENT OF POLYCYCLIC AROMATIC HYDROCARBONS ASSOCIATED WITH THE INDOOR AIR PARTICULATE MATTERS IN SELECTED INDOOR ENVIRONMENTS IN KANDY, SRI LANKA

D G G P Karunaratne[1], S.B.A.M.B.Y. Wijayarathna[2]

[1]Senior Lecturer, Department of Chemical and Process Engineering,
University of Peradeniya, Peradeniya, Sri Lanka.
[1]E-mail: dpkaru@pdn.ac.lk
[1]Telephone: +94-81-2393669

[2]Postgraduate Student, Postgraduate Institute of Science, University of Peradeniya,
Peradeniya, Sri Lanka.
[2]E-mail: bihesha2003@yahoo.com

**Abstract:** The present work investigates the health risks associated with the inhalation exposure of particulate bound polyaromatic hydrocarbons in selected indoor environment in Kandy city. Particulate samples from five different types of indoor environments were collected for chemical analysis of 16PAH priority pollutants listed by USEPA. Categorization of locations was based on the degree of urbanization, type of fuel used for cooking and proximity to road from the location. Key meteorological parameters such as wind speed, wind direction, ambient temperature and relative humidity were also measured. The collected samples were analysed using a high performance liquid chromatograph (HPLC), after the necessary extraction and cleaning up of the sample. The total concentrations of particulate PAHs are in the range of 0.386 ng/m3 to 14.65 ng/m3. The comparison of PAH levels at different categories of environment indicate that the degree of urbanization, fuel type and proximity to road influences the total concentrations of particulate PAHs. The dominant particulates of PAHs measured at the selected environments are naphthalene, acenaphthylene, acenaphthene. Then benzo[a]pyrene [B(a)P] indicating particulate PAHs are contributed by a mixture of both diesel and petrol engine type of vehicles, and biomass combustion. The total $BaP_{eq}$ concentrations at different environmental categories ranged from 0.06 to 3.08 ng/m3. The total BaP equivalency results showed the potential health risk to cancer due to inhalation exposure is of concern for residents living in high urban area with usage of wood as fuel. Since the total BaPeq concentrations for this category was higher. Very close to, or slightly exceeded the maximum permissible risk level of 1 ng/m3 of benzo(a)pyrene in other categories also.

**Keywords:** Polycyclic Aromatic Hydrocarbons, Indoor air quality, Carcinogenic risk

## 1 Introduction

Polycyclic aromatic hydrocarbons (PAHs) are a group of widespread environmental pollutants containing two or more fused benzene rings. The environmental occurrence of PAHs has been associated with adverse effects on public health, as they are considered to be the largest group of carcinogens present in the environment (Zhu, et al., 2008). These compounds are typically formed during the incomplete combustion of organic materials such as coal, oil, gas, and wood. The natural sources of PAHs include volcanic activity and forest fires. The lighter PAH (2–3 rings), which are generally not carcinogenic, are mostly found in the gas phase, while the heavier ones are mainly associated with airborne particles such as soot. Indoor emission sources of PAHs include smoking, cooking, and heating. Significantly, 50–75% of the particulate matter containing PAHs generated due to combustion of various cooking fuels are in respirable size range.

Hill capital of Sri Lanka, Kandy, is a valley surrounded by hills and air pollution due to vehicular emission in this city is becoming a major health concern. Further, a considerable proportion of houses use firewood as domestic fuel source, a situation which can lead to increased levels of air pollutants such as PAHs in these indoor environments.

The present study aims at quantifying PAHs levels associated with particulate matter in selected indoor environments in Kandy city and to investigate the influence of firewood burning and vehicular emission on PAHs levels. Further, the carcinogenic risks associated with the exposure to detected levels at different categories of indoor environments are estimated.

## 2 Methodology

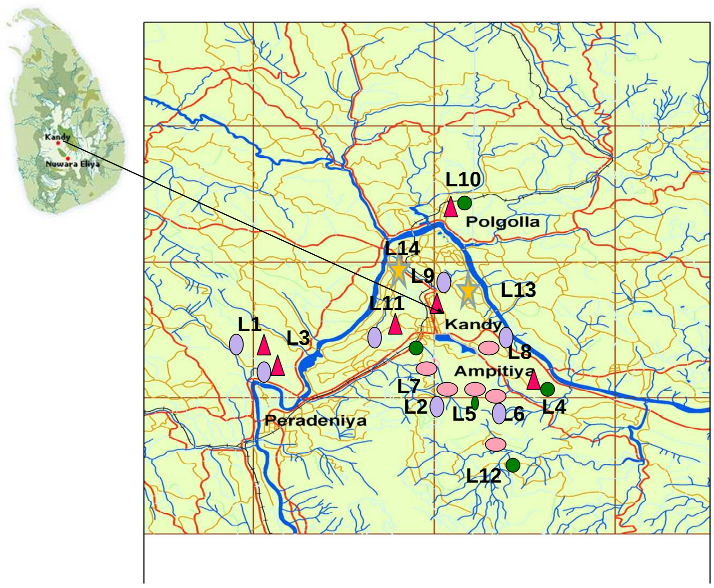
### 2.1 Sampling sites

Samples were collected from 14 sites in and around Kandy city which included 12 houses and two office buildings. The 12 houses were selected according to three main criteria; degree of urbanization, type of domestic fuel that they use for cooking, and proximity to roads. Other two sites were selected from offices around the Kandy city. The locations are shown in Figure 1. Categorization of the sampling sites, as shown in Table 1, was based on the degree of urbanization, influence of different types of domestic fuels, and proximity to roads.

Table 1 *Description of different categories of.*

| Category type | Description | Sites included |
|---|---|---|
| 1 | High urban area + Gas users | L1, L3, L9, L11 |
| 2 | Less urban area + Gas users | L2, L6, L8, |
| 3 | High urban area +wood uses | L4, L7, L10, |
| 4 | Less urban area+ wood uses | L5, L12 |
| 5 | Office premises | L13, L14 |

**Site Selection**




**High urban area Less urban area Wood Gas Office**

Figure.1 : *Locations of PAHs sampling points*

## 2.2 Sample Collection

Particulate matters in selected indoor air were collected onto 55 mm diameter glass micro fibre filters using personal air samplers. Sampling was done for 24 hrs in the selected houses and in offices only 8 hours because offices are mainly occupied during these hours. In case of office buildings, two samplers were used to collect a reasonable volume of air. The personal air samplers were chosen for this study because they were portable, less noisy, battery-operated and easy to mount inside the buildings. The flow rates were checked regularly to obtain the total flow rate throughout the sampling period. Air sampling was conducted from May 2009 to August 2009 under identical weather conditions. Indoor air samples were collected in the kitchen or in close proximity to the kitchen. This sampling location was selected because, most homes in Kandy , the kitchen has more or is closer to potential PAH sources than other rooms and is often where most family activities occur.

### 2.3 Sample preparation and analysis

The filter with collected particulate matters was extracted for 7 hours in a 250 ml soxhlet apparatus with dichloromethane. The extract was concentrated by solvent evaporation under reduced pressure in a rotary evaporator (Yamato scientific co.ltd , RE-46). The temperature of the water bath was set at 40 $^{o}$C. The remaining residues in the evaporated flask were dissolved in 1 ml of acetonitrile as required by the subsequent analytical process.

Identification and quantification of PAHs in the samples were carried out using a HPLC equipped with a ZORBAX Eclipse XDB- C8 column, (250 mm× 4.6 mm× 5 µm), under gradient elustion of deionised water and acetonitrile at a flow rate of 0.8 ml/min. Calibration data developed by Wickramasinghe et.la (2008), was used The readings were corrected to observed recovery efficiencies of the standard samples analysed.

### 2.4 Hazard and Risk assessment

The Toxic Equivalent Factor (TEF) approach has been extensively used in risk assessment of different classes of PAH mixtures. The overall toxicity of a PAH mixture is defined by the concentration of individual PAH compounds (Ci) in a mixture times their relative TEF. $BaP_{eq}$= (Ci) TEF . The TEF is an estimate of the relative toxicity compared to BaP for which a great deal of data exists. For this study the TEF values proposed by Nisbet and LaGoy (1992) were used. The $BaP_{eq}$ values can be subsequently used for cancer risk assessment.

## 3 Results and discussion

The observed PAH concentrations are given in Table 2. Accordingly, the total concentration of PAH measured in this study ranged from 0.38 to 14.65 ng/m$^3$. Highest concentration of total PAHs was present in location 13 which is located in high urban area. Lowest total PAHs was present in location 6 located in less urban area. Naphthalene (Nap) in this study was found to be the most abundant of the 16 PAHs studied. This pattern is observed in most of the reported PAHs levels in literature too. The main reason for this could be due to the fact that Nap dominates in most of the PAH emissions ( Zhu, et al., 2008, Li et al., 2005 , Kalaiarasan, et al., 2009, Mumford , et al., 1991). The Nap concentration varied from 0.10 to 7.39 ng/$m^3$ with an average of 2.813 ng/ $m^3$ and accounted for 43.49 % of the total PAHs. In addition, a previous study found that Nap emissions in indoors environments were highly correlated with the indoor activities such as the use of mothballs in china (Zhu, et al., 2008).

Table .2: *PAH concentrations at different sampling locations*

| Compound | Concentrations of PAHs at individual sampling locations (ng/m$^3$) | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | L1 | L2 | L3 | L4 | L5 | L6 | L7 | L8 | L9 | L10 | L11 | L12 | L13 | L14 |
| Nap | 2.83 | 2.13 | 1.31 | 4.69 | 0.22 | 0.1 | 0.88 | 0.48 | 7.39 | 4.12 | 0.97 | 3.14 | 4.15 | 6.98 |
| Ace | - | - | - | 3.29 | - | - | 2.28 | 0.18 | 4.05 | 1.17 | 2.58 | 1.22 | 2.05 | - |
| Acy | - | - | - | 1.78 | 0.02 | 0.11 | - | 0.37 | 1.51 | 0.46 | 1.07 | 1.06 | 6.03 | 0.04 |
| Flu | - | - | 0.01 | - | - | - | - | 0.01 | 0.13 | 0.26 | 0.18 | 0.28 | 0.17 | - |
| Phe | - | - | - | 0.06 | - | - | - | 0.002 | 0.1 | 0.06 | 0.04 | 0.03 | 0.04 | 0.03 |
| Ant | 0.37 | - | - | 0.01 | 0.19 | 0.08 | 0.23 | - | 0.19 | 0.23 | 0.34 | 0.18 | 0.02 | - |
| Flt | - | - | - | 0.34 | - | - | - | - | 0.13 | 0.45 | 0.51 | 0.38 | 0.1 | - |
| Pyr | - | - | 0.16 | 0.28 | 0.08 | 0.06 | - | 0.09 | 0.7 | 0.16 | 0.35 | 0.16 | 0.11 | 0.06 |
| B(a)A | - | 0.12 | 0.7 | 0.07 | - | - | 0.05 | 0.003 | 0.14 | 0.12 | 0.15 | 0.08 | 0.06 | - |
| Chr | 0.04 | - | 0.08 | - | - | - | 0.06 | - | 0.12 | 0.14 | 0.14 | 0.08 | 0.01 | - |
| B(b)F | - | - | - | 0.46 | - | - | - | - | - | - | 1.16 | - | 0.06 | - |
| B(k)F | - | 0.006 | - | 0 | - | - | - | - | - | - | - | - | 0.05 | - |
| B(a)P | - | 0.06 | 1.26 | 0.46 | 0.02 | 0.03 | 1.02 | 0.013 | 0.08 | 0.91 | 0.64 | 1.22 | 0.27 | 0.18 |
| DBA | - | - | 0.39 | 0.51 | - | - | 1.12 | 0.016 | 0.08 | 0.43 | 0.26 | - | 0.01 | - |
| BPe | - | 0.01 | 0.23 | - | 0.01 | - | 0.44 | - | 0.03 | 0.66 | 0.22 | - | 0.02 | - |
| Ind | - | - | - | - | - | - | - | - | - | - | - | - | 0.02 | - |
| Total | 3.24 | 2.326 | 4.14 | 11.95 | 0.54 | 0.38 | 6.08 | 1.164 | 14.65 | 9.17 | 8.61 | 7.86 | 13.17 | 7.29 |

**Nap**-naphthalene, **Ace**- acenaphthene, **Acy** -acenaphthylene, **Flu** -fluorene, **Phe** -phenanthrene, **Ant** –anthracene, **Flt** -fluoranthene, **Pyr** -pyrene, **B(a)A**- benz(a)anthracene, **Chr** -chrysene, **B(b)F**- benzo(b)fluoranthene, **B(k)F**- benzo(k)fluoranthene, **B(a)P**- benzo(a)pyrene, **DBA**-dibenzo(a,h)anthracene, **BPe**- (Benzo(g,h,i)perylene, **Ind**-ideno(1,2,3,-c,d)pyrene

According to this study, naphthalene concentration did not vary with the degree of urbanization because low concentrations of naphthalene was observed in some high urbanised locations, while in some less urban locations high concentrations of naphthalene were found. According to the results, the Nap concentration was not dependent upon the fuel type as it was found in domestic as well as office environments. Li, et al., 2005 reported that in Chicago Nap had the highest mean and median concentrations among the 16 PAHs. The medians are 177 and 168 ngm$^3$, and the maxima are 2340 and 1867 ngm$^3$, for indoor air and outdoor air, respectively. Compared with two other indoor PAH studies in the US conducted in 1986 and in 1994, indoor naphthalene concentrations found in this study are lower. They found that indoor naphthalene emission was largely associated with mothball usage, while suggesting that another combustion sources may also have been responsible for high naphthalene concentrations such as combustion of kerosene, and other fuel vapours.

Acenaphthalene (Ace) is the second highest PAH present at these 14 locations ranging 0.18 - 4.05 ng/ m$^3$. Acenaphthene found in concentrations ranging from 0.02 to 6.03 ng/ m$^3$. Then Benzo[a]pyrene (B[a]P) concentration ranging from 0.03 to 0.7 ng/ m$^3$. All PAHs with molecular weight between 128 and 202 had average concentration 5.22ng/ m$^3$, whereas PAHs with molecular weight 228–278 was found in much lower concentrations with an average of 1.03 ng/m$^3$. All PAHs with molecular mass 152–202 had average concentrations higher than 1 ng/ m$^3$ and their medians ranged from 0.5 to 19 ng /m$^3$. By comparison, PAHs with molecular mass 228 or higher exist in air in significantly lower concentrations. No median or average for these heavy PAHs is above 0.25 ng/ m$^3$ for indoor samples or 0.5 ng/ m$^3$ for outdoor samples.

As shown in Table 3, the quantitative results mean varied significantly depending on the degree of urbanization, and type of fuel used for cooking. Among five categories, as described above, the highest concentrations of total PAHs found are in houses that use wood as their cooking purposes and located in high urban area. In addition, those locations are close to road so vehicular emission may be the major source. This is due the fact that the indoor air quality in these houses is affected by both emissions from vehicles as well as from biomass combustion.

The second highest concentration was observed in office environments. Proximity to roads may be the major cause of this observed high PAHs concentration. In addition, a recent study reported the amount of dust released during operation of computers to be between 4.0 and 6.3 mg dust per day, and suggested that a significant potential for exposure to chemicals such as PAHs that are associated with re-suspended dust particles (Destaillats, et al., 2008). Some samples analysed by them had significantly higher levels of B(b)F, Chr, Flu, and Phe than domestic dust. Since these compounds are present in outdoor air and can be emitted by various indoor sources, this study could not estimate the contribution of computer emissions, but it is suggested that a fraction of measured PAHs might have been emitted from the heated plastic materials, chips and other computer components.

Table :3 *Variation of the average individual PAH compound in different categories in (ng/m$^3$).*

| Compound | Category 1 | Category 2 | Category 3 | Category 4 | Category 5 |
|---|---|---|---|---|---|
| Nap | 3.125 | 0.900 | 4.400 | 1.410 | 5.560 |
| Ace | 1.650 | 0.060 | 2.230 | 1.160 | 1.025 |
| Acy | 0.640 | 0.160 | 1.120 | 0.360 | 3.035 |
| Flu | 0.080 | 0.003 | 0.130 | 0.090 | 0.080 |
| Phe | 0.030 | 0 .001 | 0.060 | 0.010 | 0.035 |
| Ant | 0.220 | 0.020 | 0.120 | 0.200 | 0.010 |
| Flt | 0.160 | - | 0.390 | 0.120 | 0.050 |
| Pyr | 0.300 | 0.050 | 0.220 | 0.080 | 0.080 |
| B(a)A | 0.240 | 0.041 | 0.090 | 0.045 | 0.030 |
| Chr | 0.090 | - | 0.070 | 0.046 | 0.005 |

| | | | | | |
|---|---|---|---|---|---|
| B(b)F | 0.290 | - | 0.230 | - | 0.030 |
| B(k)F | - | 0.002 | - | - | 0.025 |
| B(a)P | 0.440 | 0.030 | 0.685 | 0.750 | 0.225 |
| DBA | 0.180 | 0.005 | 0.470 | 0.370 | 0.005 |
| BPe | 0.120 | 0.003 | 0.330 | 0.150 | 0.010 |
| Ind | - | - | - | - | 0.010 |
| Total | 7.415 | 1.274 | 10.560 | 4.820 | 10.215 |

Based on the values given in Table 3 and the TEF values proposed by Nisbet and Lagoy (1992), the calculated B(a)$P_{eq}$ values for the five categories of locations are given in Table 4 . It is evident from Tables 3 and 4 that even though PAH concentration in category 4 is much lower compared to category 5, the total toxicity at category 4 is higher than at category 5. This is due to the emissions from biomass burning include more carcinogenic PAHs than in emissions from vehicles.

Table 4. *TEF values for different categories of indoor environment*

| Category | B(a)$P_{eq}$ value |
|---|---|
| 1 | 1.24 |
| 2 | 0.06 |
| 3 | 3.08 |
| 4 | 2.63 |
| 5 | 2.35 |

Increased risk of lung cancer associated with life time exposure to indoor particulate PAHs was calculated B(A)P at five categories according to EPA estimated potency factor of 8.7 x $10^{-5}$ for life time exposure to a mixture with 1 ng /$m^3$ B(a)P. The estimated risk values are given in Table 5. As can be seen the risk value is higher than the acceptable level of 1 per 100000 in all categories except for category 2 which uses gas as fuel and located in less urban areas.

Table 5: *Unit risk assessment*

| Category | Unit Risk value(100000) |
|---|---|
| 1 | 6.3 |
| 2 | 0.3 |
| 3 | 15.6 |
| 4 | 13.4 |
| 5 | 12.0 |

## 4 Conclusion

This study reveals that the highest mean concentration of indoor particulate PAHs are in houses which use firewood as fuel and located in the highly urbanised areas, followed by office environments which are situated in high urban areas. Third high concentrations were at urban houses that uses gas and the least concentrations were observed in less urban houses that use gas as fuel.

The total B(a)P equivalency results showed the potential health risk to cancer is of concern for residents living in urban areas with wood use as their domestic fuel, gas uses and office environments. The total $B(a)P_{eq}$ concentrations in high urban houses with wood use as domestic fuel, in office environments, and in high urban areas with gas uses exceed the maximum permissible risk level of $1ng/m^3$ of B(a)P. The carcinogenic risk is beyond the acceptable level in all categories other than the category which are located in less urban areas and use gas as fuel.

## References


1. Destaillats, H., Maddalena, R.L., Singe, B,C., Hodgson, A.T., McKone, T.,E. (2008). Indoor pollutants emitted by office equipment: A review of reported data and information needs , *Atmospheric Environment* , 42:1371–1388
2. Kalaiarasan, M., Balasubramanian, R., Cheong, K.W.D., Tham, K.W. (2009). particulate bound polycyclic aromatic hydrocarbons in naturally ventilated multi –story residential buildings of Singapore :vertical distribution and potential health risks, Bulding and environment, 44:408-425
3. Masih, A ., Taneja, A., (2006). Polycyclic aromatic hydrocarbons (PAHs) concentrations and related carcinogenic Potencies in soil at a semi-arid region of India, Chemosphere, 65:449–456
4. Maria, R. M., Santino, O. (2008). Polycyclic aromatic hydrocarbons (PAHs) in indoor dust matter of Palermo (Italy) area: Extraction, GC–MS analysis, Atmospheric Environment, 42:1801-1817
5. Mumford, J L., (1991). Indoor air pollutants from Unvented Kerosene Heater Emissions in Mobile homes, Environ. Sci Technol., 25:1732-1738
6. Nisbet C, LaGoy P. Toxic equivalency factors (TEFs) for polycyclic aromatic hydrocarbons (PAHs). Regulatory Toxicology Pharmacology 1992;16: 290–300.
7. Wickramasinge, P., Karunaratne, D.G.G.P., “ Development of a sampling and analysis method for PAH in atmosphere ”, Annual Research Sessions, University of Peradeniya, Sri Lanka, November 30, 2008.
8. Zhu, L., Lu, H., Chen, S., Amagai, T. (2008)., Pollution level, phase distribution and source analysis of polycyclic aromatic hydrocarbons in residential air in Hangzhou, China, Journal of Hazardous Materials,

# CAPACITY BUILDING FOR SUSTAINABLE ENGINEERING PRACTICE: A CIVIL ENGINEERING EDUCATION PERSPECTIVE

S. Senadheera[1]

[1]Associate Professor, Department of Civil & Environmental Engineering,
Texas Tech University, Lubbock, Texas, USA.
[1]E-mail: Sanjaya.Senadheera@ttu.edu
[1]Telephone: +1-806-742-3523; Fax: +1-806-742-3488

**Abstract:** Civil engineers play a critical role in the planning and design of the built environment around. As a result, they wield enormous influence over the use of earth's natural resources. At this pivotal juncture of increased globalization and accelerated, widespread economic development, demand for earth's natural resources and the impact of development on our ecosystem are increasing rapidly. This is creating an unsustainable scenario, and therefore, it is imperative for educators to prepare the future engineering workforce to become stewards of a sustainable global society. This paper presents some reflections of the author based on experience from developing and teaching a course titled "Material Systems for Sustainable Design" as part of a civil engineering curriculum. The overarching course objective is to provide students with the knowledge on sustainability and create an awareness of the tools that are both available and needed to apply sustainable material selection practices in their future design work.

**Keywords:** Sustainability, Design, Engineering, Education, Materials

## 1. Introduction

A near consensus has emerged within the scientific community on the strong links between the lifestyles of people and global ecological deterioration. It does not come naturally to most societies to think in terms of the long-term impacts of their decisions. Furthermore, globalization has prompted rapid development in many emerging countries that is introducing more people to consumerist lifestyles similar to that of the western world. This has prompted countries to develop creative and innovative ways to educate communities, the younger generation in particular, to understand the long-term impacts of their actions and to find ways to create a sustainable world order. The term sustainability has evolved from an ecology-based concept to one that calls for societies to develop and sustain resources and leave the natural ecosystem in a sustainable mode for future generations.

Sustainability requires living within the regenerative capacity of the biosphere, and it has been reported that the human demand could well have exceeded the regenerative capacity of the biosphere since the 1980s [Wackernagel et al., 2002]. According to preliminary and exploratory assessments, humanity's load corresponded to 70% of the capacity of the global biosphere in 1961, and grew to 120% in 1999. The alarming increase in the rate at which Earth's resources are being utilized prompted the United Nations to initiate a Millennium Ecosystem Assessment in 2001 to assess the consequences of ecosystem change for human wellbeing and the scientific basis for action needed to enhance the conservation and sustainable use of those systems. Evidence of the consequences of non-sustainable use of Earth's natural resources was highlighted 23 years ago [UNDP 1987]. The predictions of possible outcomes of the ecological change that has been taking place include reduced standards of living in the developed world, threats to food security in the developing world and widespread famine. These scenarios may lead to increasing risk of mass migration of populations, social injustice and unrest caused by struggles to share scarce resources. The predictions and the timing of such scenarios vary between sources, but it is clear that the future workforce must be educated more intensely on the impact of the utilization of Earth's natural resources and the manner in which they are used.

## 2. Sustainable Civil Engineering Education

Current global emphasis on sustainability has prompted engineers to design products and services by incorporating principles of sustainability. Sustainability is a multi-faceted and a multidisciplinary issue. Therefore, to work towards a sustainable future, it is important to educate future engineers to develop skills needed to effectively work in multidisciplinary teams. Many techniques may be used to include sustainability within the engineering curriculum. For example, students may be required to analyze case studies and present and discuss the topics learned [Paten et al., 2005]. Additionally, promoting student creativity is an important aspect of sustainability education.

The US Accreditation Board for Engineering and Technology (ABET) updated its accreditation criteria [ABET EC 2000] to allow university engineering programs to be more creative in their education mission. In EC 2000 Criterion 3, ABET reaffirmed a set of 'hard' engineering skills while introducing a second set of six 'professional' skills that were divided by Shuman et al. [2005] into process and awareness skills. This professional skill set has been somewhat controversial, particularly due to ambiguities associated with their assessment, but many advances have been made by institutions in the teaching and learning of these skills. The 'awareness' category of skills includes the following that can be associated with sustainability education.

- A broad education necessary to understand the impact of engineering solutions in a global, economic, environmental and societal context
- A recognition of the need for, and an ability to engage in life-long learning
- A knowledge of contemporary issues

Researchers have observed a much deeper understanding of what it means to include sustainability in design. Shekar (2007) indicated that engineering design projects are active learning tools that may be used to encourage the development of problem-solving skills of students while teaching the concepts behind other issues such as sustainability. Several authors have presented ideas on appropriate ways to teach engineering design within the context of sustainability. De Ciurana and Filho [2006] proposed a novel approach for "greening" of the curricula that included ten characteristics. Morris et al. [2007] suggested that engineering students must first be introduced to design concepts and then sustainability concepts can be gradually introduced into the design process. In addition to innovative design exercises, they recommend problem-based learning as an effective way to teach sustainable engineering design, and that past research has repeatedly shown that these approaches of teaching enhance students learning experiences that motivate them for deeper learning and better retention of learning content.

According to data collected by the US Geological Survey [Figure 1], the construction industry uses by far the largest quantity of natural resources as raw materials [USGS 1998]. Most of these natural resources can be categorized as non-renewable, and therefore, the construction industry has significant impacts on sustainability of Earth's natural resources. Designers of civil engineering projects can effectively contribute towards sustainability through more effective use of new and recycled materials. This will significantly slow down the utilization of non-renewable natural resources. Civil engineering students and the society in general can benefit immensely from course curricula that provide awareness on sustainability implications of various materials available at their disposal. These implications include material supply, recycling potential and ecological implications.

The current practice of specifying construction materials for projects is primarily driven by factors such as status quo and existing pricing mechanisms. However, with the increasing emphasis placed on sustainability and the preservation of biodiversity in all facets of the society, civil engineers must take a closer look at potential benefits to society from sustainable design and construction practices.

The Accreditation Board for Engineering and Technology (ABET) has included knowledge of sustainability in its general program evaluation criteria [ABET 2000]. In addition, the ASCE Body of Knowledge (BOK) for the 21st Century document includes knowledge of sustainability as one of 11 technical outcomes to be met at the bachelor's degree level [ASCE 2004]. The second edition of the BOK recommends the incorporation of sustainability concepts in design courses and to allow students to develop specialized knowledge and skills beyond traditional civil engineering-related subject areas.

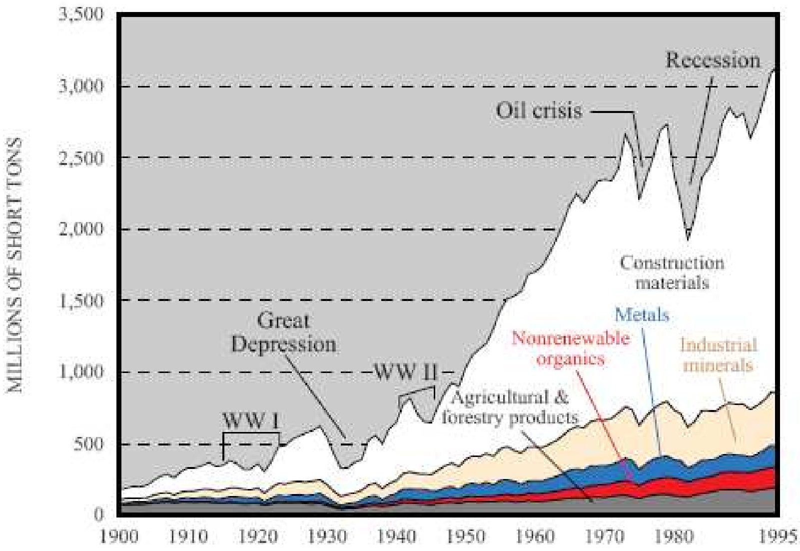



Figure 1. *Raw Materials Consumed in the United States, 1900-1995 [USGS, 1998]*

The development of sustainable built environment systems requires a coherent development strategy encompassing areas such as regional planning and development, engineering design, energy, transportation technologies, environmental quality and human health. It has been suggested that students must not only grasp the principles of these individual subject areas, but also develop skills to integrate such knowledge [Morris et al., 2007].

The typical product lifecycle in a civil engineering project consists of planning and design, material production/processing, construction, operation and disposal phases. Material selection for civil engineering projects is mostly influenced in the early phases of a project lifecycle, i.e. planning and design phases. In cultivating sustainable design practices, the amount of energy utilized and the ecological footprint of materials must be considered. During the material production/processing phase, the concerns are most notably on the energy utilized to produce the material (embodied energy) and the ecological footprint of that phase. The environmental impact is commonly expressed as the amount of greenhouse gas emissions and toxic material releases per unit mass or volume of material produced. During the construction phase, the energy consumed during the construction process (process energy), the greenhouse gas (i.e. carbon) footprint and localized pollution due to the construction process are major concerns. During the operation phase, conservation of energy and minimization of negative environmental impact are of paramount interest. During the disposal phase, it is important that material is non-toxic and recyclable.

In civil engineering design, the conventional approach to material selection typically involves maximizing the use of locally available materials due to the high transportation cost of bulky and heavy construction materials. In order to institute sustainable engineering design practices, the design process has to integrate sustainable development, environment, economy, society and the future [LSF-

LST 2007]. Students can be encouraged to take innovative approaches to material selection and the design of material systems based on a sustainability-based approach that includes material consumption availability and rate of utilization, use of energy to produce the material (embodied energy), process energy expended and the ecological footprint of the material and the related construction process. Sustainable design and material selection also requires the rational use of life-cycle analysis methods to make holistic decisions that incorporate the impact of all factors indicated above.

## 3. Material Systems for Sustainable Design: A New Course

A new course titled Material Systems for Sustainable Design was developed for graduating seniors and graduate students. This course was designed with the objective that it will meet either the senior elective or the design elective requirement for undergraduates. Additional course content was assigned to students who were taking it for graduate credit. The following factors highlighted in the literature review presented in this paper were incorporated in the design of this course curriculum and pedagogy.

1. The study of sustainability requires a systemic (global) approach.
2. Design of sustainable systems requires a holistic approach with a long-term outlook that takes into consideration the overall quality of life of societies.
3. Civil engineers can make a significantly positive impact in creating a sustainable society.
4. Sustainable engineering design is a non-linear process that requires a collaborative effort from a multidisciplinary team.
5. Sustainable design can be effectively taught by first introducing the basic principles of design, and then gradually adding sustainability concepts to the design process.
6. Case studies and team design projects greatly enhance the engineering design experience.
7. Problem-based learning (PBL) approach is an effective way to teach design.
8. The topic Sustainability satisfies the ABET ‘awareness’ professional skill category.

Considering the factors identified above, the course was divided into the following topics.

1. Engineering design process: Design criteria and constraints, working in design teams
2. Role of materials in design: Material characteristics, specifications and markets
3. Introduction to sustainability: Definitions, history, concepts, impacts
4. Sustainable use of materials: Energy, ecology and natural resources
5. Guidance documents on sustainable engineering practice
6. Material flow analysis
7. Sustainability metrics
8. Sustainable design
9. Specifications for sustainable material use
10. Design project
11. Life-cycle assessment (for graduate credit)

The above topics were selected by carefully considering the recommendations presented in the literature review conducted for this paper. Every effort was made to highlight the multidisciplinary nature of sustainable design and its impact to global sustainability. Table 1 outlines the sustainability-related topics and learning outcomes for this new course. The domains of human learning, as defined in Bloom’s Taxonomy, were used as the basis to organize student learning from this course. Learning outcomes identified in Table 1 were developed to ensure that the course addresses all domains of human learning. The course was delivered using a format that combined several techniques including the following.

1. Limited traditional classroom instruction to introduce basic concepts

2. Review magazine articles and books to understand global, holistic outlook of the topic
3. Develop a Material Flow Analysis charts for select materials
    - Each student was assigned a construction material
    - Materials selected to make comparisons for similar applications
4. Review of key publications related to sustainability
    - UN Brundtland Commission Report [UNDP 1987]
    - Sustainable Engineering Practice: An Introduction [ASCE 2004]
    - US Green Buildings Council LEED™ Certification
5. Review and discussion of journal articles on sustainable material use
6. Semester Design Project
    - Students worked in teams, with each team representing a particular application
    - Each student within group developed specifications and metrics for his/her material
7. Life-Cycle Assessment (for graduate credit)

There were twelve students enrolled in this class, and each student was assigned a commonly used civil engineering material, and each student was responsible for the development of a Portfolio for each material that included the following.

1. Existing material specifications (ASTM, AASHTO, etc.)
2. Material market information (market size, availability, pricing, sustainability)
3. Process chart for application-material combination
4. Material flow chart with inputs and outputs identified
5. Sustainability metrics for the material when used in the assigned application
6. Design of a structural component

The Read-Present-Discuss approach turned out to be a very effective method to create a highly collaborative learning environment. It was effectively used in the reading assignments of the Brundtland Commission Report and the journal articles on sustainable material use. Each student was assigned a topic to read, and each student was required to present the key points of the paper in class. Discussions followed each presentation and further discussions were also conducted at the end of all presentations. Each student was responsible for one chapter in the Brundtland Commission report, and one journal publication was also assigned to each student covering a different area of sustainable material use. The topics covered on sustainable material use are listed below.

1. Consumption of materials in the United States
2. Construction materials and the environment
3. Design principles for ecological engineering
4. Environmental taxation of raw materials
5. National materials flows and the environment
6. Natural resource requirements of commodities
7. Sustainable design and construction strategies
8. Socio-economically sustainable civil engineering infrastructures
9. Construction ecology and metabolism
10. Sustainable technologies for the building construction industry
11. Sustainable construction in the United States
12. Management of natural resources

The Project Development and Building Process for a civil engineering structure was presented to the students as comprising of three phases; Plan, Design, Build. Subsequently, the use of the construction product and a sustainable waste management strategy were introduced by using the framework shown in Figure 2. The sustainable waste management strategy presented to the students was developed based on the Reduce-Reuse-Recycle approach developed by the European Commission.

Student learning was assessed using the following assignments. The grading rubric used for this course included 25% for homework assignments, 30% for the design project, 25% for the mid-term exam and 20% for the final exam. The homework assignments included the calculation exercises indicated above as well as Read-Present-Discuss assignments.

1. Calculation of personal carbon and energy footprints
2. Calculation of sustainability metrics for materials
3. Read-Present-Discuss assignments
4. Design project
5. Exams

There were several Read-Present-Discuss assignments included in the curriculum. These included magazine articles, the UN Brundtland Commission report and Journal publications on sustainable material use. This format was a great success and it facilitated a very enriching learning process through dialogue and synergy.

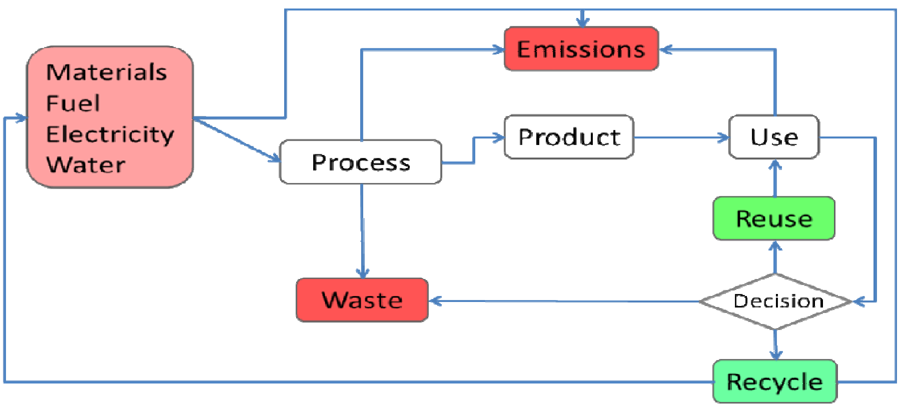



Figure 2. *Framework on Resource Utilization and Impacts for Sustainable Design*

Table 1. *Sustainability Concepts and Learning Outcomes for the Course Material Systems for Sustainable Design*

| Subject Area | Sustainability Topic | Homework Activity | Outcome |
|---|---|---|---|
| Overview of Civil Engineering Materials | • Material properties<br>• Material markets<br>• Material Flow Analysis (MFA)<br>• Embodied and process energies of materials<br>• Impact on the Biosphere<br>• Optimization of material use | For key materials;<br>a. Draw MFA charts;<br>b. Estimate embodied and process energies;<br>c. Study impact on the biosphere<br>d. Study of existing material selection practices | 6. Define MFA<br>7. Design MFA charts<br>8. Calculate embodied and process energies<br>9. Analyze material impacts<br>10. Identify key factors in material selection |
| Sustainability Concepts | • Definitions<br>• Impacts<br>• Metrics | • Study key definitions<br>• Paper review and class discussions on civil engineering design and material impacts on sustainability<br>• Calculation of sustainability metrics for select materials | 11. Define sustainability<br>12. Discuss impacts of civil engineering on sustainability<br>13. Identify parameters used in the calculation of sustainability metrics<br>14. Estimate sustainability metrics for select materials |
| Sustainable Civil Engineering Design Practice | • ASCE Policy on the Role of the Engineer in Sustainability<br>• Other guidelines for sustainable design;<br>• Sustainability metrics for materials | • Read report by ASCE Committee on Sustainability<br>• Read LEED® and other sustainability guidelines<br>• Calculate sustainability metrics for materials | 15. Discuss ASCE sustainability outlook<br>16. Discuss LEED® and other guidelines<br>17. Estimate sustainability metrics for application-material combinations |
| Life-Cycle Assessment(LCA) | • Use of sustainability metrics in LCA | 18. Selection of materials using LCA | 19. Apply LCA to material selection |
| Material Specifications | • Components of a material specification<br>• Sustainability-based material specifications | 20. Develop specifications for sustainability | 21. Apply sustainability concepts to specifications |
| Design Project | 22. Application of sustainability concepts in a real project | 23. Conduct sustainability-based material selection for a simple project | 24. Integrate sustainability to select materials for a simple project |

## 4. Student Feedback

The instructor prepared detailed questionnaires to assess student viewpoints. This questionnaire was in addition to the standard university-run evaluation of the course and the instructor. Where appropriate, student responses were collected both at the beginning and at the end of the course. The results presented below are based on student responses to the instructor's questionnaire. The questionnaires covered the following four areas.

1. Learning Outcomes from the Course
2. Student Outlook on Sustainable Practice
3. Assignment formats
4. Topics Covered

Summaries of student responses to the instructor's detailed anonymous questionnaires are presented in Figures 3-9.

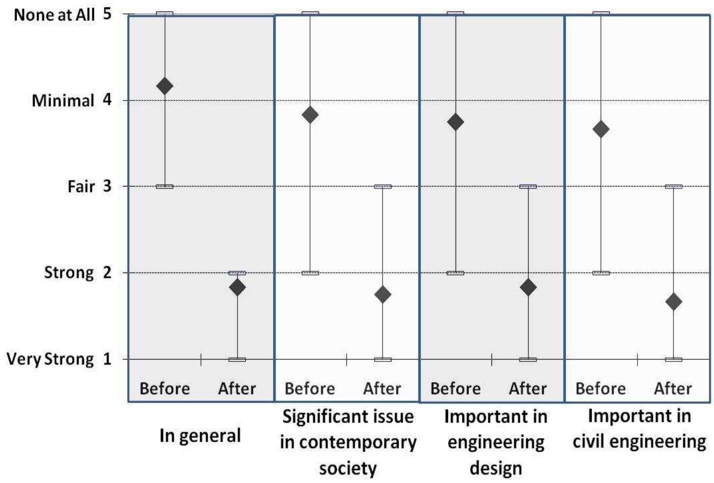



Figure 3. *Student Understanding of Sustainability "Before" & "After" the Course*

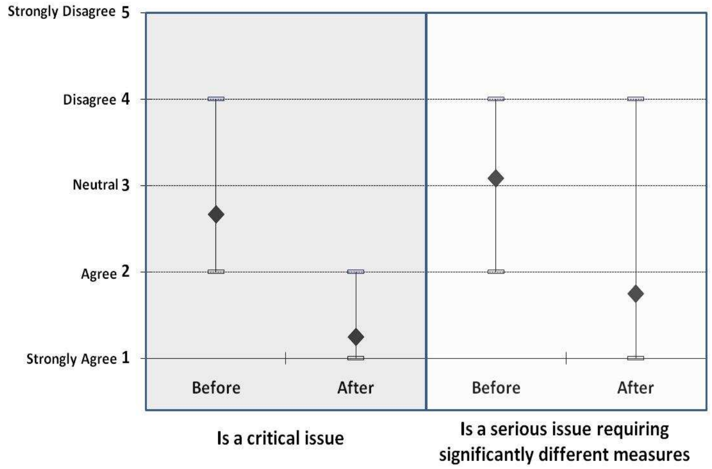



Figure 4. *"Before" & "After" Perceptions: Importance of Global Sustainability*

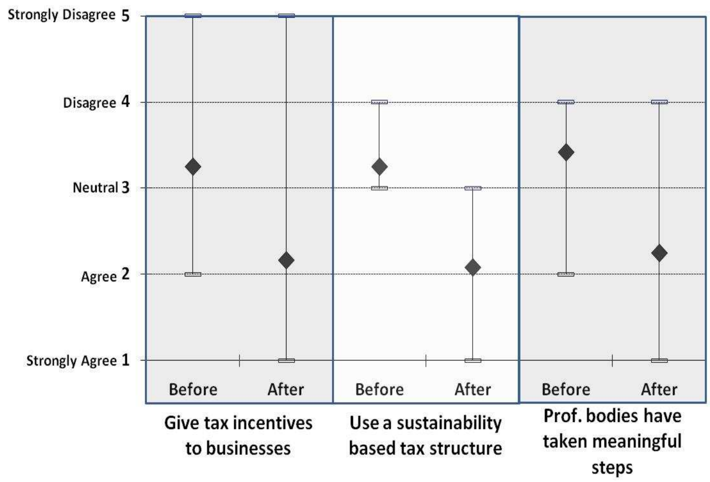



Figure 5. *"Before" & "After" Perceptions: Develop a Sustainability-Based Society*

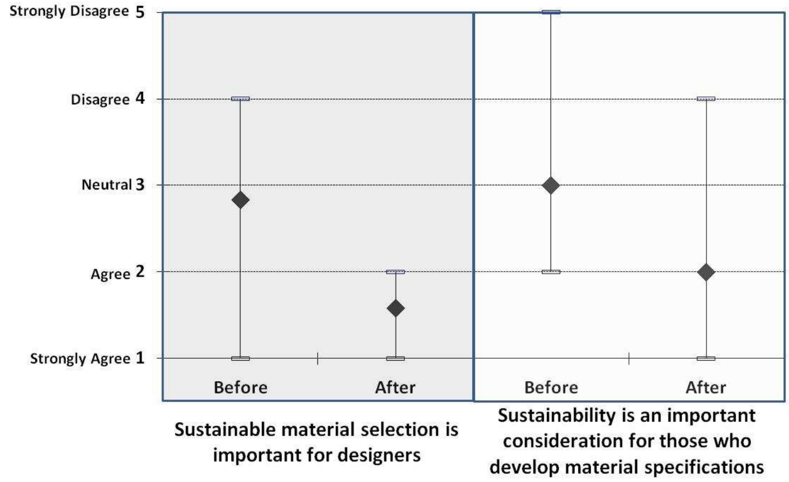



Figure 6. *"Before" and "After" Perceptions: Importance of Sustainability in Design*

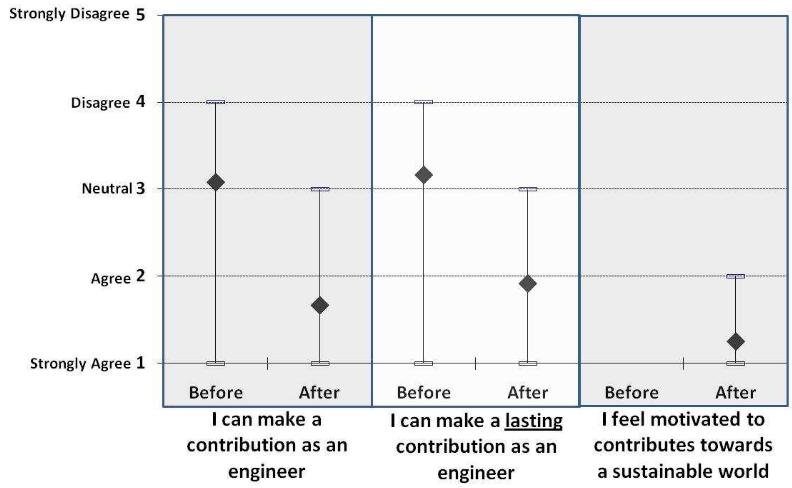



Figure 7. *"Before" & "After" Beliefs: Student's Role for a Sustainable Society*

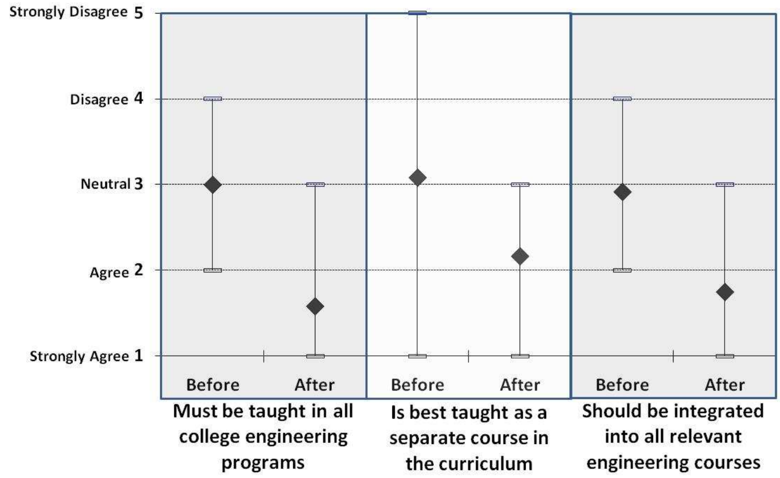



Figure 8. *"Before" & "After" Perceptions: Incorporate Sustainability in Curricula*

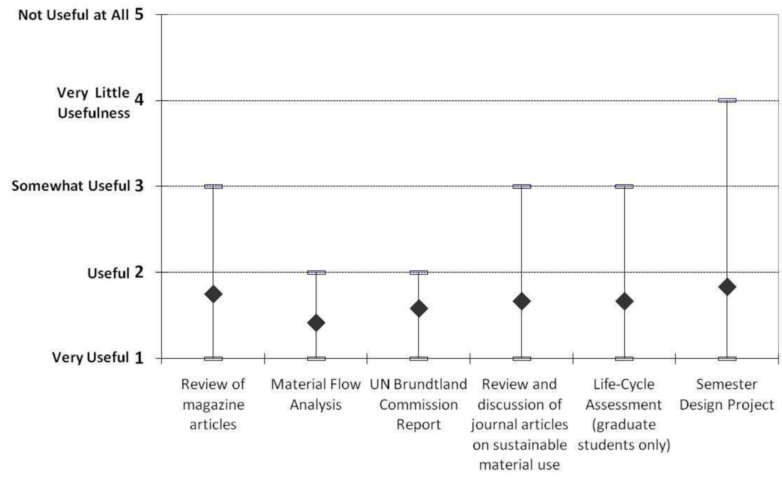



Figure 9. *Student Assessment of Course Delivery*

Based on student responses to the instructor's detailed questionnaire as presented in Figure 3-9 above, allow the following general conclusions to be made.

1. The student understanding of sustainability and its significance in the society in their chosen major field of study improved as a result of following this course [Figure 3].
2. The student group did not have disagreements on the importance of sustainability in the world today. Furthermore, as a result of following this course, students developed a more favorable perception of the importance of sustainability [Figure 4].
3. Students had diverse viewpoints on how to create a sustainable society, both at the beginning and at the end of the course. However, as a group, they seemed to look more favorably at some of the ideas currently prevalent in the literature (Figure 5].
4. The students generally agreed that sustainability is an important aspect of the design process including material selection. Student responses also indicated a strengthening of their outlook on the importance of sustainability in design [Figure 6].
5. The students indicated favorable opinions about their ability to contribute to the development of a sustainable society, and their opinions developed to a more favorable state after the course [Figure 7].
6. The students indicated that sustainability could be taught at the undergraduate level, either as modules in existing courses or as a separate course [Figure 8].
7. The Student assessment of the course generated very favorable responses [Figure 9].

The one significant negative comment the students provided was the difficulty in obtaining data needed to calculate sustainability metrics for individual materials. This course did not assign time for students to develop their own calculation metrics for the material-application combination, and therefore, the students had to rely on studies published by other researchers for such data. The students commented that data published by different researchers for the same material were often significantly different from one another. Not having either one or a few central data repositories to obtain sustainability-related data is a problem that teachers of sustainable material use have to overcome. An organized repository of data will allow students to draw from is an important resource for hands-on class projects.

## 5. Conclusions and Recommendations

This new course was well received by students, judging by student responses to the instructor's detailed questionnaire and from the student evaluations collected by the University. The instructor plans to offer this course on a regular basis.

### References


1. ABET (2000), "Criteria For Accrediting Engineering Programs," Engineering Accreditation Commission, Accreditation Board for Eng. & Technology, Baltimore, MD, 2007.
2. ASCE (2004), "Sustainable Engineering Practice: An Introduction," Jorge A. Vanegas, Ed.
3. ASCE (2004), "Civil Engineering Body of Knowledge for the 21$^{st}$ Century: Preparing the Civil Engineer for the Future," Body of Knowledge Committee of the Committee on Academic Prerequisites for Professional Practice, ASCE, Reston, Virginia.
4. Geli de Ciurana, A.M. and Leal Filho, W. (2006), "Education for sustainability in university studies: experiences from a project involving European and Latin American universities," *Int.l J. of Sustainability in Higher Ed.*, 7(1), 81-93.
5. LSF-LST (2007). "Understanding Sustainability, Learning for a Sustainable Future," <http://www.lsf-lst.ca/en/teachers/understanding.php>, York Univ., Ontario, Canada.
6. Morris, R., Childs, P., and Hamilton, T. (2007). "Sustainability by design: a reflection on the suitability of pedagogic practice in design and engineering courses in the teaching of sustainable design," *European Journal of Engineering Education*, 32(2), 135-142.
7. Paten, C., Palousis, N., Hargroves, K., Smith, M. (2005), "Engineering sustainable solutions program: critical literacies for engineer's portfolio," *Int. J. of Sust. Higher Ed.* 6(3).
8. Shekar, A. (2007), "Active learning and reflection in product development engineering education," *European Journal of Eng. Education*, 32(2), 125-133.
9. Shuman L. J., M. Besterfield-Sacre and J. McGourty (2005), "The ABET Professional Skills – Can they be Taught? Can they be Assessed?," *J. of Eng. Education*, 94(1), 41-56.
10. USGS (1998), "Materials Flow and Sustainability," *Fact Sheet FS-068-98*, U. S. Geological Survey, U.S. Department of Interior.
11. Wackernagel, M. , Schulz, N.B., Deumling, D., Linares, A.C., Jenkins, M., Kapos, V., Monfreda, C., Loh, J., Myers, N., Norgaard, R., and Randers, J. (2002), "Tracking the ecological overshoot of the human economy," *Proc.*, Nat. Acad. of Sciences, 99(14).


### About the Author

**S. SENADHEERA,** B.Sc. (Eng.) (Peradeniya); M.Sc; Ph.D. (Texas A&M) is an Associate Professor in the Department of Civil and Environmental Engineering, Texas Tech University, Lubbock, USA. His research interests are in the areas of civil engineering materials, pavement engineering and engineering education.

# INTEGRATED LAND USE AND MULTIPLE WATER SUPPLY-DEMAND MODELLING FRAMEWORK: A PERI-URBAN CASE STUDY

B. Nawarathna[1], H. Malano[1], B. Davidson[2] and B. Maheshwari[3]

(1) Department of Civil & Environmental Engineering, The University of Melbourne, VIC 3010, Australia
email: bnaw@unimelb.edu.au, PH (+614) 3330-1941.
(2) Department of Resource Management, The University of Melbourne, VIC 3010, Australia.
(3) School of Natural Sciences, University of Western Sydney, Sydney, NSW 1797, Australia.

**Abstract**: The South Creek catchment with an area 620 km$^2$ confronts increased competition between potable water, irrigation and environmental flows. Peri-urban areas also generate a large volume of effluent and stormwater and can often meet some or all the irrigation and industrial water needs provided adequate infrastructure is available. An adequate harmonisation of these multiple supplies and land use using a total system analysis approach leads to a better understanding and evaluation of the limitation and opportunities to enhance the overall performance of the system. This paper descriers the developed modelling framework to simulate water supplies and forecast future demands and integrate supplies and demands in finding water allocations with different climate change and land use scenarios. The integrated model is applied to the South Creek catchment to plan future land use and water supply in an environment of water scarcity under system harmonisation water resources management concept.

***Keywords:*** System Harmonisation, Peri-Urban Catchment, South Creek catchment, Water Resources Modelling

## 1 Introduction

Research and development in water resources management usually involves separate investigations into technical, institutional, environmental, and social spheres; however, with a primary focus on technical aspects. Hard-engineering solutions were implemented without focusing on overall economic and environmental impacts, or the social implications associated with these projects (Spingate-Baginski et al., 2003). With increasing sustainability discourse, there is a realization that the technical aspects of water resources management need to be addressed with the immediate understanding of environmental and social interactions for successful development and application of potential solutions.

Through this research, an attempt has been made to develop an alternative to past approaches for achieving more effective, equitable and efficient water resources management in heavily stressed catchments such as Western Sydney's South Creek Catchment. Different demand management options were explored from a multi-disciplinary perspective through a concept known as 'System Harmonisation'. System Harmonisation involves addressing the hydrological and economic impacts arising from alternative planning and management decisions, as well as identifying and including all affected social, cultural, institutional and policy issues to maximise benefits across the system.

Development and management of catchments require the utmost cooperation between all stakeholders involved, be it a single urban dweller, a farmer, government, corporate or academic (Prato & Gamini, 2007). The main aim of this research is to develop a dynamic generic tool for integrated water resources planning and strategy development in peri-urban landscapes using water resources and economic principles assessed from a social perspective. The tool is adaptable across peri -urban Australia and beyond. The study area has been scheduled for significant land use change due to future development, where a rapid and substantial increase in urbanized areas will be seen; however, maintenance of water supply for existing farmlands, industry, recreation, and environmental services has been highlighted as integral for sustainability of this catchment.

## 2 South Creek Catchment

The South Creek Catchment (Figure 1), located approximately 50 km west of the Sydney. It is a sub-basin of the Hawkesbury-Nepean Catchment. This catchment encompasses diversity with a mix between urban and agricultural uses, industrial, commercial and services-oriented land-use, as well as dedicated recreation areas and various other open spaces. It is approximately 620 $km^2$ and falls within portions of 8 Local Government Areas (LGAs) or councils: Baulkham Hills, Blacktown, Camden, Campbelltown, Fairfield, Hawkesbury, Liverpool, and Penrith.

Exiting population in the catchement, as of 2005, was approximately 390,000 people. With current urbanization plans, population is expected to reach one million 2030. In addition, greenfield development plans are expected to result in dramatic changes in land use (Rae, 2007). There exists the need for integrating water management approaches that considers system water supply, demands, economic impacts of change, as well as overall effects on social, cultural, institutional and political realms.

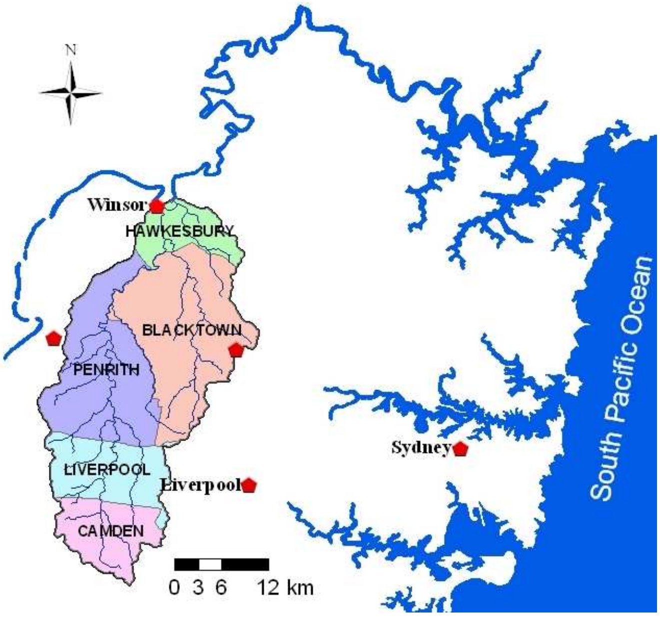



**Figure 1:** South Creek Catchment

## 3 System Harmonisation

In response to an increasing need and support for a more integrated approach to sustainable use of land and water resources in Australia, the Cooperative Research Centre (CRC) for Irrigation Futures has developed a framework (Figure 2) that seeks to align the physical, economic, environmental and social components of water resources management. System Harmonisation is a framework described by Khan et. al. (2008), that seeks to align all components of water resources management to generate and evaluate more appropriate solutions in a transparent manner. It has been recognized that a multi-disciplinary approach is ideal for water resources management, whereby each component be assessed on its own but all elements come together in the overall system framework. The main strength of this approach is that plausible scenarios, developed through extensive stakeholder consultation and social research are evaluated and compared.

The South Creek Catchment will be modelled based on the System Harmonisation method to generate potential solutions for its future with respect to management and development of water and land resources. Furthermore, to simulate management and development scenarios for water resources with regards to supply and allocation, to assess potential economic benefits, as well as the social impacts of each, alternate scenarios can be compared against one another in order to help with the decision making process.

The Social, Cultural, Institutional and Policy component of the system harmonisation program seeks to assess all identifiable social facets that will effect or be affected by any change relating to water resources. It is important that these components be considered in order to maintain transparency throughout the process, and to mitigate any potential externalities that could arise without such an evaluation.

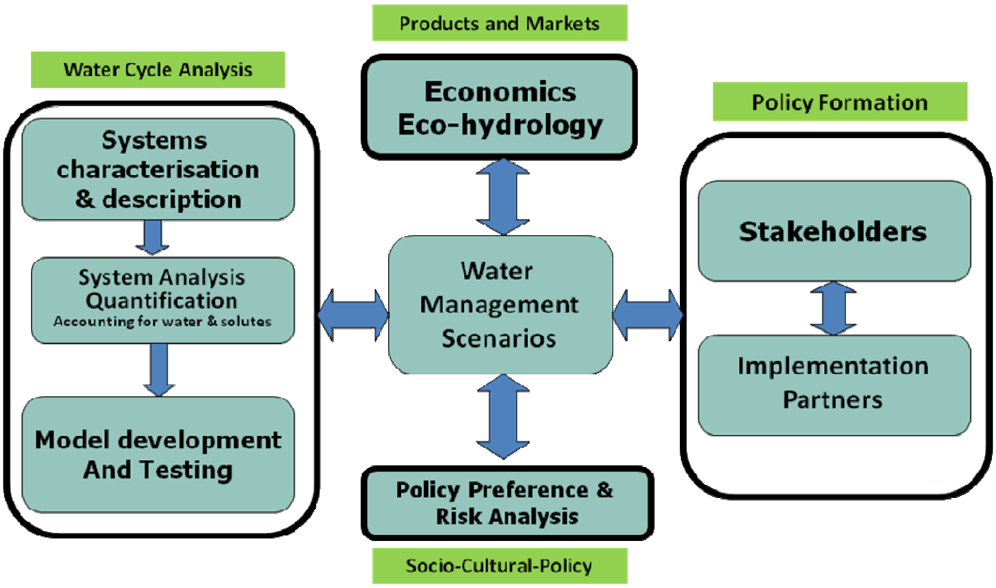



**Figure 2:** System harmonization concept

The products and markets component of System Harmonisation derives values for all uses of water in the system, drawing off outputs from both the water cycle and SCIP research. This method is a means by which impartial and comprehensive evaluation is conducted across a number of regions with a variety of uses of water over a lengthy time period. The costs and benefits of reallocating water are assessed from society's perspective, and are eventually used to evaluate scenarios developed by the catchment's stakeholders.

## 4 Water cycle modeling of the South Creek catchment

The water cycle research consists of three components designed to describe the physical system and its conceptual hydrologic modeling framework. First, the identification of the conceptual system forms the basis for the quantification of the water balance. Second, a critical water accounting process occurs that allows the key stocks and flows in the system to be identified and quantified and provides the data for calibration and validation of the various models. Third, models are developed and applied to evaluate alternative water strategies defined by the stakeholders. Following three main modules are the components related to South Creek water cycle modeling task.

(a) A distributed hydrologic model capable of reflecting the impacts of spatially distributed land use and climate changes on runoff ;
(b) A demand module to estimate water demand for multiple uses including primary production, public open spaces, industrial, domestic and environment; and
(c) A water allocation module that routes quality specific water supplies and demands based on agreed supply priorities.

In order to properly model a hydrologic system, consideration need not only be given to surface water, but also to groundwater, existing land and water use practice, as well as historical data concerning climate patterns. Future available water resource were projected using the expected change in land use and climate using the distributed hydrological model.

In the research, water cycle research consists of two simulation modelling tools: BTOPMC (Block-wise TOPMODEL with Muskingum-Cunge flow routing), a distributed hydrological model to capture the impact of land use and climate changes over the catchment and to assesses available supply, and REALM (Resource Allocation Model) – a water allocation model to link multiple sources and multiple users of water as constrained by water quality and specific legislation. Hydrologic modelling drives the overall modelling framework, as its exogenous changes in the allocation of water to different sectors and regions vary per year. The output of the hydrologic model is input to the economic model. Changes in flow of water govern the flow of net economic benefits (Davidson and Hellegers, 2008).

Modified BTOPMC, a rainfall-runoff simulation tool, is used in an attempt to discuss the influence of land use and climate changes with respect to water resources and discharge in the South Creek Catchment. Developed at the Yamanashi University (Japan), BTOPMC is a physically-based distributed hydrological model based on block-wise use of TOPMODEL with Muskingum-Cunge flow routing method that can be used for runoff simulations in different size of watersheds (Nawarathna et al, 2001).

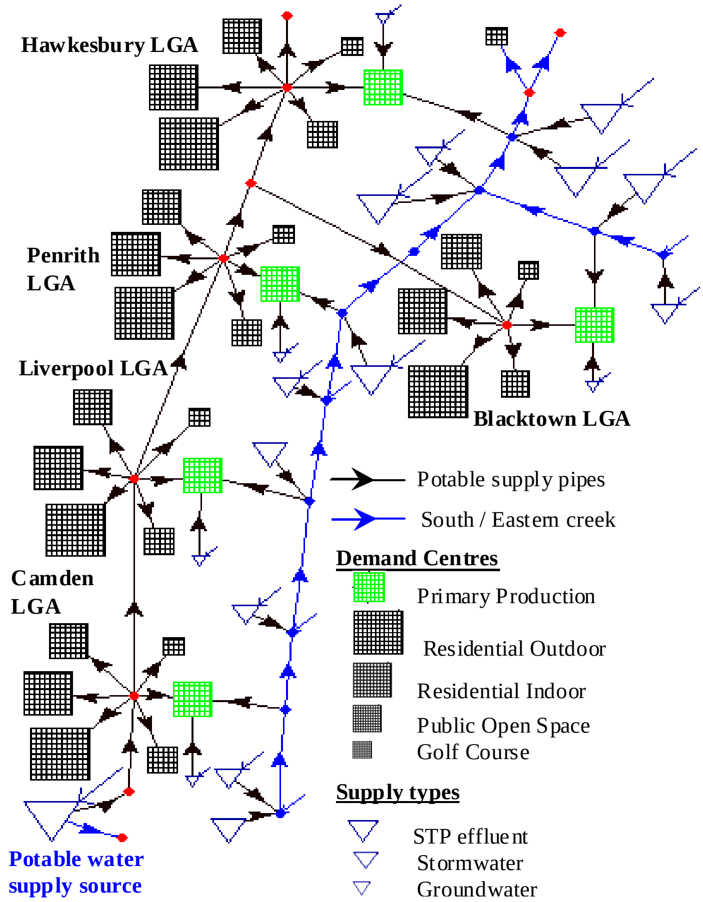



**Figure 3:** Schematic of the study region in REALM

The REALM package was developed and tested over many years by the Victorian Department of Sustainability and Environment in close conjunction with its major users. It simulates water resource distribution in a defined area. It incorporates harvesting and bulk distribution based on allocated supply and demands using mass-balance accounting at nodes in conjunction with a linear optimization algorithm. Furthermore, it operates on a set of user-defined penalties, which act as constraints to generate results leading to preferential resource use (Perera et al., 2005). Allocation modelling is a very valuable tool for planning in a catchment, whereby outcomes of alternate management scenarios are observed. REALM software can cater to environmental flows, issues relating to water entitlements and allocation, future growth in any or all sectors within the study area. Modelling a catchment in terms of resource allocation is a means by which future system requirements can be forecasted.

REALM equitably allocate and distribute water resources in the South Creek catchment based on scenario-specific supply and/or demand and on established operating rules developed through stakeholder consultation. A schematic representation of the South Creek catchment, as built into the REALM model, is presented in Figure 3.

The supply sources in the South Creek Catchment include potable water, surface water, groundwater, treated effluent and treated stormwater. Demands of residential indoor and outdoor, industrial, primary production, open space irrigation, golf and environmental flow are included. Demands were determined based on land-use details, population trends, domestic, commercial and industrial consumption records, as well as advice from the New South Wales Department of Primary Industries (NSW-DPI). Zoning of the system into demand centres, and determining water requirements of each must be determined prior to allocation modelling. Because each zone will have its unique supply and demand, and it will therefore have a unique allocation. What makes the South Creek catchment modelling exercise a unique one is that it has been zoned based on political boundaries, defined by LGAs. Five zones have been identified in the South Creek Catchment (three of the LGAs in the catchment have been included in the more prominent LGAs):

1. Camden (= Camden + Campbelltown)
2. Liverpool
3. Penrith
4. Blacktown (= Blacktown + Fairfield)
5. Hawkesbury (= Hawkesbury + Baulkham Hills)

## 5 Scenario development

The scenarios for the south creek catchment modeling as listed in the table 1 were developed through extensive stakeholder consultation. Developed scenarios are of interest to the stakeholders in the South Creek catchment and the policy makers at a wider level in the Western Sydney Region. These scenarios revolve around the need to assess future urban growth, the harvesting of stormwater, the treatment of effluent and the impacts of the Smart Farms program. Two main land use change scenario namely natural growth and growth centres are described below.

**Table 1:** Stakeholder driven scenarios of the study region

<table>
<tr><th rowspan="2">Land use</th><th rowspan="2">Smart farms</th><th rowspan="2">Effluent reuse</th><th colspan="3">Stormwater harvesting</th></tr>
<tr><th>Public Open spaces</th><th>Industrial</th><th>Residential Outdoor</th></tr>
<tr><td><u>Natural Growth</u><br>Growth predicted to remain constant in future</td><td rowspan="2">Increasing water use efficiency of irrigated agriculture across the Catchment</td><td rowspan="2">High quality effluent from wastewater treatment plants will be allocated for outdoor use, agriculture and open space irrigation</td><td rowspan="2">Use of stormwater to irrigate parks, golf courses, sporting fields and reserves</td><td rowspan="2">Use of stormwater to replace potable water for outdoor use</td><td rowspan="2">Use of stormwater to replace potable water in various industries</td></tr>
<tr><td><u>Growth Centre</u><br>Two Growth centres are considered for future developed in addition to the natural growth</td></tr>
</table>

### *5.1 The baseline scenario: Natural Growth*

Initially, a baseline will be estimated that best represent the 'as is' conditions in the study area. This means that some idea of the conditions and urban growth rates over the next 25 years is required. Constructing this baseline scenario requires extensive data collection prior to model construction, which in turn is followed by an arduous process of calibration and validation to ensure its representativeness. As suggested above, the baseline scenario is arguably the most important scenario estimated; as it becomes the one upon which all other scenarios are compared to.

It is expected that the existing population of approximately 392,000 in 2005 will reach 1 million by 2030. Plans for urban development are well under way in the South Creek Catchment. For the baseline it is assumed that the number of dwellings will expand from 91,650 to 155000 in the catchment (see Table 2).

Most of this growth will occur in the already heavily populated region of Blacktown. Growth rate were calculated based on the derived land use map for natural growth 2030 and growth centres 2030. Of interest is the annual growth, and it is assumed that the growth occurs evenly over the period in question.

**Table 2:** Expected total Number of Dwelling and population in South Creek catchment by Local Government Area under natural growth

| Region | Number of Dwelling | | Population | |
|---|---|---|---|---|
| | 2005 | 2030 | 2005 | 2030 |
| Blacktown | 55400 | 98100 | 204980 | 363000 |
| Camden | 1760 | 2900 | 6512 | 10800 |
| Liverpool | 2070 | 3900 | 7659 | 14500 |
| Penrith | 24850 | 37600 | 91945 | 139200 |
| Hawkesbury | 7570 | 12500 | 28009 | 46300 |
| Total south Creek | 91650 | 155000 | 339105 | 573800 |

### *5.2 The development of urban growth centres*

The South Creek catchment has been identified as the region where Sydney's future growth urban development should occur. In this scenario it is estimated that the population will rise to just under a million people and have nearly 269800 dwellings (see Table 3). It is not only Blacktown that grows markedly, but also Camden and Liverpool. As with the previous scenario it is assumed that the population grows evenly over the 25 years in question.

**Table 3:** Population growth and the increase in the Number of Dwellings that May result from the Growth Centres Policies of the NSW Government in South Creek Catchment

| Region | Number of Dwelling | | Population | |
|---|---|---|---|---|
| | 2005 | 2030 | 2005 | 2030 |
| Blacktown | 55400 | 113300 | 204980 | 419500 |
| Camden | 1760 | 57500 | 6512 | 213000 |
| Liverpool | 2070 | 40300 | 7659 | 149500 |
| Penrith | 24850 | 43400 | 91945 | 160500 |
| Hawkesbury | 7570 | 15300 | 28009 | 57000 |
| Total south Creek | 91650 | 269800 | 339105 | 999500 |

## 6 Results and Discussion

Through various simulations, a hydro-economic assessment can be useful in determining whether or not the implementation of particular management schemes will result in an overall net benefit (or loss) to society. In this section summary of the hydrological assessment results related to change in potable water supply demand and stream flow at the outlet of the south creek catchment in each scenarios are presented. The scenario results are compared with baseline scenario (one in which natural population growth is said to occur, but nothing else).

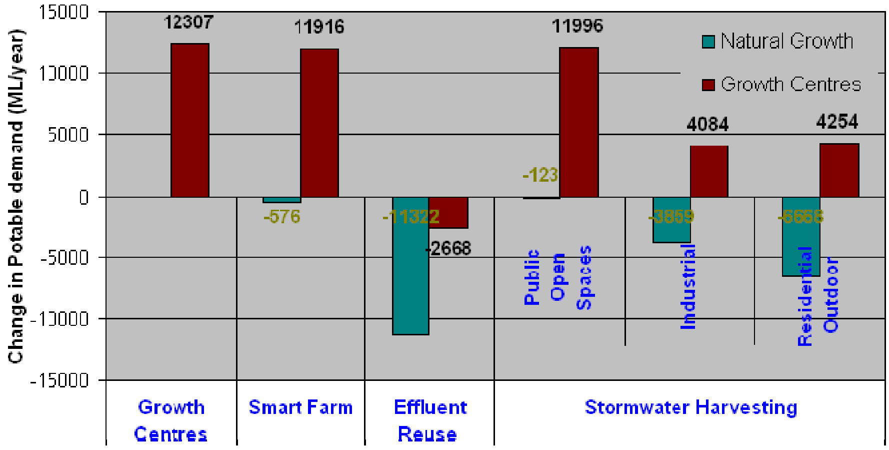



**Figure 4:** Average change of potable water demand with respect to baseline

Figure 4 shows the average potable water saving for the period between July 2009 to June 2030 by water management options like smart farms, effluent reuse and storm water harvesting in comparison to the baseline scenario. As the population increases, potable water demand is also expected increase significantly in the growth centres land use change scenario. Reusing the treated effluent to irrigate public open spaces, golf courses, agricultural lands and outdoor can save water on average 2.6 GL per year. In all the other water management scenarios the potable water demand is not decreasing with the introduction of growth centres. However a significant reduction of potable water demand can be achieved by using stormwater for industrial and residential outdoor. As the primary production demand is not significant compared to other demands, potable water saving by the reduction of primary production irrigation demand by 10% in the smart farm scenarios has not reduced the potable demand significantly. However it will reduce the surface and groundwater extractions from the catchment and increases the stream flows.

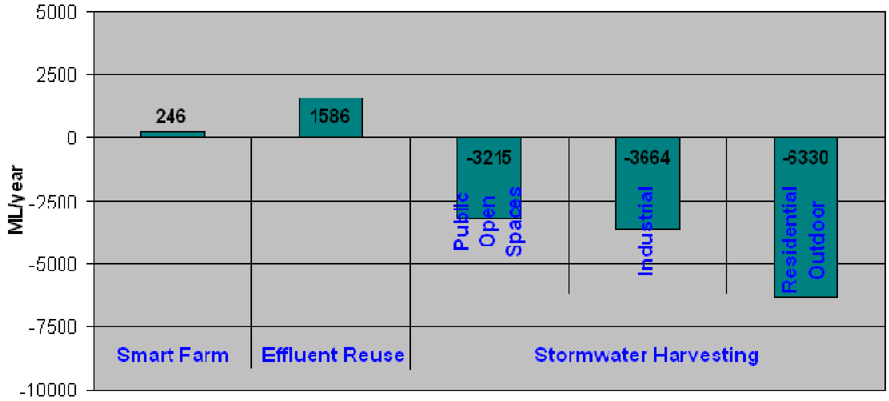



**Figure 5:** Average change of the South Creek outflow with respect to baseline

Figure 5 shows the average change in south creek outflow for the period from June 2009 to July 2030 for each scenario with respect to business as usual case of natural growth. Both in smart farm and effluent reuse water management scenario have increased the river discharge at the catchment outlet due to decrease in surface water extractions. However stormwater harvesting scenarios reduces out flows significantly. In the water allocation modeling monthly minimum flow requirement were set with highest priorities to maintain environmental flows of each LGA. Stormwater can be harvested only after releasing the sufficient volume water to meet minimum flow equipments at each river segments. Water allocation modeling was carried out in monthly time step. However, to consider environmental flow requirements, daily simulations are suggested using other water allocation tools such as Source Rivers.

The proposed modeling framework was used to simulate hydrologic and economic outcomes of different scenarios and compared with the business as usual scenario. But in this paper only hydrological results are discussed.

## 7 Conclusion

This paper discusses a system harmonisation modeling framework applicability for securing future water resources in a peri-urban catchment. The system harmonisation process establishes the physical, economic, and social position of the catchment identifies the key biophysical, economic, social, environmental, or institutional pressure points in the system as well as the system constraints. Changes in these key pressure points need to be assessed and acted upon, in a comprehensive and systematic way, to enhance the multifunctional productivity of a water resources system. With the continual trend of increasing demand and a finite supply, effective management of water resources will be needed to meet these needs in a sustainable manner. In this study, the South Creek catchment was modeled in terms of water resources availability, demand, and allocation with the aim of assessing effects of water redistribution from an economic perspective.

In this study, scenarios to address potential land use and population changes and changes in water management practices that could improve the allocation of water are discussed. A framework whereby integrating water allocation modelling and economic assessments can provide policy-makers with a tool that allows them to make more appropriate decisions with respect change of management and operations strategies. This framework was found to be capable of assessing each scenario relative to the baseline, as well as quantifying the changes in value of water in each sector arising from each hypothesized allocation

## Acknowledgment

The authors are grateful for the funding support provided by the Cooperative Research Centre for Irrigation Futures to carry out this research. The authors gratefully acknowledge the considerable assistance from the members of the WISER project team.

## References

Davidson, B., and Hellegers, P.J.G.J. (in press), Assessing the economic impact of redistributing water within a catchment: a case study in the Krishna Basin in India, *Environmental Modeling and Assessment.*

Heinz, I., Pulido-Velazquez, M., Lund, J.R. and Andreu, J. (2007), Hydro-economic modeling in river basin management: implications and applications for the European Water Framework Directive, *Water Resources Management*, 21, 1103–1125.

Khan, S., Malano, H.M., Davidson, B. (2008), System harmonisation: A framework for applied regional irrigation business planning. *Irrigation and Drainage*. 57(5), 493-506.

Nawarathna, N.M.N.S.B., Ao, T.Q., Kazama, S., Sawamoto, M. and Takeuchi, K. (2001), Influence of human activities on the BTOPMC model runoff simulations in large-scale watersheds, *XXIX IAHR congress proceedings*, Theme A, 93-99.

Perera, B. J. C., James B. and Kularathna, M.D.U. (2005), Computer software tool REALM for sustainable water allocation and management. *Journal of Environmental Management*, 77, 291-300.

Prato, T., and Gamini, H. (2007), Multiple-criteria decision analysis for integrated catchment management. *Ecological Economics*, 63, 627-632.

Rae, D.J. (2007), Water Management in South Creek Catchment: Current state, issues, and challenges. CRC for Irrigation Futures Technical Report No. 12/07, CRC for Irrigation Futures, Australia.

Spingate-Baginski, O., Reddy, V. R., Reddy, M. G., and Galeb, S. (2003), Watershed Development in Andhra Pradesh: A Policy Review. UK Department for International Development.