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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 Katubedda, Moratuwa, Sri Lanka.	University of Peradeniya Peradeniya 20400, Sri Lanka.	University of Melbourne Victoria, Australia	Institution of Engineers Colombo 07, 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 1 of proceedings of ICSBE 2010 contains the research papers that are presented on 13th of December 2010 along with the extended abstracts of keynote speeches. All the research papers of this Volume 1 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

GEOTHERMAL ENERGY IN A SUSTAINABLE BUILT ENVIRONMENT

Prof. Johnston I.W.[1], Narsilio G.A.[2], Colls S.[3]

[1] Golder Associates Chair of Geotechnical Engineering, Department of Civil and Environmental Engineering, University of Melbourne, Victoria, Australia.
E-mail: ianwj@unimelb.edu.au Telephone: +61-3-9035-8034
[2] Lecturer, Department of Civil and Environmental Engineering, University of Melbourne, Victoria, Australia.
E-mail: narsilio@unimelb.edu.au Telephone: +61-3-8344-4659
[3] Research Student, Department of Civil and Environmental Engineering, University of Melbourne, Victoria, Australia.
E-mail: scolls@pgrad.unimelb.edu.au Telephone: +61-3-8344-0193

Abstract: Geothermal energy is usually perceived to be about gushing geysers and bubbling mud pools and limited to only the small volcanically active parts of the Earth's surface. Nothing could be further from the truth. Geothermal energy is in fact an incredible store of energy found in all parts of the world which is beginning to be understood and used for our sustainable future. There are two basic forms of this energy. One form (sometimes referred to as hot dry rocks or enhanced geothermal systems), makes use of the heat (>200°C) in the rocks at depths of up to about 5 kms to produce electricity from extracted (but returnable) hot water. There are several locations around the world where "proof of concept" stage has been or about to be reached suggesting that within the next few years, these systems may be providing a significant proportion of our base-load electricity. The other form makes use of the heat (and the cooling potential) of the soils and rocks within the upper few tens of metres from the surface to heat and cool buildings. It involves the circulation of a fluid through pipes built into building foundations or in specifically drilled boreholes, and back to the surface where heat stored in the fluid is extracted by a heat pump, and used to heat a building. The cooled fluid is reinjected into the ground loops to heat up again to complete the cycle. In cooling mode, the system is reversed with heat taken out of the building transferred to the fluid which is injected underground to dump the extra heat to the ground. The cooled fluid then returns to the heat pump to receive more heat. There are many thousands of these systems installed around the world but many counties have been slow to pick up on their enormous potential. The paper explains how these systems work and looks at some of the issues which require attention in the near future for geothermal energy to become a truly sustainable, renewable and most importantly, continuous, energy source.

Keywords: geothermal energy, hot dry rocks, enhanced geothermal systems, direct geothermal heating and cooling, ground source heat pumps

1 Introduction

Geothermal energy is the heat energy that is stored in the Earth's crust. The three primary sources of this energy are the heat transferred from our planet's core of molten metal, the heat generated by the decay of naturally occurring radioactive materials, and the heat collected through the ground surface from the sun's radiant energy.

There have been many estimates made of what total quantity of heat energy is present in the Earth's crust and how much of this may be available for extraction. One such estimate is that the total heat available within the upper 5 kms of the Earth's surface is about 140×10^6 EJ (WEC 1998). If only 1% of this could be used at the current rate of world energy consumption of about 500 EJ/year, this would provide the world with all its energy for 2,800 years. However, it must be recognised that this energy is also being replaced by heat from the various sources given above. According to Bertani (2010), this amount is of the order of about 660 EJ/year, suggesting that even if we were able to provide all our power requirements from geothermal energy, it would still be fully sustainable.

Humans have made use of geothermal energy since prehistoric times with thermal pools and hot springs ("hydrothermal" energy) being used for heating, cooking, bathing and therapeutic purposes. The first district heating system was introduced to the central French town of Chaudes-Aigues in the 14th century. There are many other modern examples of this application around the world where hot water taken from depths of up to about 1 km is piped to heat a wide range of domestic, commercial and industrial buildings and other infrastructure. Although electricity was first generated using steam from a geyser in Italy in 1904, it was not until 1958 that New Zealand started commercial power generation using separated steam. This form of power generation is now common in many countries with hydrothermal resources close to the ground surface.

It would be fair to observe that the use of geothermal energy up until relatively recently is associated with areas where ground temperatures are generally in excess of what would be the normal range under the majority of the Earth's surface. These areas are mostly linked with the regions of volcanic activity that are found at tectonic plate boundaries. While these limited areas with relatively easy access to hydrothermal energy will continue to be an important source of energy, it is the areas of "normal" temperatures where the future of geothermal energy lies.

There are two basic forms of geothermal energy. One is the indirect form which uses heat extracted from rocks encountered deep below the ground surface to generate electricity. These systems go by a variety of names including "hot rocks" or "enhanced geothermal systems". The other is the direct form which uses the ground within a few tens of metres of the surface as a heat source in winter and sink in summer for heating and cooling domestic, commercial and industrial buildings.

2 Enhanced Geothermal Systems (EGS) for Power Generation

Within about 5-10m depth from the ground surface, the temperature is strongly influenced by the atmospheric temperature, and temperature variations due to daily or seasonal effects can be large. With increasing depth and down to several tens of metres below the surface, the temperature becomes relatively constant and is initially close to the mean atmospheric temperature for any particular location. Therefore, the ground is warmer than the atmosphere during winter and cooler during summer, a generalisation that applies for most locations around the world regardless of geology.

Below this relatively thin surface layer, for regions not influenced by volcanic activity, the average thermal gradient is about 25°C to 30°C increase per kilometre, due to heat flux from within the Earth. However, there are many areas where a much higher gradient has been recorded. These areas typically have at least one of the two important characteristics:

- there is significant presence of radiogenic decay, and
- there are thick highly insulating surface formations which reduce heat loss to the atmosphere.

Figure 1 shows a map of Australia indicating the estimated temperatures at 5km depth. To the east of the central part of the country there is a large region where the estimated temperatures are around 250°C or more. The high temperature gradient for this region far from a tectonic plate boundary is due to high levels of radiogenic activity buried beneath thick, insulating sediments.

Clearly, the greater the temperature gradient, the shallower the boreholes required to reach temperatures which can provide enough heat for commercial power generation. In very broad terms, the minimum temperature at depth which can be used to produce electrical power at the surface is about 150 ^{o}C. However, much more efficient systems may be developed with greater temperature, preferably significantly greater than 200^{o}C.

The basic EGS comprises an injection well to depths of about 4 to 5 kms, an "enhanced" rock mass extending out from the well, a production well and a power station at the surface. Water is forced down the injection well, through the hot fractured rock mass to pick up heat and then returned to the surface via the production well. This hot water is then used to produce steam or another more volatile gas to drive turbines to produce electricity. Figure 2 shows a schematic view of an EGS project.

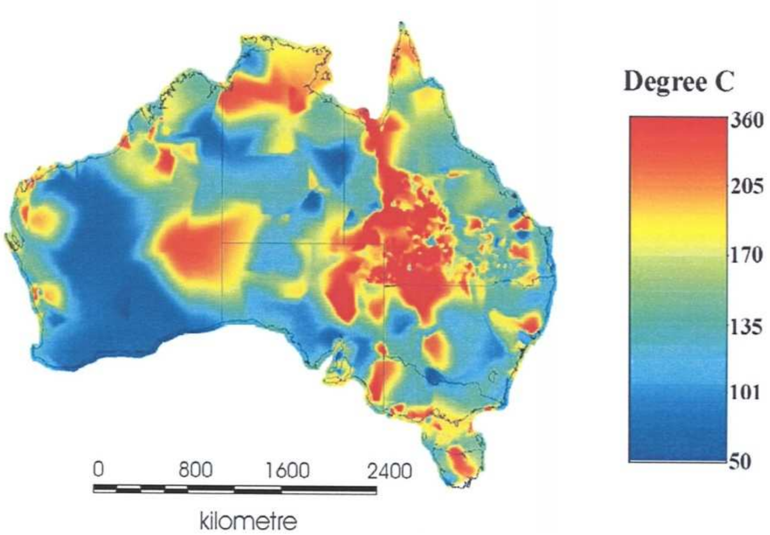

Fig. 1: Estimated temperatures at 5km depth in Australia (Sommerville et al. 1994)

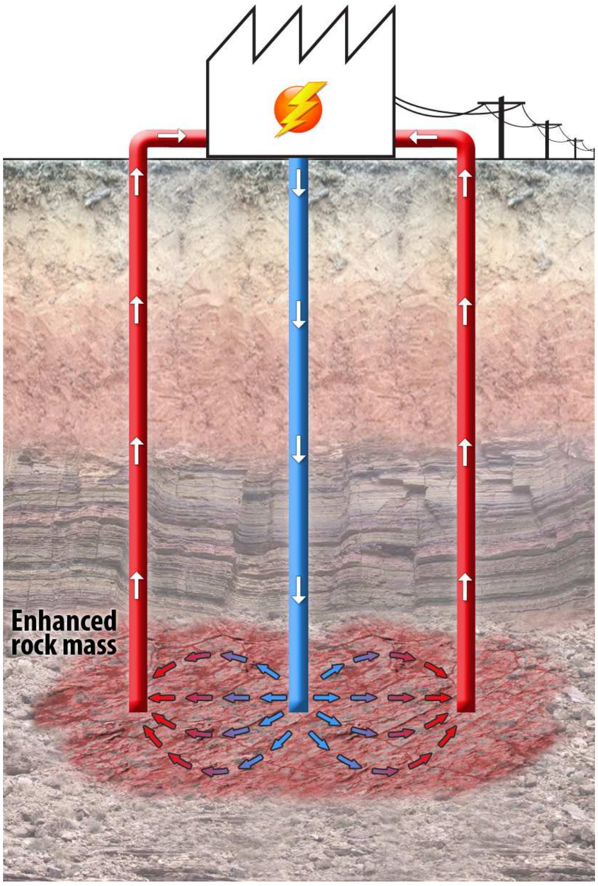

Fig. 2: Schematic view of an EGS project

The cost of drilling the injection and production wells is considerable, and would typically be of the order of $20 million to $30 million for the pair. As well drilling can often account for over 50% of the total cost of an installation, any advances in this area would have a significant impact on total costs. Areas of the technology which are advancing include the use of expandable casing, drilling with casing, more efficient under-reaming techniques to increase borehole diameter, use of more thermo-chemical resistant cements in casing, and probably most importantly, increases in the rate of drilling penetration. This latter factor is the subject of intense research by a number of organisations so that drilling bit life and effectiveness can be improved.

One of the critical steps in any EGS project is the effective enhancement of the rock mass reservoir to make it adequately permeable for economic and sustainable production. This is achieved by packing off the injection well at the target depth and increasing the fluid pressure between packers until the natural fractures in the rock mass open. By controlling the volume and pressure of injected fluid, the permeability will increase and extend out from the well. The fractures propagate in a direction which is controlled by many interacting factors and which can often vary considerably in extent and direction. The hydraulic pressures tend to cause a shearing movement of the fractures leading to small permanent shear displacements of the rough opposing faces of a fracture. As these rough surfaces do not match, the surfaces tend to stay propped open on the many individual asperities on each fracture surface, thereby “locking in” a much higher permeability. It is also common practice to introduce some sand into the fluid which enters the fractures and performs the same propping action.

The water pressures are maintained so the extent of the enhanced reservoir increases to the dimensions required for production. A key part of the enhancement phase is the monitoring of the fracture development by means of acoustic emission measurement. These geophysical techniques allow the many micro-seismic events that occur when the fractures develop to be mapped so that the volume of rock enhanced can be estimated. Once these have been completed, it is then necessary to decide the best location for the production well.

There are many factors which have to be considered, measured and assessed for any one EGS. The most critical are the temperature and volume of the fluid that can be delivered. It is vitally important that the injected fluid can pass from the injection well to the production well adequately quickly so that there is a sufficient volume of hot water. This is clearly related to the permeability and length of flow paths within the rock fracture zone. However, it is also important that the fluid has an adequately long residency time in the hot rock to absorb the heat. Problems arise when the rock becomes too permeable and there is short circuiting between the injection and production wells, so that relatively large volumes of fluid can be delivered but without an adequate level of temperature. Clearly there is a balance that must be achieved between flow rate and heat gain. As a general guide, flow rates need to be in the range of about 30 to 100 kg of water per second with the lower rate applicable to temperatures well in excess of 200°C while the higher rate is applicable to temperatures approaching the current lowest practical temperature of about 150°C.

A vitally important factor for any EGS is its effectiveness over time. For example, depending on the size of the reservoir and its rate of heat replenishment, it is possible to draw the operating temperature of the reservoir down to the extent that it cannot provide water at an acceptable commercial temperature. This may require operations to be slowed down or possibly stopped until there has been adequate recovery. Another major factor involves the fluid flowing through the fractures causing some of the rock to dissolve thereby increasing permeability. While some increase may be acceptable, indeed advantageous, there could become a point at which the flow becomes unsatisfactory. Alternatively, there may be scaling and deposition within the fractures which has the opposite effect of reducing permeability to the extent that the volume yield of water is inadequate. These effects also need careful consideration and understanding. While there are a number of remedial techniques currently under investigation, it is clear that we must develop a detailed understanding of these effects and develop an armoury of methods to ensure long serving reservoirs.

It is clear from the above that there are many factors which can influence the effectiveness of an EGS. Indeed, for the risks associated with an EGS to be adequately reduced so that it can become a commercial reality, we must develop reliable tools to allow us to predict the performance of a reservoir over time. There are many modelling techniques currently being developed which consider the different variables (including fracture network distribution and character, in-situ stress field, response of fracture network to hydraulic pressures, relationship between aperture and permeability, and, of course a number of micro and macro geological features). These then need to be calibrated and validated against the increasing amount of field data that has and is being collected from test locations such as Fenton Hill and Desert Peak (USA), Rosemanowes (UK), Hijiori (Japan), Soultz (France), Landau (Germany), Basel (Switzerland) and the Cooper Basin (Australia).

The production of electricity from geothermal energy involves the removal of hot water from the ground and the conversion of its heat energy to electrical energy. Once the heat is removed, the water is returned to the ground for reheating. As discussed earlier, as it generally appears that the supply of heat from the earth more than balances the heat removed, the system is totally renewable and sustainable. The only part of an EGS that is visible is the power station at the surface and the distribution lines, the rest is underground. There are virtually no greenhouse gases, or other emissions, no major intrusions into the ground surface or waste to be removed and stored.

The one significant environmental issue that does need serious consideration is that when an EGS reservoir is enhanced, the micro-seismic events involved with the fractures opening are in fact small earthquakes. They are the events which are monitored as acoustic emissions to determine the extent of the enhanced reservoir. It follows that seismicity is expected to occur and has been routinely monitored in most EGS sites. For example, in the Cooper Basin in Central Australia, the largest earthquake measured was about magnitude 3.7. However, this region is in the centre of a large seismically inactive continent with very little locally that could experience any damage.

The situation in Basel, Switzerland, however, is very different. A commercial EGS project was commenced in the late 90s. An injection well was drilled to a depth of about 5kms and enhancement commenced at the beginning of December 2006. Almost immediately there was a pronounced increase in seismic activity and an event of magnitude 2.7 was recorded after about 5 days of enhancement. The process was stopped but the seismic events continued for many months. The largest shock was about magnitude 3.4 which occurred a few hours after stopping the enhancement. Slight non-structural damage was reported to have occurred in numerous buildings (usually some fine cracks in plaster walls) and about US$7 million was paid out in insurance claims. It should be pointed out that according to Kraft et al. (2009), the insurance settlements were probably very generous with the damage significantly overstated. Also, events of the magnitudes recorded, although not to be ignored, must not be confused with those highly destructive earthquakes of considerable greater magnitude that regrettably occur from time to time. What must be made clear is that the good citizens of Basel had a good reason for being concerned because the city is located on an active fault. In 1356, with an event of magnitude 6.5, a significant part of the city was destroyed. The lesson which must be learnt here is that a thorough risk assessment must be undertaken with any planned EGS project so that the risk can be rationally rather than emotionally evaluated.

At this point in time, there has been no commercially demonstrated example of EGS generated electricity. However, there are many organisations around the world which are close to this important breakthrough in geothermal technology, most notably in Australia, Europe and USA.

The most comprehensive summary of the many issues involved with EGS is probably the MIT (2006) report, although the recent Stanford Geothermal Workshop (Stanford University 2010) and the 2010 World Geothermal Congress in Bali (International Geothermal Association 2010) provide specialist papers dealing with a range of issues.

3 Direct Geothermal Systems

The other form of geothermal energy is the direct form which uses the ground within a few tens of metres of the surface as a heat source in winter and sink in summer for heating and cooling domestic, commercial and industrial buildings. This highly cost and energy efficient technique is growing rapidly in Europe and North America, but is only just starting to generate interest in other parts of the world. Excellent overviews of these systems may be found in Brandl (2006), Banks (2008) and Preene and Powrie (2009).

Each direct system involves the circulation of fluid (water or refrigerant) through pipes built into building foundations, or in specifically drilled boreholes or trenches, and back to the surface. In heating mode, heat contained in the circulating fluid is extracted by a ground source heat pump (GSHP) and used to heat the building. The cooled fluid is reinjected into the ground loops to heat up again to complete the cycle. In cooling mode, the system is reversed with heat taken out of the building transferred to the fluid which is injected underground to dump the extra heat to the ground. The cooled fluid then returns to the heat pump to receive more heat from the building. Depending on several factors, about 100m to 150m of buried small diameter pipe can continuously provide for most heating and cooling requirements for the average family home. The length of pipe is usually accommodated by a number of vertical boreholes to around 50m deep, although deeper boreholes are commonly used. Fig. 3 shows a schematic view of such a system in which the ground loop system (GLS) is connected to the structure's conventional heating and cooling system via a GSHP. Note that this drawing is not to scale and the borehole would only be up to about 150mm in diameter.

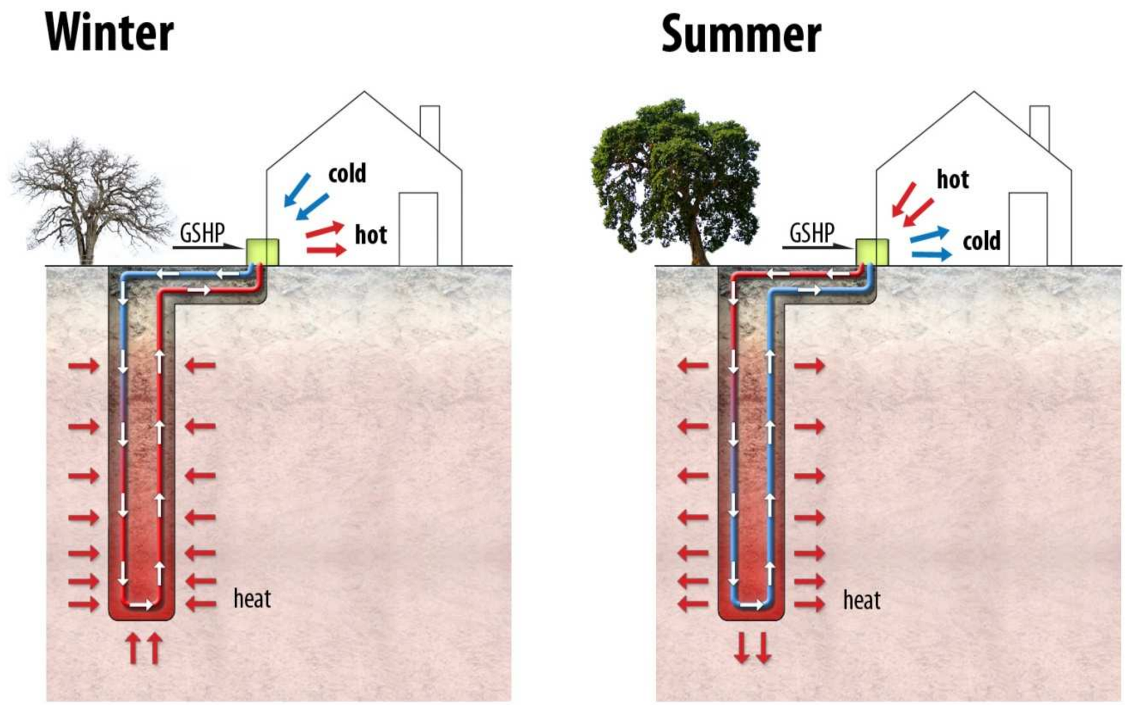

Fig. 3: Schematic view of a direct heating and cooling system (borehole not to scale).

The cost of direct geothermal heating and cooling systems for larger buildings can be reduced by incorporating pipe into their generally larger and deeper foundations, instead of the relatively expensive drilling of separate boreholes.

Figs. 4 to 9 show a number of configurations of GLSs. The installations in Figs 4, 5 and 6 would be appropriate for relatively small buildings whereas the installations shown in Figs 7, 8 and 9 would be appropriate for larger multistorey buildings.

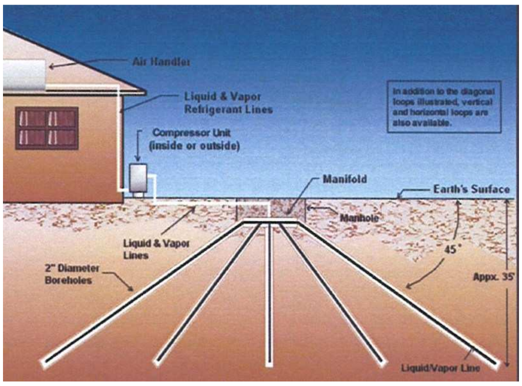

Fig. 4: Possible GLS configuration for a house (Direct Energy, 2010).

Fig. 5: Installation of copper ground loop into small diameter borehole (Payne, 2010).

Fig. 6: Shallow horizontal "slinky" ground loops in trenches (Banks, 2008).

Fig. 7: Lowering HDPE ground loops into the ground on the reinforcing cage of a section of diaphragm wall (Enercret, 2009).

Fig. 8: Horizontal ground loops in a mass concrete base slab (Enercret, 2009).

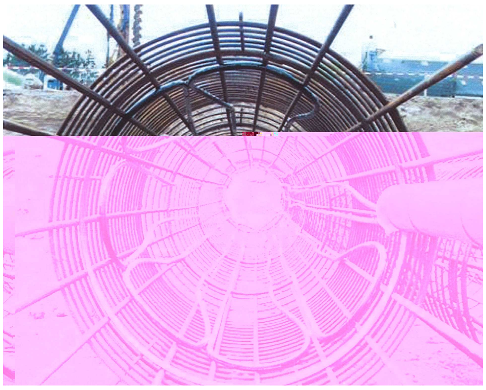

Fig. 9: HDPE ground loops in reinforcing cage of a large diameter pile before vertical installation (Enercret, 2009).

These systems can operate continuously (which wind and solar systems do not), they are relatively maintenance free over a long period of time and their costs are modest with capital expenditure recovered rapidly. There are several buildings in Europe which are supplied by well over 1.5MW of heating and cooling. They provide a very effective means of significantly reducing the carbon footprint of any building.

The two key elements of any system are the GSHP and the GLS. The GSHP is, in effect, a powerful domestic refrigerator which is capable of taking heat out of one area and delivering it to another. Therefore, when working in one mode, it takes heat out of the fluid in the GLS and delivers it to a building to be heated. When switched to the cooling mode, it takes heat out of the building and delivers it to the GLS for disposal to the ground. The operational characteristics of a GSHP are clearly linked to the performance characteristics of the GLS. GSHPs typically operate with a coefficient of performance (COP) of around 4. This means that for every 1kW of electricity used to power the heat pump, 4kW of thermal energy is produced. A slightly smaller energy advantage is obtained when operating in the cooling mode. This represents a considerable saving in power consumption (and thus the need for its generation) for heating and cooling.

Vertical or near vertical GLSs generally extend to depths well below the zone of influence of the surface air temperature. As noted above, this depth is generally about 5 to 10m below the surface where the ground temperature is effectively the same as the weighted mean annual air temperature. This is about 15°C for Melbourne, with Tasmanian temperatures a few degrees lower and Queensland temperatures generally getting above 20°C. GLSs extending well into this zone of virtually constant temperature are more efficient than GLSs placed closer to the surface where temperatures vary according the season (and within about a metre of the surface additionally according to the time of day). The near surface GLSs are generally installed in horizontal trenches or within structural ground slabs (see Figs 6 and 8). The reason for the lower efficiency of near surface horizontal GLSs is that these shallower regions will be cooler in winter when heat is required and warmer in summer when heat needs to be dumped. However, as it is generally much cheaper to place GLSs in locations closer to the surface than at greater depth, a trade off with a greater length of shallow GLS to allow for a lower efficiency may often provide a more effective overall economic balance.

As is indicated in Figs 4 to 9, GLSs can have many configurations with the energy output dependent on many factors including the type of ground involved, materials used, the installation geometry and the fluid flow rates. While it is possible to broadly estimate the energy delivered on the basis of these variables, current technology only allows a very approximate estimate which could include large variations and inaccuracies. This is not a satisfactory situation as installations could be significantly under- or over-designed, leading to systems which are neither cost effective nor competitive. There is clearly a need for more accurate design measures. The reason for the approximate nature of the performance predictions is primarily because direct geothermal applications up until recently have been driven by the heating, ventilation and air conditioning (HVAC) industry whose main concern has been with the above ground technology. It is only recently that the importance of the below ground GLS component of the overall technology has been seen as requiring significant research.

It follows that in order to develop comprehensive design data, considerably more research is required into the performance of GLSs. There is a need for answers to questions such as: i) how do different ground conditions influence the overall energy output, ii) what effect is there with different geometrical system configurations, iii) how does the pipe spacing and the materials used for the different components within the ground affect overall performance, iv) how does energy output and performance vary with fluid flow rates (laminar vs turbulent) and different fluids (water vs refrigerants vs mixtures).

4 Concluding Comments

EGS and direct geothermal heating and cooling using GSHPs are new and emerging technologies that have the potential to significantly reduce the world’s dependence on carbon based energy sources. The technologies are complementary: EGS has the potential to significantly increase supply of “clean” electricity and GSHPs the potential to reduce demand. Furthermore, and unlike the better known forms of sustainable energy, geothermal energy is available 24 hours a day, 7 days a week.

Commercial application of EGS technology is some years into the future but its enormous potential demands investment for further research. Direct geothermal energy and ground source heat pump technology is available now – policy makers, the construction industry and the public need to be educated and the potential of these systems demonstrated. Research should focus on optimising ground loop and GSHP systems to maximise economic, as well as environmental, benefits.

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About the Authors

IAN JOHNSTON, B.Sc.(Hons)., Ph.D. University of Southampton, UK, FIEAust, C.Eng. is the Golder Associates Chair of Geotechnical Engineering at the University of Melbourne. His research interests are in the areas of foundation engineering, soft rock engineering and geothermal energy.

GUILLERMO NARSILIO, B.Eng.(Hons)., National University of Cordoba, Argentina, M.Sc. Georgia Institute of Technology, USA is a Lecturer in Geotechnical Engineering at the University of Melbourne. His research interests are in the areas of the mechanics of porous media, numerical modeling and geothermal energy.

STUART COLLS, B.Eng.(Hons)., B.Sc. University of Melbourne is a research student at the University of Melbourne undertaking a Ph.D. in design of direct geothermal systems.

LOW CARBON BUILDINGS AND TRADITIONAL MATERIALS

Prof. Pete Walker
BRE Centre for Innovative Construction Materials, University of Bath, Bath, UK

ABSTRACT

The necessity, driven by Government legislation, to deliver more energy efficient buildings with significantly lower climate change impact is supporting the development of new uses for traditional materials, such as straw, hemp, timber and unfired clay. Many traditional, also called natural, materials offer low carbon and sustainable sources of materials for modern building. Plant based materials store carbon through photosynthesis. Unfired clay requires very little processing. However, traditional materials face many challenges in the reintroduction into modern construction, including: the high cost of labour (in the UK); shortage of skills in design and construction; limited performance data; and, limited supply chain.

This paper briefly summarises some recent developments in the UK in the use of unfired clay masonry, hemp-lime composites and modern prefabricated straw bale building. The BRE Centre for Innovative Construction Materials at the University of Bath is leading research into a variety of low carbon building materials.

INTRODUCTION

Plants, including trees (timber), reed, straw, bamboo, and hemp, have provided the raw materials for building throughout much of human history. Many vernacular techniques around the world continue to relay on natural and locally sourced materials. Although the industrial revolution displaced many natural technologies, concerns over the environmental impacts of building and infrastructure have stimulated a renewal of interest in plant crop based materials, such as hemp and straw, and use of unfired clay building techniques such as rammed earth and cob.

Using plant based materials reduces the climate change impact of building development, achieved through use of a sustainably grown renewable resource and the atmospheric CO_2 used up by the plants during their growth. Plant based materials offer other benefits, including very high levels of thermal insulation, passive hygrothermal regulation of building spaces, providing healthier living spaces.

The paper briefly summarises some recent developments in the UK in the use of unfired clay masonry, hemp-lime composites and modern prefabricated straw bale building. The BRE Centre for Innovative Construction Materials at the University of Bath is leading research into a variety of low carbon building materials.

HEMP-LIME CONSTRUCTION

Hemp-lime is a lightweight composite building material that combines renewable plant based aggregates (hemp shiv) with a lime based binder. It is a non-structural material used for walls, roof and under-floor insulation. It is used together with a structural frame, typically of timber construction. The lime binds together the hemp together and protects the shiv from biological decay as well as providing fire resistance.

The shiv used in hemp-lime is sourced from the stem of the hemp plant (*cannabis sativa*). Hemp is an industrial bast fibre crop grown as a break crop in the UK between April and September. After harvesting the higher value outer fibres are stripped away from the inner stem. The shiv is made by cutting the inner stems into chips between 4 and 25 mm in size. Around 60% by mass of the hemp plant is comprised of shiv, with typical total plant yields in the UK of around 8–12 tonnes per hectare.

Hemp shiv is very hydrophilic, with a capacity to absorb up to 450% of its own weight in water. Rapid de-watering of the binder by the hemp during initial mixing can significantly impair the hydraulic set. This has led to the development of specialist formulated lime based binders for hemp construction. These formulated binders are blends of hydrated lime, cement, and pozzolanic additions (such as ground granulated blast furnace slag). The binder initially sets hydraulically and then hardens further through carbonation.

Hemp-lime materials are either cast (lightly tamped) inside formwork (figure 1) or spray applied in-situ (figure 2). Mix proportions of hemp shiv, lime binder and water vary depending on use (wall, floor or roof mixes) and the method of application (casting or spraying). Cast walls are lightly tamped horizontally inside formwork. Sprayed hemp-lime is applied in vertical layers against one layer of permanent formwork. Following initial set and drying the hemp-lime is finished off with a breathable plaster or render, generally lime based.

As well as hemp-lime hemp fibres are also used in construction, to produce insulation quilted batts. Products often include polyester fibres, to improve ‘loft’ and stability. Hemp-fibre insulation quilt is suitable for a wide variety of building applications, including external walls. Like many other natural materials it is hygroscopic, which enhances its heat storage capacity.

Current research on hemp-lime construction at the University of Bath is focussing on basic characterisation of materials, better understanding of hygroscopic aspects of behaviour and investigating structural performance of composite walls. Although hemp-lime has modest structural properties (compressive strengths around 0.1-0.2 N/mm^2 typically), by encapsulating the structural timber framework it offers sufficient restraint to prevent premature buckling failure of the compression studs, enhancing compressive load capacity by up to 4-5 times.

MODERN STRAW BALE CONSTRUCTION

Straw bale construction has been used, to varying extents, for over 100 years. Bales may used to form modest loadbearing walls or as in-fill material in framed construction. Bales are a cheap and abundant renewable crop based resource. But despite success in a number of largely self-build projects, there has been little wider uptake of straw bale. Barriers to wider adoption include uncertainty over technical performance characteristics (especially durability and fire resistance of straw) and high manufacture costs of on-site construction subject delays due to inclement weather. Although modular in size (nominally 450 mm wide x 350 mm high x 1000 mm long) dimensions and compaction density can also vary significantly; bale lengths can easily vary by ±100 mm. This causes problems for construction and robust detailing. Loadbearing construction relies on a period of settlement prior to the application of the render, prolonging the period of construction.

Development of off-site prefabricated straw bale panelised construction seeks to overcome these barriers. One approach, ModCell™, uses timber framed panels in-filled with straw bales. Panels are typically 490 mm thick, with varying heights and widths to suit. The thermal transmittance of these panels is 0.19 W/m^{2}K. In construction the straw bales are compressed vertically to improve their structural robustness and dimensional regularity in preparation to receive plasters, renders or timber based fascias. On completion of straw infill and compaction the internal and external faces of the panels are covered in a formulated lime based render (figure 3). It is important that the bales remain for prolonged periods at moisture content levels below which biological decay are likely to occur (around 25%). The facings must therefore protect the bales from direct weathering but also allow water vapour to escape as part of a breathing wall system.

The University of Bath has played a leading role in development of the ModCell system. Tests completed to date include lateral wind load resistance (in and out of plane), fire resistance, thermal transmittance and acoustic resistance.

UNFIRED CLAY MAOSNRY

Although traditional earthen architecture, such as rammed earth and cob, has stimulated a renewal in interest of building with unfired clay, there is limited scope for wider adoption of these technologies in modern building in the UK. In particular their reliance of labour renders them more expensive compared to contemporary methods of building. However, the use unfired clay brick masonry offers modern cost effective alternative to fired clay or concrete blocks masonry in internal applications, where only modest structural requirements are necessary, such as low rise housing.

A focus on thin (100-150 mm) thick unfired clay walls has necessitated research on mortar development as traditional unfired clay mortars have insufficient bond strength for such slender walls. Work at the University of Bath has successfully developed alternative clay based mortars using novel stabilisers, such as sodium silicate and lignosulphinate.

SUMMARY

Natural building materials, such as hemp, straw and unfired clay, offer exciting new opportunities, broadening the range of materials available to the designer, builder and client. Although their use still remains a small fraction of mainstream construction, interest in these technologies has never been greater. Initiatives in response to climate change, including the Code for Sustainable Homes, has provided the framework and stimulus for this increased interest and encouraged growth in the use of lower carbon building materials. To enable wider market development requires further research and innovation to overcome technical and commercial barriers, supported by both government and private investment.

Figure 1. Cast hemp-lime panel

Figure 2. Spraying hemp-lime (Lime Technology Ltd)

Figure 3. Spraying lime render on ModCell panel

SUSTAINABLE DEVELOPMENT, ANCIENT WISDOM AND SRI LANKAN TECHNOLOGY

M. P. Ranaweera

Emeritus Professor, Department of Civil Engineering, University of Peradeniya, Peradeniya, Sri Lanka.
E-mail: ranaweer@civil.pdn.ac.lk
Telephone: +94-81-2393547-8 ; Fax: + 94-81-2393500

Abstract: The world is facing many problems due to the improper exploitation of its resources and lopsided developments with total disregard to the environment during the last few centuries. To overcome these problems sustainable development is advocated and many concepts have been introduced recently. However the examination of the history and religions of the world reveals that many of these concepts were embedded in the wisdom of ancients, and they practised sustainable development for thousands of years. In Sri Lanka, Buddhism had a strong influence on the lives of people and ancient Sri Lankans developed a hydraulic civilization, in harmony with the environment, and achieved significant progress in engineering and technology. This paper deals with ancient Sri Lankan technologies related to irrigation and water management, architectural and structural engineering and metallurgy.

Keywords: *Sustainable Development, Ancient Wisdom, Sri Lankan Technology*

1 Introduction

Tremendous advances in science and technology over the past hundred years or so have uplifted the quality of life for many people in the world. However these also have created many problems, such as pollution, global warming, nuclear weapons, etc. which are now threatening the very existence of the mankind.

One of the main reasons for this sad state of affairs is the exploitation of natural resources with total disregard to the environment by various organizations. Wrong notions held early times that human beings are supreme in this world and that they can exploit the natural resources in anyway they want, have also contributed to this situation. The power struggle between countries - especially between industrialized ones, and the lack of effective means to control the advances in technologies also are contributing factors.

To come out of this crisis all nations of the world should unite and decide on common strategies, which should be accepted and implemented by all, but unfortunately this does not seem to be happening. Many “solutions” proposed have also come from industrialized countries, the creator of most of the problems.

In this scenario it is important to look at the philosophy of our forefathers, their technologies and the way they came to terms with the environment they lived in. We may not be able to use their solutions directly, but we can learn much from their ancient wisdom, in finding solutions to these global problems.

2 Sustainable Development

In the recent past, dangers faced by the humanity due to wrong and over exploitation of natural resources have been discussed in many forums, and in 1983 the United Nations appointed the Brundtland Commission on Environment and Development [1] to address growing concerns about the accelerating deterioration of the human environment and natural resources and the consequences of that deterioration for economic and social development. Having studied poverty in the world, concerns about the acute pressure of population growth, modern technology and demands on the planetary fabric, global warming, depletion of world's natural resources, etc, the Commission delivered its report on *Our Common Future* in 1987 [2].

The Commission found that population growth, most of which is among the world’s poor, was not the major threat to the harmony of the planet. It was not the poor who were consuming most of the Earth's supply of fossil fuels, warming the globe with their carbon emissions, depleting its ozone layer with their CFCs, poisoning soil and water with their chemicals, or wreaking ecological havoc with their oil spills. In fact, their consumption of the world's resources was minute compared to that of the industrialized countries. Brundtland declared that poverty in the developing world was less cause than effect of environmental degradation, and only sustainable development could blend the fulfilment of human needs with the protection of air, soil, water and all forms of life - from which, ultimately, planetary stability was inseparable [1].

Thus the concept of *sustainable development* was introduced, as a pattern of resource use that aims to meet human needs while preserving the environment so that these needs can be met not only in the present, but also for generations to come. The Commission defined it as *the development that meets the needs of the present without compromising the ability of future generations to meet their own needs*. It contains within it two key concepts: the concept of needs, in particular the essential needs of the world's poor, to which overriding priority should be given, and the idea of limitations imposed by the state of technology and social organization on the environment's ability to meet present and future needs [2].

With the introduction of the concept of sustainable development, many related concepts came into prominence. Of these sustainable architecture is directly relevant to this conference theme. Sustainable architecture describes an energy and ecologically conscious approach to the design of the built environment, and seeks to minimize the negative environmental impact of buildings by enhancing efficiency and moderation in the use of materials, energy, and development space [3].

Energy efficiency over the entire life cycle of a building is a primary goal of sustainable architecture. Architects use many different techniques to reduce the energy needs of buildings and increase their ability to capture or generate their own energy. This is very important for Western countries where energy cost, for heating and cooling, is considerable. Other considerations are sustainable building materials and the use of recycled or second hand materials, waste management, water management and reducing water demand by means such as grey water reuse and rainwater harvesting.

It should also be pointed out that a universal solution to fit all countries of the world is not possible. Different countries should find their own solutions based on their environmental, cultural, political, economic and social factors.

3 Ancient Wisdom

Human beings have been in existence in the world for about 200,000 years, reaching full behavioural modernity around 50,000 years ago. Until around 10,000 years ago, most humans lived as hunter-gatherers, and about 6,000 years ago, the first proto-states developed in Mesopotamia, Egypt's Nile Valley and the Indus Valleys. The late Middle Ages saw the rise of technologies, and the Industrial Revolution in the 18^{th} –19^{th} centuries promoted major innovations in transport, energy development etc. With the advent of the Information Age at the end of the 20^{th} century, modern humans live in a world that has become increasingly globalized and interconnected.

Although interconnection between humans has encouraged the growth of science and technology, it has also led to conflicts and wars and the development and use of weapons of mass destruction. During the last few centuries so called human developments have led to environmental destruction and pollution, producing an ongoing mass extinction of other forms of life that has been accelerated by global warming. According to some worst case scenarios we are not sure of the existence of human beings for another millennium.

In the ancient world the human beings have lived in harmony with the environment for thousands of years. The Australian Aborigines were able to maintain a stable life style, although primitive from our standards, for more than 40,000 years in a very hostile environment. Ancient Egyptians maintained a civilization for more than 3000 years, and produced engineering marvels, some of which we still fail to understand fully. Ancient Chinese were the masters of invention and discovery for three millennia, and many of the greatest inventions have their foundations in China. Hence it is useful to look at the wisdom of our ancients which embodies many concepts which we think are modern.

Judge Christopher Weeramantry, former Vice-President of the International Court of Justice, has referred extensively to sustainable development and ancient wisdom in many of his writings [eg. 4], speeches [eg. 5] and judgments [eg. 6]. His latest book, Tread Lightly On The Earth [4], deals with the environmental wisdom contained in the teachings of some great religions of the world; Buddhism, Hinduism, Christianity, Islam, and Judaism.

Referring to Buddhism, he points out that “its basic teachings integrate with concerns for the future and the case of future generations, and that the respect for all living things – including plants and animals, is an integral part of its attitude towards the environment”. He refers to the sermon of the Buddhist missionary Thera Mahinda who brought Buddhism to Sri Lanka from India in the 3rd century BC, preached to the King Devanampiyatissa of Sri Lanka (this is given in Mahawamsa – the Great Chronicle of Sri Lanka [7]). “O King” said the Thera, “you may be the king of this country but you are not the owner of this land. You are its trustee and you hold this land for the benefit of all those who are entitled to use it both now and the generations to come”. Weeramantry points out that “here succinctly stated over two thousand years ago, was a cardinal principle of environmental law which we moderns tend to think we have recently invented or discovered”

Referring to Hinduism, Weeramantry points out another example of the relevance of ancient wisdom regarding the legality of weapons, such as nuclear weapons, which go beyond the purpose of war. He points out that “in the Indian classic Ramayana this was expounded in the context of the availability of a hyper-destructive weapon which could decimate the population of the enemy and devastate his landscape. In a far more civilized manner than is now the case, Rama, the prince to whom the weapon was offered, was told that before he used such a weapon he should consult the sages of the law. The sages of the law advised Rama that he could not use this weapon in his conflict with Ravana, King of Sri Lanka, because the purpose of war is to subjugate your enemy and live in peace with him thereafter – not to massacre his population and ravage his countryside”.

The Buddha has specifically referred to recycling in giving instructions to his monks regarding the preparation and the use of the robe. Visuddhimagga or The Path of Purification [8] gives details regarding the preparation of a robe from refuse material such as; a rag dropped on a charnel ground, a cloth thrown into a place for rubbish, a rag thrown away after wiping up blood stains, etc. Cullavagga [9] gives a conversation between the King Udena and Thera Ananda regarding the use of the robe. The Thera told the King that once the robe is worn thin after use, it will be used as a bed spread. When it is also worn out it will be used as a mattress covering, afterwards as a ground covering, then as a door mat until it is worn out. Afterwards it will be shredded, kneaded with mud, and used for plastering.

The above is only a few examples where the ancients, all over the world, in the past have adapted environmentally friendly and sustainable solutions in their developments. We will not be able to use these solutions directly, but we can learn a lot from them in tackling current environmental problems faced by the world.

4. Ancient Sri Lankan Technologies

Sri Lanka has a recorded history dating back more than 2500 years. The ancient Chronicles Mahawamsa [7] and Chulawamsa [10] cover the period from around 6th Century BC to 12th century AD. Foreign visitors to the Island and officers from occupied forces, especially the British, have written accounts of the Island, its people, and their technological achievements. Of particular interest are the books by British civil servants Tennent [11] and Parker [12]. More recent books by Sri Lankan authors also cover the history of the Island during ancient times [eg. 13, 14, 15], and its history of engineering [16].

These accounts show that the ancient Sri Lankans had great respect to the environment and all living things, and that they lived harmoniously with the environment. Remains of ancient engineering works also show that ancient Sri Lankans had engineering skills, very advanced for their times, related to irrigation and water management, architecture and structural engineering, as well as materials and metallurgy. Remains of large irrigation works with reservoirs dams embankments and channels, mega structures like Stupas, monasteries and palaces, rock fortresses and pleasure gardens, as well as weapons and appliances, amply illustrate this. Buddhism which was deeply established in Sri Lanka in the 3rd century BC, had a profound influence on the lives of people and the developments which had significant sustainability. Following is a brief account of some of the major technological developments of ancient Sri Lanka.

4.1 Irrigation and water management.

Sri Lanka traditionally has been an agricultural country and ancient Sri Lankans have demonstrated superior skills in irrigation and water resources development. In the dry zone of the country which gets rain for only 3-4 months of the year, they built reservoirs (also called tanks) to store rain water directly falling on catchments as well as to store water diverted from perennial rivers (Fig. 1).

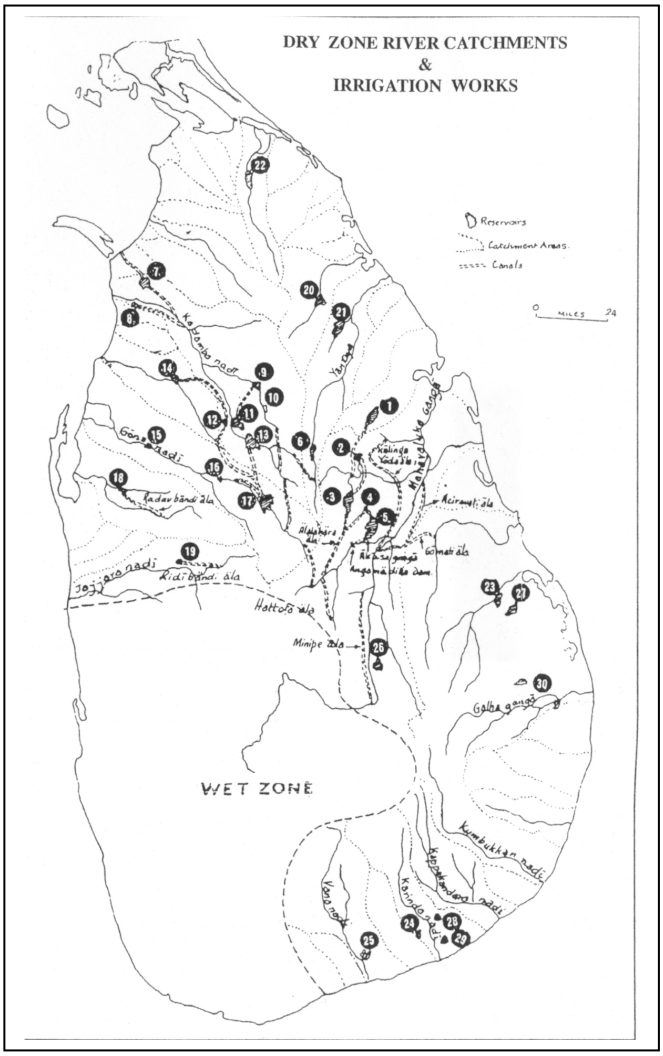

Figure 1 Dry Zone River Catchments and Irrigation Works [15]

As a result a Hydraulic Civilization heavily dependent on irrigation systems for its agriculture systematically developed in the country from about 6th century BC to form a large and complex system to exploit land and water resources. “It is a system which, while recognizing the need for development and vigorously implementing schemes to this end, at the same time specifically articulated the need for environmental protection and endured that the technology it employed paid due regard to environmental considerations”[6].

Chronicles [7, 10] and others writings [esp. 12, 17] give wealth of information on the irrigation works carried out from time to time by ancient rulers of the country. King Pandukabaya (5th century BC) is credited with the construction of the Basawakkulama reservoir, which is still used at present. From then onwards the construction of reservoirs and irrigation systems continued up to the end of the reign of King Parakramabahu I (1153-1186 AD), the great reservoir builder. His Parakrama Samudra, or the Sea of Parakrama (Fig. 2) is the largest ancient tank, covering a water spread area of 2100 hectares, with an embankment 14 km long having an average height of 12 m. In all around 30 large reservoirs (Fig. 1) and more than 25000 small ones have been built during this period.

Fig. 2 Parakrama Samudra (Sea of Parakrama)

King Parakramabahu had declared that “no drop of water should flow into the sea without serving the interest of man”. Chronicles also have records such as “this irrigation system was undertaken for the benefit of the country and out of compassion for all living creatures”. These statements have notions of optimal use of resources for the benefit of all living things, and demonstrate that ancient Sri Lankans had practised sustainable development. In his separate opinion on the case concerning the dispute between Hungary and Slovakia over the use of waters of river Danube [6], Judge Weeramantry has used Ancient Irrigation System of Sri Lanka as an example of sustainable development some 2500 years ago.

Ancient Sri Lankans have used much originality and ingenuity in developing their irrigation systems. Rainwater which fell on a catchment was collected in a cascade of small tanks (Fig. 3), and used and re-used many times before coming to a large reservoir, thus giving expression to the royal dictum of Parakramabahu [18]. In addition to supplying water for cultivation of paddy, the water stored in the cascade system was used for domestic bathing needs, livestock needs, inland fisheries, etc. These series of small tanks also reduced the silting problem in the large reservoir, and they were maintained collectively by the villagers.

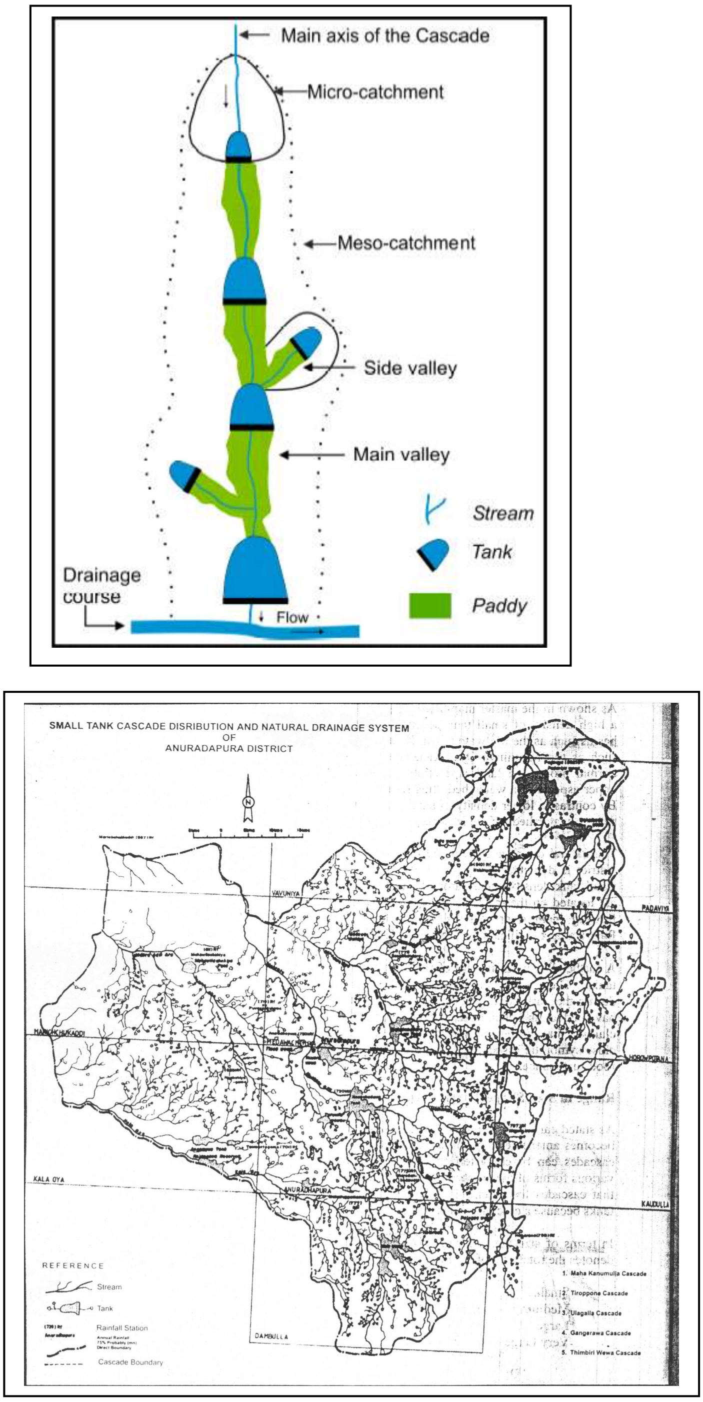

Fig. 3. A Small Tank Cascade and a Distribution of Cascades [18]

Issue of water from the large reservoir, which will have water depths in excess of 10 metres was a difficult task. This problem was overcome by ancient Sri Lankans with an ingenious creation called the Bisokotuwa (Fig. 4) built near the point where the water level meets the inner slope of the dam bund. It is an open well with rectangular section having faces lined with stonework and timber flanks, and an inlet culvert and an outlet culvert placed at its bottom. Sluice gates regulated the water inflow and outflow. Bisokotuwa also served as a place where de-silting can be done. Parker [12] states " whatever form the design took, it was a triumph of the ingenuity of the ancient Sinhalese engineers. It was this invention alone, which permitted them to proceed boldly with the construction of reservoirs that still rank among the finest and greatest works of this kind in the world". Thus the ancient Sri Lankans were the inventors of the valve-pit more than 2000 years before it was used in the West in the 19th century.

In the case of river diversions to feed tanks, the ancient Sri Lankans used oblique dams rather than square dams across the rivers (Fig. 5). Parker observes " the Sinhalese possessed profound practical knowledge of the best methods of dealing with water, and realized that an oblique dam would have greater stability; and the blow of a log would have much less tendency to displace a stone of an oblique dam" than a square one built perpendicular to the current. To convey the diverted water from the river to the reservoir and from one reservoir to another, canals were used, some of them very long and ingeniously located. Jaya Ganga, the right bank canal of the Kalawewa reservoir conveying water to Tissa Wewa reservoir, is 91 km long and 12 m wide. The canal feeds a number of small tanks on the way, and over its first 30 km, the canal has a gradient of 1 in 10,000 (6 inches to a mile), an accuracy hard to achieve even today.

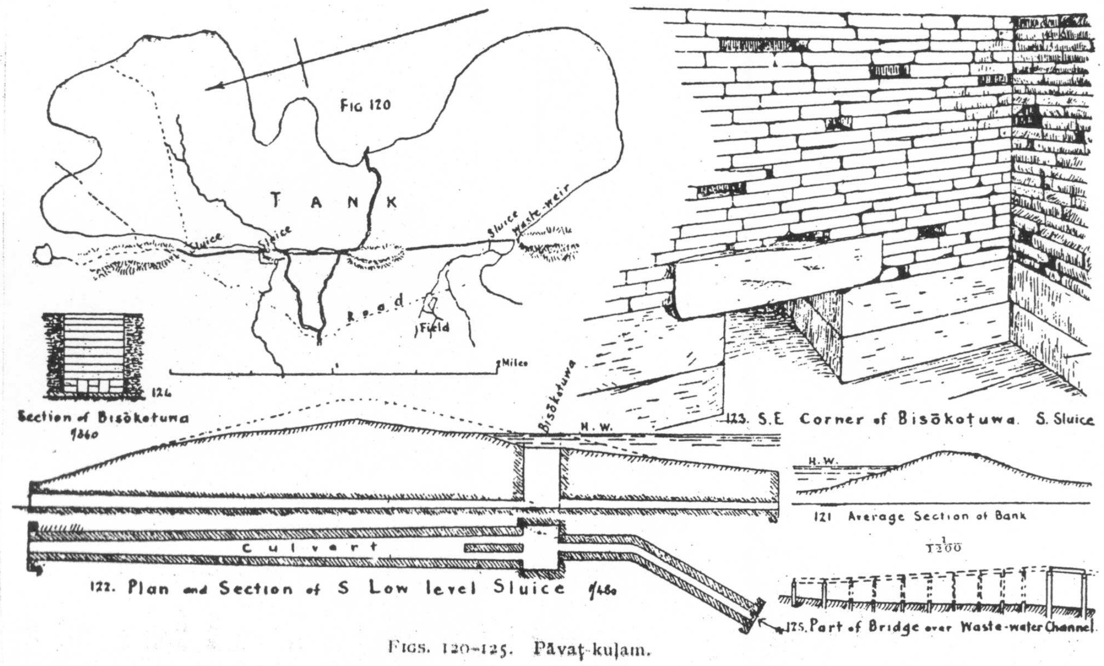

Fig. 4. Pavatkulam Bisokotuwa [12]

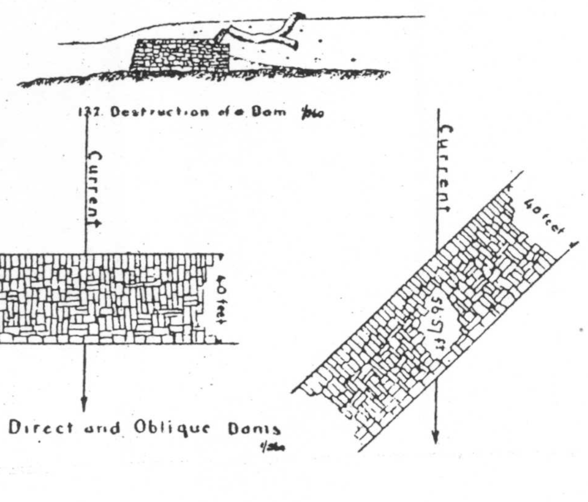

Fig. 5. Oblique Dam at a River Diversion [12]

The location of the reservoirs and their sluices were also done in the best possible way. When the modern engineers, after a thorough study of the terrain selected a place to locate the dam for Maduru Oya Project, they found an old dam and a sluice at precisely the same place!

Ancient Sri Lankans not only built colossal reservoirs, river diversion structures, canals and canal structures, but also established procedures and proclamations for proper management of water resources. There were inscriptions regarding good practices, rights and obligations of cultivators, seasons for crop harvesting, etc.

As Weeramantry puts it in his Separate Opinion [6], “This system of tanks and channels, some of them two thousand years old, constitute in their totality several multiples of irrigation works involved in the present scheme”. “They were executed with meticulous regard for environmental concerns, and showed that the concept of sustainable development was consciously practised over two millennia ago with much success”, by ancient Sri Lankans.

4.2 Architecture and structures

Ancient Sri Lankan architecture was significantly influenced by Buddhism, and most of the important buildings are of religious types. Significant constructions are Stupas [19], monasteries and temples [20], palaces and fortresses. Materials used for construction were indigenous and consist mainly of earth, stone, brick and timber.

Fig. 6. Ruwanveli Stupa(2nd century BC)After Reconstruction.

Stupas designed and constructed in Sri Lanka are among the largest brick structures in the world [21, 22]. They are solid structures and house corporal remains of the Buddha or mark an important event or place associated with his life. From the time Buddhism was established in the 3rd century BC, Kings of Sri Lanka built Stupas to honour the Master, and these are venerated by the Buddhists. At the beginning Sri Lankan Stupa was similar to the Indian one, but with the passage of time it changed to a style of its own which later spread to other Buddhist countries like Thailand and Myanmar. Ruwanveli Stupa (Fig. 6), built by King Dutugamunu in the 2nd century BC, is the most venerated Stupa in the Island and it has a height of 103 m and a base diameter of 91.4 m. The Jetavana Stupa (Fig. 7) built by King Mahasen (276-303 AD) attained a height of 122 m making it, at that time, the tallest brick structure in the world, and the third tallest structure in the world (surpassed only by the two pyramids in Giza). With a total volume of bricks in excess of 300,000 m^3 it, arguably, is still the largest brick structure in the world.

Fig. 7. Jetavana Stupa (3rd century AD)After Conservation

Ancient Sri Lankans showed many skills in constructing these mega structures. The bricks used were of very high quality, much stronger than the modern day bricks, and the mortar used was a very thin slurry (butter clay), which did not weaken the brickwork. The structure was water proofed by a thick plaster giving good weather protection (Fig. 8). Much care was taken in selecting the site and preparing the foundations. Ruwanveli Stupa foundation was constructed with layers having crushed stones, clay, cement, brick, metal, and impregnated with chemicals. Hence it is essentially a reinforced concrete foundation with damp and insect proofing. Many Stupas were located on bedrock and the brickwork started from the foundation level. Setting out and raising the structure was done very precisely. For the domes of mega Stupas, paddy-heap shape (ellipsoidal or paraboloidal), which produces no tension under self weight, was used. Mahawamsa also mentions about a device called Vajrachumbata, fixed at the top of the Stupa, to prevent damage to the pinnacle by lightening. This is fifteen centuries before Benjamin Franklin studied lightening in the West.

Fig. 8. Kirivehera (12th century AD) Showing Original Plaster

Some Stupas of small size were provided with a roofed circular structure called Vatadage. This is a construction unique to Sri Lanka, and remains of the stone columns used to support the roof are present in several places. Fig. 9 shows remaining columns of Thuparama Stupa, built in the 3rd century BC by King Devanampiyatissa, and the conjectured form of its Vatadage [19].

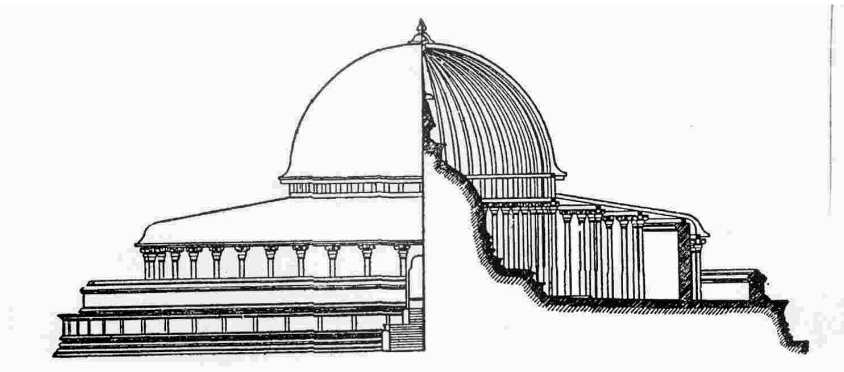

Fig. 9. Thuparama Stupa (3rd century BC) & Conjectured Vatadage [19]

Sri Lanka's ancient monastic architecture [20] displays a rich variety of architectural forms and styles. Monasteries were designed using guidelines given in manuscripts, which outline the layout of various components such as shrines, image houses, assembly halls, etc. They blend well with the environment causing minimum intervention. Efficient and environmental friendly systems were used for water supply and drainage, sanitary requirements, waste disposal etc. Some monasteries were very large accommodating thousands of monks. According to the Chinese monk Fa-Hsien, who visited Anuradhapura in the 5th century AD, there were 5000 monks residing at the Abayagiri monastery and 3000 at Mahavihara.

Mahawansa gives a description of a tall building - the Lovamahapaya (brazen palace) built by King Dutugemunu in the 2nd century BC, and renovated by other Kings. It was used as an assembly hall as well as a residence for monks, and its ground floor was used as a preaching hall too. According to Chronicles it was a 9 storey building built with stone and timber with a copper tile roof. Each floor had 100 chambers, and it could accommodate 9000 monks. With a height of around 49 m, it would have been one of the earliest high-rises in the ancient world. A conjectured drawing of Lovamahapaya [23] is shown in Fig. 10. Presently only the lower stone pillars added by King Parakramabau are remaining.

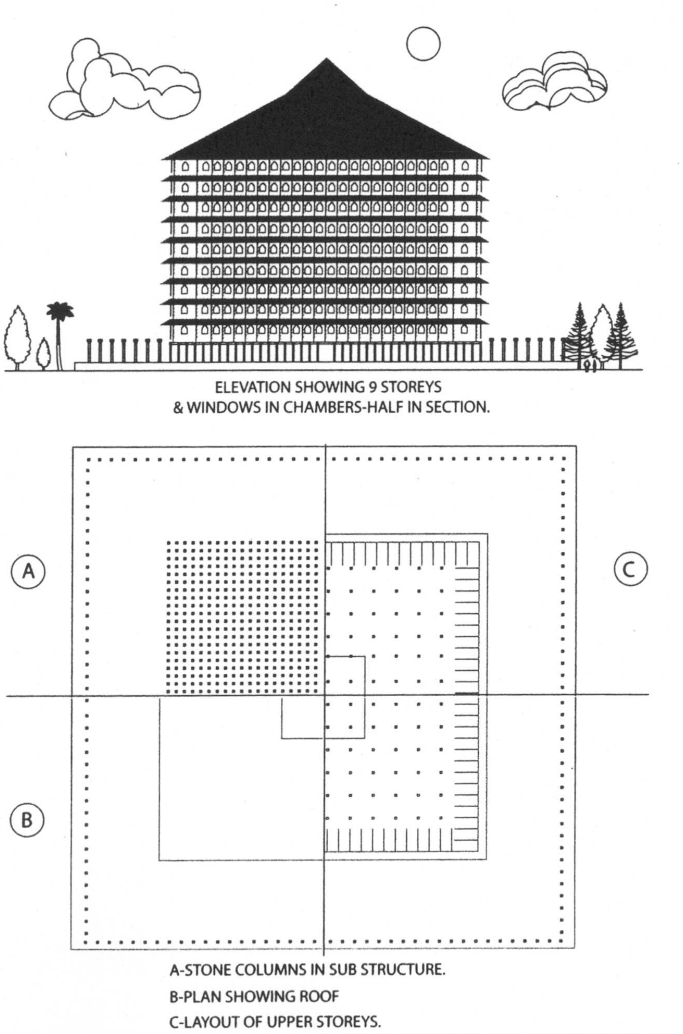

Fig. 10. Lovamahapaya-Conjectured View[23]

Remains of royal palaces are found in the ancient kingdoms of Anuradhapura and Polonnaruwa. The royal palace of Parakramabahu in Polonnaruwa (Fig. 11) was a nine storey building with very thick brick walls. Timber was used for beams as well as for columns.

Fig. 11. Remains of King Parakramabahu's Palace

The rock fortress Sigiriya (Fig. 12), used by King Kassapa (5^{th} century AD) is a meticulously planned royal complex with palaces, water gardens, ponds, and supporting infrastructure. The rock rises 183 m and the palace complex is located on its summit. The water garden in the ground is served with water, conveyed entirely under gravity, using underground conduits made of clay. Its exceptionally engineered hydraulic inflow and outflow conveyance system is a marvel even by today's standards. Sigiriya is one of the oldest landscaped gardens in the world.

Fig. 12. Sigiriya Rock and Water Garden

4.3 Iron and steel manufacture

Historical records show that Sri Lanka was a source of high quality steel, and the presence of slag heaps in many parts of the Island, are evidence of an iron smelting and steel making industry that existed in ancient times. It has been said that Serendib (the name given to the Island by Arabian traders) steels were used for the manufacture of swords in the Arab world [16].

Research by the British archaeometallurgist Juleff [24] has revealed the presence of wind-driven iron smelting furnaces on the hills in the central highlands of the Island, which are exposed to monsoon winds. These furnaces (Fig. 13) are unique to Sri Lanka, and are wind-driven, not wind-blown. It is not the direct wind but the suction created by the wind blowing over the top of the furnace that produces the air blast required to keep the furnace going. Field trials [24] have shown that the furnaces were very efficiently designed and that they were capable of producing high quality steel to sustain a large-scale industry which would have supplied the Islamic world with steel for sword making.

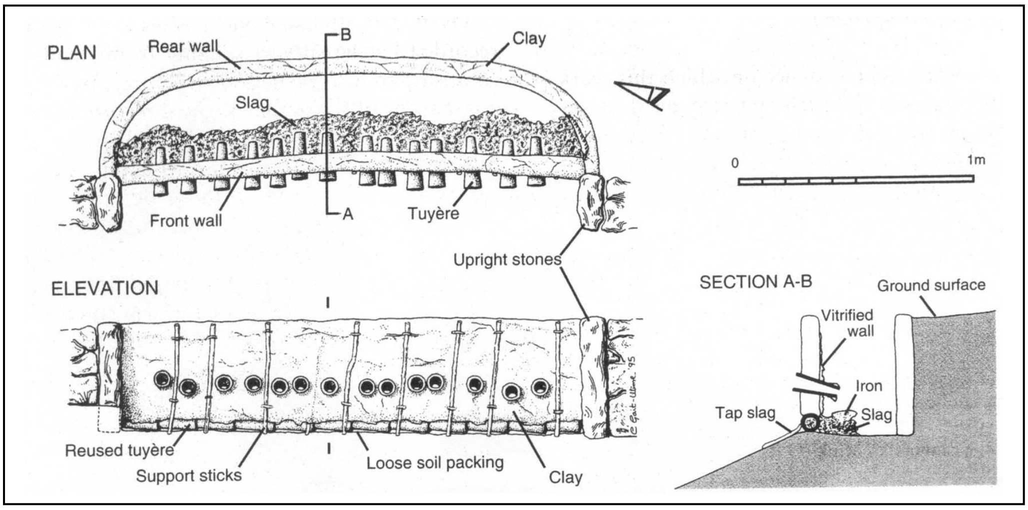

Fig. 13. Wind-powered Iron Smelting Furnace Reconstructed From Archaeological Data[24]

6. Conclusions

Our ancients were much more responsible and wise in dealing with the environment. They respected and protected the environment and practised sustainable development. Many concepts which we think are modern were known to them. In searching for sustainable solutions for development, we should not forget the wealth of knowledge and wisdom of our ancients, all over the world.

While practising sustainable development, ancient Sri Lankans produced significant innovations in irrigation and water management, architectural and structural engineering, and metallurgy.

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About the Author

M. P. Ranaweera, B.Sc.Eng. Ceylon., Ph.D. Cambridge., FIE (SL), FSSE (SL), MASME, C.Eng. is an Emeritus Professor at the Department of Civil Engineering, University of Peradeniya. His research interests are in the areas of computer analysis of structures and conservation and restoration of historical structures.

SUSTAINABLE BRIDGES

Adj. Prof Wije Ariyaratne
B.Sc Eng; M Eng S; Grad Dip Bus; FIE Aust.

Principal Bridge Engineer, Roads and Traffic Authority of NSW, Australia.

Introduction

One of the hottest topics discussed in the international engineering community at present is global warming and climate change. Global warming and climate change would result in frequent extreme events such as high intensity rain fall, severe storms, floods and droughts and sea level rise causing coastal line moving inland, reducing valuable land area and adversity affecting people's living conditions.

In most of the countries, some infrastructures such as bridges, highways, buildings, dams and other structures are approaching their end of design life. Also, the current infrastructure management practices are not geared to increase the sustainability of such infrastructure to an acceptable standards or to meet the demands of future sustainable infrastructure development.

Therefore, strategies need to be developed for keeping existing infrastructure sustainable and building new sustainable infrastructures with minimum consumption of energy, reduction of carbon dioxide emissions and minimum impact on the environment. This can only be achieved through developing a life-cycle management plan that addresses sustainability issues at feasibility, planning and design, construction, operation, maintenance and decommission and/or removal stages.

Sustainable Infrastructure

Sustainability is generally defined as follows:

- Sustainable development meets the needs of present without compromising the ability of future generations to meet their own needs.
- Sustainable development is about achieving economic growth while protecting the environment and ensuring that economic and environmental benefits are available to all of society now and in the future.

By these definitions sustainable infrastructure can be achieved through a life-cycle management plan that addresses environment sustainability, social responsibility and economic growth at present and into the future.

This sustainable infrastructure should possess the following characteristics:

- Durability and longevity
- Preservation of natural environment
- Minimum impact on cultural heritage
- Minimum life-cycle cost
- Safety over whole life
- High performance
- Use of renewable energy

The paper will examine sustainability taking bridges as examples. When studying the sustainability of bridges, it is important to understand why bridges are important to community.

The road network is a critical component of a country's economic infrastructure and it binds communities together and bridges are a vital part of that network. Good roads are a country's measure of civilisation its state of economy and ensure its social cohesiveness and facilitate commerce. This has been true from ancient times and is just as important today and in the future.

The contribution of the transport sector to a country's economy is, and will continue to be significant as it moves into world market. In industrial nations, the use of large vehicles with higher axle loads is being promoted by the transport industry as a means of providing significant cost savings, resulting in increase of national productivity and greater economic benefits to people. This is the situation in Australia, as its transport cost accounts for approximately 20% of average household expenditure. This puts enormous pressure on road authorities in Australia to allow larger vehicles with higher axle loads on its road network.

In Australia, the mass limit review of 1996 recommended that gross mass of six axle articulated vehicles (commonly used heavy vehicles) be increased from 42.5 tonnes to 45.5 tonnes. The review also identified that bridges emerged as the greatest impediment to improve transport efficiency through mass limit increases.

There are over 60,000 bridges in the Australian road network of nearly 900,000km. In the state of New South Wales (NSW) there are 5093 bridges with a replacement value of $13.4 billion under the justification of the Roads and Traffic Authority of the NSW (RTA). These bridges were built with different materials at different times to different standards and loads in varying environments.

Some of these bridges were identified as limiting factors to allow further increases in axle loads necessary to increase Australia's national productivity.

In order to overcome this constraint, the RTA developed a method for load capacity assessment of bridges including load testing to determine the actual load carrying capacities of bridges identified as being under capacity to carry increased loads. By conducting this load assessment process, the RTA minimised the number of bridges to be strengthened or replaced saving millions of dollars to the community, yet keeping the road network open for increased loads.

Life–Cycle Management of Sustainable Bridges

The life-cycle management plan for sustainable bridges, as for any other infrastructure, contains the stages of feasibility study, planning and design, construction, operation, maintenance and demolition or reuse.

Feasibility Study

This stage is where high level decisions are made to meet government strategic objectives and anticipated economic growth. The project needs to be scoped in keeping with these objectives and different options developed and evaluated to minimise the following:

- Energy use
- Carbon dioxide emissions
- Life-cycle costs
- Resource use
- Design and construction cost
- Environment impact
- Community impact
- Heritage impact

Planning

During planning stage different routes are evaluated and a suitable one is selected after extensive value management and risk management studies are carried out by engaging all stakeholders.

Design

Design is a multi-disciplinary process that generally involves bridge engineers, environmentalist, geologist, geotechnical engineers and architects to address the following issues:

- Traffic at present and expected future growth
- Waterway requirements
- Climate change impact
- Mining subsidence
- Geotechnical impact
- Site constraints
- Durability
- Material selection
- Construction operation and maintenance
- Community impact

Having considered the above issues a bridge type is selected that is durable and long lasting with a minimum life-cycle cost, minimum impact on environment, heritage, energy consumption and community.

Construction

Construction industry has a large and direct impact on the economy, society and environment and therefore has a major role in delivering sustainable bridges. During this phase, the energy saving, carbon dioxide emission reduction and environment protection can be achieved by adopting the following measures;

- Minimise amounts of excavated materials, balance cut and fill in earth works
- Reuse building materials and construction waste
- Increase durability and minimise maintenance cost by enforcing strict quality controls
- Use energy efficient and high performing construction equipment
- Promote use of construction automation technologies
- Protect environment by preventing industrial discharge to environment

The sustainability indicators for construction measure the success of construction in achieving sustainability and they are:

- Environmental protection
- Impact and benefits to society
- Economic benefits

Environment protection is measured by how construction impacts on climate change, land, ecology and water use and how construction is carried out by minimising energy use and carbon dioxide emissions. Also processes need to be developed and implemented to minimise dust, noise and traffic delays during construction as these result inconvenience and health hazards to public.

In addition good construction practice needs to be implemented to construct durable and sustainable bridges. Some of the measures for good construction practices are:

- Enforce adequate construction quality assurance or quality control
- Implement high-performance construction specifications
- Use suitable materials for concrete
- Proper concrete placement
- Proper concrete curing and formwork removal
- Suitable surface protection system for steel elements

The Sea Cliff Bridge in NSW is presented as an example of design and construction of a sustainable bridge in an aggressive marine environment. Figure 1 shows four options out of the 14 considered, where as Figures 2 and 3 show different stages of construction and the completed bridge respectively.

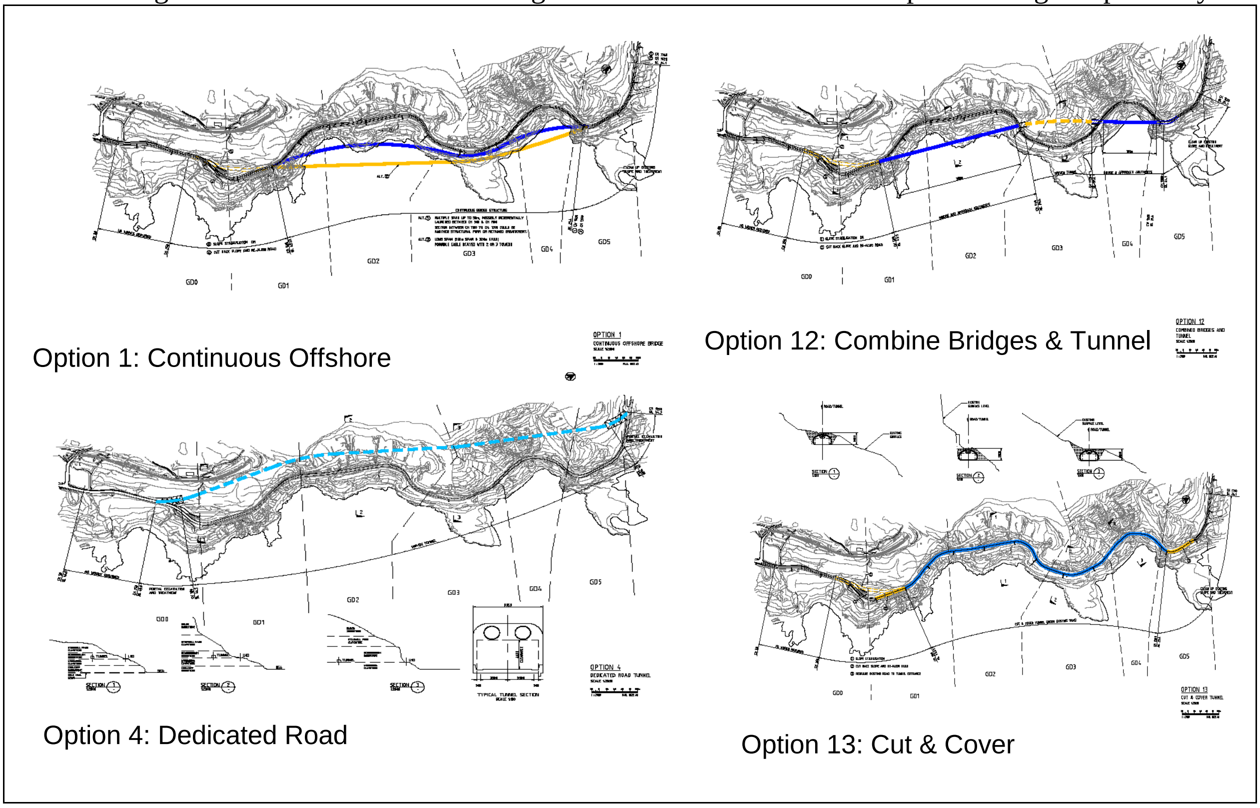


Figure 1 – Some Options Considered

A suitable option was selected based on the following criteria:

- Direct design and construction cost.
- Restore two lane road.
- Road user - risk
- Time for project delivery
- Minimal closures for geotechnical events
- Whole of life cost.

The bridge types considered were:

- Cable stay
- Suspension
- Combined Balanced Cantilever and Twin-Tee incrementally launched bridge.

Having considered the above, Balanced Cantilever and Twin-Tee Incrementally launched bridge was selected.

This bridge option was selected based on:

- Geography
- Ground Condition
- Access for pier constructability
- Geometry of the route
- Aesthetics

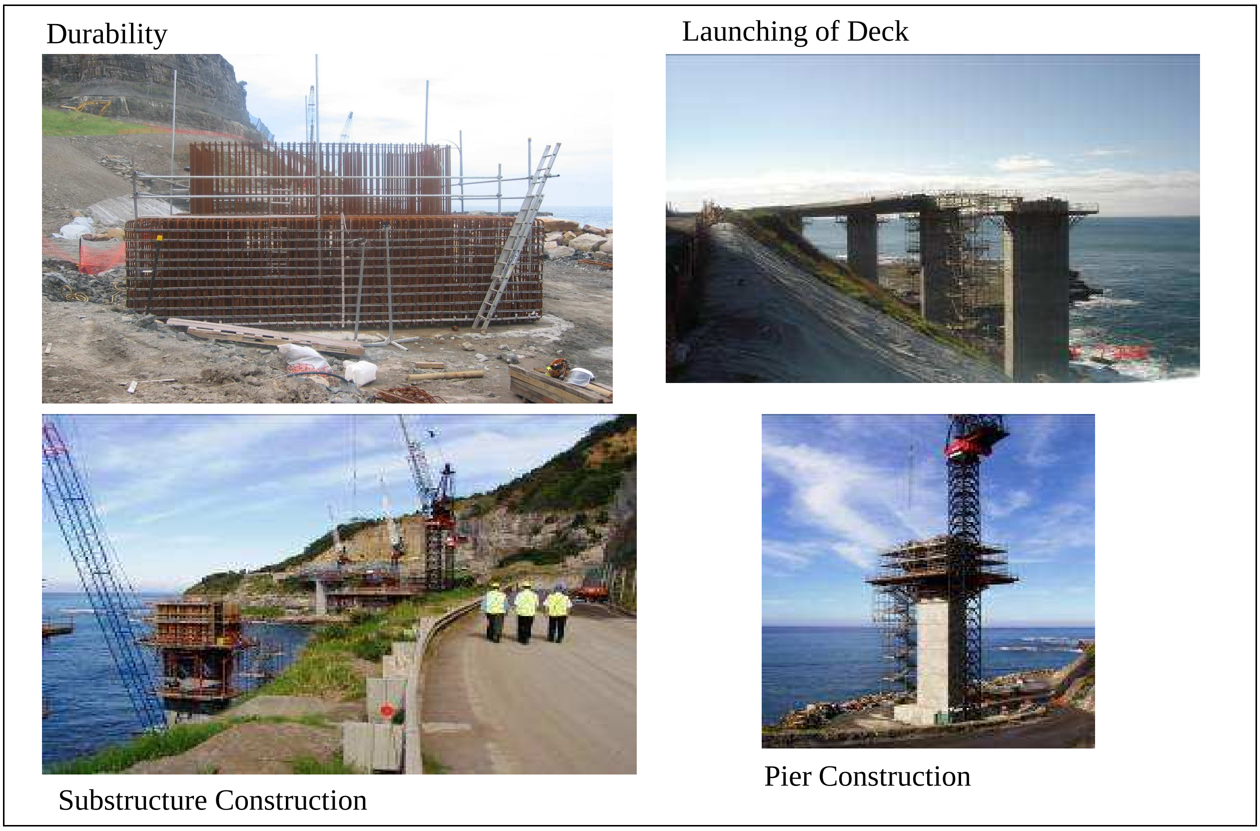


Figure 2 – Different Stages of Construction

Figure 3 – Completed Sea Cliff Bridge

Operation and Maintenance
All bridges after construction pass to the phase of operation and maintenance and this phase lasts until the end of its services life.

Sustainability during this phase can be achieved by implementing a well planned asset management cycle. This will extend the service life of bridges and eliminate the need for their replacement, extensive rehabilitation or strengthening.

Asset Management cycle for Sustainable Bridges

Earlier in the paper it was stated that the bridge infrastructure in Australia is subjected to significant increase in mass, volume and frequency of heavy vehicles. This has resulted in a accelerated deterioration of the bridge stock, particularly ones constructed before 1948.

The RTA has developed an Asset Management cycle to minimise deterioration of its bridge stock and keep them sustainable whilst allowing them to carry higher loads without compromising their safety and performance.

The Asset Management cycle has 3 phases. They are:
Phase 1 – Routine Activities
Phase 2 – Investigation and Assessment
Phase 3 – Decision Making and Action

The three phases are detailed in Figures 4, 5 and 6.

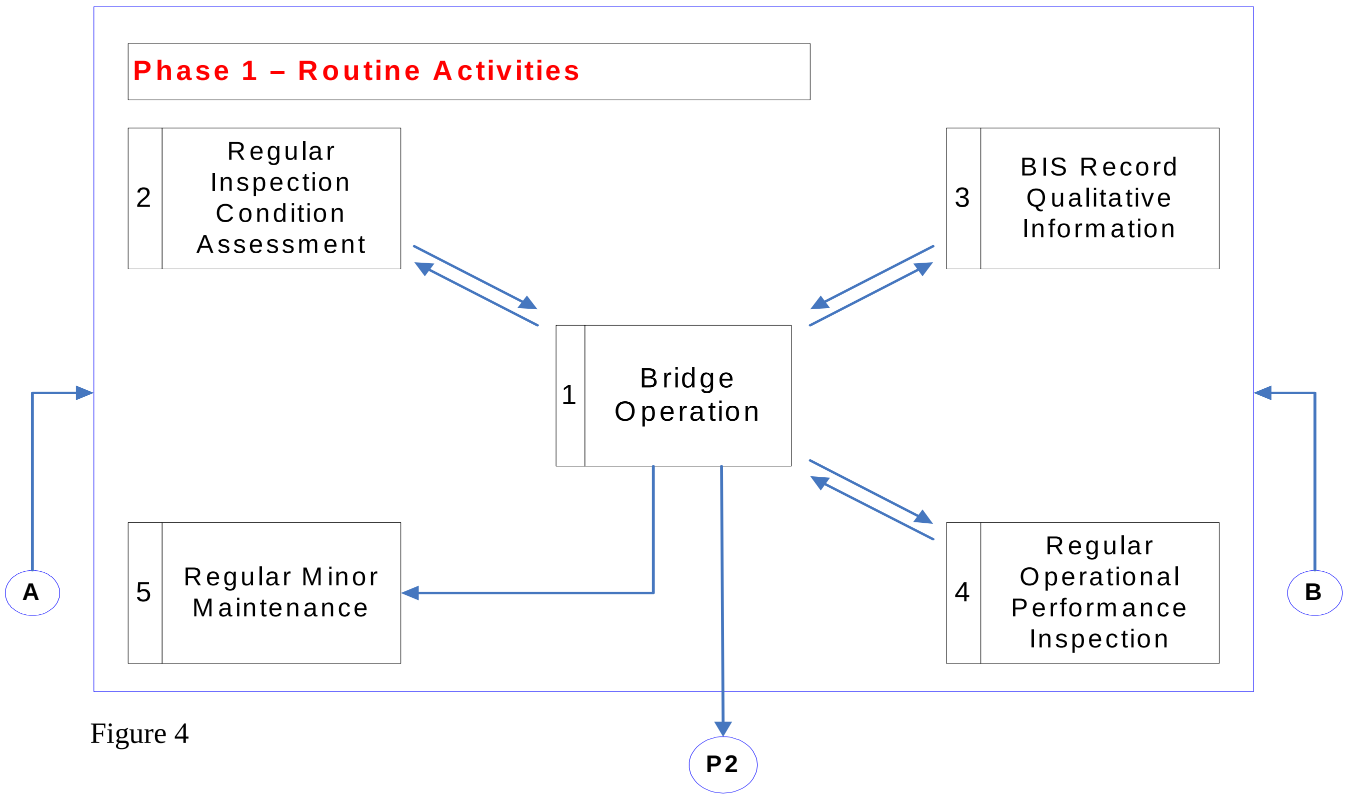


Figure 4

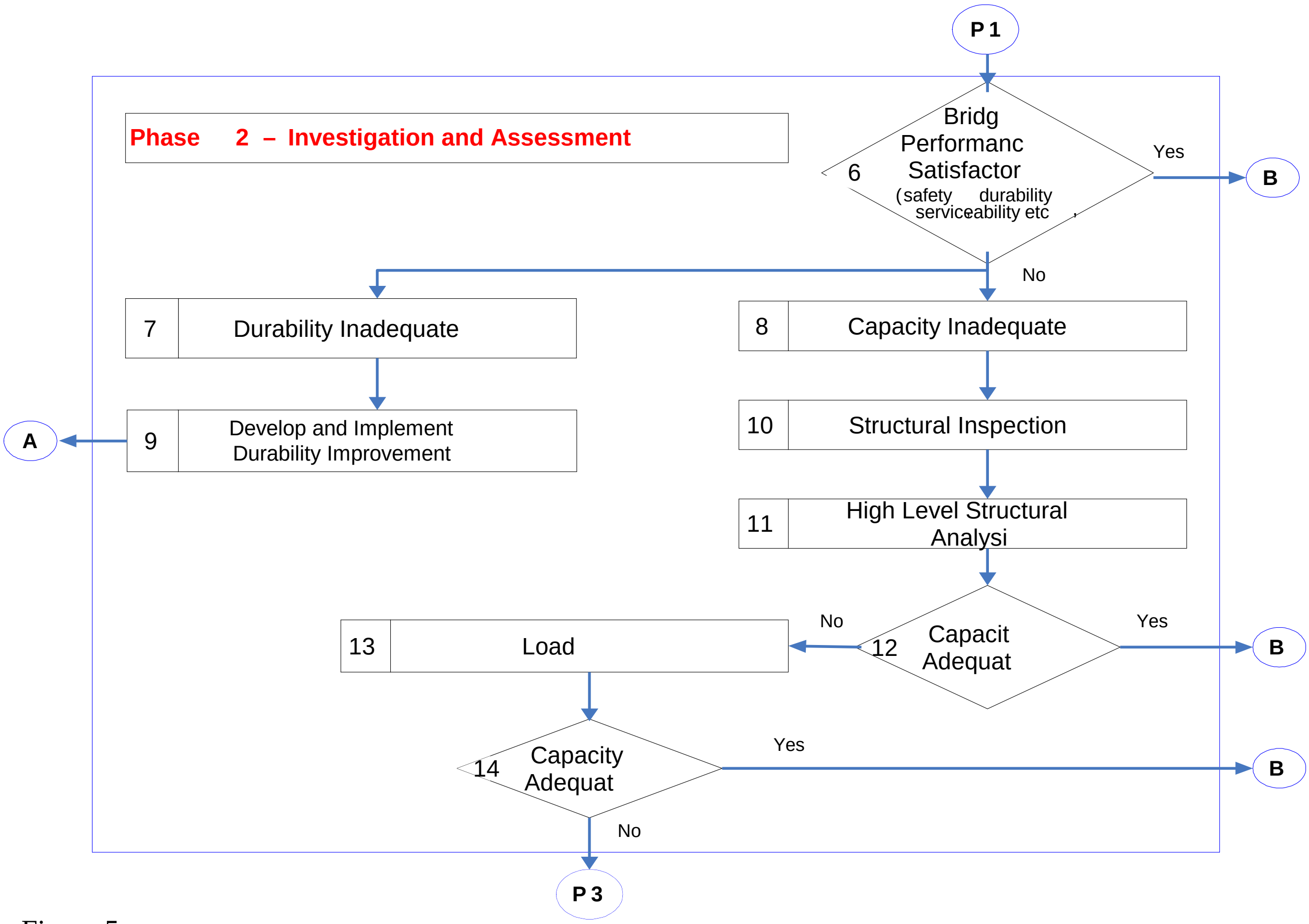


Figure 5

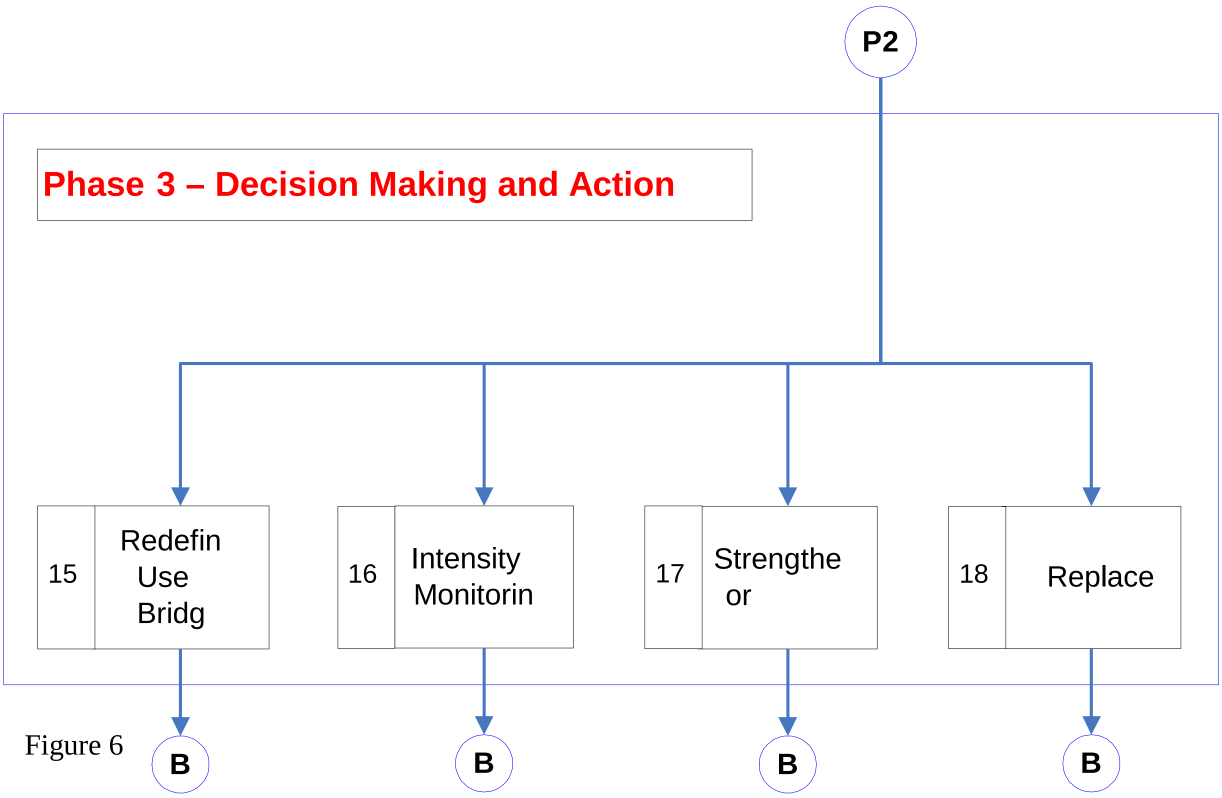


Figure 6

The Phase 1 covers routine activities such as regular inspection and regular minor maintenance work. The Phase 2 has two streams, one is the durability assessment and the other is the structural capacity assessment. The Phase 3 is where decisions are made to redefine the use of bridge, monitor its performance, strengthen or repair or replace the bridge.

Bridge Information System (BIS)

It is through a properly developed BIS that a good asset management cycle can be implemented. BIS provides a database to store, update and access the necessary information for effective management of bridges. The information in BIS should cover information on design, construction, inspection, load capacity, maintenance, strengthening and attached utility services.

Bridge Inspection

Inspection is the process by which information on physical and structural conditions are collected and updated to effectively manage bridges. Inspection should commence at the handover of a new bridge and continue through its service life at predetermined regular intervals dependant on type of bridge. The RTA has a four (4) level inspection process.

Level 1 inspection applies to all bridges and it is the basic drive-by inspection carried out on a regular basis.
Level 2 inspection is a more detailed visual assessment of element conditions carried out at two yearly intervals by trained bridge inspectors.
Level 3 inspection is a detailed structural inspection carried out by a practicing bridge engineer. It is based on a reported or suspected deterioration or damaged critical elements generally arising out of level 2 inspection.
Level 4 inspection is conducted before carrying out a load capacity assessment of a bridge to determine the extent of deterioration or section losses of its critical elements, so that these information (‘As is’ condition of a bridge) can be fed into the model for assessment.

Durability of concrete Bridges

Concrete deterioration due to the corrosion of reinforcement commonly referred to as ‘concrete cancer’ presents a significant risk to the integrity of the RTA’s concrete bridges along the coast. The costs to rehabilitate such affected structures are very high and increases exponentially as the condition of the bridge worsens.

As a more pro-active approach the RTA, has undertaken a global review of the durability condition of its coastal concrete bridge stock which is located within the aggressive marine environments that present the greatest corrosion risk to its bridge stock.

Some of the findings from the review are:

- Long term remedial solutions need to be implemented to prevent structural performance of these bridges being compromised by reinforcement corrosion.
- If long term remedial solutions cannot be implemented within 5 years due to lack of funding or resources, remedial options need to be put in place in the next one to two years to halt or at least inhibit the on set of further concrete deteriorate.
- The RTA is presently investigating the financial and technical viability of the Sacrificial Cathodic Protection (CP) system as a potential interim measure.
- The RTA’s in-house trial data suggests that such systems offer corrosion control and thus offer a cost effective short to medium ‘holding’ solution until long-term solutions can be implemented.

The management cycle for of durability of concrete bridges is shown in the flowchart in Figure 7

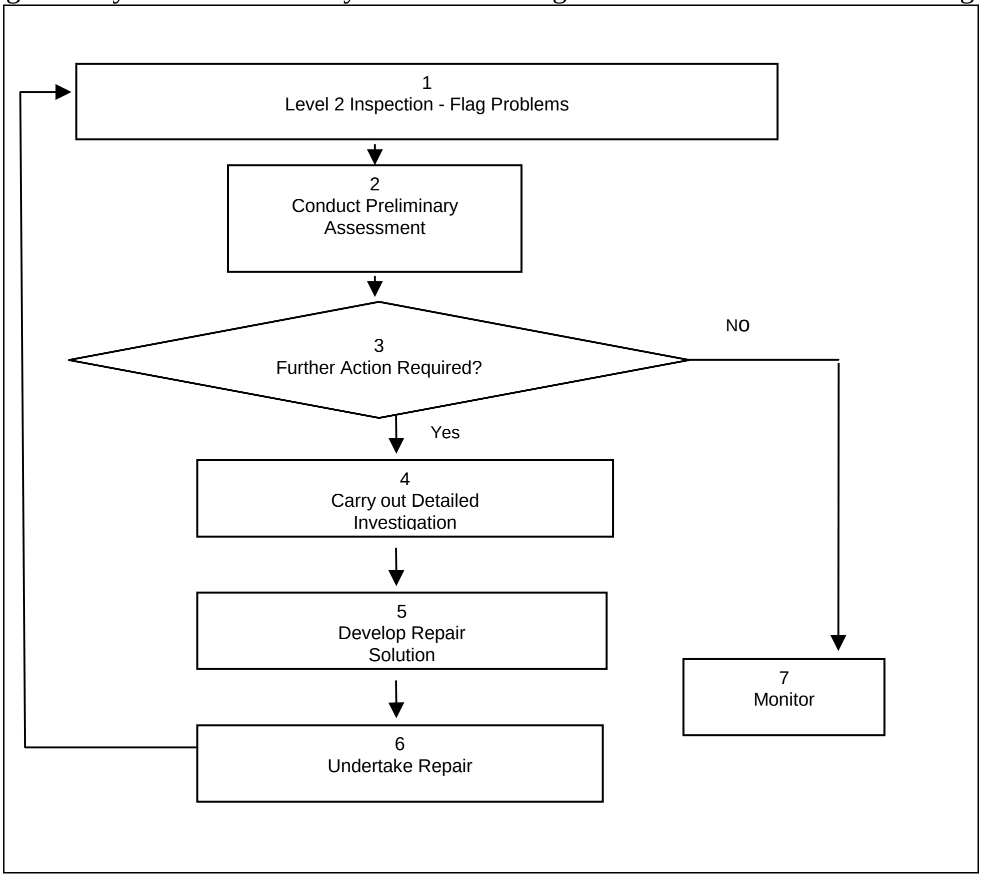


Figure 7 – Management Cycle for Durability of Concrete Bridges

Preliminary Assessment of Concrete Bridges

The steps for preliminary assessment are:
- Carry out testing of concrete
- Determine cause of deterioration
- Assess whether deterioration is widespread

Detailed Investigation for Concrete Bridges

The detailed investigation is necessary only if deterioration is widespread and the steps for detailed investigation are:
- Undertake investigation of areas with widespread deterioration.
- Confirm cause of deterioration by testing for chloride ingress, carbonation, resistivity and potential mapping.
- Determine areas of corrosion activity.
- Determine repair options including costs.

Having undertaken the above investigation a suitable repair solution is selected based on life-cycle costs analysis.

Boyd's Bay Bridge in Teed Heads, NSW is presented as an example.

Figure 8 – Boyd’s Bay Bridge. Located on the Tweed River, Tweed Heads

Problem Flagged

Level 2 (2 yearly) inspection identifies concrete damage.

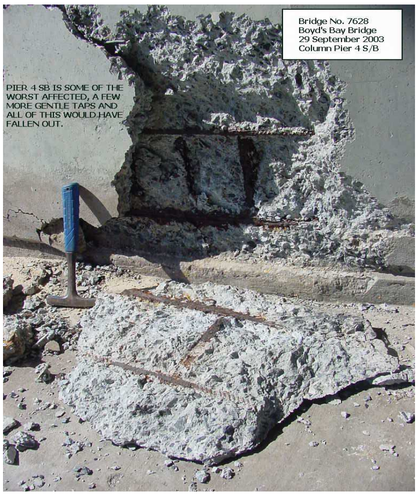


Figure 9 – Damaged Concrete

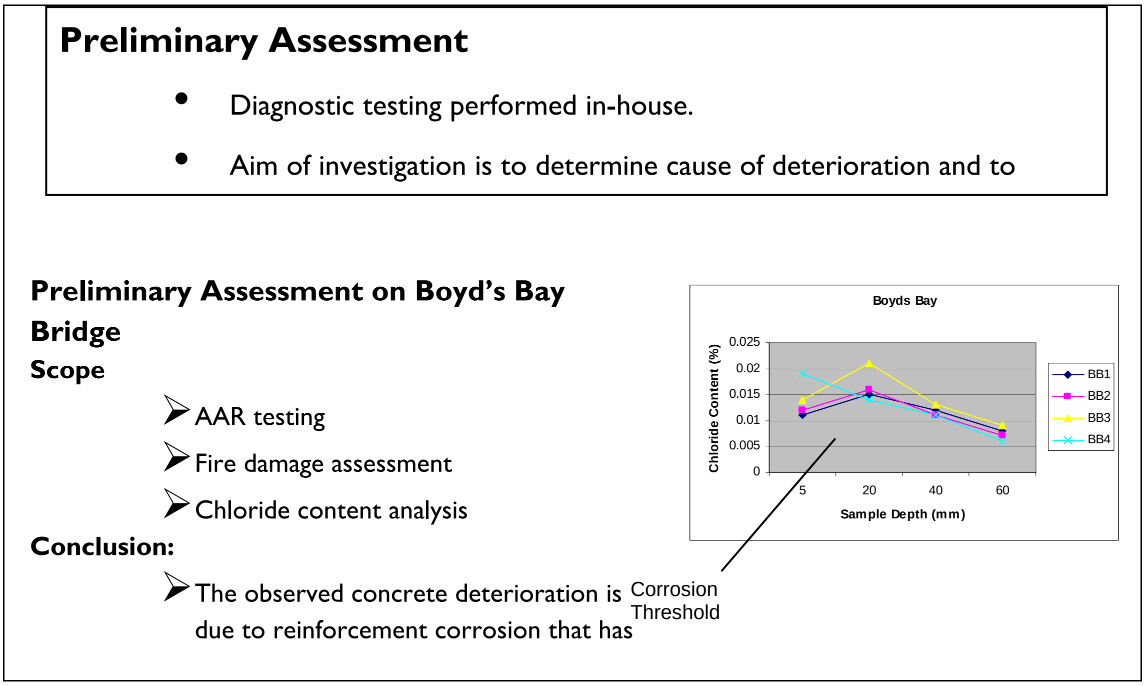


Figure 10 - Preliminary Assessment

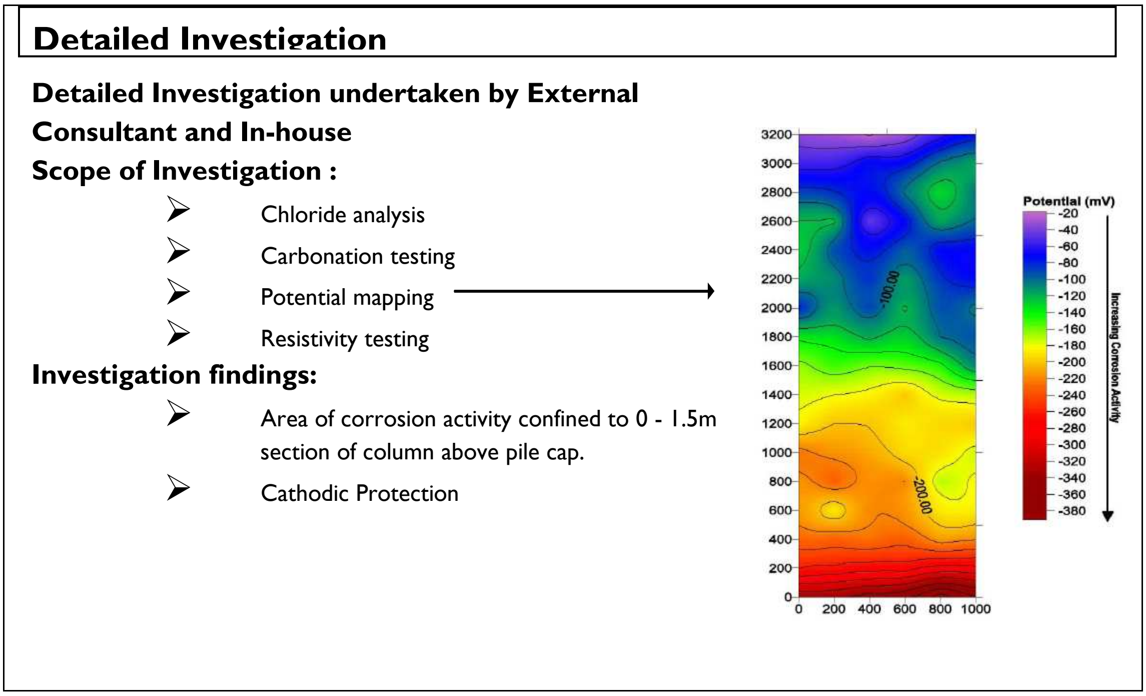


Figure 11 – Detailed Investigation

Develop Repair Solution

- Repair solution selected based on life-cycle cost analysis (LCCA) of repair options.

Option 1 - Conventional patch repair and coat
Option 2 – Sacrificial Cathodic Protection
Option 3 - Demolish and re-cast up to 1.5m

Option 2; Sacrificial CP identified as the most cost-effective long-term solution

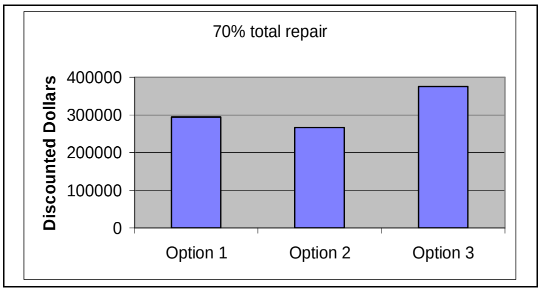


Figure 12 – Repair Solution

Undertake Repairs

- In-house surveillance and testing undertaken to confirm that repairs meet the specified performance criteria.

Reference electrodes to confirm that corrosion protection is being provided

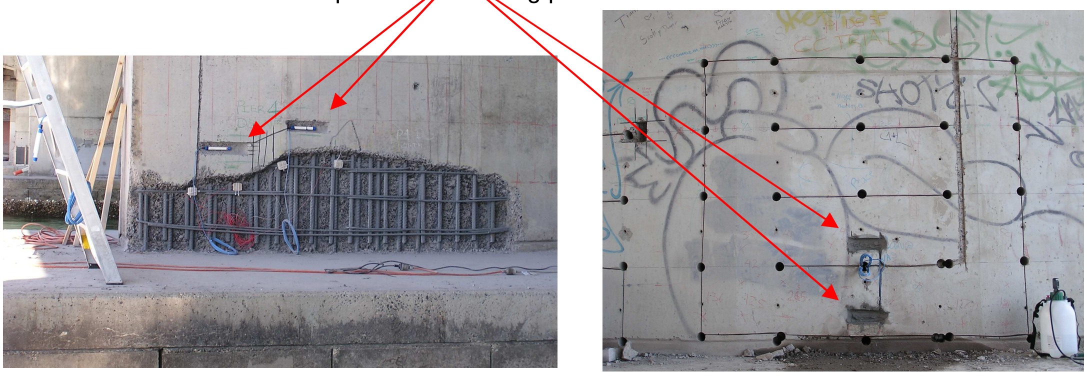


Figure 13 – Surveillance to Identify Performance

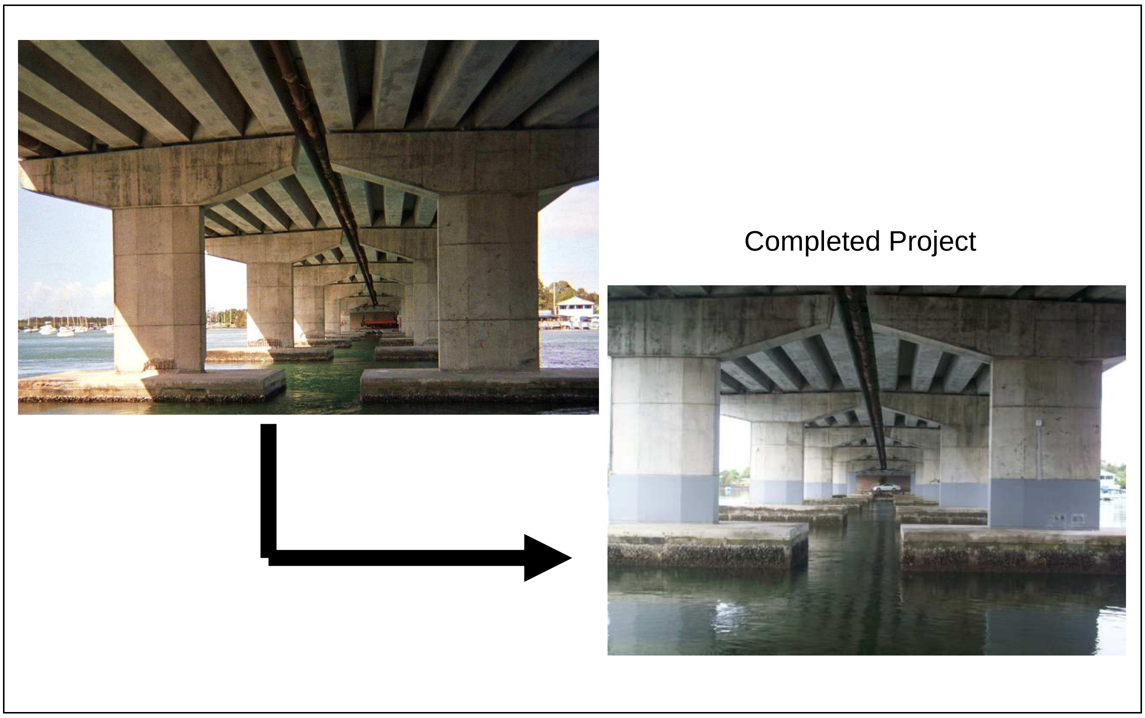


Figure 14 – Completed Bridge

Durability of Steel Bridges

The process developed by RTA for managing steel bridges is shown in Figure 15

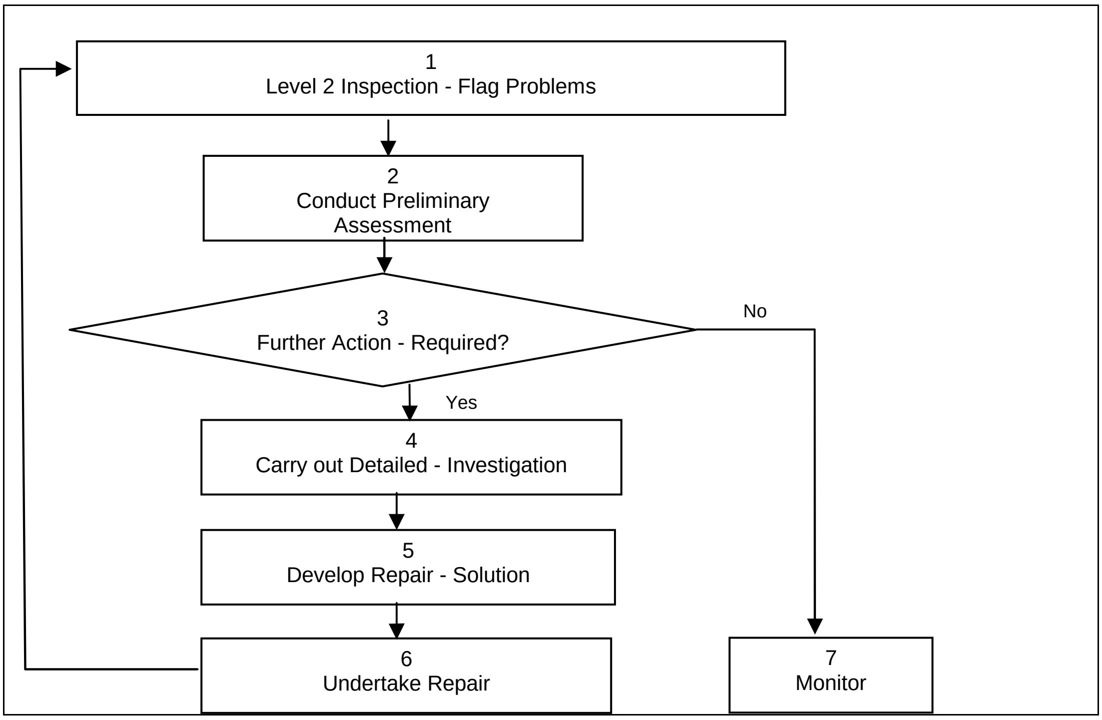


Figure 15 – Process for Management of Steel Bridges

Preliminary Assessment for Steel Bridges

The process for preliminary assessment is:
- Inspect elements for surface cleanliness.
- Establish extent, intensity and method of surface preparation required
- Carry out Level 2 inspection to determine steel condition state and paint condition state of elements.
- Conduct routine maintenance by cleaning, removal of debris and provision for drainage, if the condition of the bridge is in state 1 (as per BIS).
- Carry out further investigation if the bridge is in any other state.

Detailed Investigation for Steel Bridges

The process for this investigation is:
- Map areas of paint in condition state 2 to 4.
- Test coating samples.
- Provide a report identifying paint failures and defining required surface preparation and option for paint systems.

Having completed these investigations develops repair options identifying extent of repairs and a suitable paint system to protect the elements from the environment.

Bridge Load Capacity Assessment

Bridge load capacity assessment is a very effective tool to manage a complex bridge infrastructure, particularly in an environment of increasing live loads.

Bridge Load Capacity Assessment

Bridge load capacity assessment can be conducted at different levels depending on type and age of a bridge.

Different levels of load capacity assessment with management outcomes are shown in Figure 16.

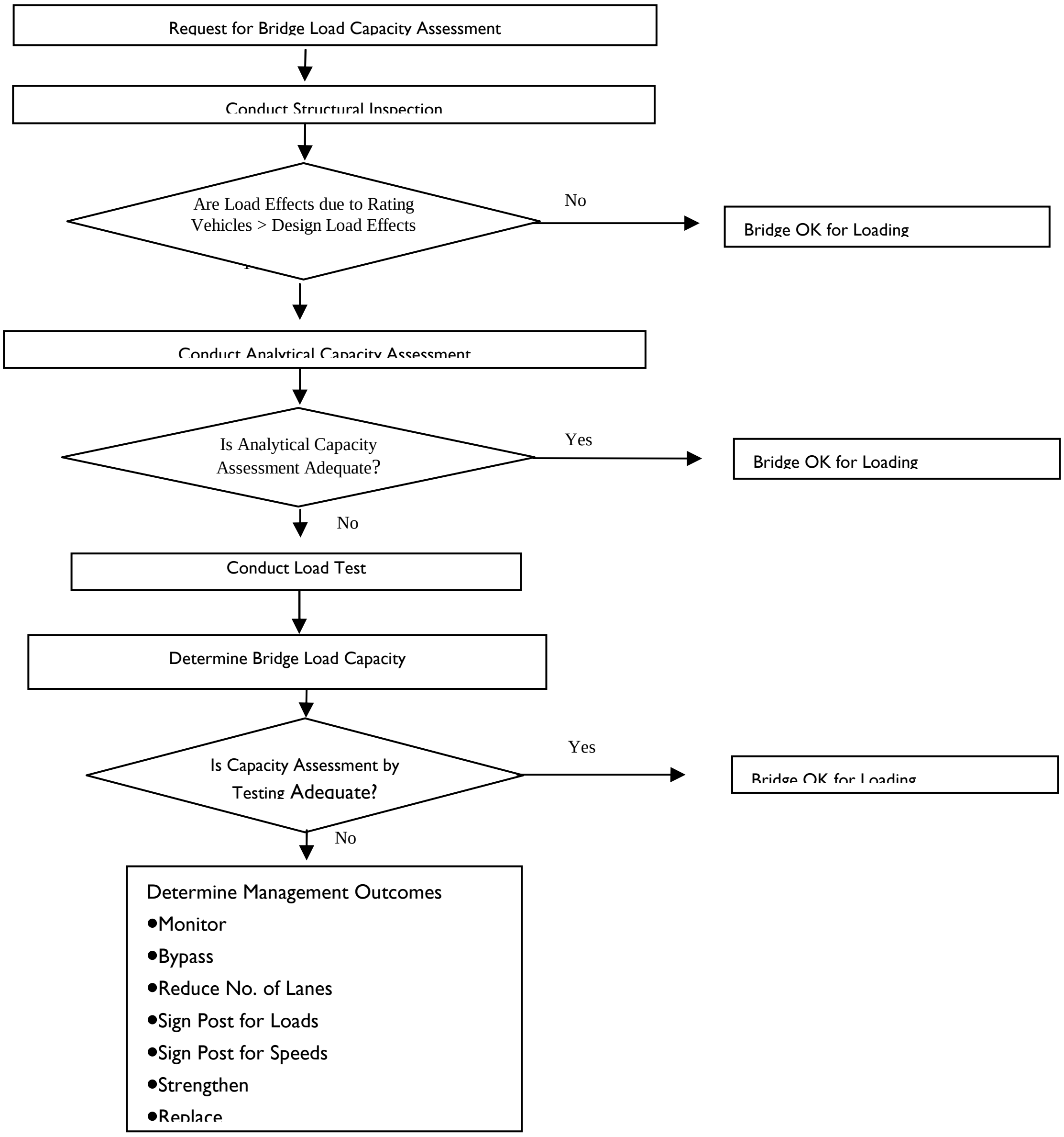


Figure 16 – Bridge Load Capacity Assessment

In 1995 the RTA developed a Bridge Proof Load Testing Facility to enable "deficient" bridges identified by analytical assessment to be evaluated at higher levels of loading, to determine their "true" load capacity without compromising their safety or performance.

Before conducting Proof Load Testing or any other bridge testing discussed in the paper, it is necessary to follow the process detailed in Figure 16. The process consists of structural inspection, material testing, structural analysis and then determine the modes of failure for increase in live loads before carrying out any type of load testing.

Structural Inspection

Structural inspection is a very important part of Load Capacity Assessment and is carried out by competent practicing bridge engineers. Prior to inspection, information is collected from Work as Executed (WAE) drawings and Past Inspection Reports.

Structural Inspection consists of 2 parts, namely Inspection for Loading and Inspection for Resistance.

Inspection for Loading is a Geometric Survey to determine the self weight, superimposed dead load and identify installation of services that may or may not be shown on the WAE drawings.

Inspection for Resistance is carried out to determine all parameters needed to determine strength of the bridge. They are:

- Member sizes
- Cracks
- Corrosion
- Settlements
- Defective Materials
- Damages
- Bridge Articulation
- Section Losses

Material Testing

Material testing is carried out to determine the actual material strength of concrete, steel or timber used in bridges, as these may vary from those shown on the drawing.

Structural Analysis

Prepare structural model of bridge taking in account its ‘as is’ condition determined from inspection. Then carry out structural analysis of the ‘as is’ condition of the bridge to determine the load capacity of the bridge to carry nominated load.

Load testing of the bridge is only carried out if its load capacity by analysis is less than the capacity required to carry the nominated load.

Non-Destructive Load Testing

Load Testing is an effective means of determining the actual load carrying capacity of a bridge. It is particularly suitable for bridges that cannot be accurately modelled for analysis or whose analytical load capacity is less than the capacity required to carry legal loads or nominated loads.

Types of Load Tests:
- Proof Load
- Performance Load
- Health Monitoring
- Dynamic Frequency analysis
- Fatigue Load
- Dynamic Load

Performance Load Testing

This is a serviceability limit state test. Bridge is carefully and incrementally loaded in the field to a pre-determined live load level, marginally higher than the legal load current at the time.

This load level is determined by multiplying the pre-determined live load by the dynamic load allowance and the serviceability limit state live load factor.

Proof Load Testing

Figure 17 – 1st Proof Load Testing (1995)

In these tests, the bridge is carefully and incrementally loaded to a pre-determined target proof load or until the bridge approaches its elastic limit, which ever occurs first.

The Target Proof Load is the lower of the theoretical ultimate live load or 2 to 2.5 times the current legal load.

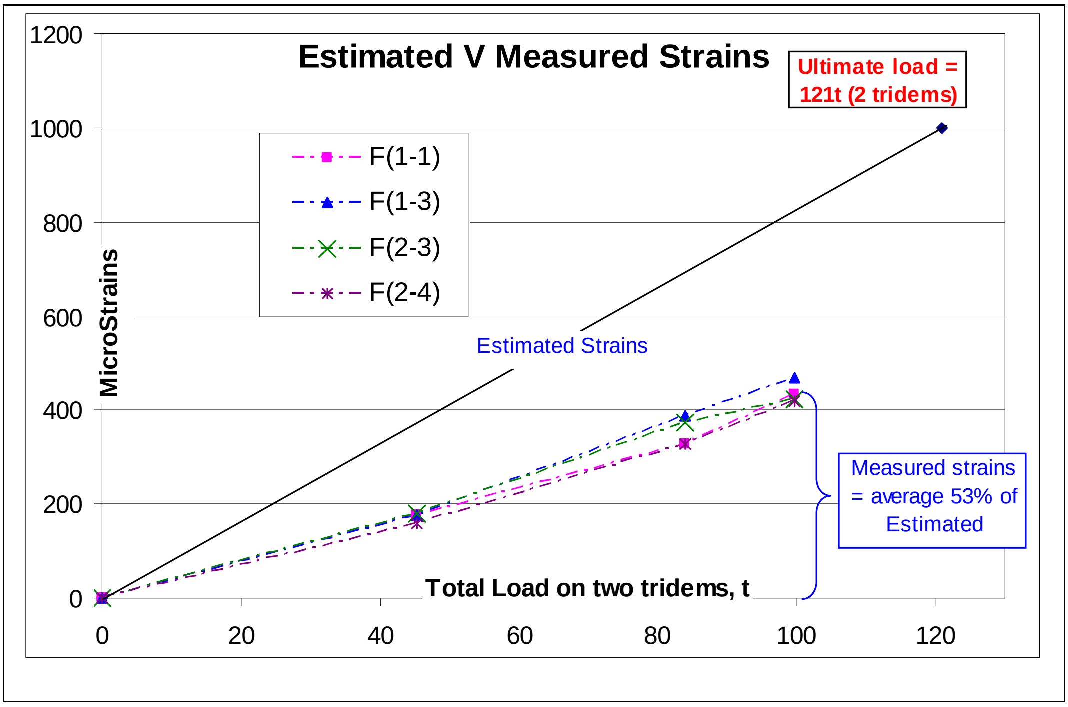


Figure 18 – Results of Proof Load Test:
Load versus strain for girders in span 1 & 2 of Red Bank Creek Bridge

Figure 18 shows the results of comparison of measured strains versus estimated strains against load for a proof load test carried out on Red Bank Creek Bridge in NSW.

Health Monitoring

Stresses strains and deflections of critical elements are measured at ambient traffic over a pre-determined period. Then same effects on these critical elements are measured for a known vehicle. From these results the maximum safe load bridge can carry is determined.

Dynamic Frequency Analysis

Bridge is excited by dropping a drop hammer on deck and dynamic frequency and stiffness of critical members are measured. Using these stiffnesses the Finite Element (FE) Model developed for the bridge is calibrated and its load capacity is determined.

Fatigue Load Test

This is a serviceability limit state test. Measure strains or deflections of critical elements for ambient traffic for a pre-determined period. For this period, determine stresses and number of cycles, and extrapolate these results for the past and into the future. From these results and using Miners rules determine the remaining fatigue life.

Dynamic Load Test

This test is carried out by running test vehicles of known axle configuration and Gross Vehicle Mass over bridge at varying known speeds, including at crawl speed. Dynamic strains, deflections and acceleration of the bridge for these speeds are measured and from these results Dynamic Load allowance (DLA) is determined.

The RTA has carried out Proof Load Testing of 56 bridges and other types of load testing for a larger number of bridges. The results from these tests have confirmed that bridges identified by analysis as being inadequate to carry current legal loads have significantly higher capacity to carry legal loads.

Results of some bridges tested are given in a table 1.

Table 1 – Comparison of Test Results

Name	**Type**	**Description Load**	**A/Rate Load**	**T/Rate Load**
Road over Rail on Pennant Hills Rd, Carlingford NSW	Jarch	17t	40t	47t
Upper Warrel Ck. On SH10 at Macksville, NSW	Sbeam	33t	40t	65t
Road over Rail on Weeroona Rd. Strathfield-West, NSW	Jarch	16t	29t	49t

Benefits – Load Capacity Assessment

Conducting structural inspection and load capacity assessment of bridges, asset managers will be able to proactively manage the aging bridge infrastructure, keep them sustainable and bring the following benefits:

- Minimise strengthening of bridges.
- Delay replacement of bridges.
- Priorities replacement and strengthening of bridges perceived as weak links in the road network.
- Establish a basis to safely increase volume, mass and length of road freight vehicles.
- Allow more liberal movement of heavy loads across the network.
- Maximise benefits from limited funds.

In addition it also brings the following global benefits:

- Improved utilization of country's bridge infrastructure.
- Improved national transport efficiency and productivity.
- Improve industry competiveness.
- Reduced cost of living.

Heavy Vehicles on RTA Road Network

Below are some heavy vehicles on the RTA road network.

Figure 19 – In 1900, 18 bullocks, Carry 29t

Figure 20 – General Access Vehicle Semi Trailer (44.5 Tonnes)

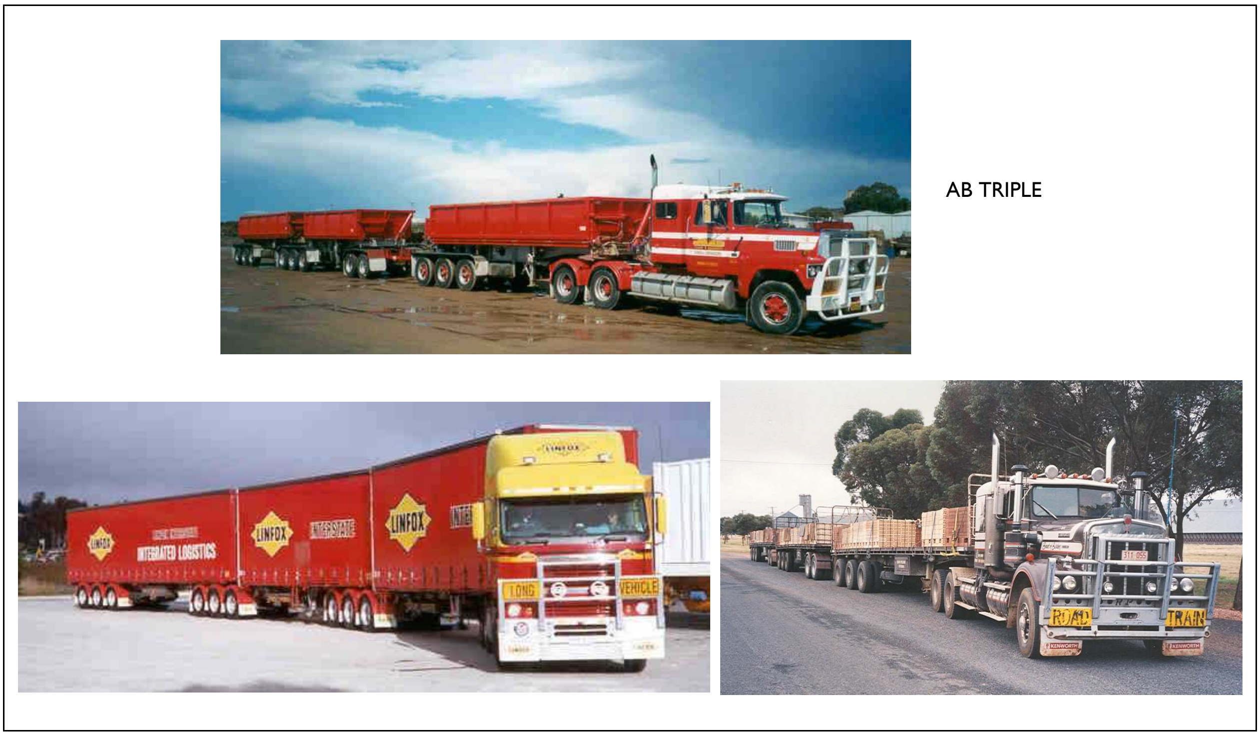


Figure 21 – Restricted Access Vehicles

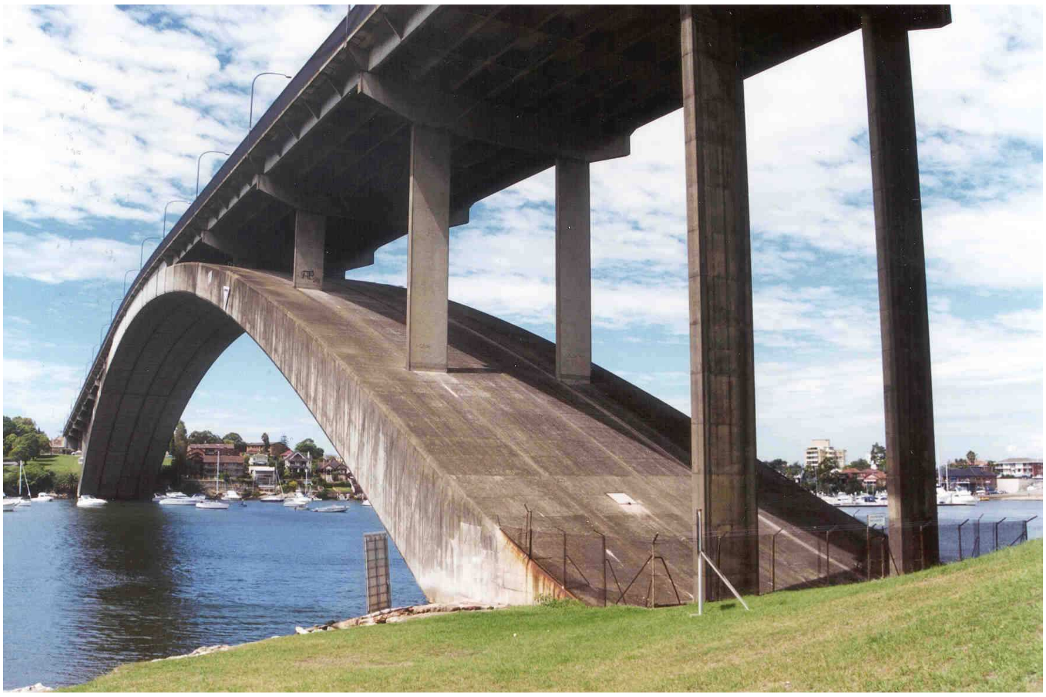
Figure 22 – Permit Vehicle, Crane

Figure 23 – Permit Vehicle

Managing Challenge

The RTA is managing effectively a very complex bridge infrastructure of different vintage bridges with different types in different aggressive environment and in a regime of significant increase of heavy loads.

The following are some examples from the RTA bridge infrastructure:



The Gladesville Bridge

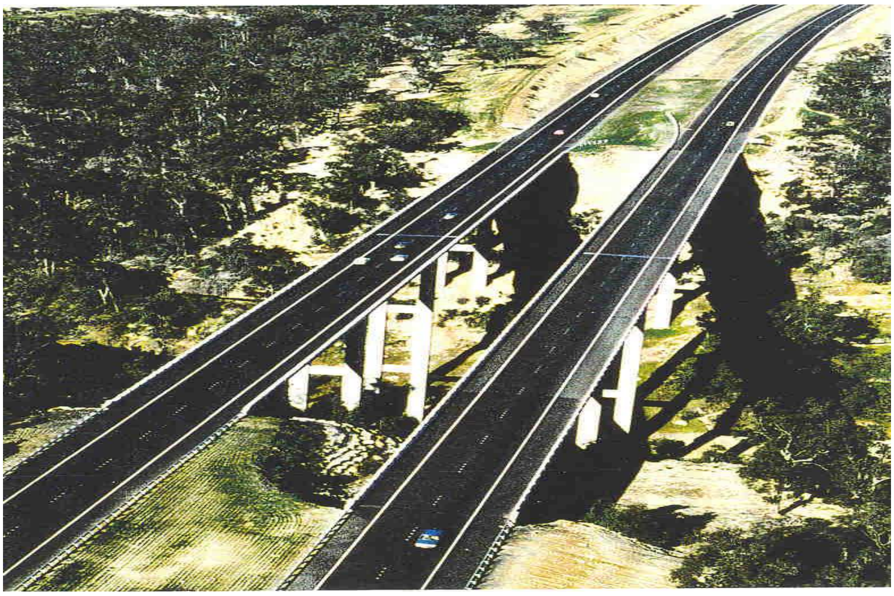
The Anzac Bridge

Woronora River Bridge

Twin Bridges over Mooney Mooney Creek

The Harbour Bridge

The Rip Bridge



Twin Bridges over the Nepean River at Douglas Park

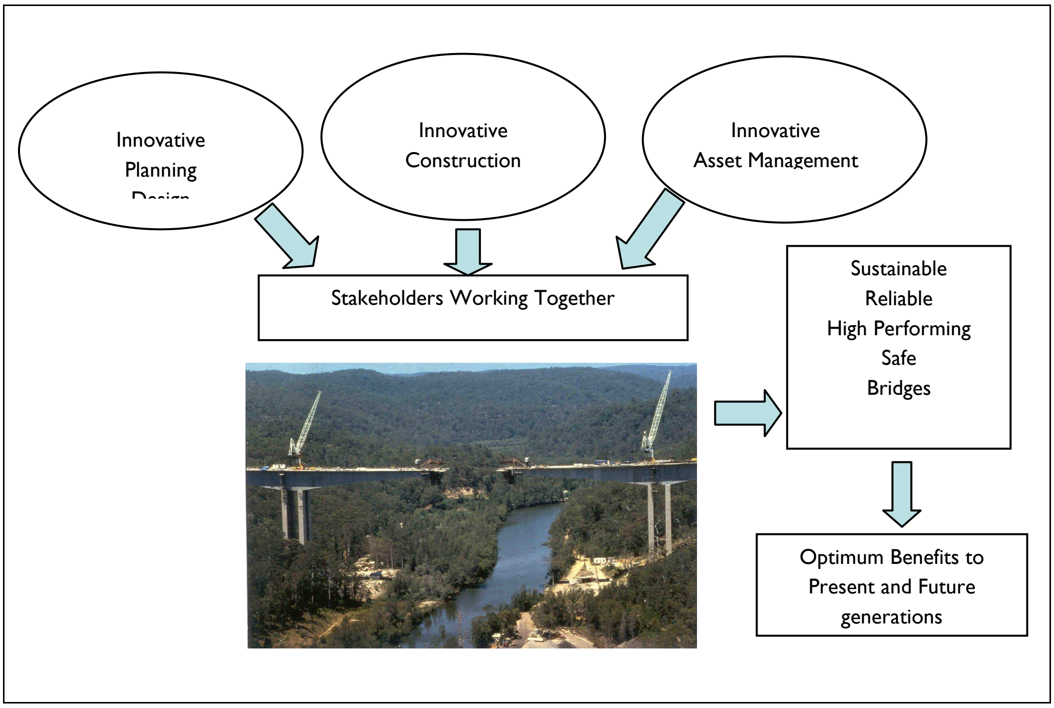
- The 1.8km long viaduct was opened to traffic in 1986.
- The viaduct consists of prestressed concrete cast-in-place voided slab spans continuous over five and six span sections.
- There are 58 spans with span lengths from 30 to 40m.
- The piers are of reinforced concrete (RC) columns and are supported on cast-in-place RC piles founded on shale.

Viaduct between Granville & Parramatta on M4

Conclusion

Sustainability is about keeping cleaner air and water, greener earth and healthier living for the present and future generations.

All stakeholders working together innovatively thought the phases of planning, design, construction and asset management we can achieve sustainable bridges. Sustainable infrastructure bringing in benefits to the present and future generations.



SUSTAINABLE DEVELOPMENT THROUGH APPROPRIATE TECHNOLOGIES IN CIVIL ENGINEERING

[1]Dr. Ishwar P. Sonar

Assistant Professor in Civil Engineering
College of Engineering Pune-5
[1]E-mail: ips.civil@coep.ac.in, ishwarpsonar@rediffmail.com

1. Introduction

Appropriate technology is small-scale technology which is simple enough that people can manage it directly and on a local level. Appropriate technology makes use of skills and technology that are available in a local community to supply basic human needs such as gas and electricity, water, food, and waste disposal. Although the technology involves simple, easy-to use and repair designs, it is based on sophisticated, 20th-century technologies. Appropriate technology may involve judicious use of high energy materials to improve the properties of resulting composite materials. It will also involve development of eco-friendly processes to solve environmental problems. The development of appropriate technology in materials, products and services may demand simple cost effective equipments, machineries and process. Therefore, this technology may involve other disciplines like mechanical, chemical and electronics.This technology will help to set up medium to small scale industries, transfer of technology from lab to land, up-gradation of local skill, proper use of agricultural waste, waste materials and bi-products from industries, local development of people through participation in it directly or indirectly. This technology will sensitize the common people to understand about the alternate materials & technologies which will go a long way in achieving goal of sustainable development towards affordable housing. This technology will improve the scope for self development, practical experience, interaction with industries and entrepreneurship development opportunities. Thus, appropriate technologies will help sustainable development of the region through use of local materials and skills and hence reduce the pressure on conventional costly materials and process. The paper presents areas in civil engineering where appropriate technologies can be successfully used where cost reduction in construction and energy is of prime importance. Some experimental works have been carried out in order to develop cost effective products. The paper also presents some test results of cost effective materials and structural elements suitable for low cost construction. The role of various agencies associated with these appropriate technologies and their interaction is presented in the paper.

2. Challenges to the Environment and Sustainable Development[3]

Environmental and developmental challenges facing the nations today are complex but well known. One side of problem is like the situation in regions such as South Asia where high population levels, rapid population growth and unbalanced population distribution are already overloading the capacities of various natural systems. The increasing inability of these systems to provide for basic needs is resulting in increased poverty. Countries are caught in a vicious cycle of poverty, rapid population growth, environmental deterioration and more poverty. The other side of problem is well exemplified in the pacific- rim region where rapid industrialization is taking place. Number of negative environmental impacts due to industrialization is rapidly affecting the human life and benefits received from the growth of industries. Haphazardly planned industrialization could lead to rapid natural resources depletion, air, water and noise pollution, accumulation of hazardous wastes, deadly industrial accidents, urban congestion and damage to human health.

Environmental trends and projects are generally used as the bases for anticipating changes in the region and to identify challenges that must be faced if the region is to sustain the long term needs of both present and future generations. This has triggered the concept of sustainable development. [2]Sustainable development meets the needs at present generations without compromising the ability of future generations to meet their own requirements. It involves wise use of nature and its resources to promote the greatest sustainable benefits to the present generation while maintaining the potential to meet the needs and aspirations of future generations. Sustainable development is an inter-generational concept, seeking equity over time and minimization of disparities between generations. Even now only a very small fraction of humanity enjoying the good life even within developed countries and there are wide disparities in the less developed countries. Since present standard of living is low in most of the low developed countries, people aspire for a higher standard. Sustainable development cautions that there are limits to such growths due to finite stocks of natural resources and energy available on other side pollution of environment, exploding population is escalating aspirations and conflicting interests.

3. Role of Civil Engineering for development and its effect on environment

Civil Engineering is responsible for multidimensional growth in economic, social, environmental, industrial and business areas. Civil engineering is backbone of for the development of any country due to its characteristics which supports directly developmental activities like infrastructure facilities irrigation system, water supply and sanitary system, transportation systems like roads, railways, airports, docks and harbours, building construction systems and indirectly to all disciplines. Civil engineering provides largest scope for employment next to agriculture. Research and developmental activities different branches of civil engineering have benefited to mankind in proper utilization of natural resources and every efforts are being taken to achieve objectives of sustainable development. With the increase in population, demands for buildings, roads, transportation and other infrastructure facilities are increasing. The cost of important construction materials such as cement, steel, bricks is increasing due to high pressure on demands, high processing and transportation cost and limited availability of resources. The over exploitation of natural resources such as stone, sand, wood, clay and other materials results in environmental deterioration.

To reduce the cost of constructions, research works related to low cost materials, utilization of waste materials for value added products and cost effective techniques to solve the problems related to water shortage and environmental problems, are being carried out by scientists and engineers. There is need to develop appropriate technologies in the area of development of cost effective materials using locally available suitable natural materials or waste materials and the area of cost effective processes to solve the need based or environmental related problems through participation of local people and their skills.

4. Appropriate Technology and its Characteristics

Appropriate technology is small-scale technology. It is simple enough that people can manage it directly and on a local level. Appropriate technology makes use of skills and technology that are available in a local community to supply basic human needs, such as gas and electricity, water, food, and waste disposal. It is important to realize that use of appropriate technology does not mean turning the clock back to the 18th or 19th century. Although the technology involves simple, easy-to use and repair designs, it is based on sophisticated, 20th-century technologies. Appropriate Technology involves a search for technologies that have, for example, beneficial effects on income distribution, human development, environmental quality, and the distribution of political power as well as productivity in the context of particular communities and nations.

The appropriate technology movement in the rich countries such as the United States got started due to the convergence of a variety of concerns. These included the need to find a more harmonious and sustainable relationship with the environment, identify a way out of the accelerating energy and resource crises, reduce alienating work disconnected from its products and goals, develop more democratic workplaces, bring local economies back to health with diverse locally owned and operated enterprises, and revitalize local communities and cultural traditions. Thoughtful, careful social choices are needed to correct the excesses and imbalances of an industrial culture driven by materialism. An essential quality of the appropriate technology movement in the United States can therefore be expressed by the word "restraint".

The appropriate technology movement in poor countries has, on the other hand developed in a very different fashion. In the poor countries the small amounts of capital available have usually been concentrated in the small industrial sector, creating very few jobs due to the high investment required per workplace. The appropriate technology movement in poor countries has come out of the recognition that industrialization strategies have not been successfully solving the problems of poverty and inequality. Indeed, in many cases "modernization" efforts have been massive assaults on local culture. The result for hundreds of millions of people has been the modernization of poverty—the neglect or construction of traditional craft occupations, the consolidation of farmlands into fewer and fewer hands, and the division of communities, leaving these people to eke out an existence on the fringe of economic activity. The appropriate technology movement in the developing world has developed as "the art of the possible" among the world's poor, seeking ways to solve pressing basic problems and create jobs with resources consisting of local skills and materials but little surplus cash.

From these different origins, the appropriate technology movements in rich and poor countries have been moving towards each other. The development of renewable energy technologies has long been a chief area of activity among U.S. appropriate technology groups. It moved high on the list of priorities in oil-importing poor countries in the late 1970's, as they faced high prices and scarcity of fuel for buses, tractors, and irrigation pumps. Similarly, environmental protection has gained increased attention in poor countries as pesticides have created major health risks for farmers and farm workers, and deforestation has reached a critical level.

Appropriate Technology

1. requires only small amounts of capital;
2. emphasizes the use of locally available materials, in order to lower costs and reduce supply problems;
3. would be relatively labor-intensive but more productive than many traditional technologies;
4. would be small enough in scale to be affordable to individual families or small groups of families;
5. can be understood, controlled and maintained by villagers whenever possible, without a high level of specific training;
6. can be produced in villages or small workshops;
7. supposes that people can and will work together to bring improvements to communities;
8. offers opportunities for local people to become involved in the modification and innovation process;
9. would be flexible, can be adapted to different places and changing circumstances;
10. can be used in productive ways without doing harm to the environment.

Some of the reasoning that underlies the concept of appropriate technology may be summarized as follows:

1. it permits local needs to be met more effectively because local people are involved in identifying and working to address these needs; for the same reasons, it is likely to be in harmony with local traditions and values;
2. it means the development of tools that extend human labor and skills, rather than machines that replace human labor and eliminate human skills;
3. it represents a comprehensible and controllable scale of activities, organization and mistakes, at which people without management training can work together and understand what they are doing;
4. it allows more economical operation by minimizing the transport of goods in an era of expensive energy, allowing greater interaction of local industry and permitting greater use of local resources—both human and material;
5. it makes unnecessary many expensive or unavailable finance, transportation, education, advertising, management, and energy services; avoids the loss of local control that use of such outside services implies;
6. it helps to establish a self-sustaining and expanding reservoir of skills in the community which begins from already existing skills;
7. it provides a region with a cushion against the effects of outside economic changes (e.g., the collapse of the world sugar market or the sudden unavailability of fertilizer);

8. it helps to reduce economic, social, and political dependency between individuals, between regions, and between nations, by recognizing that people can and will do things for themselves if they can find a way.

Appropriate technology emphasizes the use of renewable resources, like the energy from the sun, wind, or water. Appropriate technology makes it possible to satisfy our basic human needs while minimizing our impact on the environment. We are running out of the natural resources necessary to sustain ourselves. In addition we are limited in our ability to deal with the social and environmental problems that result from continuous growth. There seems to be a growing dissatisfaction with the complexity and hectic lifestyle of 20th-century society. Many people would prefer to return to a simpler way of life. Appropriate technology is attractive because it makes households and industries more self-sufficient, and most things can be managed at a local level. We may have to do more hand labor instead of depending on automation to satisfy our basic needs. However, there are many advantages to simplifying our lives. By growing more of our own food and producing and buying goods in our own communities, we spend less time and money on transportation, produce less waste and consume fewer environmental resources.

Appropriate technology may involve judicious use of high energy materials to improve the properties of resulting composite materials. It will also involve development of eco-friendly processes to solve environmental problems. The development of appropriate technology in materials, products and services may demand simple cost effective equipments, machineries and process. Therefore, this technology may involve other disciplines like mechanical, chemical and electronics.This technology will help to set up medium to small scale industries, transfer of technology from lab to land, up-gradation of local skill, proper use of agricultural waste, waste materials and bi-products from industries, local development of people through participation in it directly or indirectly. This technology will sensitize the common people to understand about the alternate materials & technologies which will go a long way in achieving goal of sustainable development towards affordable housing. This technology will improve the scope for self development, practical experience, interaction with industries and entrepreneurship development opportunities. Thus appropriate technologies will help sustainable development of the region through use of local materials and skills and hence reduce the pressure on conventional costly materials and process.

4.1 Obstacles in Appropriate Technology[1]

The emphasis on self-reliance and local production for local needs, no need of well-developed infrastructure and of highly trained human power may be the reasons for that the concept of appropriate technology is so popular. The elements of self-reliance, local initiative, and local control that are essential parts of this approach present a challenge to conventional thinking in the development institutions. Up to 80% or more of the population in most developing countries lives in rural villages. Many of the people in urban areas fled the stifling lack of opportunities that tends to characterize rural areas. Thus successful rural appropriate technologies might concern some 90% of the population. An important voice in the dialogue about village technology can and should be provided by educated people working in rural areas in small projects. However, many of these people seem hesitant to get involved in experiments with technology, perhaps because this is seen as the work of engineers and scientists, and therefore as too difficult for others to undertake. Yet, the development of appropriate technology in not solely or even primarily a question of engineering design it involves a wide range of considerations. Appropriate technology work cuts across traditional lines of expertise, and benefits from the insights of local farmers, technologists, educated generalists and business people.

5.0 Appropriate Technology in Civil Engineering

In today’s context when India is heading towards economic growth and entering into an era of overall development, it is more than essential that we create an enabling environment for affordable housing for one and all. Over the years, we have realized that the technology exists but it does not reach to the common people. Most of the time, we are either not aware of these innovative interventions or do not know how to proceed. It is now widely recognized that the cost of housing can be reduced and speed and quality of construction stepped up through the use of emerging innovative building materials and technologies. Despite a number of innovative cost-effective building materials, components and construction techniques developed through research the housing and building agencies have not adopted them in their construction practices. The extent to which lack of standards and specifications has been instrumental in hindering the adoption of homegrown innovative building material technologies has long been a matter of concern. Since non-listing of these new techniques in Indian Standards and Codes is quoted as one of the foremost reasons by construction agencies for non- adoption in their practice, the Bureau of Indian Standards (BIS) has been constantly striving to cover new technologies within the fold of standardization. While quite a few of new materials and techniques have attracted attention of the building industry and several housing agencies and have also been gradually identified in Codes of Practices, these have not percolated to the practices of organizations like CPWD, MES, State PWDs and others in public and private sectors. This indicates that there is wide scope for research and development of new cost effective, durable materials .

By and large, conventional building technologies like burnt bricks, steel and cement are high in cost, utilize large amount of non-renewable natural resources like energy, minerals, top-soil, forest cover etc. These increase dependence on external materials and manpower, harm the local economy and are generally polluting in nature. The materials and technologies chosen for construction must, in addition to functional efficiency, fulfill some or more of the following criterion, for the cause of sustainability and a better quality environment

- non endanger bio-reserves and be non-polluting;
- be self-sustaining and promote self-reliance.
- recycle polluting waste into usable materials
- utilize locally available materials
- utilize local skills, manpower and management systems
- benefit local economy by being income generating
- utilize renewable energy sources
- be accessible to the people
- be low in monetary cost

Details of some Environment-Friendly, Energy Efficient, Cost-Effective Composite Materials/Products for Low Cost Housing are summarized in the Table 1

Table 1: *Environment-Friendly, Energy Efficient, Cost-Effective Composite Materials/ Products for Low Cost Housing*[4]

Product	Basic Raw materials	Materials for production	Applications
BAMBOO MAT BOARD	Bamboo Plantation wood veneer	polymeric resin, chlorinated hydrocarbons and boron and cashew nut shell liquid	Flooring, walling, structural membrane, false ceiling, door/window frames.
BAMBOO MAT VENEER COMPOSITE	Bamboo Plantation wood veneer	polymeric resin, chlorinated hydrocarbons and boron and cashew nut shell liquid	Door skin in flush doors, structural use as roofing, web construction, prefab and portable shelters, packing, modular partitions, furniture.
BAMBOO LAMINATED COMPOSITE	Bamboo plant, wood Bamboo mat,	waste wood chips, polymeric resin	Flooring, walling and partitions.

BAMBOO CORRUGATED ROOFING SHEET	Bamboo grass (Plant), Species (Meleannca baccifera	Bamboo, polymeric resin, chlorinated hydrocarbons, boron, cashew nut shell liquid, coating for UV protection and to improve impermeability to water.	Roofing sheets as substitute to corrugated Asbestos Cement sheets, Galvanized Iron sheets, Aluminum sheets and Fiber-reinforced Plastic (FRP) sheets.
BAGASSE COMPOSITE PANEL/BOARD	Waste from Sugar industry	General purpose, polyester resin, methyl ethyl ketene and cobalt napthenate	For variety of building and furniture applications. Properties closely resemble with the wood but lighter in weight. Stackable and can be easily chiseled and sawed.
JUTE POLYESTER COMPOSITE	Jute industry (Jute plant is grown in several developing countries)	Woven jute fibbers, and polyester amide polyol as interfacing agent	Chip boards, roofing sheets, door shutters, partition panels and door/window frames.
COIR COMPOSITE BOARD	Coconut – Plantation in Coastal Regions	Core fibers, mineralized water, cashew nut shell liquid, para-formaldehyde	Medium Density Fiber boards, can be used as wood substitute for paneling, cladding, surfacing and partitioning and door/window shutters
CELLULAR LIGHT WEIGHT CONCRETE BLOCK	Fly Ash from coal based power generating plants	Fly Ash, cement, sand, water and foaming agent	Concrete blocks, densities ranging from 400 kg/m3 to 1800 kg/m3. Filler walls. In-situ cellular walls & partitions. Very good insulation at roof tops for reduction of heat load in buildings.

Appropriate Technology will play important role in the development of base for research and development of cost effective and eco-friendly technology in various areas related to civil Engineering are suggested below.

1. Indigenous Material Technology : Use of different natural materials such as bamboo, coconut coir, jute fibers, banana fibers, sisal , sugar baggase, etc; in efficient way, application of bio-technology in development of required properties in these natural material.
2. Energy Efficient Technology: It includes cost effective technology applicable to green building, energy saving system (natural ventilation) , use of local materials in construction, use of solar, wind energy , bio gas energy etc.
3. Disaster resistance Structures : It includes earthquake resistance structures using light weight materials, portable structures, Temporary sheds in disaster affected areas, floating structures near coastal areas
4. Rain water harvesting techniques : Water conservation through roof top water collection and utilization system , Collection of rainwater from open spaces, parks, roads and using it for storage and ground water recharge. Reuse of water through recycle systems in processing units and factories like sugar, clothes, paper mill etc and service industries like automobile, washing hotels etc.
5. Waste Material Technology : conversion of waste materials into value added products.

Waste may be solid waste from treatment plants industries, sewage, processing units, wastes from construction sectors like recycled aggregates, road materials, waste from pipe, tile and other precast factories, waste from furniture, plastic wastes agricultural wastes, bio-composting etc.

6. Cost effective Transportation systems: Low cost roads in rural and remote areas, Temporary and emergency bridges, use of rope way systems in remote hilly areas, Use of Inland water ways for transportation of goods and persons with water cleaning techniques.
7. Bamboo and other useful tree plantation in open land.

With this in mind, it becomes essential to introduce a subject related to appropriate Technology and low cost technology in civil Engineering discipline in Engineering and research institutes so that short term research projects can be taken. This will create awareness about need and importance of sustainable development and they will come forward to develop the technology for common people and technology for our nation to reduce the pressure on demands on energy intensive materials. Role of government and local authorities in this area will be very important. Participation of local people, non-government organizations, and industries with vision of sustainable development with appropriate training will gear up the movement of appropriate technology. This technology will help to set up medium to small scale industries, transfer of technology from lab to land, up-gradation of skill, proper use of agricultural waste, waste materials and bi-products from industries, local development of people through participation in it directly or indirectly.

6.0 Conclusions

The vicious cycle of poverty, rapid population growth, environmental deterioration and more poverty and negative environmental impacts due to industrialization have triggered the concept of sustainable development. Civil Engineering is responsible for multidimensional growth in economic, social, environmental, industrial and business areas. Civil engineering is backbone of for the development of any country due to its characteristics which supports directly developmental activities infrastructure. Deterioration of environment due to over exploitation of natural resource materials and increasing cost construction , have created awareness in development of cost effective materials and processes related to civil engineering and hence appropriate technologies suggested in this paper will help the sustainable development of the region. There is vast scope of research in the area of development of appropriate technology in various area which is possible through interaction of various agencies and participation in projects related to appropriate technology.

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DESIGNING TO MEET CLIMATE CHANGE CHALLENGES

Dr. R. B. Draper[1], **Dr. P Attanayake[2]**.

[1]Principal Environmental Engineer, Bechtel Corporation, 50 Beale St, San Francisco, CA 94105, USA
[1]E-mail: rbdraper@bechtel.com
[1]Telephone: 415-768-3282, Fax: 415-768-6167

[2]Chief Environmental Engineer, Bechtel Corporation, 50 Beale St, San Francisco, CA 94105, USA
[2]E-mail: pmattana@bechtel.com
[2]Telephone: 415-768-0454, Fax: 415-768-6167

Abstract:

We identify decision points in standard engineering work processes where adjustments should be made to assist design teams to evaluate climate change related design impacts. Early in the project, design options should be evaluated to optimize the combined greenhouse gas (GHG) reduction benefits of interfacing building systems. In the conceptual design stage, GHG emission reducing alternatives can be evaluated and incorporated into the design. In addition, sustainable GHG reducing engineering, procurement and construction approaches should be evaluated and adopted early in the project for cross-functional benefits to the project and to the client

In the preliminary engineering phase emission monitoring and control of greenhouse gases may be considered. Closed-loop processes may also be considered to minimize potable water use and wastewater generated. In this design phase, future site climatic conditions should be considered, in addition to developing criteria based on historical norms. In the final design phase, when specifications and material requisitions are developed, they should include GHG reducing requirements for high performance building systems including: energy efficient HVAC, lighting and building envelope systems; water conserving utility systems and processes; recycled or rapidly renewable materials content; and fuel efficiency standards for construction site vehicles.

Keywords:

climate change, adaptation, mitigation, sustainable design, greenhouse gases, standard engineering work processes, emissions, carbon dioxide, global warming

1 Introduction

Growing and potential impacts of climate change (Karl *et al* [1]), such as flooding in coastal areas, change in weather patterns, and melting of the permafrost have created new challenges for the engineering and construction industry. These challenges involve adaptation in the design and construction of projects to address these impacts, as well as developing ways to reduce and controlling greenhouse gas (GHG) emissions to mitigate climate change.

Engineering has the lead responsibility for determining the technical feasibility and cost parameters to overcome these challenges. Engineering and construction projects are implemented with the help of a set of standard documents that lay out the work process of the projects. They include standard design detail drawings, standard design criteria, standard specifications, design guides and work process flow diagrams. Incorporating in these standard documents materials and processes which assist project engineers to identify and assess climate change related impacts can be a major step in effectively preparing to meet the challenges of climate change mitigation and adaptation.

2 Optimizing the Design Process

In many respects designing to meet climate change challenges is sustainable design. A project execution approach integrating the following concepts for sustainable engineering, procurement and construction (S-EPC) is directly relevant to designing for climate change:

- Site master planning and design for ecology

- Potable water conservation, and minimizing waste water discharges
- Process design to conserve water, energy and other natural resources
- Design provisions for phased construction to meet current needs with provisions for meeting future facility requirements; provisions for adding sustainable design measures in future phases of construction, if not funded in the initial phase
- Passive design of facilities to save energy in plant and building operations, e.g. Energy Star® roofs or green (vegetated) roofs; adequate insulation of building walls, roofs, pipes, ducts and vessels, to minimize fossil-fueled power consumption and emissions
- High-efficiency HVAC and electrical systems including high-performance lighting systems integrated with daylighting and smart controls
- Energy Star® appliances and equipment
- Onsite renewable energy with energy storage for peak use, meeting the power demand that has been reduced by all of the above concepts, and resulting in reduced fossil fuel demand / emissions.
- Eco-purchasing and contracting: "greening" the supply chain to minimize climate change impacts of the supply chain.
- Managed construction to protect the site's natural resources, minimize pollution and waste and recycle or salvage surplus materials.
- Neutralizing any additional capital costs by combining "hard" benefits (life-cycle cost savings and returns on investment) with "soft" benefits (intangible but real and significant).

Similarly formulated sustainable design concepts can be found as the Hannover principles (William McDonough & Partners [2]), the Twelve principles of green engineering (Anastas and Zimmerman [3]) and ten tenets of structural sustainability (D'Aloisio [4]).

Designs informed by an integration of these concepts, with inter-discipline and cross-functional trade-offs for overall optimization of the project, are more energy efficient, minimize GHG emissions and adapted to a changing climate. They add value to the standard engineering work execution by better integrating the design of systems. The combined benefit of the overall optimization is greater than if the systems are optimized individually. The benefits include significant reductions in energy and water requirements leading to lower GHG emissions and resulting in cost savings in construction and facility operations, as well as reduced impact on the site and the environment.

To implement sustainable design, the standard engineering work process needs to be enhanced to foster closer collaboration among disciplines and functions. For cost-effective success, sustainable design should be embedded as an integral part of the engineering work process. Designing to meet climate change challenges does not have to be an "add-on" to the current work process. Figure 1 provides an overview of the standard work process with some notes on sustainable design related activities that can be integrated into the standard process.

3 Conceptual Design

The conceptual design phase is when sustainable design, climate change mitigation and adaptation features can be most easily incorporated into a project. Establish a multi-disciplinary team of project personnel and hold an integrated sustainable design team planning meeting early in project development. This integrated team does more than coordinate; it collaborates throughout all project phases and includes engineering, procurement, construction, the client, operator and other stakeholders. The team can be established as an integrated project team following U.S. Department of Energy (U.S. DOE [5]) or U.S. Department of Defense (U.S. DOD [6]) or similar guidelines.

This group should establish sustainability and GHG emission performance goals: e.g., 25-percent less energy use than required by code, 30-percent reduction in GHG emissions relative to a baseline facility providing the same products or services. The team should document the goals in the project execution plan and/or relevant discipline engineering design criteria.

The integrated design team ensures collaborative work among all disciplines to embed sustainable design concepts, sustainable systems, green fabrication and construction techniques into the development of the project design and systems selection. It helps the project team evaluate alternatives to optimize the overall systems / process / facility design and realize the benefits of sustainable design.

Expertise in sustainable design methods, including energy modeling and building information modeling, may or may not be available "on-project". Depending on the project and scope of sustainable design, experts in energy modeling, wetland design, stormwater management, energy-efficient lighting should be made available to the project from off-project staff or consultants, as needed. Accreditation by the U.S. Green Building Council for LEED™ is highly desirable for members of the integrated design team.

Table 1 provides examples of climate change related sustainable design performance metrics that can be used to establish these goals. A broader set of metrics is available from the Institution of Chemical Engineers (IChemE). The Leadership and Energy and Environmental Design (LEED™) building rating system also provides a framework for setting project performance goals

Table 2 provides a list of GHGs associated with specific industrial sectors. Table 3 provides the major emission categories, with a representative list of devices that fall into each of these categories.

During conceptual design, the integrated sustainable design team evaluates design alternatives. Project facilities, process and mechanical equipment, and building components or features should be evaluated based on their sustainability as well as feasibility and cost-effectiveness. The team should consider the maturity of the technology of the building, facility or process feature; the capital expenditure (i.e., first cost) required to procure, install, and implement the facility, building or process feature under consideration; the acceptable time horizon to repay the initial capital expenditure through savings made via operating and maintenance costs over the life of the feature; and the carbon emitted during the construction and operation of the facility, building or process feature.

Consider alternatives to:

- Maximize energy efficiency and minimize GHG emissions by:
  - Selection of a suitable location and orientation for the facilities on the site and their configurations and proportions to minimize energy loads (and thus GHG emissions) on the building due to climate at the site and to take advantage of passive solar and wind opportunities
  - Preservation of natural site features, restoration of degraded habitat areas, and minimize facility footprint to preserve the maximum open space and undisturbed land
  - Providing for natural daylighting, renewable energy, and natural ventilation
  - Selecting high-efficiency HVAC systems that exceed ASHRAE 90.1 requirements (ANSI/ASHRAE [7])
  - Providing light-colored roofing to reflect light and heat
  - Sealing joints and airlocks prior to sizing mechanical conditioning systems to avoid over-sizing equipment
  - Reclaiming waste heat from equipment and return air and water
  - Installing vegetated rooftops
  - Maximizing the efficiency of electric power distribution; size transformers close to the actual anticipated load; distribute power at the highest practical voltage at the maximum power factor
  - Maximizing service water heating and cooling efficiency, and considering solar hot water heating
- Maximize water efficiency by:
  - Collection and reuse of rainwater

  - Closed-loop processes to conserve water and other resources
  - Alternative wastewater treatment system selection. Consider waste water from processes for possible treatment and reuse as grey water, if permitted.
  - Developing self-sustaining landscapes using native plants tolerant of soils, climate, and drought with minimal irrigation using harvested rainwater or grey water
  - Specifying water-efficient plumbing such as ultra-low-flow or waterless plumbing
  - Protect or restore the existing site hydrology
- Minimize the embodied energy and carbon content of materials by
  - Considering off-site pre-fabrication to minimize onsite cutting and waste.
  - Designing for constructability and potential reuse
  - Develop energy and emissions criteria to include in materials requisitions

The conceptual design should include development of a simplified energy model, a preliminary estimate of GHG emissions and embodied carbon content, a preliminary lifecycle cost estimate and a 3-D model with preliminary building information. These design documents and models are used throughout the design process to assess progress toward meeting the sustainable design goals. Upon completion of these and other design deliverables, conduct the first review of the sustainable design aspects of the project during the project's first design review. This review should check that the sustainability goals established earlier, including energy efficiency, GHG emissions, water efficiency and materials content are being met. The National Renewable Energy Laboratory has a handbook for planning and conducting charrettes for high-performance projects that may be helpful in preparing for the sustainable design portion of the design review (NREL [8]).

Compare preliminary energy model results with sustainable design performance benchmarks, using readily available software for: energy requirements simulation; renewable energy; water conservation; and lighting and daylighting analysis.

GHG emissions may be estimated for sources classified according to a scheme similar to that provided on Table 3 for the petroleum industry (API [9]). Table 3 provides the major emission categories, with a representative list of devices that fall into each of these categories. The GHG Protocol for Project Accounting developed by the World Business Council for Sustainable Development (WBCSD) and the World Resources Institute (WRI/WBCSD [10]) appears to be the protocol most suitable for use developing design basis GHG emissions estimates. The U.S. Energy Information Administration provides emissions coefficients for a number of fuels combusted for energy generation here: www.eia.doe.gov/oiaf/1605/coefficients.html (last accessed 30 Aug 2010).

Useful guides to life-cycle cost estimates are provided by ASTM International and the U.S. Department of Energy. ASTM E 917-05 is a standard practice for measuring life-cycle costs of building and buildings systems (ASTM International [11]). The U.S. DOE's guidance is for life-cycle cost analyses required by Executive Order 13123, Greening the Government through Efficient Energy Management (U.S. DOE [12]).

A number of resources are available to check the embodied energy and carbon content of materials. The University of Bath, Department of Mechanical Engineering, maintains a database of the embodied carbon dioxide and energy content of building materials. A summary of the coefficients in the database is periodically published and is available here: www.bath.ac.uk/mech-eng/sert/embodied/ (last accessed 30 Aug 2010). The embodied energy and carbon dioxide refers to the total primary energy consumed and carbon dioxide released over the life-cycle of the construction material. With the life-cycle boundary being defined as all energy and carbon dioxide emissions from the extraction of raw materials through manufacturing up to the point where the materials leave the manufacturing facility.

4 Preliminary Design

During preliminary design develop the facility energy model to confirm the design meets the established performance goals; calculate facility operations GHG emissions and materials embodied carbon content; develop a facility life-cycle cost estimate; include building information in the 3-D model. Periodically update these calculations and verify the project continues to meet the sustainable design performance goals as design progresses.

Continue to promote an integrated work process among all disciplines with early inputs from procurement, project controls and construction to assure continued implementation of the established sustainable design scope during systems selection. The following tasks are included in this design phase:

- Include sustainable engineering concepts in system design descriptions and facility design descriptions. Right-size systems and facilities using software models (not conventional rules-of-thumb), avoid over-design.
- Identify energy consumption by category, e.g., internal loads from the processes, building envelope loads (heat losses / gains through walls, roofs, etc.), ventilation requirements, and others.
- Identify energy interactions between systems and opportunities for reductions in energy requirements and cost savings through energy efficiency measures.
- Develop alternative design solutions to reduce energy loads and evaluate systems as a whole.
- Iterate these optimization steps and refine the system selection / design to arrive at the optimized combination of systems for energy efficiency and emissions reduction.
- Update the energy model, emissions calculations, cost estimate and 3-D model to reflect the design, as it develops.

Conduct a second review of progress toward meeting energy and emissions goals on the project, after the design concept is developed. This review can be concurrent with other required design reviews and is intended to confirm continued progress toward meeting the established sustainable design criteria.

5 Detailed Design

Continue to promote an integrated work process among all disciplines to assure continued implementation of the established energy efficiency and emissions reduction goals. Specify low embodied CO_2 and energy content materials. Include embodied energy and CO_2 evaluation criteria in technical bid evaluations. Specify materials available locally.

Develop a construction execution strategy that minimizes construction energy consumption and greenhouse gas emissions. Consider construction waste management options, construction vehicle options, etc.

Finalize the:

- Energy model
- 3-D model with building information
- GHG emissions calculations
- Life-cycle cost estimate

Conduct a third and final review of the design relative to the energy efficiency and emissions reduction goals.

Assist procurement with evaluating and pre-qualifying potential bidders for materials, systems and sources to support implementation of sustainable design goals. Discuss sustainable design requirements in pre-bid meetings. Include criteria for vendors to conserve energy, water and other natural resources during field construction activities.

Incorporate sustainable design requirements in standard project specifications and material requisitions (MRs) to be included with purchase orders and contracts / sub-contracts, including sourcing from suppliers practicing sustainable manufacturing practices, which are located near the construction site.

6 Construction

Prepare a site master plan for orderly development of infrastructure and land use on the site for the construction phase and operations phase. Prepare designs and specifications for temporary field facilities following sustainable design principles. Locate temporary facilities in non-sensitive areas of the site and minimize temporary facilities by building permanent facilities early in the construction phase and using them for construction needs. Include sustainable engineering considerations in responses to requests for information, field change requests and other design change documents.

During construction, track embodied energy and CO_2 content of construction materials; review supplier submittals and substitutions for impacts on energy and emissions performance goals. Review field change requests for impacts on energy and emissions performance goals and plan for energy and emissions systems commissioning to verify performance during operation. Commissioning activities may begin as early as the conceptual design phase when the commissioning agent reviews design documents for conformance with the sustainable design criteria. During construction and start-up the commissioning agent verifies installation of energy and emissions related systems and confirms that they operate as specified.

7 Conclusions

Designing to meet the challenges of climate change does not require a completely new design process. Incorporating sustainable design considerations into the conventional design process can result in more energy efficient and lower GHG emitting designs if sustainable design performance goals are set early in the project development and regularly monitored to assure the evolving design continues to support achieving the goals. Establishing an integrated sustainable design project team comprised of representatives from each engineering discipline, procurement, construction, the client and other stakeholders provides a working group to collaborate in evaluating design alternatives to optimize energy efficiency and minimize emissions.

This integrated team develops sustainable design criteria early in the project, is involved in developing and maintaining an energy model and 3-D model with building information. The team track GHG emissions and the embodied carbon and energy contents of materials by periodically updating relevant design calculations. The team participates in development and periodic updates of the project life-cycle cost estimate to inform decisions about the feasibility of implementing sustainable design alternatives.

All of these activities can be conducted in the context of an established design process and provide added value to clients in terms of energy and water efficiency. While it may not be possible to fully embed sustainable design in the standard engineering work process without some cost and potentially schedule consequences for engineering, it is likely that when considered in the context of the overall life-cycle cost of a project, sustainable design will reduce life-cycle costs and produce significant benefits for climate change mitigation.

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Table 1. *Climate Change Related Project Performance Metrics*

Category	**Metric**
Materials	Materials used by weight (kg) or volume (m^3)
	Weight or volume percentage of materials used that have recycled input materials
	Total weight of waste materials by type and disposal method
Energy	Direct energy consumption (kJ) by primary energy sources
	Indirect energy consumption (kJ) by primary source
	Energy exports (kJ)
	Energy use per weight or volume of product produced (kJ/kg or kJ/m^3)
Water	Total water withdrawal by source (m^3)
	Total water discharges by quality and destination (m^3)
Emissions	Total direct and indirect greenhouse gas emissions by weight (tons CO_2-equivalents)
	Indirect greenhouse gas emissions by weight (tons CO_2-equivalents)
	Greenhouse gas emissions per weight or volume of product produced (tons CO_2-equivalents/kg)
	Emissions of ozone-depleting substances by weight (kg)
	Emissions of NO_x, SO_x, PM_{10}, ozone, carbon monoxide and lead by weight (kg)
	Emissions avoided (kg)

Table 2. *Industry Specific Greenhouse Gases as Identified in 2006 IPCC Guidelines for National Greenhouse Gas Inventories*

Industry	**Activity**	**Greenhouse Gases**
Power	Fuel combustion	CO_2, CH_4, N_2O, NO_x, CO, NMVOC, SO_2
	Fugitive emissions from fuel	CO_2, CH_4, N_2O, NO_x, CO, NMVOC
Mining & metals	Industrial processes and product use	CO_2, CH_4, N_2O, HFCs, PFCs, SF_6, Other halogenated gases, NO_x, CO, NMVOC, SO_2
	Metals industry	CO_2, CH_4, N_2O, HFCs, PFCs, SF_6, Other halogenated gases, NO_x, CO, NMVOC, SO_2
Oil, gas and chemicals	Industrial process and product use	CO_2, CH_4, N_2O, HFCs, PFCs, SF_6, Other halogenated gases, NO_x, CO, NMVOC, SO_2
	Chemical industry	CO_2, CH_4, N_2O, HFCs, PFCs, SF_6, Other halogenated gases, NO_x, CO, NMVOC, SO_2
Other	Waste	CO_2, CH_4, N_2O, NO_x, CO, NMVOC, SO_2

NMVOC: non-methane volatile organic compounds

Table 3. *Source Classifications*

Category	**Principal Sources Include:**
Combustion Devices	
Stationary Devices	Boilers, heaters, furnaces, reciprocating internal combustion engines and turbines, flares, incinerators, and thermal/catalytic oxidizers
Essential Mobile Sources	Barges, ships, railcars, and trucks for material transport; and planes/helicopters and other company vehicles
Indirect	Off-site generation of electricity, hot water and steam for onsite power and heat
Vented Sources	
Process Vents	Hydrogen plants, amine units, glycol dehydrators, fluid catalytic cracking unit and reformer regeneration, flexicoker coke burn
Other Venting	Crude oil, condensate, and petroleum product storage tanks, gas-driven pneumatic devices, chemical injection pumps, exploratory drilling, loading/ballasting/transit, loading racks
Maintenance/Turnaround	Decoking of furnace tubes, vessel and gas compressor depressurizing, well and pipeline blowdowns, tank cleaning, painting
Non-Routine Activities	Pressure relief valves, emergency shut-down devices
Fugitive Sources	
Fugitive Emissions	Valves, flanges, connectors, pumps, compressor seal leaks
Other Non-Point Sources	Wastewater treatment, surface impoundments

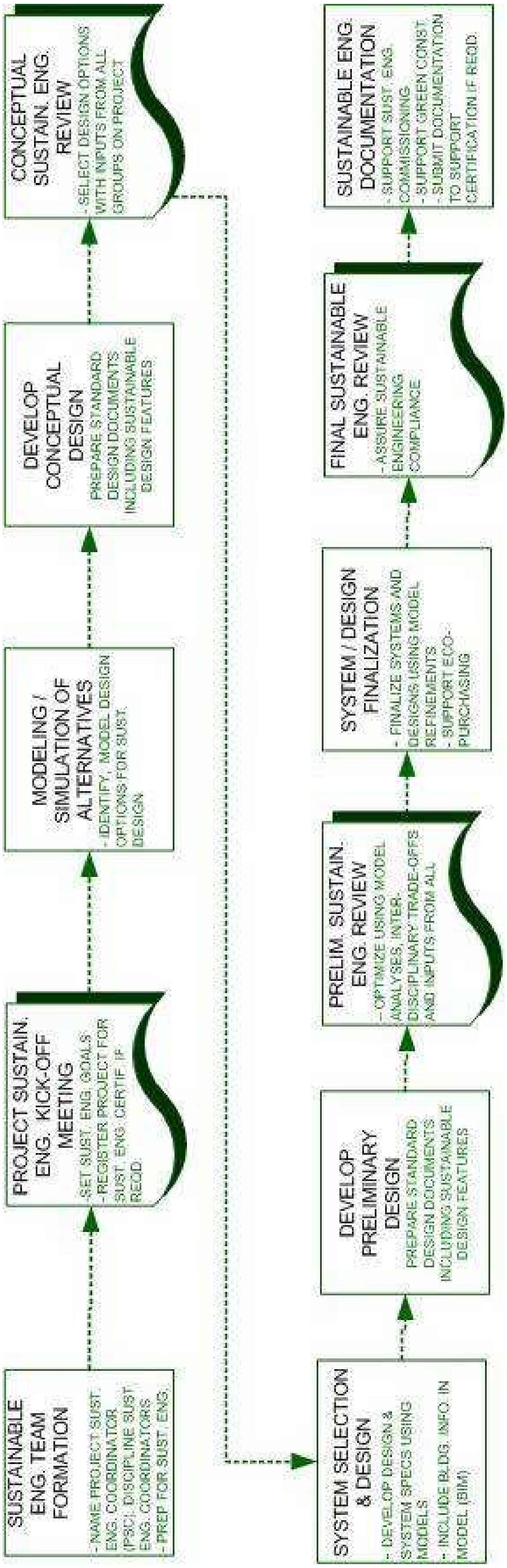

Figure 1. *Overview of Design Work Process with Sustainable Design Elements Embedded*

SUSTAINABLE DEVELOPMENT THROUGH GREEN BUILDING CONCEPT IN SRI LANKA

B.A.W.P. Bombugala[1], A. Atputharajah[2]

[1]Central Engineering Consultancy Bureau, Colombo, Sri Lanka
Telephone: +94-11-2668800; Fax: +94-11-2668951
bpriyantha@yahoo.co.uk

[2]Department of Electrical & Electronic Engineering, University of Peradeniya, Sri Lanka
Telephone: +94-81-2393408; Fax: + 94-81-2385772
atpu@ee.pdn.ac.lk

Abstract: The construction sector accounts for a large percentage of the world's total energy consumption and green house gas emissions. One of the effective and intelligent initiatives in infrastructure sector is sustainable development through green building concept. "Green Building" concept is a practice of creating structures and using processes that are environmentally responsible and resources-efficient through the building life cycle. Implementing the green building concept can result in reduction of carbon emissions by 35%, water usage by 40%, energy usage by 50% and solid waste by 70%. This paper also discuss the factors to be considered for green building construction and the Leadership in Energy and Environmental Design (LEED) assessment method to declare a building as a green building. The unique departure from traditional building model to green model delivers savings in energy cost, reduction in water consumption, reduce in water cycling in green building constructed in Sri Lanka are highlighted in the paper.

Key Words: Sustainable development, Green building

1. Introduction

Sustainable development is development that meets the needs of the present without compromising the ability of future generations to meet their own needs [1]. Sustainable development ties together concern for carrying capacity of natural systems with the social challengers facing humanity. Three most important indicators of sustainability are Environment, Society and Economy [2].

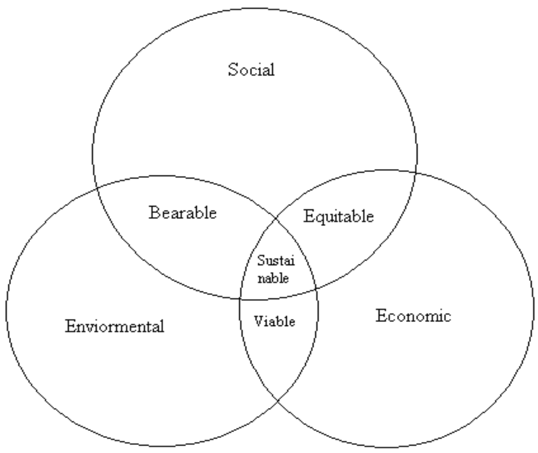

Figure 1: *Components of Sustainable development*

One of the main indicators of environmental sustainability is the process of ensuring that the existing method of human interaction with the environment is as pure as naturally possible. Also the economic sustainability is the economic development with minimal environmental degradation or equitable development that environmentally and socially sounds. Social sustainability indicates that the future generations should have the same or greater access to social resources as the current generations.

The concept of green building emphasize saving water, energy and material resources in construction and maintenance of buildings that can reduce or eliminate the adverse impact on the environment and occupants. It can be highlighted that implementing the green building concept can result in reduction of carbon emissions by 35%, water usage by 40%, energy usage by 50% and the solid waste by 70% [3].

A representation of elements of green building taken into consideration at the design stage [2]

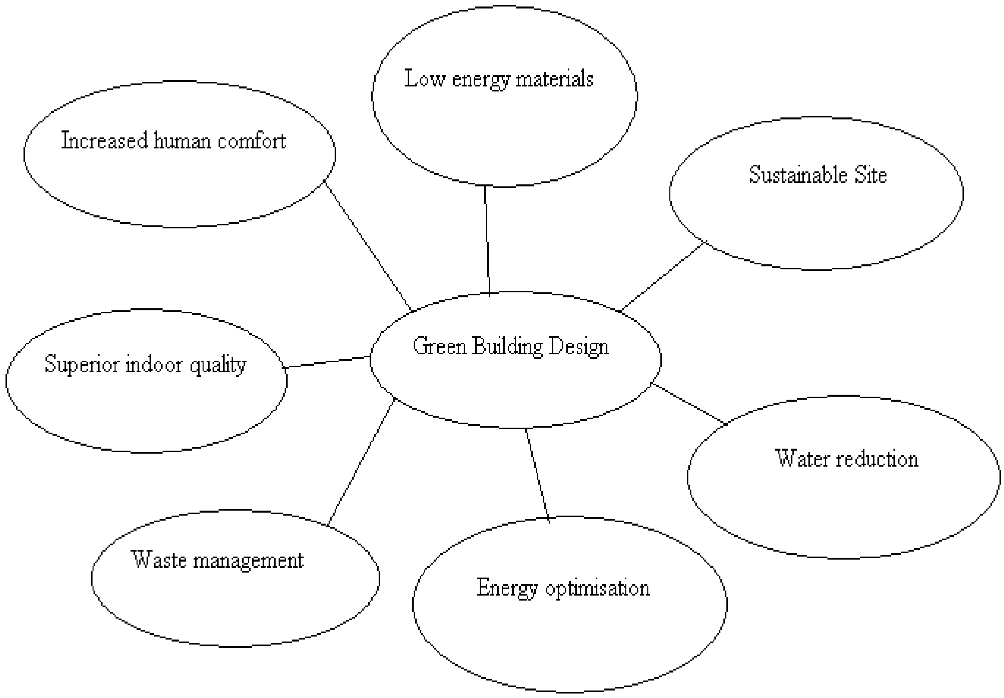

Figure 2: *Components of Green Building Design*

The benefits of green building can be illustrated as follows: (i) reduce energy consumption, (ii) reduced destruction of natural resources, (iii) reduced water consumption, (iv) limited waste generation, (v) increased user productivity and (vi) corporate image enhancement.

2. Benefits of the green building construction.

Conserve the external environment at the building location, Improve internal environment for the occupants, Preserve the environment at places even far away from the building

2.1 Green building conserves the external environment at the building location

When planning to construct any type of building, the site should be selected after taking into consideration the conservation of local vegetation, wildlife, natural water resources etc. Specially a site with biodiversity should be either avoided or the building should be planned to reduce site disturbances.

2.1.1Land

The landscaping and exterior design in a green building shall be done to ensure more shaded area, the light trespassing can be eliminated and local species of plants can be grown.

2.1.2 Water

The green building design shall not disrupt natural water flow. Rainfall in the catchments shall be harvested fully to either replenish the ground water table in and around the building or to be utilized in the services of the building. The toilet shall be fixed with low flush fixtures and the plumbing system should have separate lines for drinking and flushing, Grey water from the kitchen, bath and laundry shall be treated and reused for either gardening or cooling towers of air conditioning. Bidets help eliminate the use of toilet-paper reducing sewer traffic and increasing possibilities of re-using water on site. The use of non-sewerage and grey water for on-site use such as site irrigation will minimize demands on the local aquifer. Waste water from sources such as dish washing or washing machines can be used for subsurface irrigation or if treated for non-potable purposes e.g. flush toilets and wash cars.

2.1.3 Waste

Green architecture also seeks to reduce waste of energy, water and materials used during construction. Well designed building also help reduce the amount of waste generated by the occupants as well, by providing on-site solutions such as composite bins to reduce matter going to landfills. By collecting human waste at the source and running it to a semi-centralized biogas plant with other biological waste liquid fertilizer can be produced.

2.1.4 Energy

The solar energy at the top of a green building is harvested supplement conventional energy. Solar water heating further reduces the energy load. Natural light is harvested in intermediate floors to minimize electricity usage. In addition, effective window placement (day lighting) can provide more natural light and lessen the need of electric lighting during the day. Sunlight is restricted by the high-growing trees outside lower floors of the building. High efficiency light fittings make pleasant lighting in addition to saving energy. High efficiency windows and insulation in walls, ceiling and floors ensure better temperature control. The use of low-energy sources such as compact fluorescent lamps and solid state lighting with light emitting diode bulbs (LED) lamps and ‘low voltage’ electrical filament-incandescent light bulbs, and compact metal halide, xenon and halogen lamps can be used. Also timers, motion detectors and natural light operation sensors reduce energy consumption and light pollution even further.

2.2 Green building improve internal environmental for occupants

2.2.1 Light

In a green building, occupants shall feel as if they are in a natural environment. Interior and exterior designs shall go hand in hand by blending natural and artificial lighting.

2.2.2 Air

A comfortable atmosphere at workstations improves staff attendance and increases productivity. In and air-conditioned environment, a green building shall be specially equipped to ensure in indoor air quality necessary for a healthy atmosphere. The inhabitants can breathe air free from any odor of paints, polish or varnish.

2.3 Green buildings preserve the environment at places far from the building

Buildings are constructed using cement, sand, steel, stones, bricks and finishing materials. Collectively these are responsible for about 20% of the green house gases emitted by a building during its lifetime. Green buildings use products that are non-toxic, reusable, and recyclable wherever possible. Locally manufactured products are preferred which also save the fuel ordinary used to transport materials. Preference should also be given to recycled materials. Materials with higher recycled content should be selected in order to reduce the embodied energy of the buildings, thereby decreasing the environmental impact of extraction and processing of energy extensive materials. Ecology blocks, recycled stone, recycled metal, recycled concrete, garbage, wood, rice husk ash, compressed earth blocks, baked earth, foundry sand, demolishing debris in construction projects are few of materials that can be use for green building.

3. Impacts on Green Building construction & operation

The most criticized issue about constructing environmental friendly buildings is the price. Photovoltaic, new appliances and modern technologies tend to cost more money. Most green buildings increase the construction cost by 20-25% but the yield 10 times as much over the entire life of the building. The statics have shown green building reduce 30-40% in operational cost of the building.

4. Green building rating system

Buildings constructed based on green building concept should conform to prescribed standards. There should be continuous assessment and monitoring from the planning/design stage up to the completion of construction, in order to declare a building a green building. The LEED (Leadership in Energy and Environmental Design) green building rating is followed in this assessment of a building. In this system, points are awarded for adopting green concepts in various categories and the buildings are certified green at levels such as Silver, Gold, or Platinum based on the total number of points they get in this LEED rating.

5. Constructed green buildings in Sri Lanka

5.1. The MAS intimates Thurulie factory in Thulhiriya [4]

Energy cost reduce by 40%

Low energy “evaporative cooling” system used in place of air conditioners and this alone saves 65% of the energy consumed. Light usage is minimized by depending amply on daylight to light the premises and individual sewing machines are kitted out with an LED based task light. 10% of the plant’s power provide by solar panels.

50 % reduction in water consumption

Rain catchment tanks are used to collect water for non-drinking purposes, such as flushing toilets and landscaping.

95% waste recycling

All sewage is treated by on-site anaerobic digestion sewage treatment facility and bio-gas which is a by product of this will be used in the kitchen.

Reduced absenteeism

Green roof with grown vegetation and cool roofs with high solar reflectivity ensure a cooler interior. Amenities such as relax-stations, picnic areas and a holistic centre ensure better comfort the employees.

5.2 Brandix Causualwear, Seeduwa [3]

Energy cost reduction by 50%

New screw-type chiller unit provide energy –efficient air conditioning for the entire factory. Square ducts were converted to round ducts to reduce distribution loss in air conditioning. Sophisticated new LED used as task lights provide light to the sewing machines.

60% reduction in water consumption

A serious of measures exists to reduce water consumption through recycling. Direct rainwater is recycled for all use except for drinking. A tertiary filtration system and a disinfection process allows the used water to be recycled again for toilet flushing and gardening.

95% waste recycling

The factory has a solid waste disposal system by recycling or reusing the solid waste it use. Canteen waste is being composed and contributes to biogas generation.

Reduced absenteeism with improvement health standard to 2%

An advanced intelligent building management system controls relative humidity and carbon dioxide levels to improve comfort levels for workers. The green areas in the gardens have been increased.

5.3 HNB, Nittambuwa
Reduce energy cost by 30% and 56% reduction in portable water

5.4 CKT Apparel, Mihila
Reduce energy consumption by 48%, reduce 60% total water by harvesting of rainwater

6.0 Calculating number of luminaries require for an area of a green building

Energy consumption of a green building from luminaries can be further reduced by rearranging & selection of luminaries for the area.
The number of luminaries needed for a particular area can be calculated from Utilization Factor method. First, Room Index (K) of the area must be calculated.

$$K = \frac{LxW}{(L+W)xH_m} \quad (1)$$

Where L= length of room
W= width of room
H_m = height of luminaries above working plane

Utilization Factor can be found from light fitting manufacturer's tables [5], when the room index (K) and the reflectance of the room are known.

Calculation of number of luminaries needed to provide required illuminance level can be found from

$$N = \frac{(ExLxW)}{(NLx\Phi xMFxUFxLLD)} \quad (2)$$

Where E= Required illuminance (Lux level)
N= Number of Luminaries
NL= Number of lamps in each luminaire
Φ= Lamp flux
MF= Maintenance Factor
UF= Utilization Factor
LLD= Lamp Lumen Deterioration Factor

6.1 Proposals to reduce energy consumption of luminaries
Energy consumption for lighting can be saved by reducing number of luminaries needed for a space. Further from the equation 2 for a particular lamp type MF, LLD & Φ are constants at the time of installation. Therefore by getting higher UF, the number of luminaries needed for the space can be reduced. Higher UF can be achieved by higher room index (K) for a correspondent reflectance of the space from fitting manufacturer's table. [5]

At the building design stage following steps are recommended to follow to achieve higher room index (K) which benefits for energy savings further in addition to using energy efficient lighting fixtures

1) Design a space to have high area to perimeter ratios means that the room should be more rectangular corridor shape
2) Decrease the height of the luminaire to the working plane
3) Increase reflectance from luminary by introducing polished or mirror like reflector fitting rather than opal or prismatic diffuser fitting in use.

7.0 Conclusion

Several benefits including environmental, economic, health, safety and community benefits can be achieved through adopting to the green building concepts instead of traditional building concept in the sustainable building construction and maintaining processes. Altogether these benefits lead to reduce operational cost and make the building sustainable. Rearrangement & reduction of lighting fixtures would be used as a tool to reduce energy consumption of these buildings too. Although construction cost of green buildings is 20-25% higher than the traditional buildings, the gaining advantages yield 10 times as much over the entire life of the building. Therefore for the sustainable development, green building concept must be integrated in all building codes.

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HAZARDS ASSESSMENT AND MANAGEMENT IN THE CITY OF ALGIERS (CAPITAL OF ALGERIA)

Djillali Benouar

Director, Built Environment Research Laboratory (LBE)

USTHB, Faculty of Civil Engineering, BP 32 El –Alia, Bab Ezzouar, Alger 16111, Algeria
Tel & Fax: +213 21 247 914 / E-mail: dbenouar@yahoo.com

ABSTRACT
As many other countries of the world, the northern African countries also suffer from environmental and geological problems, among others, the large cities and their suburbs. The capitals, particularly, represent gravitational poles constitute true economic metropolises of them, recording a considerable migratory flow in addition to one important demographic growth, a fast industrialization and an anarchistic urbanization, which make of it the receptacle of various sources of pollution, where ground, air and sea do not escape the consequences of these plagues. Furthermore, Algiers have had also suffered from storms, floods, landslides and earthquakes. Algiers in this research work, capital of Algeria, is taken as a case of study because it introduces most of the risks met in the other countries of North Africa. Algiers counts more than 3 million inhabitants for an area of 809.19 km^2. From the independence of Algeria in 1962, Algiers was found constituted of a dense urban fabric where various functional scales were overlapped. The town of Algiers experienced a significant development as well on the urban level as industrial and of this fact it is seen confronted with a degraded environment and a multiform pollution. The industrial sector and the factories established in urban fabric and its periphery are at the origin of the existing or potential sources of pollution in addition to the consumption of space.

Keywords: *Algiers, natural and Environmental Hazards, Disaster Management.*

1. Introduction

For the North African countries, geological, environmental and technological hazards constitute a constant threat to human life and property, sometimes causing major economic losses and disruption. The rapid urbanization, development of critical engineering works such as dams, decaying building stock, implementation of various industries within and around the main cities, industrialization of cities with modern types of buildings and the large concentration of populations, with a heavy dependency on infrastructure and services, living in large cities and/or settling in hazardous areas are matters of growing concern, as they contribute to heavier loss of life and seriously increasing the economic losses in future disaster damage. The environment concerns and an increased official and public awareness of various hazards have, in the last decade, led to a rapid rise of interest in hazard and risk evaluations and thus in disaster risk management.

In fact, there are various reasons for investigating the Mediterranean Africa as a unit and for evaluating the various hazards of the entire zone under similar criteria: (1) In terms of geological process: the countries limiting the southern part of the Mediterranean Sea and its adjacent continuation in the Atlantic Ocean have had, since hundred million years ago, the same tectonic process marked by a relative motion alternating between left and right lateral along the border of the African and Eurasian plates, (2) Similar present level of development: the actual state of development in the whole zone is dominated by a rapid urbanization, high density of population in most important cities, degradation of the environment, (3) at the present pace and patterns of rural-to-urban migration and unplanned urbanization are causing increased population densities in urban centers, such areas are a prime reason for increasing vulnerability, (4) Similar historical development: the historical development of the countries in the region shows many common factors, such as cultural background. Similarities in population settlements, building stock characteristics and socio-economic and demographic conditions, various types of pollution, climate, etc., are very important parameters in the whole process of disaster risk studies in most cities in the zone under consideration. For all these reasons, this research work is concentrated on the city of Algiers, which presents a general case of most of the geological, environmental and technological threats found in all the main cities across the Mediterranean Africa.

Algiers, the Capital of Algeria, including its surroundings communities, has a population of approximately three millions, represents the most important concentration of investment, government institutions and population in the whole country. It is the intellectual, social, political and economic center of the country. In recent years the disaster risks have increased due to overcrowding, faulty land use planning and construction, inadequate infrastructure and services, and environmental degradation and technological plants within and surrounding the city. In the last two decades, the city of Algiers and its surroundings have known an important development in the urban domain as well as in the industrial one and thus it is actually confronted to rapid environmental degradation and to a multiform pollution. The industrial sector implemented within the city itself and its surroundings increases considerably the risk of disasters. Algiers is confronted seriously to all risks as earthquakes, floods, landslides, as well as the industrial pollution (4 Industrial zones), atmospheric pollution (road traffic, main industries, Public dump Oued Smar), Solid wastes pollution (Public dump Oued Smar, units for wastes treatment), Hydro pollution (superficial and underground water), marine pollution, soil pollution, forests and green spaces degradation, as well as to geological risks. The topography, the waterfront location and the ancient neighborhoods (Casbah) make it difficult to affect radical solutions to the most of its problems. Decision-makers need adequate integrated information on the likely (probabilities) intensity of the disaster the city will face if they are to reduce disaster vulnerability. This research work shows the need of an integrated disaster risk management system in megacities.

Industrial Pollution

The industrial activity constitutes a most significant source of pollution and harmful effect in the city of Algiers. It is the origin of several forms of pollution as hydrous pollution and atmospheric; it is also generating dangerous and toxic wastes posing serious problems for the health of the people as well as their elimination.

Great Algiers represents one of the most significant areas of the country, from the point of view of the industrial activity. The total number of industrial units is approximately 735 public and private in the Wilaya of Algiers, that is to say 7.2% of the national total estimated at 10 200 units, of which 5 242 would be located on the coast.

The polluting industrial activity is localized with the periphery of Algiers, mainly in the East at the industrial zones of Gué de Constantine (8 km of the center town), of El Harrach (11 km of the center), Oued Smar (15 km) and Rouiba - Réghaia (27 Km).

The most polluted industries are:

- Paper mills,
- Manufacture of batteries,
- Oils and soaps,
- Yeasts, detergents, metals heavy,
- Pharmaceutical products
- Fertilizers,
- Petroleum products: gas, gasoline, asphalt,
- etc.

Air pollution

The principal sources of air pollution in Algiers are:

- Very intense road traffic,
- Rejections of pollutants by the industrial units established in urban fabric and by the various industrial zones,
- The combustion of the wastes at the public damp of Oued Smar.

2. Road traffic

The automobile park of Algiers counts approximately 560 000 vehicles all categories mixed (with a daily traffic of 1500 to 2000 vehicles/hour), which represents the quarter of the national automobile park. This pollution consists approximately to CO2 15%, 60 à.70% of CO, 40 to 50% of NO, 30% of the hydrocarbon none burnt residues, SO2 5%, black fume, dust, lead, …

A quantity of 180 tons lead/year is emitted in the streets of the capital at a rate of 0.5g of lead/liter of gasoline for the vehicles, that is to say an annual average of 3.8mg/m^3 (average higher than that of the capitals of the other countries). The annual average recommended by WHO, as a standard of quality of the air not to be exceeded, ranges between 0.5 and 1 mg/m^3. Whereas lead concentration in the agglomeration of Algiers east of 2.01 mg/m^3 (Aoudia, 1996).

3. The main industries

The most polluting industries are:

- the cement factory of Rais Hamidou which emits cement dust charged with combustion gas NO, CO2 and of CO, is 30 tons of dust/day (the standard ranges between 15 to 250 tons/Km^2/year) (Service environment and forests, 1990). This cement factory, localized within the heart of an urban zone (to 10 km west of the center of Algiers), unquestionably constitutes a significant harmful effect for the populations resident and the whole environment in the vicinity. Producing 750 tons of cement per day, this cement factory emits fine particles, made up mainly by products limestone, which generate respiratory diseases. The most visible impact is certainly the deposit of cement dust on the roofs, the vegetation and on the entire zone close to the cement factory. The threshold of 1000g of dust/100m^2/month, determined by a model of dispersion as a norm, is largely exceeded on a radius of 3 km (INGECO, 1997).
- The tobacco production units of Bab El Oued and El Hamma (both located within the center of Algiers) emit harmful fume due to the use of fuel.
- the unit of cable-making of Oued Smar emits dust charged with lead,
- the industrial unit of the greasy substances in the harbor of Algiers emits fume with nauseous odors,
- the manufacturing plant of batteries of Oued Smar emits lead oxide dust,
- the manufacturing plant of painting within Oued Smar emits gas dust charged with asbestos and lead,
- the refinery of Baraki (localized to 12 km of the center of Algiers) releases various hydrocarbon gases,

4. Public dump of Oued Smar

It releases nauseous fume, odors and gas emissions (CH4, CO2, NH3) from the combustion of the household refuse of the city of Algiers, its surroundings and the industrial zones. This pollution is visible at Oued Smar, El Harrach, Bab Ezzouar, Hamiz, Dar El Beida and Eucalyptus.

Pollution by solid waste

Solid urban wastes can be defined like the whole of the solid waste generated by the urban activity. In Algeria, in particular in Algiers, household refuses pose serious difficulties to the public services to collect them correctly, as they encumber the streets of Algiers and its surroundings. Wild waste dumps on which the inhabitants come to deposit their waste without paying any attention to the impact on the environment. In fact, the lack of information and awareness, and the non respect of the law, made that the citizen pours his solid waste anywhere, more particularly in the isolated places and thus the creation of wild discharges. The estimate of the quantity of the waste generated by the agglomeration of Algiers is about 1 408 tons per day (ANAT, 1996). The quantity of 2 500 tons per day is produced on the whole of Wilaya of Algiers, that is to say 910 000 tons/year. This quantity has risen to 3200 tons/day in 2005.

As for the industrial facilities, they pour daily more than 2 000 tons of not controlled wastes, particularly asbestos, acid, cyanide, phosphorus, etc. It should be also noted that waste of the slaughterhouses is rejected directly into the public dump of Oued Smar.

The nature of solid waste is:

- domestic Waste: 657 000 tons/year;
- industrial Waste: 930 000 tons/year;
- Waste of the administrations (paper, paperboards, etc.) : 90 000 tons/year;
- Waste of the markets: 50 000 tons/year;
- Waste of hospital (syringes, bandages, etc.) : 3 900 tons/year;
- Waste of trade: 80 000 tons/year;
- special Waste (toxic): 30 000 tons/year;

(Service CPVA in Inspection of environment of Algiers).

Public dump of Oued Smar

Solid waste of the Wilaya of Algiers is forwarded to the single public dump of Oued Smar just as well as the waste of the industrial zones. The initial surface was 10 hectares and reached 37.5 hectares today. It is located at 13 km of the center of Algiers on a ground of a unit of clay. This public damp reached a very advanced degree of saturation where the monticules of stored waste exceed 6 meter high above the ground level. An investigation of a engineering and design department (Kaoula, 1996) estimated at 4 000 tons/day the quantity of waste which arrives on the discharge including 1 600 tons/day of domestic waste, that is to say approximately 1 000000 tons/year coming from Wilayates of Algiers, Boumerdes and Tipaza.

Hydrous pollution

Hydrous pollution is related to the disposal of the liquid and solid wastes in the ground. Pollution has reached most of the hydrographic networks and poses serious problems, on the one hand by the insufficiency of the water resources, and on the other hand, because of the degradation of the living conditions in the aquatic environment.

Superficial waters

The superficial water pollution is caused mainly by the sewage systems deteriorated and not maintained where also pour the effluents in the rivers. Sewage water of the of the city of Algiers and that of all the other communes of the Wilaya is rejected to the sea, either directly, or by the means of

Oued El Harrach. Oued El Harrach in which flow all the waste water of the regions of El Harrach, Baraki, Eucalyptus, Bab Ezzouar, Dar El Beida, Oued Smar, Gué of Constantine and even part of Wilaya de Blida. The total volume of the wastewater poured in the Oued El Harrach is approximately 57 000m^3/day (IEA, 1997).

Underground waters

The water table of the plain of Mitidja constitutes the main water tank of the area. It is prone to many contaminations:

- Pesticides, nitrate fertilizers;
- the liquid infiltrations of the water table;
- Nitrates of the catchments basin of Oued El Harrach.

The pollution of the water tables by hydrocarbons is a serious problem since it can expand to surrounding collecting fields. Indeed, drillings of the water table of Algiers were subjected of pollution by hydrocarbons. Most of drillings analyzed present a very high degree of pollution, showing a largely higher index than the maximum permissible concentration (10mg/litre) for the water intended for human consumption (Bruchet, 1985).

The discharge of domestic and industrial waste water without treatment in the receiving medium constituted by a hydrographic network which is characterized by nonpermanent rivers and relatively low flow not allowing a process of self - purification, dangerously threatens the water table, the beaches, the dams as well as the public health.

Marine pollution

The bay of Algiers covers a water area of 184 ha, characterized by a pollution of urban and industrial origin. Most of the cities in the Wilaya and industrial facilities established on the of Algiers littoral pour their waste water either directly to the sea, or by the means of the Oueds, without preliminary processing which causes the deterioration the sea water quality. At the Algiers harbor, 25 outlets of urban and industrial waste water were listed, with presence of oils and greases coming from the harbor maintenance workshops, the sewage waters of Hamma, the hospital Mustapha Bacha, the factories of the fatty corps, the pasta production unit, oil company and electricity production unit in addition to the draining operated by the tankers and other ships either within the harbor or at large (Urbanis, 1998).

Soils Pollution

The pollution of the soil may have its origin from the industrial activities. The massive use of the artificial fertilizers, the use of certain organic soil conditioners and the systematic use to the pesticides result in a very significant increase in the agricultural outputs; unfortunately this rise of the productivity of the grounds is often accompanied by the increase in the contents of heavy metals in these grounds. Indeed, the contamination of the grounds by heavy metals constitutes a phenomenon which results mainly from the various human activities as the agricultural use of fertilizers (phosphates, fertilizers, organic soil conditioners,..), the industrial wastes, etc. The analysis carried out in various places of the Mitidja plain shows a content of nitrates between 50 and 250 mg/l, whereas the standard set by WHO is 45 mg/l (ANAT, 1996).

Deterioration of the forests (Deforestation)

The forests of Wilaya of Algiers know actually a very advanced deterioration covering a surface of 633 ha for a total surface of 5 338 ha, including 37 forests whose majority were arranged as entertaining forests.

The principal causes of deterioration of the forests are:

- Attack of insects;
- Fires;
- Atmospheric pollution of origin generated by the industrial activity; case of the cement factory of Rais Hamidou which devastated the forest of Bainem;
- Neglect of the sector of the forests in the land use decisions;
- anarchistic urbanization;
- Proliferations of the cities.

Floods

Rapid urbanization is a major factor in the increase of floods. Flash floods is a growing concern due to concrete which absorbs little water, the decline of open spaces, engineering works that divert river flows and weak city drainage systems (neglect, lack of maintenance). inappropriate housing on river banks or near delta is a major concern.

The last Algiers flood and mudflow of 10 November 2001 which caused the loss of 712 human lives, injured 350, 116 are missing, and 1800 housing units suffered damage, 56 schools, scores of bridges, roads, public works were damaged. 1000000 m3 (up to 10 m thick) of mud in the street of Bab el Oued, more than 350 vehicles (cars, trucks and buses) with passengers buried under mud.
•Preliminary cost U.S.$ 250 000 000.

Landslides

•Rocks and soil sliding rapidly downhill. Mudflows and rock falls triggered by earthquakes, storms, water logged soil and heavy constructions.
•growing amounts of badly built housing on/below steep slopes, on cliffs, or at river mouths of mountain valley. Landslides have occurred with or without the help of earthquakes. During the last Algiers flood and mudflow of 10 November 2001, several landslide cases were recorded within Algiers and its surroundings.

Earthquakes

•Algiers, densely built, densely populated city located on seismic zone. The city suffered several damaging earthquakes in the past, occurring within the city or in adjacent zones. The earthquake catalogue for the Wilaya of Algiers goes back to 1365.
•The building stock of the capital Algiers presents a high vulnerability to earthquake loads and thus seismic risk management is really needed if the government wants to avoid surprises

5. Conclusions and Recommendations

Decision-makers need adequate integrated information on the likely (probabilities) intensity of the disaster the city will face if they are to reduce disaster vulnerability. This research shows the need of an integrated disaster risk management in megacities.

In a country which, regrettably, is a disaster-prone as Algeria or in other country, it is of crucial importance, at the macro-level, for the country to have a well established and well regulated disaster management plan. This will enable the government to avoid undue crisis management when future emergencies occur. It is also of crucial importance, again at the macro-level, to integrate disaster management in all its facets with government's mainstream policies and plans for national development.

Disaster management and economic development are not two separate disciplines that conflicts for resourcing. They are synonymous and their resourcing should be a combined administrative process.

To fulfil these goals, the proposal of the establishment of a national disaster research and management agency in Algeria has two objectives (1) to prepare the national disaster management plan and (2) to create a sustainable cadre of disaster management staff at all levels, and to promote institutional and public awareness of disasters, their effects and likely relief activities. The need of permanently established national disaster management organisation is a must today. The organisation chart describes the structure, the chain of control and reporting, and the main working relationships (fig.1). It allows having a permanently established and functioning integrated data collection system to gather information relevant to disaster management in all its aspects. However, it has a wider application and provides an important step forwards an effective national data collection system, and this will require extensive research work. Several partial data bases are already available and these will be drawn upon in the creation of the agency which will attempt to provide a more general view within a single framework. The structure of the agency will then incorporate existing government, non-government and community information/data sources in order to provide an overall picture of potential danger zones, multi- sectoral early warning indicators and available resources. This enables particular attention to be paid to problem geographic sectors or problem functions, and the consequent mobilisation and allocation of resources in advance of disasters. A proposed permanent structure for disaster risk reduction management integrated in the Algerian government is presented in Figure 1.

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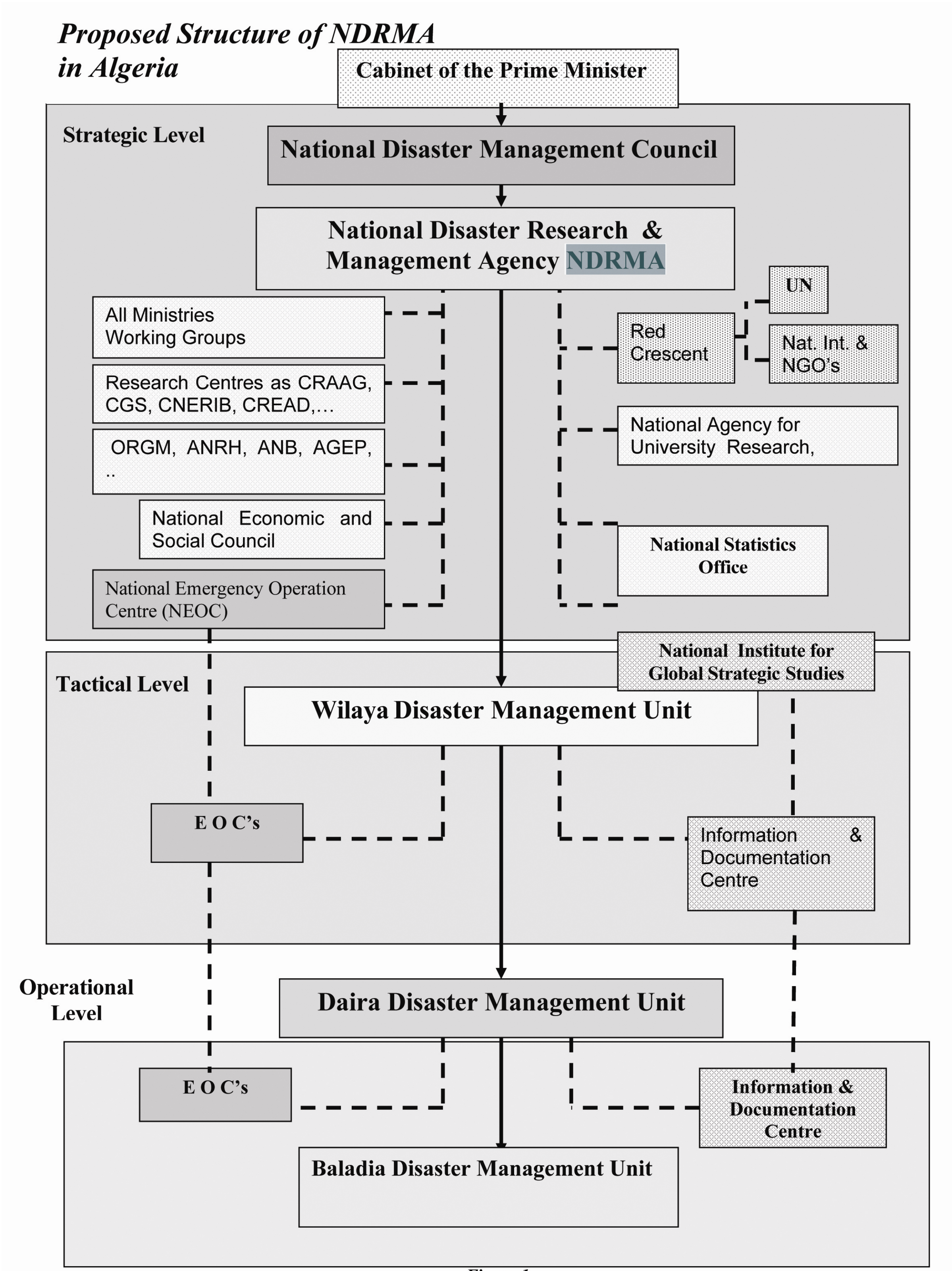
Proposed Structure of NDRMA
in Algeria
Cabinet of the Prime Minister
Strategic Level
National Disaster Management Council
National Disaster Research & Management Agency NDRMA
UN
All Ministries Working Groups
Red Crescent
Nat. Int. & NGO's
Research Centres as CRAAG, CGS, CNERIB, CREAD,…
National Agency for University Research,
ORGM, ANRH, ANB, AGEP, ..
National Economic and Social Council
National Statistics Office
National Emergency Operation Centre (NEOC)
National Institute for Global Strategic Studies
Tactical Level
Wilaya Disaster Management Unit
E O C's
Information & Documentation Centre
Operational Level
Daira Disaster Management Unit
E O C's
Information & Documentation Centre
Baladia Disaster Management Unit

Figure 1

THE SIMULATION OF THE POST FLOOD DRYING OF DWELLINGS IN LONDON

Taylor, J.[a,b], Davies, M.[a], Lai, K-M[b]

[a]Bartlett School of Graduate Studies, UCL, Gower Street, London, WC1E 6BT, UK
[b]Department of Civil, Environmental, and Geomatic Engineering, UCL, Gower Street, London, WC1E 6BT, UK

Corresponding Author: Jonathon Taylor
E-mail: j.g.taylor@ucl.ac.uk
Phone: +44 (0) 20 3108 4047
Fax: +44 (0) 20 7679 3042

Abstract
Climate projections indicate that the UK will experience more frequent extreme precipitation events and a rise in sea levels, with the greatest relative sea level rise occurring in the South-East of England. As a result, flooding is expected to occur more frequently in London. Flood simulation modelling has previously been done to predict the impact on the built environment for a variety of scenarios for London. However, such models have not taken into account the detailed hygric behaviour of the building stock under flooding and drying conditions which can vary significantly between property types, and can lead to prolonged damp and health problems. To address this problem, a building stock model of central London has been developed in a Geographic Information System (GIS), with information on the age, structure, and height of each dwelling and assumptions about built form based on historical building standards and surveys. This paper describes a novel methodology for the development of a building stock model with the necessary information to carry out hygrothermal simulations of the penetration of water into the building envelopes of flooded homes and the subsequent drying under different environmental scenarios. It discusses how the risk of damp in flooded homes can be modelled using hygrothermal methods and supplemented with mould models. By combining the multidisciplinary components of GIS, building simulation, and microbiological modelling, a holistic picture of the potential health implications of flooding at a population level across different temporal and spatial scales can be developed.

***Keywords**: Climate Change, Flood, GIS, building stock, Hygrothermal, mould*

1. Introduction

The world is currently experiencing a change in climate, which is predicted to bring changes in climate variability and extreme events, such as more frequent heat waves, less frequent cold spells, and a greater intensity of rainfall events. The impacts of climate change on the UK have been studied in the UKCPO9 climate change scenarios [1], and are predicted to lead to more frequent intense rain storms, which can cause surface and fluvial flooding. Surface flooding due to heavy rains occurs frequently in London; recently 320 London properties were flooded in the Autumn of 2000, and over 1,100 London households were flooded in the summer of 2007 [2], indicating that flooding is a problem even with current rainfall patterns. A combination of rising sea levels, sinking land, and large changes in winds and storms also means that the South East is predicted to experience the largest sea surface rise in the UK, and an increased frequency of tidal surges. This would lead to an increase in the risk of tidal flooding in London and the Thames Estuary. An increase in flood frequency and severity will test the resilience of the building stock and local communities to water damage and contamination.

Following a flood, improperly dried buildings can remain damp for long periods of time. This is significant as dampness in building materials in homes is consistently associated with respiratory illnesses, such as asthma, wheeze, cough, respiratory infections, and upper respiratory tract infections [3]. While most literature has focused on mould growth, bacteria [4-6], protozoa [7, 8], and viruses [9, 10] have also been found on damp surfaces or in elevated levels in the indoor air of damp houses.

Microbes such as mould and bacteria can release spores or fragments, and toxins into the indoor air, which may impact the health of occupants. During flooding, building surfaces can become contaminated with pathogens from the floodwater. The survival of pathogens on surfaces can also show a dependence on water availability and temperature [11], meaning the ability of a material or envelope design to dry quickly can impact the contamination levels.

Different building materials and envelope designs will react differently to moisture in both liquid and vapour form. Understanding how the building stock will respond to changes in the climate and extreme events can help predict how the long-term health of the population may be impacted under different scenarios. The objective of the study presented in this paper is to examine how existing building stock information developed for energy studies can be used to develop building archetypes suitable for use alongside building physics and microbial models to inform the flood vulnerability of existing buildings to damp and post-flood microbial persistence. This paper presents a novel methodology of combining building stock data and hygrothermal inputs in order to simulate the flooding of London. Future work on building simulation and the application of microbial models to the building archetypes and stock model will be discussed.

2. Theory

2.1 Hygrothermal and Mould Models

Heat, Air, and Moisture (HAM) computer models simulate the transport of heat, air and moisture within building assemblies. HAM models vary widely in their complexity, from simple steady-state models that do not take account of changing external and internal conditions, to complex whole building simulation models that calculate air and heat flow throughout the building and take into account transient heat, air, and moisture levels (eg. EnergyPlus)[12]. The family of HAM models of relevance to the current work operate by dividing wall assemblies into 1, 2, or 3D cells, and using numerical techniques to calculate the flow of heat, and liquid and vapour phase water from cell to cell. By taking into account the hygrothermal properties of the materials in the assembly, and the internal and external environmental conditions, such HAM models can calculate the moisture content and temperature at various positions throughout the assembly, which can then be used to predict the risk of microbial growth.

HAM models are widely used to assess the hygrothermal performance of buildings, and have been extensively validated and such tools have been used for simulating the drying of flooded buildings. For example, the modelling of the drying of a wooden church after a flood [13], and the simulations of the forced drying of historic wall constructions [14] have been done using EnergyPlus and WUFI, respectively. Delphin has been used to simulate walls of buildings in Dresden following the flooding of the River Elbe in 2002 [15]. HAM simulations offer a quicker and cheaper method of examining the absorption and drying of flooded buildings as compared to the alternative of physically constructing walls and buildings, flooding them, and then measuring the rate of drying [16-18].

As noted, mould growth is a serious problem in damp and flooded buildings. Relative humidity, temperature, nutrient availability, and colonising species are all important factors in mould growth on building surfaces. A number of models have been developed in order to predict the risk of mould growth in buildings, and are used alongside HAM models [19-22]. These models can be used to help predict the risk of mould growth in damp houses, but may have difficulty modelling the survival of viable spores when conditions dry. Further work is required to adapt these to the mould and bacteria species that are specifically found in flooded buildings.

2.2 Building Stock Models

Any potential building stock model for flood recovery will be complex due to the variations in the physical form of dwellings and the wide range of internal and external climates that can occur. In the UK, the housing stock has been constructed over a long period of time, with a wide range of materials and building techniques used, meaning that a range of assumptions are required in the development of a model. Many of these assumptions are already in use - building stock models are widely used for energy consumption estimation and energy policy development in the UK - and these models can be adapted for hygrothermal simulation.

Building stock models can be classified as bottom-up or top-down models. Bottom-up building stock models are built from a hierarchy of disaggregated components, which are then combined according to estimates of their individual impacts. Bottom up models based on building physics tend to consider dwelling archetypes, which are considered to be representative of the dwellings within a building stock. Building physics calculations are then used to estimate, for example, the energy consumption based on utilisation scenarios. There are a wide number of different bottom-up building stock models for the UK, which range from 2 to 1,000 building archetypes [23]. Top-down models are built from aggregated data, for example fitting historical energy consumption data, and so do not have the level of constructional detail or historical flood recovery data required for this study.

Of the existing building stock models in the UK, most bottom-up models divide the building stock into age-use groups, and define building archetypes to represent these building types. Buildings of different ages have different built forms, and can use different building materials. In studies of energy consumption, this is reflected, for example, in the differing heat transfer characteristics (U-values) for walls of different ages [24], which are based on assumptions about materials and material thicknesses. The structure of the building is used to infer the area of external walls and the internal layout of buildings, in order to calculate heat loss. Consequently, building stock information available for analysis tends to be divided into age brackets and building types that have similar layouts, and wall and floor constructions. Many of the assumptions made about building envelope design and construction are based on surveys, such as the English Housing Condition Survey (EHCS) [25] and knowledge of historical building trends [26]. The EHCS is a survey of the condition and energy efficiency of housing in England and describes the building fabric characteristics of surveyed buildings. Allen and Pinney [27] describe standard dwelling dimensions, construction, and occupancy schedules for building simulation modelling, while the Standard Assessment Protocol (SAP) for energy performance rating of buildings [24] and the Building Research Institute’s BREDEM model [28] also lists a number of building envelope assumptions used in building stock modelling.

While energy models are concerned with the thermal performance of the building envelope, hygrothermal simulations are interested in the *combined* thermal and hygric performance of the buildings. Building stock models for hygrothermal simulations require many of the same inputs as energy simulations. Age-structure archetypes can be used to estimate the wall constructions for internal and external walls, and the surface area of the floors and walls exposed to flooding and drying scenarios. The age of the dwelling can also be used to infer the air change rates, which can be used to model the internal drying of unventilated spaces buildings. However, hygrothermal simulations require additional information about the materials within the envelope that are not contained in the harmonised standard hygrothermal design values used for energy simulations [29]. Parameters such as porosity and detailed water absorption and desorption functions up to material saturation are important for HAM simulations. Harmonised design values for the hygrothermal properties of building materials in energy studies has previously been determined by statistically analysing existing material data [30].

Geospatial databases have been developed in previous studies for energy simulation data at levels ranging from the individual building to administrative area. Geographic Information Systems (GIS) enable the storing of building information in a spatial database, the mapping and dissemination of the building stock data, and can act as a platform for energy simulations. Energy simulation algorithms require a large amount of input data, which is costly and time consuming to collect via house by house surveys, so GIS can also be used to infer building characteristics using remotely sensed data such as aerial imagery, A range of different sources of geographic data are available that contain building stock data relevant to hygrothermal simulations. These databases can be used to develop a bottom-up building stock model with information that can be used to model the flooding and drying of geographic areas.

3. Methodology

The research area for the present study was selected to be an area of London extending from Richmond in the west to Greenwich in the east – an area covered by approximately 250,000 homes, and at risk of flooding. The main GIS database used was Ordnance Survey Mastermap [31], a continuously updated cadastral map, that has a Topographic Layer with individual building footprints and crude land use information. As it is the most recently updated topographic map available, the building footprints were used as a basis for the model. OS Address Point data was used to determine the number of dwellings in each residential building [32] based on matching Topographic Identifiers (TOID). Cities Revealed [33] produces a landuse database with building footprints classified as one of 15 building types and 17 age categories. The building classification can be used to fit suitable building archetypes to individual properties within the GIS system. The Cities Revealed building classifications were joined to the OS Mastermap data through a spatial join (Figure 1).

Figure 1. *Ordnance Survey Mastermap with Cities Revealed building age characteristics*

Building height information is required to determine the depth of any flood water. Total height was derived for individual properties from a LiDAR (Light Detection and Ranging) -generated Digital Surface Model (DSM) raster provided by Cities Revealed. The ground level heights of the dwellings were determined by filtering the DSM to create a Digital Terrain Model (DTM).
Additional relevant spatial building data is required to match archetypes to building footprint data. The HEED database [34] is a collection of information from energy suppliers, government scheme managers, local landlords, Energy Saving Trust (EST) energy checks, and EST programmes on wall types, cavity wall insulation levels, and building ages. This information is valuable because it provides information on the frequency of insulation in aggregated areas down to census output area detail, or around 125 households. In the present study, the HEED data on insulation and wall type was used to supplement the topographic data.
While the focus of this paper is on building characteristics, many other factors are vital to hygrothermal flood modelling. The depth and duration of the flood, the water temperature and salt concentrations, and the drying conditions, such as external weather and internal conditions will impact building behaviour. Understanding the impact of a flood on the building stock requires information on the geographic extent and height of any floods. The Environment Agency's (EA) flood risk map [35] shows areas with risks of 1 in 100 years for river flooding, 1 in 200 years for sea flooding, and a 1 in 1000 years for extreme sea or river flood events (Figure 2). In this study, the EA flood risk map was used to determine the height above ground that individual properties would be flooded under different scenarios by comparing the flood heights to the Digital Terrain Model (DTM) (Figure 3).

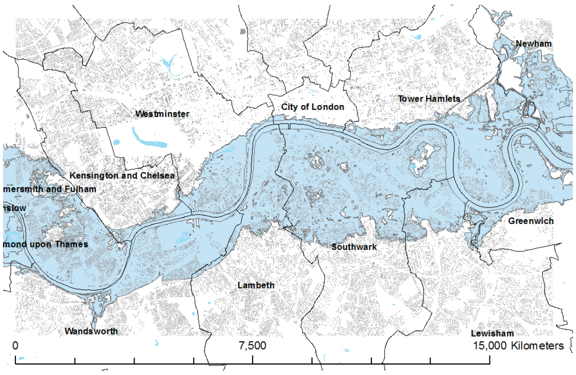

Figure 2. *Environment Agency tidal and fluvial flood map of the research area.*

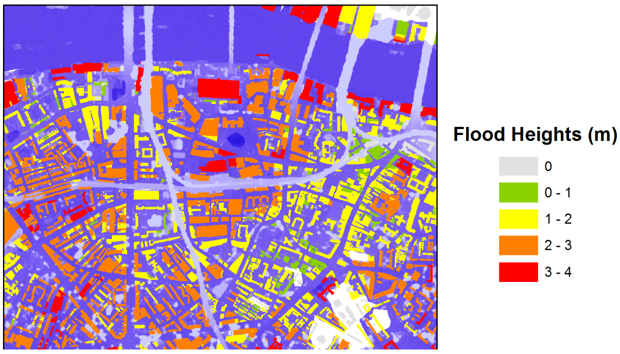

Figure 3. *Flood heights assigned to individual properties in the flood zone for a 1 in 1000 year event.*

Different flood durations and heights will be simulated in hygrothermal models for walls and floors in 2D (WUFI, Delphin) and for whole-buildings (EnergyPlus) for each building archetype under different environmental conditions. These models will simulate the movement of water into the building envelope, and the subsequent drying of the building under a range of internal and current and future external weather conditions. 2D models will be used to compare simulated data against previous laboratory and field measurements of flooded walls and floors, while whole-building simulation will enable the prediction of water vapour movement throughout the building zones causing damp in areas that were not physically contacted by flood water. EnergyPlus will also be used to calculate ventilation rates within the buildings under different scenarios (e.g. windows open, dehumidifier), as well as within any wall cavities, in order to help determine the most effective means of drying. The temperature and relative humidity at surfaces within the envelope where microbial growth can occur and impact indoor air, such as the interior surface and inside the cavity, will be determined.

4. Results

Presented here is the novel methodology that has been developed in the first phase of the work (**Error! Reference source not found.Error! Reference source not found.**). The proposed model outlines the necessary inputs required to develop and implement a hygrothermal flood simulation model. Later phases will implement the methodology, and the outputs from the integrated simulations will be presented in future papers. Microbiological models are not detailed in this methodology, but will be combined with the model outputs at a later stage.

There are many uncertainties associated with such complex simulations and work is necessary to understand how the assumptions made about the built form and material parameters impact the model. For example, the hygrothermal material properties are assumed to be homogenous, while in reality materials can vary significantly. Consequently, further work is required to understand how variations in the input parameters in Figure 4 can impact drying characteristics. A sensitivity analysis in the next phase will help to identify the impact of such uncertainties.

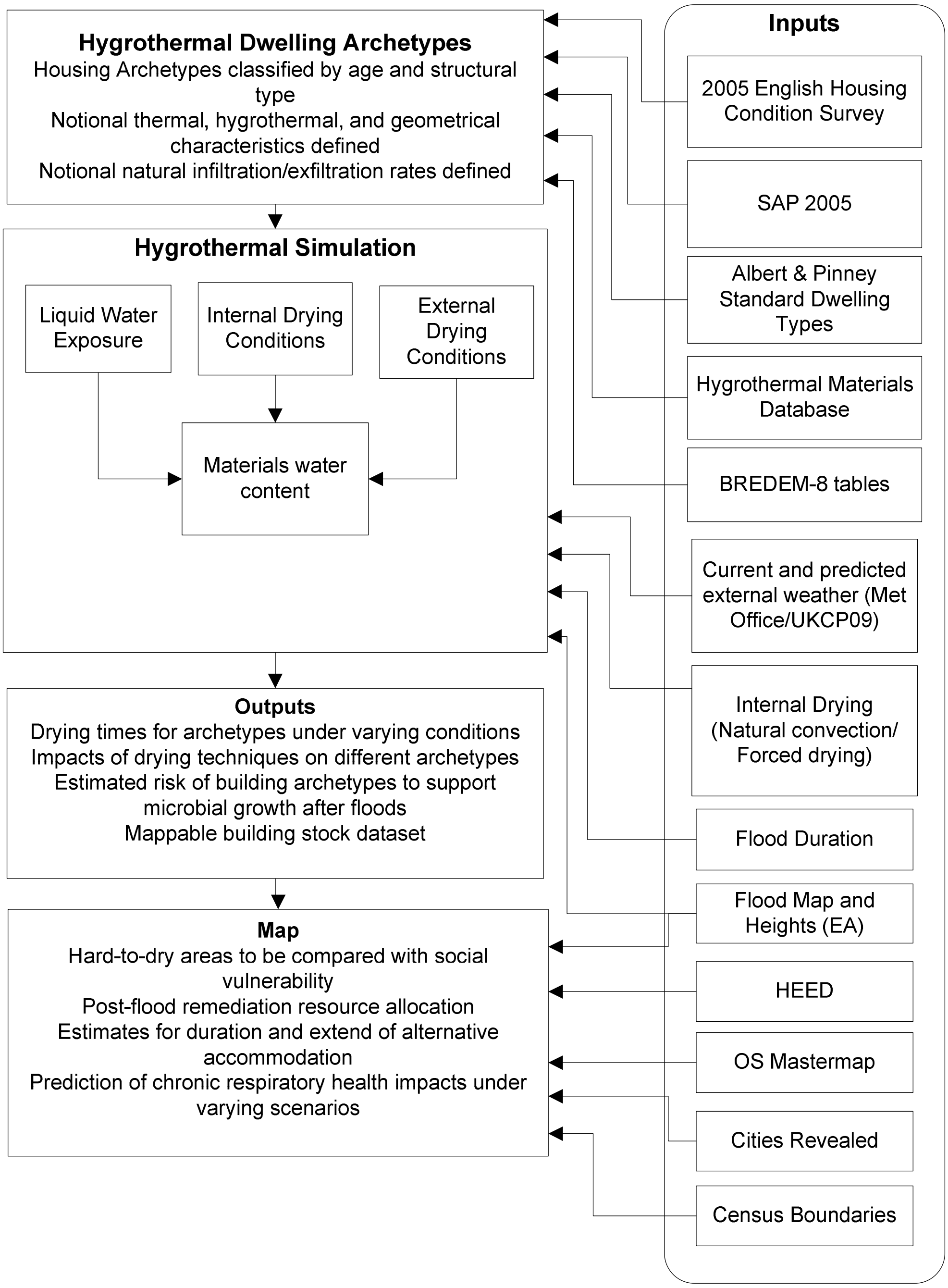

Figure 4. *Schematic of the hygrothermal flood modelling process*

5. Conclusion

Building archetypes can inform the hygrothermal simulation engine in order to understand the drying of the building stock under a range of different flooding scenarios and drying scenarios. As the model is composed of notional building types which are intended to represent average buildings within each archetype, the model will not be able to calculate the actual hygrothermal performance of *individual* buildings. However, the great value of the work is that the building stock model can be used in order to determine the relative risk of post-flood damp at small and intermediate areas.

Application of biohygrothermal models can then help identify how long buildings will be at risk of mould growth following a flood based on the material substrates and the temperature and relative humidity values output by the HAM simulation. Adapting these models to show persistence, or declination of mould, and bacterial behaviour will further enhance the power of the model.

The objective of this phase of the work was to develop a novel methodology for performing hygrothermal simulations of flooding scenarios for the London building stock. The outputs from this research will inform the drying times of different buildings and localities, and their potential for lingering damp and contamination, which can be used alongside analyses of social vulnerability to flooding of the population. The results will be of great interest to flood remediation organisations, health agencies, insurance organisations, and social housing providers.

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SUSTAINABLE, SMART, SAFE - A 3'S' APPROACH TOWARDS A MODERN TRANSPORTATION SYSTEM

Md. Mazharul Haque[1], Ashim Kumar Debnath[2], Hoong Chor Chin[3]

[1]Research Fellow ; Email: *cvemh@nus.edu.sg*
[2]Research Fellow ; Email: *cveakd@nus.edu.sg*
[3]Associate Professor : Email: *cvechc@nus.edu.sg*

[1]Corresponding Author, Telephone: +65-66011165
[123] Dept. of Civil Engineering, National University of Singapore, Singapore-117576

Abstract: Sustainability, smartness and safety are three sole components of a modern transportation system. The objective of this study is to introduce a modern transportation system in the light of a 3'S' approach: sustainable, smart and safe. In particular this paper studies the transportation system of Singapore to address how this system is progressing in this three-pronged approach towards a modern transportation system. While sustainability targets environmental justice and social equity without compromising economical efficiency, smartness incorporates qualities like automated sensing, processing and decision making, and action-taking into the transportation system. Since a system cannot be viable without being safe, the safety of the modern transportation system aims minimizing crash risks of all users including motorists, motorcyclists, pedestrians, and bicyclists. Various policy implications and technology applications inside the transportation system of Singapore are discussed to illustrate a modern transportation system within the framework of the 3'S' model.

Key Words: *Sustainability, Smart, Safety, Transportation System, Singapore*

1. Introduction

Now-a-days the development of a sustainable transportation system is a global issue mainly because of an increasing concern on environment issues. At the same time, recent technological revolutions made smart or intelligent technologies readily available to be incorporated inside the transportation system to make it more efficient to users while maintaining and/or improving sustainability. In addition to sustainability and smartness, safety should not be ignored to have the system feasible. Hence a modern transportation system should promote social and economic welfare in a safe and efficient way without damaging the environment or depleting environmental resources. The concept of this novel approach along with major components is shown in Figure 1.

A sustainable transport system has three major targets: 1) economic development, 2) environmental issues, and 3) social equity. In light of this, it has been defined [1] as one that firstly, allows for the safe and environmentally harmless basic means of access and development on the individual, business and societal level, while promoting equity within and between generations; secondly, is reasonably priced and runs efficiently, providing choice of transport mode as well as support for a competitive economy and good regional development; thirdly, keeps production of emissions and waste within the carrying capacity of the natural environment and keeps the consumption of renewable resources and non-renewable resources respectively within the rates of generation and development of renewable substitutes, while minimizing the impact on the use of land as well as production of noise.

Recent advancements in technological developments have made urban transportation systems more efficient through the use of smart technologies. A smart technology is defined as a self-operative and corrective system that requires little or no human intervention. It has three basic elements – sensors, command and control unit, and actuators, contributing to three basic capabilities - sensing, processing and decision making, and acting [2]. Having sensing ability, a smart technology able to process and interpret the received information and capable to execute decisions into actions through actuators. These activities follow a cyclical pattern and make a smart technology forming a closed-loop monitoring and action-taking process.

In the transport system, safety is a significant component that ensures viability. Road safety is an important public health issue and traffic incidents are not only costly for the injured, but a burden on society and national health systems. Since a transportation system is dynamic and has a human component, incidents are an inevitable consequence. Hence a transport system targets to minimize the injuries and casualties of its users, promotes safe road usages through the development of proper infrastructures and environment, and encourages safe road behaviours through education and legislations.

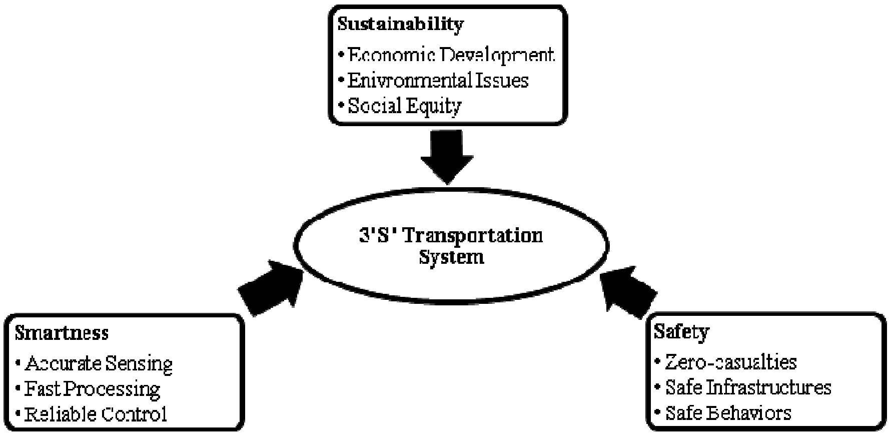

Figure 1: *Framework of a 3'S' Transportation System*

In light of the above, it appears that sustainability, smartness, and safety are three important components (3'S') of a modern transportation system. The objective of this study is to illustrate how the transportation system of Singapore is progress towards this three pronged approach. In subsequent sections, this paper discusses important steps and initiatives of the transportation system of Singapore to achieve sustainability, smartness, and safety. Lastly, the paper discusses some concerns and future aspects in development of a modern transportation system.

2. Sustainable Transport Planning in Singapore

Over the years, various strategies and policies have been implemented into the transport system of Singapore on its path towards sustainable development. The sustainability issues from the experience of land transport development of Singapore can be broadly categorized into four key areas: 1) integration in land use and transport planning, 2) transport supply measures, 3) transport demand management, and 4) incorporation of environmentally-friendly technologies for vehicles.

2.1 Integration in land use and transport planning

Singapore's small land area has consistently been a major constraint on land use and transport planning and hence an optimal balance and integration among those were required. The first strategic development plan in 1971 decentralized population by developing residential blocks further away from CBD and connected by roads, expressways, and MRT (Mass Rapid Transit) lines. Later, the revised concept plan of 1991 [3] aimed to further decentralize commercial and economic activities by developing regional and sub-regional centres around MRT stations. The locating of employment centres, industrial estates, business parks, and commercial centres near residential areas reduced the people's need for travel, while resulting in a better utilization of the MRT network. For a better integration, a hierarchical system with well-defined roles for each transport mode was also designed. While railways serve the long-haul travel, LRT (Light Rapid Transit) and buses provide services feeder services to connect areas in housing states to MRT stations. Those strategic plans not only restricted the development of urban sprawl but also reduced the number and length of trips of commuters and hence were the key steps towards sustainability.

2.2 Transport supply measures

The increase in population and economic activities will increase the daily travel demand from the current 8.9 million to about 14.3 million journeys by the year 2020. To meet the mobility needs of people in a sustainable way, transport authorities are planning to double the public transport trips (about 10 million) by 2020. Therefore, recent transport supply measures on land transport master plan *(4)* mainly focus on initiatives promoting public transports including rapid transit system, buses and taxis.

Rapid Transit System (RTS): RTS network in Singapore consists of MRT and LRT. The current 148 km long RTS network consists of four MRT lines (68 stations), three LRT lines (33 stations) that accommodate a total about 1.7 million passenger trips daily [5]. Ibrahim [6] has reported that more than 60% of Singaporeans use the MRT for commuting and other trips. To accommodate the projected demand, the MRT network has been planned to be doubled to 278 km by 2020 including two new lines and extensions on existing lines that will lead to a total 36 new stations.

Buses: Singapore's two bus operators currently operate a fleet of 3,268 buses on about 344 bus routes accommodating an average daily ridership of about 3.09 million [5]. Recent policies to improve and promote bus services include peak hour bus lanes, full day bus lanes, priority at signalized junctions, and mandatory give way to exiting buses from bus bays. A 120 km of peak hour bus lanes and 7.6 km of full day bus lanes has improved the bus speed by about 16%. In addition to normal bus services, many special services have been introduced over the years to cater diverse needs of people especially who prefer a more comfortable and luxurious level of service. Those include premium bus services, express bus services, intra town services, night bus services, and niche public bus services.

Integrated rail and bus services: To make journeys more seamless and convenient for commuters, there has been a deliberate move towards integrating rail and bus services through coordinating network, physical facilities, fares, and information. Bus stations are strategically located near MRT stations and connected with well designed walkways for the convenience of commuters. A common ticketing system in the form of a universal fare card (“EZ-link”) was developed for use on both trains and buses. To further improve transfers by removing the current fare penalty, a distance-based through fare structure has been implemented recently. The integrated information system for public transport will be discussed in section 3.3.

Taxis: Taxis bridge the gap between basic public transport modes and private cars, by offering commuters the choice to have personalised high-end door-to-door service. There are currently 7 private taxi companies with a total fleet of 24,300 taxis catering for about 0.8 million trips daily. While fares and supply are maintained by companies, quality standards are regulated and enforced by the Land Transport Authority (LTA).

Other facilities to improve the usage of public transport: The LTA maintains well designed footpaths, sheltered link ways, overpasses and underpasses to provide pedestrians a comfortable and conducive walking environment between transport nodes and residential, commercial and institutional buildings, as well as serving as connections between the various transport modes themselves. Singapore has currently 40 major public transport nodes with park and ride sites (about 5000 parking lots) where motorists can park their vehicles and take public transport to travel to the CBD. Similarly most of the MRT stations and bus interchanges have bicycle parking facilities that encourage commuters to cycle from the housing estates to public transport nodes. MRT and buses now also allow foldable cycles on board to further promote this environment-friendly mode.

Meeting diverse needs of people: Equity is a key component of a sustainable transport system which must be ensured by providing transportation needs for various user groups including elderly and physically challenged people. For this group of people, bus stops and MRT stations as well as buses and trains are being redesigned to have wheelchair accesses. It has been targeted that 100% public buses will have wheelchair accesses by 2020. Recent initiative to improve the walkways for elderly and physically challenged people include installation of tactile guidelines, at-grade pedestrian crossings, thickened road crossing lines for vision impaired, and audio-alert crossing facilities for hearing impaired.

2.2 Transport demand management

Enhancing transport supply alone is not sufficient to maintain a smooth flow of traffic, especially since continuous road addition is unfeasible in land-scarce Singapore. Transport demand management policies also play a vital role in achieving sustainability that has mainly been done by controlling growth of motorization and imposing road pricing policies.

Controlling growth of motorization: Singapore maintains a sustainable rate of growth (about 3% p.a.) of its vehicle population by vehicle quota system (VQS) policy since 1990. The VQS works by determining a suitable number of new vehicles allowed for registration annually and subsequently letting market forces determine the price of ownership via bidding. Recently the vehicle growth rate is set to 1.5% p.a. to ensure long term sustainability [4]. In light of high costs of cars due to the VQS, the off-peak car scheme (OPC) allows people to buy cars at a cheaper rate but restrict the usage of cars from 7:00 am to 7:00 pm on weekdays. OPCs currently make up about 8% of the total car population.

Imposing road pricing: In addition to purchase-based constraints, road pricing is a usage-based tax system discouraging use of expressways and main roads towards CBD during peak hours to prevent congestion. Congestion prices are collected by an electronic road pricing (ERP) system that functions with gantries on roads and in-vehicle units at vehicles and capable to collect chargers at operating speed. Through regular reviews and rate adjustments, traffic on priced roads is relatively smooth-flowing as such charges divert traffic from busy roads. ERP has been effective in maintaining an optimal speed range of 45 to 65 km/h for expressways and 20 to 30 km/h for arterial roads. The technology for road pricing will soon be updated ("ERP II") with the incorporation of Global Positioning System (GPS) technology that enables distance-based congestion charging without the need for physical gantries. It will be a more flexible and efficient method of managing congestion, and thus more effective and sustainable in the long run.

2.4 Incorporation of environment-friendly technologies and policies

To address environmental sustainability, green technologies and policies are also incorporated to promote green vehicles. This is done by raising emission standards for vehicles and encouraging the use of cleaner fuels and more energy efficient vehicles.

Through strict emission regulations, Singapore's ambient concentration of most air pollutants has stayed within international standards except for particulate matters smaller than 2.5 microns in size (PM2.5). To restrict those particles, more stringent emission standards (Euro IV) have been planned to be implemented for all vehicles by 2020 leading about 70% less PM2.5 than current time [4]. Another policy to control emissions is that vehicles older than 10 years are subjected to a surcharge tax on their annual road taxes.

To encourage green vehicles, Singapore has introduced green vehicle rebate scheme which offers an offset on their registration fee of about 40% and 10% of open market values respectively for cars and motorcycles. Both bus and taxi companies are now renewing their fleet with more environment friendly and greener vehicles. The number of green vehicles in total has increased from a mere 713 in 2006 to 4582 in 2009, of which 30 are buses and 1859 are taxis [7]. Currently one Taxi Company is operating 1000 taxis on CNG and intends to bring another 3000 within next two years.

3. Smart Technologies in Singapore

Smart technologies in Singapore's transport system can be broadly categorized into four divisions according to their primary functions: Control systems, Monitoring and enforcement systems, Information management systems, and Revenue management systems. Note that the classification has been done in city-level. The user-level technologies which do not require an infrastructure, such as the smart vehicle control systems, collision avoidance systems, and the driver safety monitoring systems are left out.

3.1 Control Systems

Smart control systems manage traffic flow efficiently using automated traffic signals. An adaptive traffic signal system (i.e., GLIDE) continuously collects traffic information (through detection of vehicles and pedestrians) and automatically allocates signal timing at intersections, while allowing signal coordination along a corridor. Some intersections are also equipped with a transit signal priority scheme (i.e. B-signal) which detects approaching buses and facilitates their movements by extending green time as well as turning on the B-signal for an earlier start. Detection technology of some traffic signals enables the recognition of elderly pedestrians via the tapping of senior citizen concession cards, and increases their green time. Some are also equipped with countdown timers and audio signals to aid the disabled. Another recent improvement is the installation of intelligent road studs at 17 major intersections [8] which alert motorists to the presence of pedestrians crossing by blinking on the pavement.

3.2 Monitoring and Enforcement Systems

Technological advancements allow transport authorities to continuously monitor transport facilities, sense disturbances in traffic flow and identify traffic violations. Singapore has speed cameras at 45 locations and red light cameras at most major intersections. They automatically detect respectively speeding and red light running vehicles, and take snapshots of the registration plates for identification. For smooth operation and safety, expressways are equipped with Expressway Monitoring and Advisory System (EMAS), which is a smart incident management system. It automatically detects incidents and congestion, allowing authorities to take quick action (e.g. dispatching recovery vehicles and disseminating congestion information). The service is planned to extend to 10 major arterials by 2013 [8]. Outside of expressways, about 280 advanced surveillance cameras (J-Eyes) operate at major signalized intersections [8] to help traffic control centre operators detect irregular traffic situations, including congestion, illegal parking and loading/unloading. To regulate Bus lane and Mandatory give-way to buses policies [4], LTA utilizes a smart bus lane enforcement system, whereby cameras are installed on-board the buses to detect violations of the bus lanes. A total of 90 buses of 12 routes were fitted with the cameras in 2008 [8].

3.3 Information Management Systems

Information management systems collect, process, and share real-time traffic information with travellers via fixed and mobile platforms, so that they can plan (and modify) their routes in advance. Smart information management systems can be categorized into two: traffic news broadcasting and public transport information sharing.

Traffic News Broadcasting: The smart system TrafficScan uses data from taxis that equipped with GPS to provide average travel time along roads, while incidents and congestion information from EMAS and J-Eyes is provided to travellers through LTA portals, in-vehicle devices (e.g., radio) and variable message signs. To further improve, a smart traffic prediction tool is being developed for better traffic advisory information that will use advanced statistical techniques for predictions. Other than those, the smart parking guidance system in CBD collects and processes information in a central computer and displays real-time information on available parking spaces through 24 electronic panels [8].

Public Transportation Information Sharing: The four attractive features of this system are: an integrated public transport map, a public transport travel advisor, on-board information services, and an advance taxi booking system. The online integrated map provides travellers with transit alternatives (rail and bus) and a web-based multi-modal journey planner determining the best routes for travellers. The public transport travel advisor facilitates the convenience of travel by mainly providing arrival timings. Bus arrival timings are available through internet, SMS (currently 215 stops), and at bus stops equipped with electronic display panels (currently 76 bus stops). For MRT, arrival timings of trains are provided in the proximity of MRT stations and platforms. On-board real-time information on next stops and routes are also provided on trains and buses. For taxis, service providers use a smart taxi booking system, where passengers can book using the internet, SMS, or phone. Contact centres wirelessly connect taxis using General Packet Radio Service technology and in-vehicle mobile data terminals to process the booking requests. To further improve the service, recently a common telephone number for all taxi services has been introduced [9].

Apart from the above initiatives, Singapore is anticipating the use of next-generation electronic road pricing system (ERP II) with GPS technology which will also be helpful for the enhanced collection and dissemination of traffic information [4]. That information will help to make a better journey plans, enable dynamic fleet management of logistics and taxi companies, give priority to emergency vehicles, and relieve congestion.

3.4 Revenue Management Systems

Managing fast and accurate transactions for public transport fare and toll payments is important for a transport system to be efficient. Singapore has been utilizing smart technologies for better management of revenue systems, such as public transportation fare payment, parking charge payment, and toll collection. The contactless tap-and-go fare card for public transport (i.e. EZ-link card) allows for paying of fares in all transport modes including MRT, LRT and buses. The recent upgrade of that smart card - the Symphony for e-Payment (SeP) - now includes payment for other usages like ERP, parking, and many retail outlets. As discussed in the previous section, the ERP is also a smart technology that automatically collects tolls via gantries during real-time traffic operations. The next generation of this system (ERP II) will remove physical gantries and implement a distance-based congestion charging via GPS technology.

4. Road Safety Initiatives

The road traffic crash rate per 100,000 registered vehicles in 2008 was 31.1 and the fatality rate per million population of Singapore was 45.7. The contribution of cars, motorcycles, pedal cycles and goods & other vehicles to total road traffic crashes were respectively 43.6%, 33.0%, 4.1%, and 19.3% [10]. While traffic crashes are low by international standards, road safety remains a concern in the efficiency-conscious nation. Singapore has included world class traffic safety legislations, regulating and monitoring systems into the transport network.

Despite of mandatory helmet laws and day time headlight laws, motorcyclists are the most vulnerable user groups accounting for about half of road fatalities for many years. To enhance safety of motorcyclists, recent initiatives include paving high-skid resistant materials at crash-prone sites, installing better vehicular impact guardrails appropriate for motorcyclists, and providing more rain shelters to encourage motorcyclists not to ride in the rain [4].

Pedestrians are another vulnerable road user groups accounting for about 28% of road traffic deaths. Recent engineering solutions to promote pedestrian safety include the installation of intelligent road studs at pedestrian crossings to warn motorists; personal electronic devices for elderly pedestrians to allow more crossing time; and advance road markings, real time speed advisory signs, and traffic calming markings to reduce the speed of vehicles at pedestrian crash-prone areas.

The Road Safety Engineering Unit of LTA is responsible to ensure good and sound road engineering practices, enhance road safety, and work with other agencies involved in road safety. Recent initiatives of this unit include identification and improvement of black spot locations, 'Enhanced School Zone' design to improve traffic safety around schools, installation of crash cushions at high-risk locations to reduce injury severity, installation of real-time speed display signs, erecting concrete bollards at selected bus stops to protect waiting commuters from runaway vehicles, and installation of Platform Screen Doors at MRT stations above the ground to promote safety of commuters [4, 8].

While LTA looks into road safety through various engineering solutions, the Traffic Police is responsible for enforcing traffic laws and regulations on roads and promoting road safety by influencing behaviour and skills of road users. Besides enforcements, various education and safety campaigns have also been organized by the Traffic Police. Every year, the Singapore Traffic Police in collaboration with other agencies develop a myriad of public education outreach programmes primarily targeted at vulnerable road users. Some of these include safety education of primary school children couple with traffic games at Road Safety Park, ride safe programme, the road courtesy campaign, the anti-drink drive campaign and other educational programmes to encourage the proper use of child seats. Trade associations, non-government organizations, and various private companies also play a vital role in organizing safety campaign and awareness programmes [11].

5. Discussions

Achieving sustainability is very challenging for a transportation system. Singapore experience shows that promoting public transport may be best way to achieve sustainability. However it is still very challenging to accommodate future travel demand within the public transport. Although the average maximum passenger loading on the trains is low (3.7 persons per square metre) by international standards, MRT are very crowded during the peak hours. Authorities should look forward to improve this condition. For buses, in addition to the crowd, boarding on buses still takes a lot of time due to on-board tapping of smart cards. Implementation of off-board fare collection system may be a suitable alternative to decrease this time wastage.

Other than public transport, non-motorised forms of transport such as cycling should also be promoted. A caveat for cycling is the limited land area that does not allow building cycling tracks over the island. However, an increasing amount of attention is being directed to it as it is recognized to be a green form of commute that promotes an active lifestyle too. More research can be done in this area to find innovative ways to better incorporate cycling into the transport system, including using it as a link to major public transport nodes. Another non-motorised form of commute is walking, and currently, more seamless ways of connecting pedestrians to transport nodes are being developed. Since Singapore has a tropical rainforest climate condition, walkways should be fitted with shelters to promote walking and facilitate usage of public transports during adverse weather condition.

It has been observed that restriction on vehicle ownership and imposing road pricing are some viable schemes to promote sustainability. However complete restriction on private transports can never be achieved. Hence promoting green vehicles through various policies and government incentives may encourage more motorists towards sustainability. Moreover bus and taxi companies should put more efforts to promote environment-friendly vehicles in their fleet.

With regard to safety, road traffic safety is still remaining a concern. A scientific estimate [12] put the total cost of road traffic crashes in Singapore at about S$610.3 million for the year 2003 which was about 0.3% of the annual GDP. In particular, motorcyclists and elderly pedestrians should be targeted for safety improvements. More innovative researches should be conducted to investigate the root causes and countermeasures for those problems. Overall, to further raise the road safety standards of Singapore, there has to be a change in mind-set for all road users such that individuals understand and undertake their own social responsibility toward safety, and begin to act more safely on roads. To achieve a higher level of safety awareness, authorities should be more conscientious in promoting and enforcing road safety. Moreover there must be greater coordination and dialogue among the different road safety stakeholders.

In general smart technologies facilitate implementation of different policies promoting a sustainable and safe transport system. For example, to promote public transport as a viable alternative to private transport, many smart technologies have been implemented such as the bus priority signal system, bus lane enforcement system, availability of real-time service information and an integrated multi-modal fare payment technology. Availability of traffic and travel related information also has the potential for enhancing motorists' flexibility in route planning so as to ensure a less congested, faster and safer trip. To enhance safety of motorists and pedestrians at intersections, a set of smart traffic control systems is also used which increases efficiency of road networks and environmental health. The electronic toll payment system is another smart technology which has been successfully implemented to facilitate the road pricing policy for managing congestion and hence promoting sustainability. The development and inclusion of newer and smarter technologies with enhanced abilities will make the transportation network more efficient and solve many transport problems.

6. Conclusion

A 3'S' approach is highly desirable for a modern transportation system to be viable and efficient. Sustainability, smartness, and safety are three major components to enhance mobility and accessibility, ensure safety and social equity, improve system efficiency, protect the environment and importantly foster the economy. As learnt from the Singapore experience, sustainability, smartness and safety are not only the three most critical components of a modern transportation system, but are actually closely related to each other too. To develop a sustainable and safe transport system, smart technologies should also be incorporated to increase efficiency and reliability. The success of Singapore's 3'S' approach for its transport system may serve as a good reference for other cities in creating sustainable, smart, and safe modern transportation system.

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APPLICATION OF ‘GREEN CONCEPT’ TO SEGREGATE PEDESTRIAN AND VEHICULAR LAYERS TO REDUCE URBAN TRAFFIC CONGESTION
A Case of Commercial-Pedestrian Underpass Network in Kandy, Sri Lanka

[1]H.A.D.S Buddika and [2]V.S.K Wickramasinghe

Department of Civil Engineering, University of Peradeniya, Peradeniya, 20400, Sri Lanka
[1]samithbuddika@gmail.com, [2]vskw@pdn.ac.lk

Abstract: This paper presents the direct and indirect benefits of the feasibility studies carried out for a proposed commercial-pedestrian underpass network in Kandy CBD. Field observations and condition assessment process together with layout design of the underpass have been done by linking economic, social, and commercial centres. Services such as ventilation, water and sanitary, lighting, and security were designed following the green concepts. Natural ventilation, novel lighting techniques such as LED was introduced. Safety alarms and emergency exits were designed and well placed within the system. Further, greenery curtain concept and solar powered street lighting system were introduced to the city.

Keywords: *commercial-pedestrian underpass network, green concept, conceptual design*

1. Introduction

Kandy, the hill capital of Sri Lanka, is a compacted historical city where vertical expansions are restricted. City has an aggregated population density of 540 persons per sq.km. Besides, Kandy serves as a commercial hub for over two million people residing in adjacent areas. Though sufficient pedestrian walking facilities (e.g., sidewalks and crosswalks) are provided, many of them are occupied by street venders, leaving hardly any space for pedestrians to walk along the sidewalks. Thus, pedestrians tend to walk on the road creating many vehicle-pedestrian conflicts creating life threats and disturbing the smooth traffic flow. Emission of carbonic gases and wasting of useful human-hours create massive environmental and economic impacts

The central business district (CBD) of the city is seeking of well integrated city planning for its road network (both vehicle and pedestrian) to overcome such barrier to stay still as one of the worlds greatest historical and prestige city. However, the increasing trend of traffic congestion in the commercial area of the city is hard to mitigate with ordinary solutions (with traffic circles and traffic lighting system) as most of the pedestrian walkways have been disturb by street venders (Fig. 1) resulting pedestrians in the vehicular layer to cause traffic problems and accidents (Fig. 2). Further to this, the pollution caused by the emission of gases from vehicles in the traffic jam, loss of lives due to accidents and time wastages are hard to convert into financial terms. The economy of the country is also negatively affected and businesses are the first to suffer when customers find it is difficult to reach to CBD.

Figure 1: *Conflict due to street venders.*

Figure 2: *pedestrian- vehicle interaction*

The impact on the aesthetic appearance should also consider prior to implementation of any modification as it merge with the great history of the city as well as the country. Underpass network is the one of the recommended option to solve the problem associated with Kandy as it act as graded subway for the pedestrians with safe use. Moreover, it also can be used as a commercial and recreational centre for peoples to enhance their quality of lives.

2. Methodology

During the comprehensive literature surveys, following key factors have been identified as important for the safe design of commercial pedestrian underpasses. Namely, pedestrian behaviour (Pedestrian Level of Service), identification of control points and the lay out (Public and commercial places), lighting and ventilation, designing of the entrances and methods use to attract pedestrian and their safety. In the implementation stage of the project each scenario was addressed and solutions were proposed [1].

First, the CBD area that needs to be address during the project was identified (as shown in Fig.03) considering commercial and public locations as well as pedestrian behaviour. It includes Kandy hospital, Good shed, central market, post office, Telecom building, railway station, existing underpass, Bank of Ceylon building, Commercial bank building, Peoples bank building, Kandy commercial city, Cargill's food city, Hatton national bank building together with some other public locations.

Secondly, condition assessment survey was conducted in between predefined nodal points to count the number pedestrians and their moving speed to estimate existing level of services (Fig.04). Other than the pedestrian counting, measurement of existing pedestrian walkway dimensions was taken and used to calculate the existing level of service. Moreover, pedestrians and street venders were interviewed on site and their feedback was recorded.

Thirdly, relevant government authorities were consulted and the details of existing service lines such as water supply, sewer, electrical and communication lines were collected for the feasibility studies.

As the next step, desk studies were carried out to select possible alternatives by considering both pedestrians' movement and technical feasibility .Then the best alternative was selected to cater for the intended purpose.

Finally, proposal was made on the lay out of the underpass to locate shops and other basic facilities and services (electricity, water and lighting) to serve the pedestrians and attract them in to the commercial pedestrian underpass. Parallel to those modifications greenery concept was introduced to Kandy city to preserve its' historical value.

3. Observations and results

First of all, existing pedestrian lane widths were measured and effective lane width was identified due to street venders. In most of the locations, the presences of street venders disturb the flow of pedestrian movement. Apart from that, poor quality pedestrians' disciplines were identified at certain locations and cause of action for such incidence was identified as lack of road signs and inadequacy pavement width. As a result, most of the people tend to use the rail road as a pedestrian path though it is illegal.

Other than that, pedestrian vehicle accidents are the worst that can be happened due to bad attitude of both pedestrians and drivers. Data was collected regarding resent accidents and used for calculations. Several street venders were interviewed during the site visits and feedback was taken on the existing under pass and the proposed alternative. Majority of venders showed positive respond towards the proposed commercial pedestrian underpass because they will get permanent places to do their business as they are now in temporary shops made with timber. They also conclude that, the disturbance due to bad weather conditions will be completely vanished with the proposed underpass. From the pedestrian point of view, many of them accepted the proposal in an informal manner just because they wanted to have greenery Kandy city for their future generation.

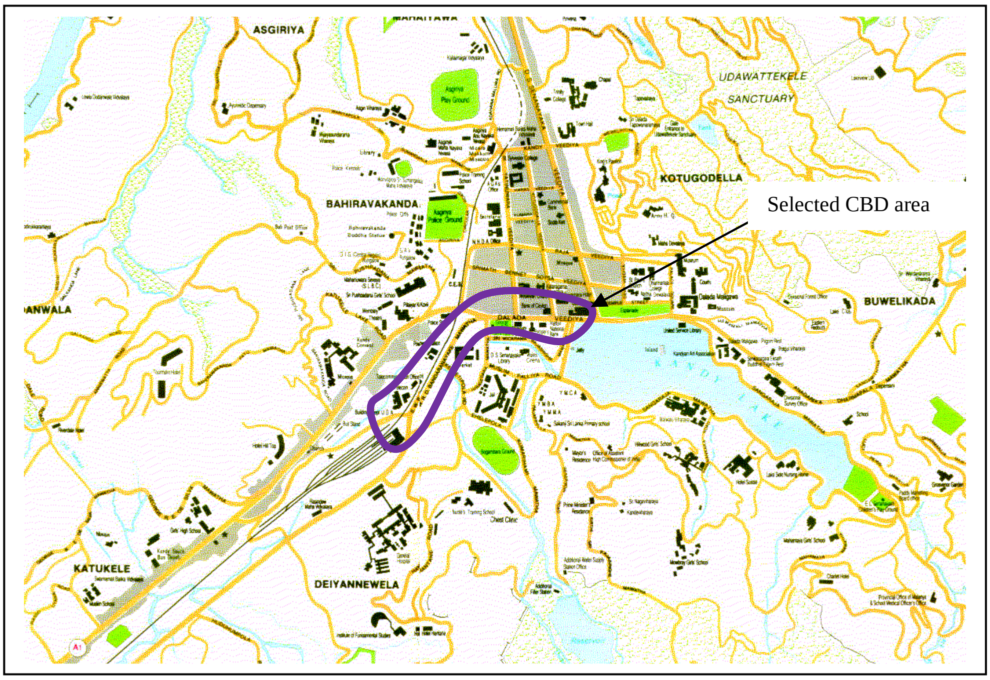

Figure 3: *Kandy Central Business District*

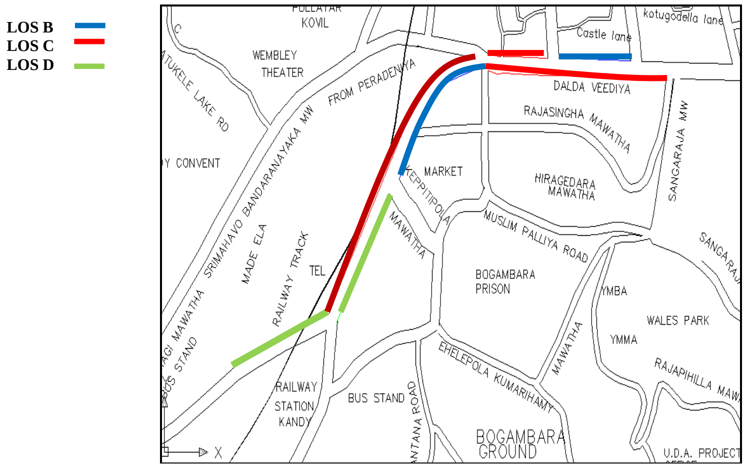

Figure 4: *Existing Level of services*

4. Conceptual design

During the conceptual design, priority was given to relocation of street venders in a comfortable manner in order to do their business parallel to the greenery Kandy city concept. The direct and indirect benefits of the changes can be highlighted as bellows [1, 2].

- Both pedestrian-vehicle accidents and conflicts are avoided
- Fuel consumption of vehicles is reduced
- Environmental pollution can be significantly controlled
- Time savings due to traffic can be improved
- Aesthetic appearance of the city can be improved

Selecting the locations and size of the entrances and exits, alignment of underpass were carried out by considering both space availability and existing service layout plans found during the condition survey. The dimensions of the underpass and other facilities including lighting, ventilation and sanitary facilities were designed according to standard guide lines. Modern available technologies were introduced to the underpass towards sustainable design [7].

4.1Dimension of the underpass

Pedestrian level of service was considered as the key design criteria in deciding the dimension of the underpass and its, shops and other facilities. Selections of the dimensions were performed according to standard guideline found during the literature survey. Conceptual design of the entrance was done and separate lane was provided for disabilities [10].

4.2 Lighting of the underpass

Natural lightning is almost impossible and it is proposed to have completely artificial lighting system with novel LED lighting. The intensity of the lighting was arranged as shown in the Fig.05. The intensity of lighting was maintained closer to the natural sunlight at the entrance and exit locations.

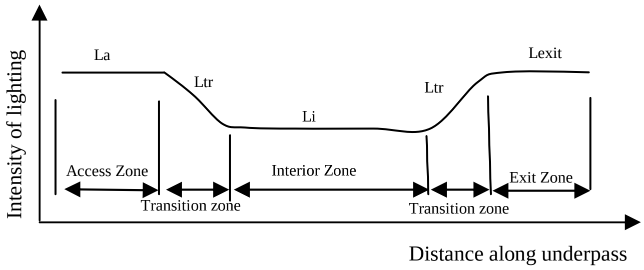

Figure 5: ***variation of the intensity of lighting***

Figure 6: ***Proposed LED lighting***

The LED lighting is a new energy saving product (Fig. 6) that utilizes high power LED's as a light source and it can be connected to direct power supply or can be powered with the optional power source. Other than that, there is no ultraviolet light, no infrared rays, no heat and no radiation products. As a result led tunnel lighting is a convenient "green' lighting source [6]. Moreover, emergency lighting systems with solar panels were proposed and reliability of lighting was guaranteed in a failure of supply from national grid.

4.3 Ventilation

It is proposed to have a natural ventilation system for the underpass except in between node 2 and 5 (See Annex "A") where there is no intermediate open area for free air flow. Calculation of the fan capacity was carried out and minimum air velocity through the underpass was considered as 1.25ms-^{1} and maximum 1.50 ms^{-1} [5].

Table 01: *Calculation of fan capacity*

Air velocity /(ms^{-1})	Existing area with entrance and exits/ (m^2)	Required additional Inlet area/(m^2)	Total fan Capacity/(m^3/s)
1.25	22.30	13.80	28.32
1.50	22.30	16.50	33.88

4.4 Water supply and waste water system

Supply of water was planned to extract from public water supply located closely to the underpass system as identified during the survey. The soakage of waste water is not always possible and it is proposed to dispose it to the proposed sewer system by Kandy Municipal council (Annex "B").

4.5 Safety within the underpass

The safety within the underpass was considered as a one of the key design criteria since it plays a vast role to attract pedestrians into the underpass. The design for safety was carried out to flavour a range of peoples with different disabilities. Key areas were identified and addressed in the conceptual design [8, 9].Namely, fire safety, safety at the entrance and exit, operating time for the underpass, lighting and ventilation within the underpass, emergency exists.

4.6 Kandy city surface

Greenery curtains were proposed for building structures within the selected CBD as shown in fig. 7. Further, street lighting system with solar panels was proposed and green Kandy city concept was established.

Figure 7: *Green curtain concept*

Figure 8: *Fences with greenery area*

5. Impact on environment

The initial environmental impact assessment was carried out and following `areas were addressed during the project.

- Geomorphology
- Ecology and natural recourses
- Socio economics
- Utility services

It could be concluded that the proposed project has positive yield towards the development of the city. However, the key issues that have to be addressed during the implementation of the project were identified during the IEA process. In particular, traffic caused by the construction of the underpass has to be addressed and close investigation of service ducts should be examined prior to the construction of the project. Other than that, effect of the project on the natural resources and bio diversity should be minimized. Finally, the effects on socio economic factors have to be addressed with careful inspection.

6. Conclusion

This paper summarises the direct and indirect benefits of a conceptual design of a proposed commercial pedestrian underpass system in the Kandy CBD. Factors such as, pedestrians' behaviour, technical and economical feasibility of proposed underpass was discussed with appropriate solutions. Relocation of street vendors in the either side of the sub-way road was given the priority and green garden was proposed to construct on that particular space. It can be concluded that the construction of underpass will directly reduces the vehicular- pedestrian conflicts. Other than that, smooth flow of vehicles can be achieved. This will yield to a low fuel consumption of vehicles. Apparently, wastage of time will be minimizing. Street venders will get a well-projected zone for doing their businesses.

In conclusion, implementing the proposed commercial pedestrian underpass system, the dream of greenery Kandy city concept can be achieved and the same scenario can be implemented for other major cities in Sri Lanka.

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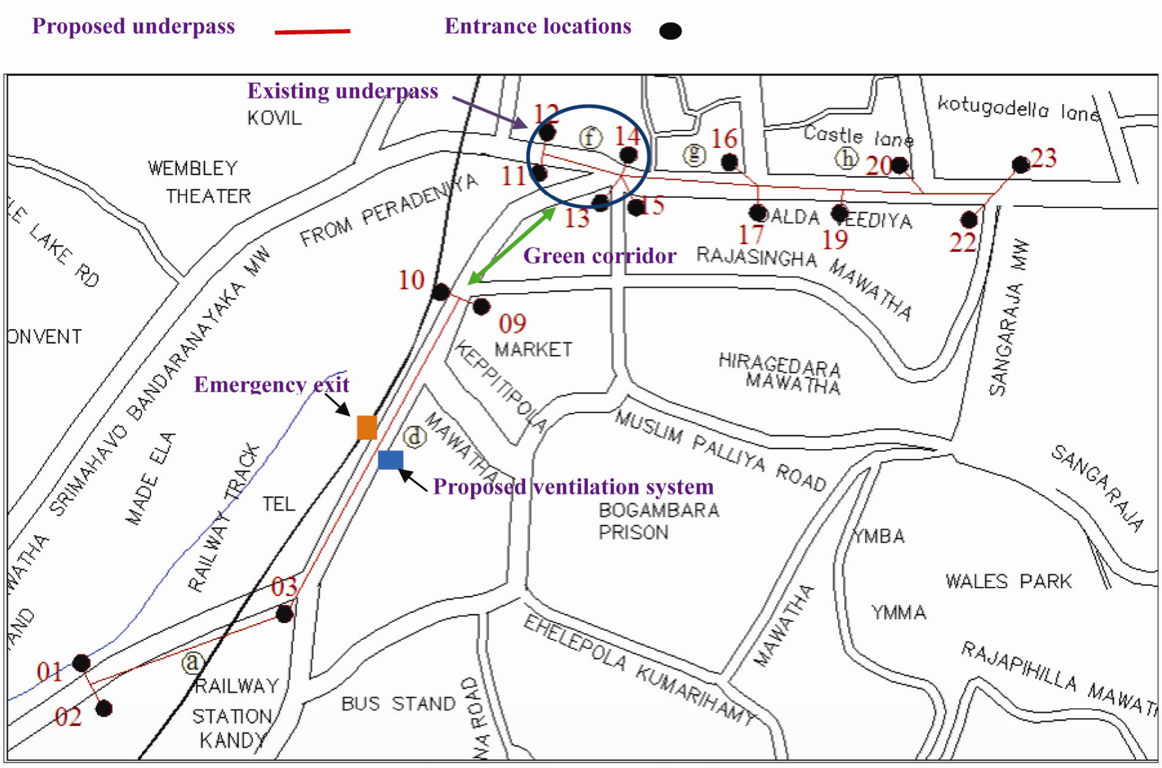
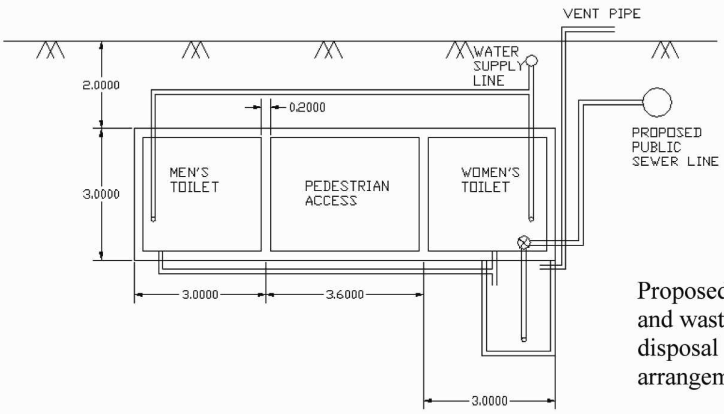
Appendix "A"



Layout of the proposed underpass

Appendix "B"



Proposed water and waste water disposal arrangement

SUSTAINABLE INFRASTRUCTURE DEVELOPMENT - A NEW ZEALAND EXPERIENCE

Kuruwita, S.S.
C.Eng. (IESL), M.Sc., M.Eng., B.sc. Eng. (Hons.)

Aurecon, Level 4, 139, Carlton Gore Road, P.O. Box 9762, Newmarket, Auckland 1149, New Zealand
Tel: 0064 - 09 - 5206019 ext.7446 ; Fax: 0064 - 09 - 5247815
E-mail: skuruwita@gmail.com, kuruwitas@ap.aurecongroup.com

Abstract:
Sustainable Infrastructure has been defined as "…Future friendly, resource efficient …. With pedestrian and public transport oriented systems…."[1] (Back ground paper, UN Expert Group Meeting, 2007)

Wide arrays of strategies are available to achieve the above. Creation of transport corridors, higher capacity transport modes, Increased levels of service and future proofing projects are four of them. Using these in an eco efficient manner will provide sustainable transport systems.

This paper concentrates on the surface transport strategies adopted in New Zealand. More specifically it wishes to provide an insight to the exclusive busway and buslane operation in New Zealand

Keywords: *Sustainable, Transport, Bus way, Bus Lanes*

1.0 Introduction

Sustainable infrastructure system has been defined as "one that facilitates the delivery of transport, energy and water services to support social and economic development in an integrated, eco-efficient and socially inclusive manner. It contributes to the achievement of the Millenium Development Goals and the reduction of green house emissions and the ecological footprint" [1] (Dr. David Ness, Background paper, UN Expert Group meeting, Bangkok, 11-13, June, 2007)

Alternatively, The Ministry of Economic Development, New Zealand has made the following definition about the Infrastructure in a Report to the Ministry of Economic Development, May 2004 [2]. This report has been compiled by Peter Clough, Ian Duncan, Doug Steel, Joanna Smith and John Yeabsley and reviewed by Brent Layton of NZIER.

Based on the McMillan Dictionary of Economic terms (1996) the above report defines the infrastructure as "The structural elements of an economy that facilitates the flow of goods and services between buyers and sellers".[2] The report then divides the infrastructure in to three categories.

- Economic Infrastructure (Physical assets)
- Social infrastructure (assets supporting a healthy workforce with adequate skills)
- Institutional infrastructure (legal system, culture and capital markets)

Both these definitions are complementary to each other. They have common characteristics. Both emphasize the need to be;

- eco-efficient
- Reducing green house gases
- Contributing to development goals.

In the context of transport, sustainability can be loosely defined as keeping the transportation of goods and passengers as long as possible without a reduction in the level of service. Alternatively, any effort to increase the level of service or to lengthen the service period of transportation facilities with unchanged level of service will help make them sustainable.

There are consequences that humans are ready to live with, to enjoy a certain level of service in their current transport facilities. Any attempt to increase the level of service or lengthen the period of service will therefore increase the consequences that humans tolerate.

The impact of these consequences can be minimized or better managed if appropriate strategies are followed.

This paper attempts to highlight some strategies that have found success in minimizing the above consequences. Two case studies from New Zealand are used to highlight these strategies. This paper concentrates on the area of physical infrastructure and specifically transport sector.

2.0 The Current Situation

The world is facing an increase in population. Countries are finding it a struggle to keep and protect the populace. Noteworthy trend is that there is increase in urbanization parallel to this increase in population. Roberts, B. and Kanaley, T., report that “Urbanization in Asia involves around 44 million people being added to the population of cities every year.” [3] (Sustainable Urban Development in Asia, Australian planner, Vol44, No1, March, 2007)

People are attracted towards the better living conditions offered by the urban centers. One adverse effect of the urbanization is the creation of slums or low income housing at the periphery of the cities.

This extra population is serviced by the limited transport services of the city. They share the road space, the rail space, vehicle space and recreation space. They produce increased tonnage of waste that needs to be disposed. They consume more energy and increased amounts of water.

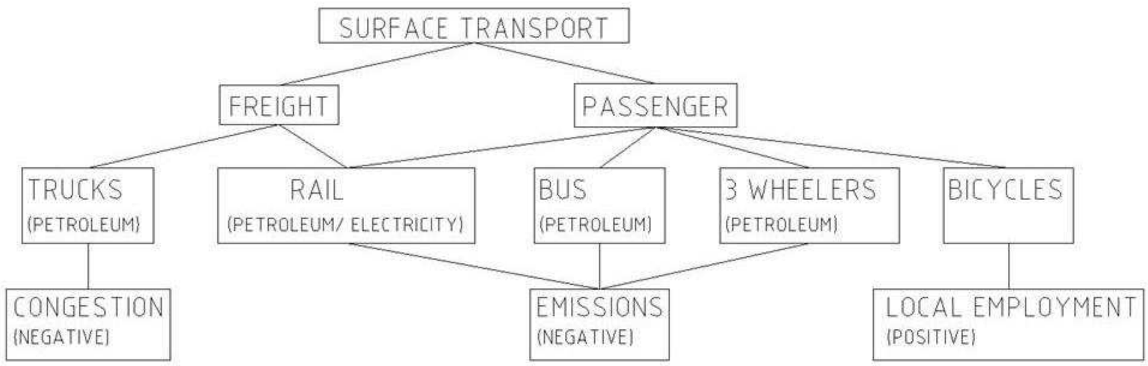


Figure 1 - *The Current Transport Scenario*

If better ways of managing are not implemented, all this extra pressure will result in unbearable environmental degradation and resource consumption.

An increase of services to meet these demands will release increased amounts of pollutants, waste products and will reduce the livable space. (i.e. environmental degradation and excessive resource consumption) The particular choice for the service increase can lock the consumption patterns for a long time. “ Urban roads and freeways in preference to mass transit systems imply heavy fossil fuel demand for personal modes of transport and continued growth in greenhouse gas emissions” [1] (Dr. David Ness, Background paper, UN Expert Group meeting, Bangkok, 11-13, June, 2007)

In transport sector the symptoms of these adverse factors are;

- The congestion of road space

- Higher concentrations of pollutants
- Reduction of livable space

The higher concentration of pollutants is an example of environmental degradation. The congestion of road space and the reduction of livable space are the examples for higher resource consumption.

The livable spaces can be defined as the areas where people could reside safely, conveniently and with sufficient recreational spaces.

3.0 The Future Scenario

From this relatively unplanned chaos, it is better to move towards a better planned urbanization. Better planned urbanization will yield lower or managed environmental degradation and lower resource consumption. In a better planned urbanization the pressures on the infrastructure will be far less than that of an unplanned one.

In the transport sector, a better planned urbanization will yield

- Reduced congestion on the roads
- Lower concentrations of pollutants
- More land available for livable spaces.

4.0 Challenges of Getting There

If the increase in road and rail space can be achieved in an eco efficient and future friendly manner, the environmental degradation and the resource consumption can be minimized.

The first challenge is making the infrastructure eco efficient. Reduction of resource and energy consumption and emissions will make the infrastructure projects more eco efficient. Making the infrastructure eco efficient will also be consistent with the definition of Sustainable Infrastructure Development.

The second challenge is highlighted by the following statement. “Transport and urban infrastructures become traps if they can only operate on large footprints. In contrast, future friendly infrastructure-cities designed as resource efficient, with carbon neutral buildings and pedestrian and public transport oriented systems-can support a high quality of life with a small footprint” [4] (WWF,2006). Therefore, future proofing the transport projects (in terms of eco efficiency) during planning, can overcome this challenge.

5.0 Strategies to Meet the Challenges

Innovations to the current way of planning are needed to meet the above challenges. Ways to minimize ecological foot print and to make the transport projects future friendly needs to be discovered.

Transport corridors, higher capacity transport modes, increased levels of service (eco friendly), future proofing projects are a few strategies that will reduce the ecological foot print and make the transport projects more future friendly.

5.1 Transport corridors

This concept has been identified some time ago and is made use of increasingly. It is composed of combining or relocating different transport modes to flow in a fewer corridors. This concept will give the following advantages:

- More land will be freed for recreational and residential land use.

- Dissipation of pollutants will be concentrated at a fewer number of corridors and could be more economically and easily managed.
- Switching between the modes is made more convenient for the passengers.

These corridors tend to increase the service throughput by eliminating or minimizing the time lag between the loading and unloading. The time and the costs of travelling between the modes are also minimized.

Down side of these transport corridors is the production of higher concentrations of pollutants.

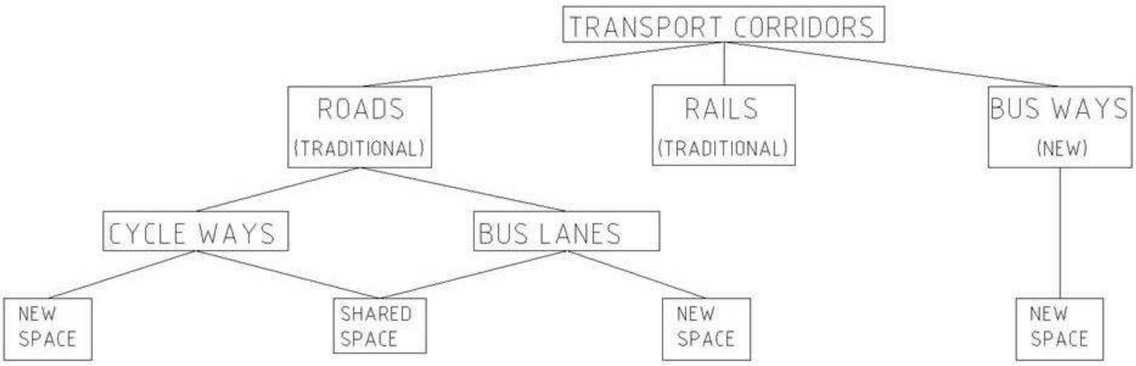


Figure 2 - *The concept of Transport Corridors*

This concept can be extended to include the networks of other services of energy and water within the same corridor.

5.2 Higher Capacity Transport modes

Busses, Trucks and the Trains are the conventional higher capacity transport modes. Increasing the use of these instead of the passenger car will help to increase the service throughput. Popularizing the use of these modes will lead to increased patronage. Some innovations to popularize these modes would be:

- Creating more accessible modal interfaces (modal interface-Railway station, Bus Station)
- Creating more convenient modal interfaces (more comfortable stations, better parking facilities etc.)
- Introducing more frequent trips
- Introducing faster trips between stations (segregating the passenger transport from the road transport mix - busways, monorails etc.)
- Introducing more comfortable vehicles (light rails, smaller busses etc.)

5.3 Increased levels of Service (eco friendly)

Mobility is inversely proportional to accessibility. Keeping mobility / accessibility ratio constant, it is still possible to increase the service throughput. This could be achieved by using higher capacity transport modes in higher mobility corridors. A number of innovations are being tried out and are being found acceptance.

- Busways
- Bus lanes
- HOV lanes
- Light rails

5.3.1 Busways

Exclusive busways are usually run between urban centers or between cities. No other vehicles or pedestrians are allowed on them. Bus Stations are located near the centers of Bus Trip catchment areas. Due to their exclusivity they do not face traffic congestions faced by the busses on normal roads. Therefore, they will be faster networks. They can throughput greater number of passengers more rapidly.

Bus Stations are designed to streamline the transfer from the land side to the vehicle side smoothly. Eco friendly designs help in minimizing the foot print and also in attracting more passengers. Free parking for the passengers can be an added incentive.

5.3.2 Bus Lanes

Bus lanes are exclusive lanes assigned for bus travel. These are usually assigned during the peak travel periods. The existing traffic lanes are rearranged to produce the bus lanes. The best location for the bus lanes are the lanes adjacent to the kerbs. However, there are median adjacent bus lanes in densely commercial city centers. (The freight trucks may occupy the kerb side for loading and unloading operations. Special accesses at higher frequencies may be available for the passengers to access the medians).

At times other than peak periods, these lanes may be opened for other traffic to use.

5.3.3 HOV Lanes

These are operated in situations where the high occupancy vehicles are relatively low in number compared to single occupancy vehicles. When the HOV lane is not congested, the high occupancy vehicles can move at greater speeds than the adjacent conventional lanes. This is an increase in the service throughput. The down side is that these require more disciplined drivers and more policing. Usually these are single directional and are separated from the other lanes. The entries/ exits are spaced out at suitable locations. Reversible HOV lanes are also operated occasionally. The main justification is the speed differential between the HOV lane and the adjacent lanes. A barrier between the HOV lane and the adjacent lane is highly desirable as this speed differential can be very high. These are used on freeways. [5]

5.3.4 Light Rails

Light rails are of two basic types. Running along urban streets sharing space with other traffic one type is known as Super Trams. They behave like trams with smaller number of cars and frequent stops.

The second type runs in its own right of way with less frequent stops. They behave more like trains with a larger number of cars.

Light rails typically have a passenger carrying capacity in excess of 200 for each car. They offer large increase in service throughput. [6]

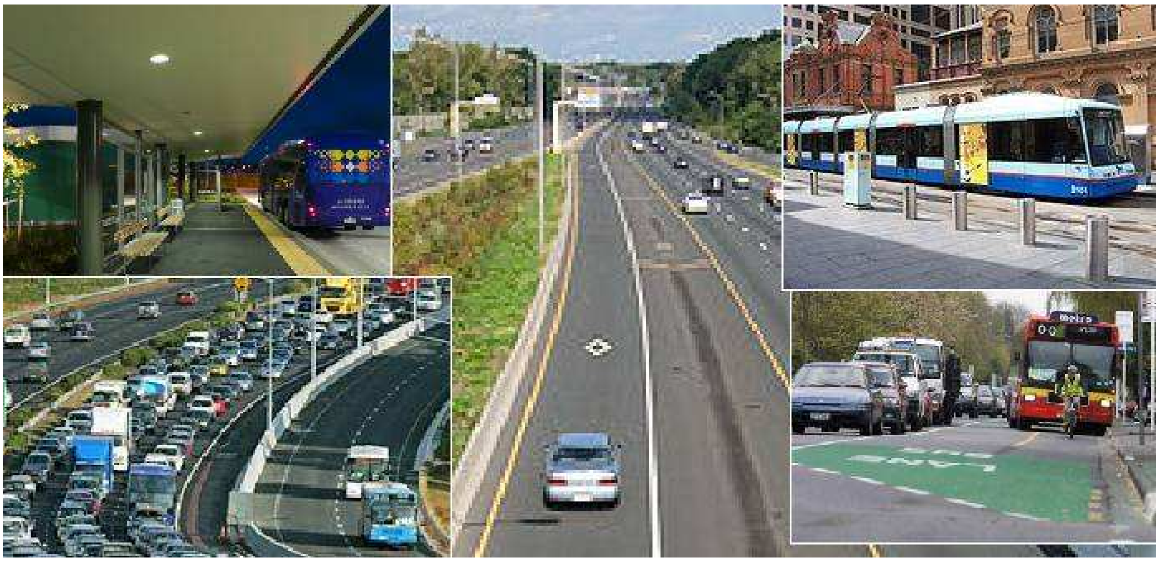


Figure 3 – *Clockwise from Top Left Corner, Light Rail, Bus Lane New Zealand, Middle - HOV Lanes, Northern Busway New Zealand, Bus Station-Northern Busway New Zealand*

5.4 Future proofing the Projects

Future proofing the transportation projects against more eco efficient methods will eliminate the trap of large eco footprints. Selecting for a busway, an alignment which is also suitable for a light rail will serve as an example of future proofing. Such an alignment will minimize the costs of retrofitting a light rail at a future date.

6.0 New Zealand Experience

New Zealand has been trying a few of these innovative strategies pursuant to the Ministry of Economic Development policy, and others are in the pipe line. New Zealand is a country with one fourth of her population in the city of Auckland. This imposes a huge pressure on the transport network of Auckland. Therefore it is no surprise that most of the projects where these strategies are tried out is around Auckland.

Two projects highlighting above strategies are the Northern Busway and the Bus lanes. Northern busway is known as a huge success story with its use increasing by 20%, in one month alone. The bus lanes on the other hand have some criticisms leveled against them. But both are contributing to an increased throughput.

There are investigations being planned for a light rail link in the Auckland Central Business District.

6.1 Bus Lanes

According to Auckland City Council website there are 19 roads with bus lanes. All of them are operated at peak hours (i.e. 7-9AM and 4-6PM). Motor cycles and bicycles are permitted on them as they do not infringe on the capacity of the bus lane. Outside the peak hours other vehicles are permitted the use of these. Wellington and Christchurch are other cities operating bus lanes in New Zealand.

The road space was reallocated by permitting the busses to use the outermost lanes during peak hours. The rationale was that uncongested bus lanes increase the speed of busses. These conform to all three strategies enumerated above.

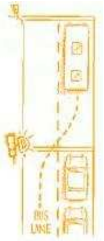
The start and the ends of bus lanes are clearly sign posted. The start sign additionally carries the operating times. The pavement of the bus lanes are marked in green at both the start and at the end. These colored areas carry a logo “BUS LANE” in white to avoid any ambiguity.

Very stiff penalties are imposed for unauthorized use of the bus lanes. The use of bus lanes are monitored by video cameras. Private vehicles turning off the main road are permitted the use of 50m. of the bus lane immediately prior to the turning.

To further facilitate bus movement there are proposals for bus priority measures. Bus advance, bus bypass, bus boarder, signal pre-emption are being considered actively as bus priority measures. [7] [8](Courtesy of Auckland City Council Web site.)

6.1.1 Bus Advance

These enable the busses to get to the front of a queue. Additional set of signals are mounted approximately 50m. in advance of the normal signals, where the general traffic is halted. The bus can then move to the front of the queue and the first to move when green lights on.



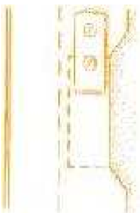
6.1.2 Bus bypass

These are special lanes that allow busses to move around the waiting traffic at an intersection. As shown in the illustration, busses are allowed to move along a left turn only lane and get to the limit line at the intersection

6.1.3 Bus Boarder

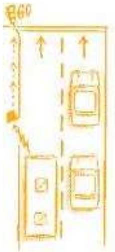
These intrude in to a traffic stream allowing the busses to stop (to pick-up passengers) without going off stream.

The trailing vehicles will have to wait behind the bus when that is stopped. Advantage is that no time is lost in finding headways to move into a traffic stream, as from a bus bay. This is the exact reverse of a bus bay.



6.1.4 Signal Pre-emption

These detect when a bus is approaching, and allows the bus through the intersection without any hindrance. The detectors on the road (linked to the signal) send the message to the signal, of the approaching bus. If the current phase is red then green phase is brought forward to allow the bus through. If the current phase is green, next phase is further delayed until the bus has passed.



6.2 Busway

The Northern Busway is an exclusive, two lane two way roadway for busses. The terminii for this project are Akoranga and Constellation Drive bus stations. Akoranga bus station is the southernmost bus station on this line. Between the Akoranga Bus Station and the Constellation Drive bus station there are 8.5 kms. of busway. The busway has been located adjacent and to the east of the State Highway 1 (Motorway). Stage 1 of this project is using the motorway between Constellation drive and the Albany bus stations. This has been in operation from November 2005.

There are four Bus Stations in this line. The Smales Farm and The Sunnynook are the other two Bus stations. They are serviced by the local bus services making the passenger transfer smoother.

The busway in practice fits with the strategies outlined in this paper. By being adjacent to the motorway it fits with the strategy of Transport corridors. Being a busway upholds the next two strategies of high capacity transport modes and increased levels of service.

Transit New Zealand (currently part of New Zealand Transport Agency-NZTA), North Shore City Council, Auckland City Council and the Auckland Regional Transport Authority (ARTA) collaborated to bring the Busway into fruition. There was a total of 12 years spent in planning the busway from inception of proposals to the awarding of the earthwork contract in 2003. Various options were assessed and the final alignment was accepted by the Transit New Zealand in 2001. Further design work was undertaken and developed by OPUS, Beca and Aurecon (formerly, Connell-Wagner), all primary consultants in New Zealand. In November 2005 the first two busway stations were opened. The busway was opened for operations with all stations in February 2008.

It had a cost greater than NZ $300 millions. This cost was shared by the collaborators. The Stage 1 of Northern busway project (An express bus service between Albany and Auckland CBD) has produced following benefits. These were expected to increase post commissioning the Busway in 2008.

Benefits of the Busway

Daily reduction of Car trips	500 approx.
Annual savings in fuel	40000 liters
Annual reduction Carbon dioxide	>1000 tonnes
Annual reduction of Carbon Monoxide	13.6 tonnes
Annual reduction of fine diesel particles	0.14 tonnes
Annual reductn of car Travel	3.94million kms.

The daily use of the Northern busway is over 7000 passengers according to ARTA. This can be translated to a daily reduction of 3100 car trips assuming a car occupancy of 1.2 persons per car. In the first two months the use has grown by 7%, and in 2009 may it has shown a growth of 20%.

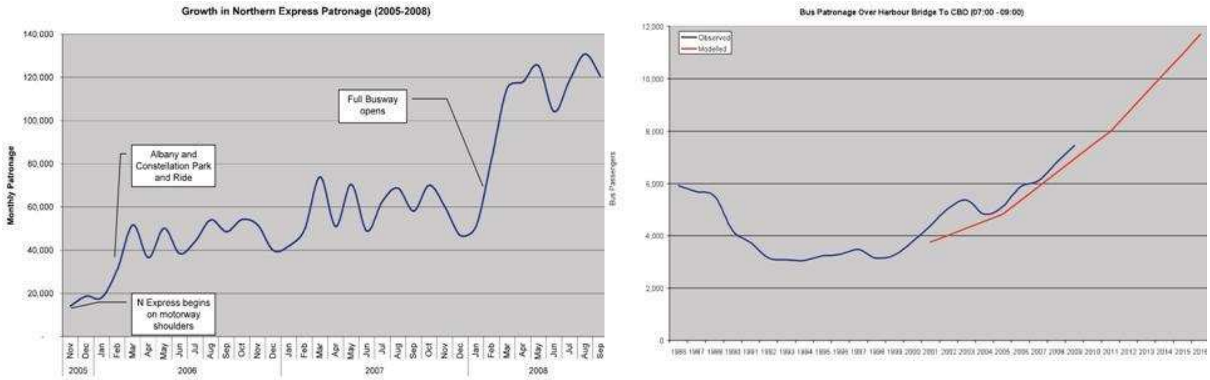


Figure 4 *The increased patronage and the patronage increasing rate of the Busway. (Courtesy of a Presentation to New Zealand Society of Sustainability Engineering and Science, 3rd July 2009) [9]*

The northern busway is being planned to be extended another 18kms. to Silverdale from constellation drive. NZTA has commissioned Zormac consultants with Aurecon for this study in 2008. It has been found that continued use of the eastern side of motorway is favorable except near the Albany station. Due to its orientation and to optimize retro fitting work, it was decided to cross the motorway by means of a tunnel before the Albany station. Once past the Albany station the busway extension reverts back to the eastern side of the motorway and remains on the eastern side until Sliverdale.

Maps showing the location of bus stations and the extensions proposed are in appendix A. [10] [11]

7.0 Concluding Remarks

The following conclusions are drawn from the above case studies:

- Busways grouped together with other transport corridors and Bus lanes increase the service throughput without adversely affecting the livable spaces.
- Higher utilization of busways and bus lanes indicate that they are eco-efficient transport projects.
- To increase operational efficiency between modes, combining stations for the transport modes should be considered.
- To attract more patronage of public transport better parking facilities adjacent to stations of the transport modes should be considered.
- Public transport priority measures should be adopted to attract more patronage.
- Aligning land use and transport is a sustainable transport outcome.
- To ensure sustainability of transport projects energy use and emissions needs to be reduced

All the above conclusions underline the acceptance and popularity of public transport systems in New Zealand. The higher acceptance and the use of public transport systems in New Zealand reflects that correct strategies have been adopted. The sustainability of the transport system will be improved by the public transport systems due to their smaller ecological footprints.

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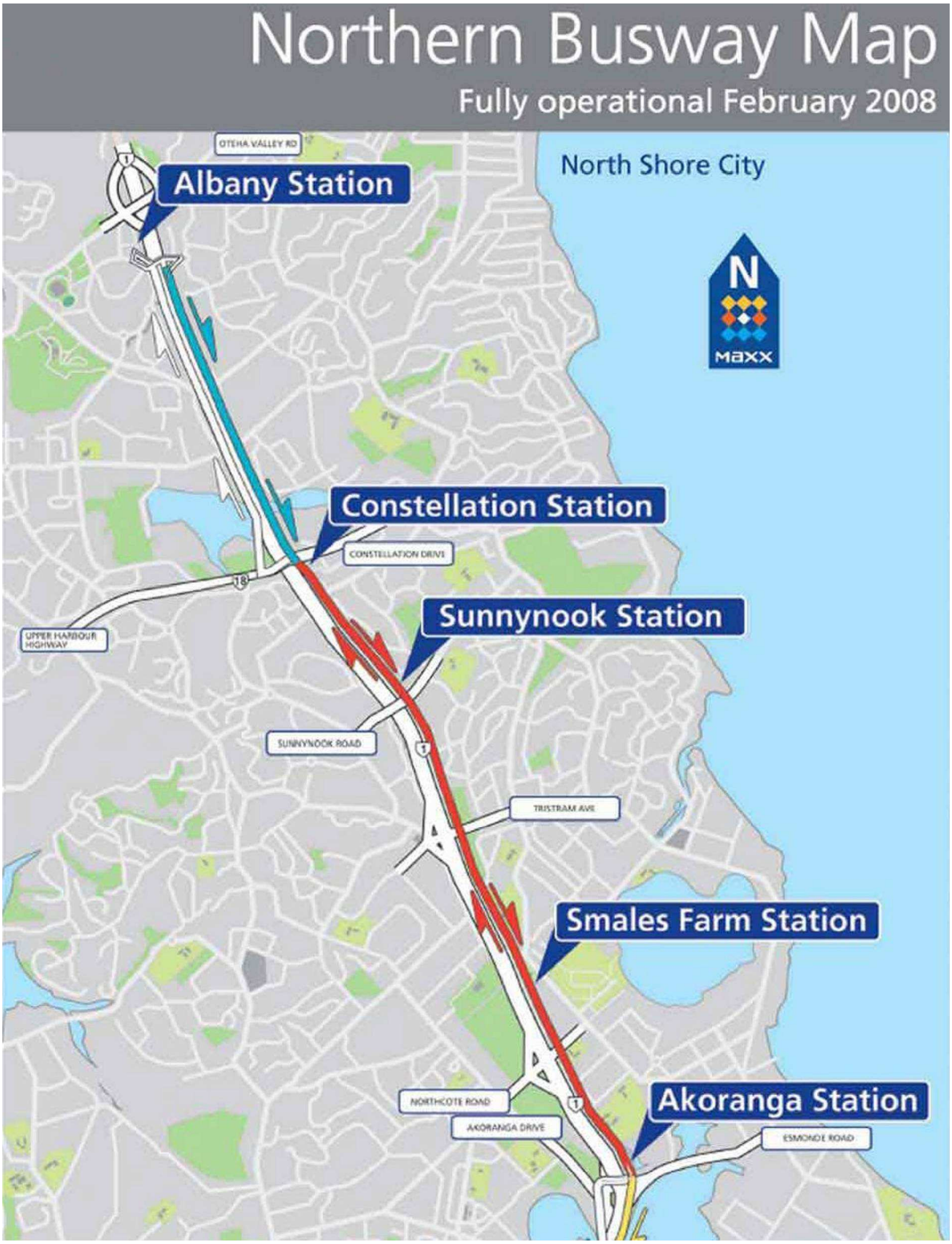
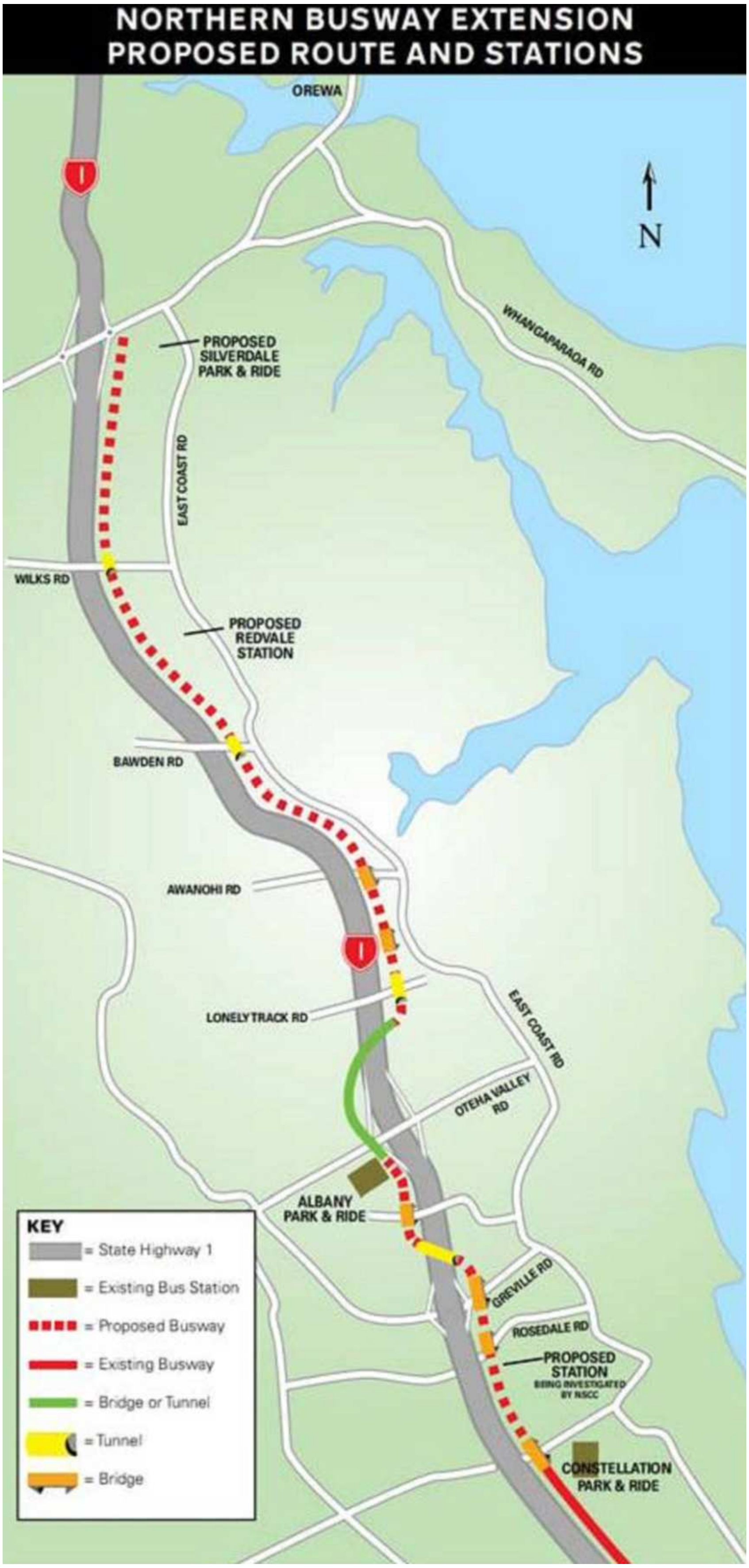
APPENDIX A



LOCATION PLAN OF THE NORTHERN BUSWAY



LOCATION PLAN OF THE PROPOSED EXTENSION TO THE NORTHERN BUSWAY

ACCESSIBILITY TO TRANSIT STATION IN MULTI MODAL TRANSPORT FRAMEWORK FOR DELHI

1. Pawan Kumar*
Ph.D. Scholar (CTRANS),
Centre for Transportation Systems, Indian Institute of Technology, Roorkee- 247667, India.
Tel: +91-1332-286305, +91-9012723521; Fax: +91-1332-283560
E-mail:pawan612@gmail.com, pkumar612@yahoo.co.in

2. Prof. S.S. Jain,
Prof. & Head (CTRANS),
Centre for Transportation Systems, Indian Institute of Technology, Roorkee- 247667, India.
Tel: +91-1332-285435; Fax: +91-1332-273560
E-mail: profssjain@gmail.com

3. Prof. S.Y.Kulkarni,
Prof. & Head
Department of Architecture and Planning, Indian Institute of Technology, Roorkee -247667, India.
Tel: +91-1332-285214; Fax: +91-1332-273560
E-mail: archplng@iitr.ernet.in

4. Prof. M. Parida,
Associated Faculty (CTRANS)
Centre for Transportation Systems, Indian Institute of Technology, Roorkee- 247667, India.
Tel: +91-1332-285448; Fax: +91-1332-273560
E-mail: mparida@gmail.com

* Corresponding Author: Pawan Kumar
E-Mail: pawan612@gmail.com, pkumar612@yahoo.co.in

ABSTRACT

Recently, Multi Modal Transit Station has been recognized as a symbol of 'Urban Identity' and 'Urban Mobility' in Delhi which integrates built environment with multiple modes and provides an important link to complete the journey. It is either as a single station or an interchange hub, accessed by human, mechanical and vehicular system. Hence, it is important that such transit station must meet minimal level of service and be a part of overall efforts to improve transit services for increasing rider ship. In this context, Multi Modal Oriented Design (M_2OD) is used which defines neighborhood character in design and provides mobility friendly environment. It also encourages a mix of mobility options to cater needs of both present and future travel demands. Further, the role and responsibilities of transit operators, facilitators and users are crucial to extend better accessibility to transit station.

Key words *: accessibility, multi modal transport, transit station, multi modal oriented design.*

1.0 Introduction

A Multi Modal Transit Station (MMTS) is defined as mobility friendly built environment which accommodates choice of modes i.e. public transport (metro, bus, etc), auto mobiles (two/three wheelers, cars, etc) and non-motorized mode (pedestrians, bicycles, etc).The design concept of such built environment requires supportive access pattern of multiple modes with proper integration, transfer facilities, safety and ease of use for all commuters. Similarly, the planning, design and engineering aspects of accessibility to multi modal transit station encompasses public space, streets, road environments, transport vehicles & infrastructure such as bus stops, ticket machines, parking, information system, etc. Such accessibility brings built environment and multi modes much closer and provides an important link to the whole journey.

The accessibility to transit station is affected by travel pattern by modes, traffic volume in the vicinity of the station, traffic volume destined to the station, potential characteristics of the surrounding areas, etc. Generally, multi modal transit station has accessibility of almost all modes and design of transit station must provide physical facilities to each mode. Similarly, segregation between modes is also essential particularly between pedestrians & other modes; and between public & private modes for better and safe mobility of commuters in the station area. At least, two independent access routes are required to each station. Similarly, access route for emergency and service vehicles are also essential for each station. Emergency vehicles such as fire brigade vehicles, ambulance, etc must have access to all sides of the station. When the system is in operation, service vehicles such as feeder bus services, RTV, etc must be accommodated in areas other than passenger entry zone.

2.0 Access Vs Accessibility for Multi Modal Public Transport System

Access and Accessibility are two distinct characteristics of any public transport system. Access to public transportation is the opportunity to use the service. It may be interpreted in terms of proximity to and cost of using transport services. If the distance or barriers to access a service are too great at either the trip origin or destination, then it is unlikely to be utilized as a mode of travel. Similarly, if cost is either too expensive (i.e. cheaper modes exist) or unaffordable then utilization of the service is also unlikely. Contrarily, accessibility is the suitability of the public transport network to get individuals from their system entry point to their system exit location in a reasonable amount of time. Thus, accessibility encompasses the operational functioning of a system for regional travel. Access greatly impacts the public transportation system and complements service accessibility *(Murrya et al., 1998).*

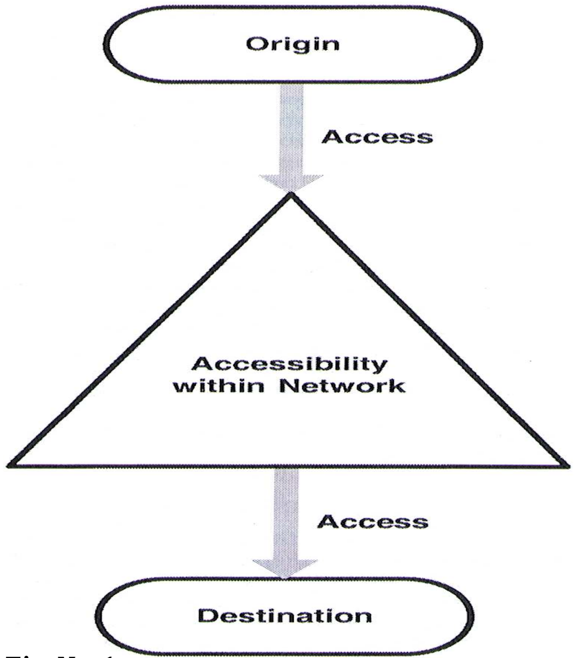


Fig. No. 1:
Access Vs Accessibility in Public Transport System

Figure No. 1 illustrates the relationship between 'access to public transport' and the 'accessibility of the public transport system'. Further, access and accessibility are dependent upon each other if the public transport system is to be successful and well utilized. Similarly, 'origin access' and 'destination access' have different impacts on public transport.

3.0 Multi Modal Transport Systems for Delhi

Multi Modal Transport System (MMTS) is Public Transport which relates to single trip consisting of combination of modes i.e. vehicle modes (bus, metro, car, tram, etc.) or service modes (private/public) between which the traveler has to make a transfer . Transfer is an essential part of multimodal trip and traveler has to change modes at transfer nodes. Hence seamless travel is an important characteristic of the system *(Kumar et al., 2007).* The Master Plan for Delhi - 2021 also advocates about multimodal transport system as future transport mode in the capital city *(Delhi Development Authority, 2007).* The Ministry of Urban Development, Govt. of India formulated National Urban Transport Policy, 2006 with the broad objective to ensure safe, affordable, quick, comfortable, reliable and sustainable access for the growing number of city residents to jobs, education, recreation and such other needs within cities. One of the methods to achieve such objectives is to *"enabling the establishment of quality focused multi modal public transport systems that are well integrated, providing seamless travel across modes" (Govt. of India, 2006)*

Fig. No. 2 : Delhi Metro : An Image of Public Transport in India

In Delhi, there are four options for accessing multi modal transit stations either at a single station or at interchange station:

i. Accessibility by Non-motorized Transport (NMT),
ii. Accessibility by Intermediate Para Transit(IPT),
iii. Accessibility by Public Transport (PT),
iv. Accessibility by Private Transport (i.e. car, two wheelers, etc.)

The performance of a public transport system is largely affected by the proximity of public transport stops to the zonal population. The performance of public transport access can be improved by incorporating more dynamic proximity measures, service considerations, demographic and socioeconomic factors.

4.0 Transit Station and Stops: Concept of Level of Service Factor

Transit station and stops are also called as Transportation Terminals. Their rider ship can be enhanced by increasing convenience, comfort and attractiveness. For a typical transit trip, 10-30% of travel time is spent in waiting and transfer. Hence, it is important to assess various features to evaluate 'Transit Station and Stops' based on level of service. These factors can be prioritized from high to low importance, and graded from A (best) to F (worst). Table No. 1 describes "Transit Station and Stop: Level of Service Factors" as shown below *(Victoria Transport Policy Institute, 2010):*

Fig.No. 3: View of Transit Station and Stop

Table No. 1: Transit Station and Stop: Level of Service Factors

S.N.	Features	Description	Indicators
i.	Weather protection	Users protected from sun and rain.	• Bus shelters and covered platforms. • Shade trees and awnings. • Enclosed waiting rooms.
ii.	Sense of Security	Perceived threats of accidents, assault, theft or abuse.	• Perceived transit passenger security. • No. of accidents and injuries. • Reported security incidents. • Visibility and lighting. • Official response to perceived risks.
iii.	Comfort	Passenger comfort.	• Seating availability and quality. • Space (lack of crowding). • Quiet (lack of excessive noise). • Fresh air (lack of unpleasant smells) • Cleanliness of stations and nearby areas.
iv.	Efficiency	Ease and speed of station activities.	• Ticket purchasing. • Baggage checking and collecting. • Security inspections.
v.	Accessibility	Ease of reaching transit stations and stops.	• Distance from transit stations and stops to destinations. • Walkability (quality of walking conditions) in areas serviced by transit. • Park & Ride facilities. • Bicycle parking availability. • Taxi service availability.
vi.	Transit Oriented Development	Quality of development in areas near transit stations and stops.	• Quality and density of development within 500 meters of transit stations. • Walkability (quality of walking conditions) in areas serviced by transit. • Affordability of housing within 500 meters of transit stations.
vii.	Universal Design	Accommodation of diverse users including people with special needs.	• Accessible design for stations and nearby areas. • Ability to carry baggage. • Ability to accommodate disable commuters.
viii.	User Information	Ease of obtaining information on transit routes, schedules, fares, connections, and destinations.	• Accuracy and understandability of information at stops, stations, destinations, as well as availability on Internet, telephone. • Real-time transit vehicle arrival information. • Availability and quality of way finding signs, maps and other information for navigating within the station and to nearby destinations. • Quality of announcements. • Availability of information for people with special needs (audio or visual disabilities, inability to read or understand the local language, etc.). • Availability of pay telephones.
ix.	Courtesy and responsiveness	Courtesy with which passengers are treated.	• Behavior of staff with passengers.. • Ease of filing a complaint. Speed and responsiveness with which complaints are treated.
x.	Aesthetics	Attractiveness of transit stations and stops.	• Attractiveness of stations and stops. • Attractiveness of station areas.

Fig. No. 4 : Waiting and Circulation Space inside Transit Station

It is important to ensure that all transit stations and stops must meet certain in minimal level of service. Multi modal transit station must have full services in integrated manner for safe& easy accessible to users. Hence, transit stations and stops improvement are considered as part of overall efforts to improve transit services and increases transit rider ship.

5.0 Access to Transit Station by Non-Motorized Transport

5.1 Pedestrian Access

All passengers, regardless of access mode, enter the station as pedestrians. Hence, provisions for pedestrians and their safe movement are an essential access consideration. Direct and safe approach for pedestrians should be provided from all adjacent streets and major developments into the station. Pedestrian access facilities should be designed to accommodate the estimated "Peak Five Minute Patronage" at "Level of Service E" ***(****Fruin, 1971).*

- Pedestrian networks should be designed to ensure that 60% of all residents living within 500 metre of a transit station have only a maximum 500 metre walk to transit station.
- Cul-de-sac design should include pedestrian access-ways that reduce distances to bus services on the adjacent higher order bus streets.
- Continuous and direct footpath networks that permeate neighborhoods increase the catchments for public transport because it takes less time to walk from home to bus/metro.
- Pedestrian networks should be most developed in multi modal transit areas.
- Safe, convenient and/or controlled road crossing points should be provided to stops with high passenger usage.

Direct pedestrian connections to surrounding major developments, existing proposed, should be encouraged to improve passenger comfort and convenience and to attract additional patronage. Connections can be underground, elevated, or sheltered walkways at grade.

5.2 Bicycle Access

Good bicycle access can increase the service area of transit stations. The demand for bicycle facilities is very much dependent on demand and supply as well as encouragement policy of ULBs in terms of reasonable parking charges for longer duration, less charges for metro smart pass holders, etc. Generally, roadways that are adequate for motor vehicles may not be suitable for bicycles. The ability of existing access roads is also important to provide a safe and convenient bicycle route between the transit station and main roads of the neighborhoods. Bicycle parking facilities should be located to provide protection from weather, theft and conflicts with other modes. Parking areas i.e. active areas with high pedestrian activity, or under the observation of station attendants, are preferred. The location should be well lighted and designated by a sign as bicycle parking facility. The parking facility should be closer to station entrance and nearby roadway so as not to require the bicyclists to walk the vehicle long distances through pedestrian areas.

Several types of bicycle parking facilities are available. They vary in their cost, space requirements, protection affordability, and utilization. They fall into three categories: i. Locker facilities; ii. Racks which lock the frame and wheels; iii. Racks or fixed objects which require the bicyclists to use their own chain. Parking facilities that secure both wheels and the frame are preferred. Delhi metro provide free bicycle parking for Delhi metro pass holders on priority basis.

5.3 Cycle Rickshaws Access

Cycle rickshaws has significant share in urban mobility. It is pollution free, low cost access mode to multi modal public transport. Legislation and traffic enforcement regulations have to be introduced to prohibit the operation of cycle rickshaws along specific routes or within particular areas of the city as per prevailing land uses. The level of traffic enforcement is a direct function of various Urban Local Bodies (ULBs) which are invariably limited. It is constraint of ULBs efforts at supervising, operation and monitoring of cycle rickshaws in the municipal limits *(Rao et a.,1990).* However, sufficient on-street parking for cycle rickshaws at metro station and improved cycle rickshaw design may provide better accessibility and safety to the commuters.

6.0 Accessibility by Intermediate Para Transit (IPT)

Various modes of IPT such as auto, taxi, phatphat, etc are considered as feeder modes which brings the passengers from various parts of the neighborhood to the multi modal transit station. Hence, IPT zone is desirable, preferably with loading on the left hand side, adjacent to the main entrance of the station and weather protection should be provided. IPT area must be covered and seating may be provided.

Fig. No.5: Role of Auto as Feeder Mode at Pragati Maidan Transit Station

7.0 Accessibility by Public Transport

Bus loading/unloading zones is considered as a part of transit station design. Bus stop design capacity for a station is based on the individual requirements for each station either as a single station, interchange station or integration point. Loading zones for buses should be located to provide the most direct and safest multimodal transfer. Excusive bus loading/unloading areas should also be provided. The following designs may be used for various types of bus loading zones, depending on specific conditions:

i. **Recessed Bus Bay:** It is used where more mixed traffics are on the road. The bus loading zone is recessed from the through traffic lane. A recessed bus bay is designed parallel to close enough to the curb so that passengers may enter and leave any door by an easy step to the curb. Upon leaving, the merging lane enables the bus an easy re-entry into the through traffic lane. The loading zone should have a 3mt wide lane, and total length for a two bus loading are should be 73.2 mt *(Refer Fig. No. 6)*. For each additional bus required, an additional 24.4 mt length should be added at curbside.

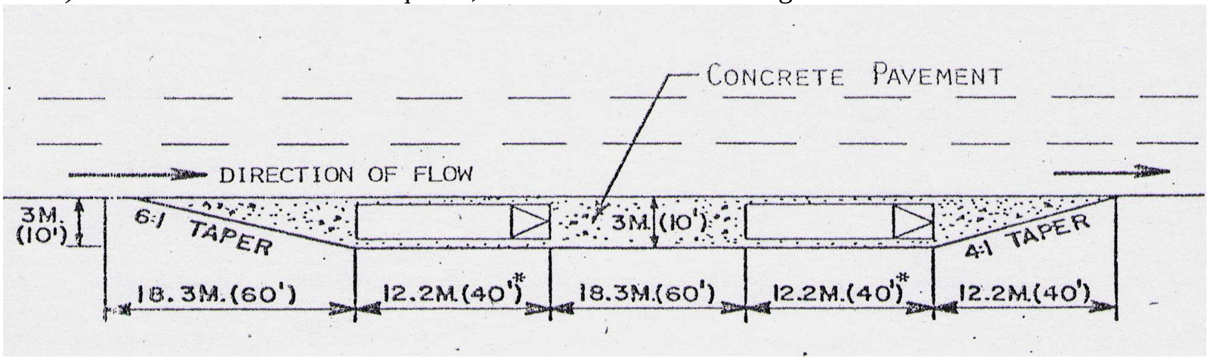


Fig. No.6: Recessed Bus Bay

ii. **Parallel- to-curb Bus Bays**: It should have 3 mt wide lanes and an overall length of 73.2 mt. This layout for bus loading area provides the minimum roadway width but requires the longest length for a bus loading zone. The critical movement in this layout is moving into position ahead of the parked bus. This leaves the rear door of the bus offset from the platform curb by approximately 0.45 mt.

iii. **Sawtooth Bus Bays:** Sawtooth bus bays reduces the length of loading zone and therefore reduces walking distances but increases the width of the roadway.

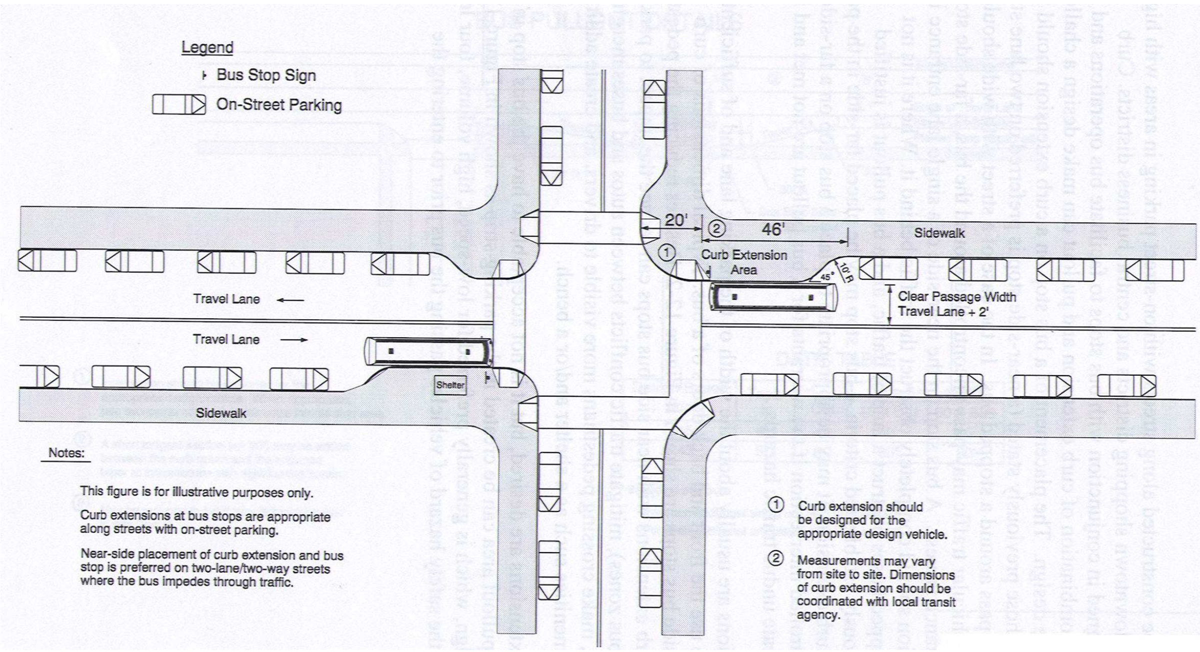


Fig. No. 7: Design of Bus Stop with Curb Extension.

In Delhi, bus loading zone is not a part of "paid area" of the station. If the concept of flat fare structure exits (i.e. no charge for transferring from bus to metro, no barrier intermodal transferring) then bus loading zone can be designed as an extension of the "paid area" of the station but these area must be fenced off or otherwise separated from parking lot, and some degree of surveillance or entry control is required to prevent pedestrians from entering the area. If bus loading zone is a part of transit station, then travel time and inconvenience is reduced by eliminating the need to obtain and turn in transfers or to pass through fare gates. Bus boarding time is reduced as both doors can be used.

Similarly, operating cost is also less (because fewer transfers to print/encode and collect/count can be eliminated).

Bus zone in transit station are must be sheltered, well lighted and visible from the street and adjacent buildings. It is also desirable that shelter or canopy must be connected to the station entrance. ‘Bus Zone’ area should be clearly marked. Signs in ‘bus zone’ and ‘transit station’ , directing patrons to the proper bus loading area, and bus stalls, displaying specific bus routes, etc should be prominently displayed.

8.0 Accessibility by Private Modes

Delhi metro provides parking space for car, two wheelers at the station. Entry and exit points should be at mid block and in no event less than 46 mt from an intersection. Parking aisles should be lined perpendicular to the station entrance/exit are to minimize the no. of potential conflicts between pedestrians and automobiles. If perpendicular aisle geometry is not possible, pedestrian walkways can be created across the aisles by well marked 1.5 mt wide paths.

Collapsible posts or signs should be used to delineate drives and pedestrian. Major pedestrian walkways should be raised 0.15 mt above the parking pavement. Since, queuing of automobiles is more severe when leaving parking facilities, the lots should be located so that there will be some ‘stringing out’ of drivers as they walk to their cars. When fee is charged for parking, the preferred collection methods are either by permit or by central fee deposit boxes. Fee collection upon entrance is likely to lead to short queues while collection upon exit to long queues. Right angle parking should be used because it allows better circulation, more orderly parking, and in most cases has lower average area requirement per space.

9.0 Improvement of Access to Transit Stations /Stops

Every commuter needs safe and convenient routes to get to and from transit station. A commuter prefers about 5 to 10-minute walk to and from transit station. Typically, a commuter walks to a transit stop, board the bus or train, get off, and then walk to their final destination. Thus, the commuter’s needs as pedestrians extend beyond the transit stop to and from the surrounding neighborhood.

Generally, transit agencies take responsibility only for their stops, stations, and parking lots, and not for sidewalks, crossings, or other pedestrian elements on nearby streets. As a result, pedestrians must often cross busy streets and cut through parking lots to get to the bus stop or train station. Hence, provision of sidewalks makes bus stops and train stations more accessible. Safe and convenient crossings are also essential, especially for midblock bus stops. New stops and stations can be placed with pedestrian (and bicycle) access in mind.

9.1 Access to Transit Stops located on Surface Street (Ground)

Transit stations at surface may be hub of bus, rail, tram, BRTS, light rail, etc. Each such transit station must fulfill the location requirement of Passengers (i.e. stops must be near places where there's an expectation of riders), Access (if a stop can't be located right where riders are, they must be able to get to the stop conveniently) and Traffic characteristics (buses can't always stop where riders want to be because of complex traffic patterns, especially at intersections).Therefore, access to transit also involves selecting the right location for stops, especially for bus stops located on surface streets.

(a) ***<u>At Transit Stops</u>***

The safety of pedestrians can also be enhanced using a variety of transit operation improvements (such as consolidating, relocating or eliminating stops) usually implemented by the transit agency in cooperation with the road authority. Convenient access by passengers must remain at the forefront of all transit stop.

(b) ***<u>At Mid Block</u>***

When a transit stop is located midblock, a single crossing should be provided to serve both directions of bus travel. If a crosswalk is marked midblock, it should be behind the bus stop for several reasons: Pedestrians cross behind the bus, where they can see oncoming traffic (crossing in front of a bus blocks visibility); Bus driver can accelerate as soon as passengers have left the bus; Bus driver won't accidentally hit a pedestrian crossing in front of the bus, out of the driver's cone of vision,; etc.

(c) ***<u>At Intersection</u>***

At intersections, farside stops are usually preferred for a variety of safety and operational reasons. One safety advantage is that pedestrians cross in back of the bus. Operationally, a far side stop often improves intersection capacity by allowing motor vehicles to make left turns even when the bus is in loading and unloading position.

9.2 Access to BRT Stops on dedicated Rights-of-Way

Transit agencies provide ‘Park-and-Ride’ facilities at BRT stations for riders who commute from distance places to the station. Once these riders park their cars, they become pedestrians as they walk through the parking lot to the station itself. These parking lots can present challenges for pedestrians walking to the station. Pedestrians can be at risk of being struck by motorists looking for, driving into, and backing out of parking spaces; they must also park cars and buses on access roads and passenger drop-off areas.

‘Park-and-Ride’ facilities can be designed to reduce these risks to pedestrians. Sidewalks can be built between rows of facing cars so that pedestrians don't have to walk in the aisles. Pedestrian routes should cross access roads where drivers can expect and see them. Bus loading areas should be positioned so that pedestrians don't have to cross between parked buses. Because hundreds of people may get off a train at once, there must be enough sidewalk space adjacent to the station entrance so that no one is forced to walk along the roadway.

9.3 Access to BRT on Surface Streets

Bus rapid transit often operates in a hybrid mode; it can run on dedicated rights-of-way on special tracks and also act like a bus on streets. Often the special tracks are laid in the median of a wide thoroughfare or boulevard.

Fig. N0. 8: Accessibility to BRT on Surface Street in Delhi

Stations are typically far apart to improve operational efficiency. This creates situations where stops attract a large number of riders within a busy street environment. It needs to provide enough waiting area for passengers; safe and convenient street crossings; ensuring that waiting, crossing, boarding, and de-boarding passengers don't interfere with the flow of pedestrians; etc.

9.4 Design of Transit Stations /Stops

At transit stations/stops, waiting for the bus or metro can be made more pleasant. Shelters with seating can offer protection from rain, snow, wind, and sun. Similarly, shelters at frequently-used bus stops and at outdoor rail stations can be provided. The shelters should be positioned so riders in wheelchairs have enough room to enter and exit the shelter. The sidewalk behind the shelter should be wide enough for two wheelchair users to pass each other and to handle the expected levels of pedestrian activity, including those who are just walking by. The best location for bus shelters is in the furniture zone, away from the walking zone.

Schedules and route maps should be placed at transit stations/stops to orient riders. Current technology makes it easy to have video monitors with bus arrival times in real time, displaying the number of minutes until the next bus or train and its destination. Nighttime lighting is important for passenger safety and security. Lighting makes it easier for riders to watch their step so they don't trip on station escalators or while boarding the bus. With lighting, drivers are more likely to see riders crossing the street. Riders are more secure while they're waiting because they can see their surroundings and watch for suspicious activity. Transit station must be made accessible to riders with disabilities.

Fig. No. 9: Accessibility to Delhi Metro for Riders with Disabilities.

10.0 Accessibility to Transit Station as Multi Modal Interchange

Transit station acts like a multi modal interchange where physical action of transferring between services or modes as part of the passenger's journey or it can be the physical location that provides access to the public transport system. Interchange can be either an inconvenience imposed by the configuration of public transport network or an opportunity for passengers to take advantage of reduced travel time and/or costs. Infact, interchange is a strategic public transport network which allows 'services to be connected', 'volume of passengers', balance between 'those using the interchange as a point of access& egress' and 'those using the interchange to transfer between services'.

Multi modal interchange facilities serve metro to metro, bus to metro, car to metro, rail to metro movements. In the vicinity of public transport interchange facilities, the interface between large vehicles (bus) and small vehicles (cars) should be planned for and managed. There should be safe and convenient covered waiting areas and paths between two modes for commuters. Where park-and-ride facilities are provided, access should be from the higher order street (if possible) to reduce impact on the local street network and to make access easier. The various ways of making use of park and ride facilities during non-peak periods should be considered. It will increase passive surveillance by creating 'public eyes' and can generate extra revenue. Off-line facilities such as park-and-ride, kiss-and-ride, bus lay-over facilities, IPT parking, etc also provide better accessibility to transit station. All interchange locations having high frequency services may have shelter & shade, seats, lighting, pick off/drop off site, telephones, extensive system information, etc.

11.0 Accessibility to Transit Station/Stops: Current Practices in Delhi

11.1 "Cycle-for-Hire Scheme" was initiated by Dellhi Metro and Delhi Bicycling Club which encourage people to use bicycles for short distances at Delhi University Metro station and near by areas. The charge of a bicycle on rent is Rs 10/= for 4 hours. After being popular "Cycle-for-Hire Scheme ", Delhi Metro started the same in other Metro stations. The Delhi Bicycling Club was started by ITDP India, an NGO engaged in research and advocacy for green, sustainable and equitable transport policies. It is estimated that Delhi with its wide roads can become a heaven for cyclists, if the Govt. spends 1/1000 part of its expenditure on building cycling tracts *(Indian Express, New Delhi edition dated 01.01.2009).* However, such scheme attracts commuters to transit station for further journey in neighboring areas.

11.2 Pedestrian way to the BRT Corridor in Delhi

Pedestrian way to the BRT Corridor in Delhi is one the examples to provide safe and comfortable sidewalk to access public transport. In Delhi, BRT is a component of multi modal transport system. The following provisions make pedestrian movement safer:

●Roadway design has retained the continuity of the sidewalks. It has wide and well surfaced sidewalks and is disable friendly.

●Sidewalks are easily negotiable by women, children, senior citizens, as the height is close to 15 cm. Width of sidewalks varies from 1.5 mt (minm) to 4.5 mt (maxm) along the corridor. Sidewalks are well lit.

● Crossings are easily accessible with kerbed ramps and there is a holding area for people to want at the side and at the pedestrian refuge islands.

●Pedestrian path on the BRT corridor has the least permanent and temporary obstructions on the sidewalks

The sidewalks are continuous. The pedestrian don't have to get off and on the footpath as they used to before the corridor was constructed. Its salubrious environment invites more pedestrians to get easy and safe access to BRT services *(*Centre for Science and Environment *2009).*

11.3 Park and Ride Facility at Delhi Metro Station

Park-and-ride facilities provide parking for people who wish to transfer from their personal vehicle to public transport. Such facilities are available at Delhi Metro stations. Different parking rates for car, two wheelers, cycles, etc are available according to parking duration. Site design of such facility is very crucial for easy entry and exit. Design features must be in compliance with applicable design standards, specifications, operating standards, and any other local requirements that may apply.

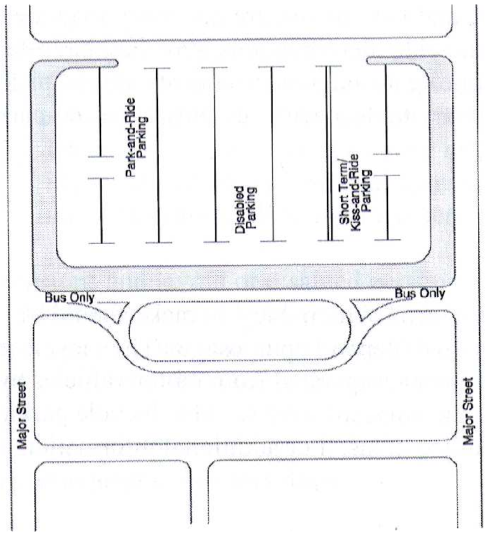


Fig. No.10 : Park -and-Ride Facilities along Two Adjacent Streets

For Park and Ride Facility

Mode	**Stall size**	**Aisle width**
Standard car	2.6 mt x 5.5 mt	18.3 mt to 18.9 mt
Compact car	2.3 mt x 4.3 mt	14.6 mt to 15.8 mt.

Design features such as entrances and exits, internal circulation, shelter location, illumination, landscape preservation and development, and passenger amenities are generally site specific and used to maximize the efficiency and usefulness of the facilities.

12.0 Transit Station Area Development

Transit station particularly in the framework of multi modal transport system is mainly based on transit oriented development which involves the coordination between land use planning and transit planning. Multi modal transit station is always planned as high density mixed land use areas with transit friendly design. Higher densities have been permitted for office use because offices generate more transit rider ship per sq.mt than residences. Similarly, zoning regulation must permit some floors to retail commercial uses. However, front of transit ways can be permitted for restaurants, coffee houses, etc.

Fig. No. 11: Surrounding Environment at and around Transit Station in Delhi

Parking management is also an integral part of transit station area development. Proper parking facilities at the station reduce transfer time attracting the personalized vehicle users to the transit services. A lesser parking fee at station increase the share of transit mode. The parking should preferably be underground as it permits higher development densities and free the street level for retail uses and other development that encourages pedestrian use. In Vancouver, at Metro Town Station, 10,000 free parking spaces have been provided to support the station area development *(Jain & Parida 2001).*Design of park-and-ride lot locations should not preclude development opportunities. Similarly, automobile traffic in and around station areas should be managed to facilitate station access. Topographical constraints should be overcome through steps, ramps, grading, etc and shade trees and other weather protection features should be provided.

Multi Modal Transit Oriented Design (M_2OD) encompasses a diverse family of land use planning and site design concepts designated to encourage a mix of mobility options. It does not give preference to any mode but create a balance between various modes so that they may work to complement each other. M_2OD consider the following attributes to enhance the accessibility to transit station:

- **People Place:** Transit station area is a place for people/commuters. Hence, it must be well used, safe, comfortable, and attractive; and need to be distinctive and offer variety, choice and fun.

- **Streetscape Design:** M_2OD should promote and enrich the qualities of existing urban places at neighborhood and street level. It is important to develop streetscape design elements for each of the transport corridor by incorporating various architectural elements.

- **Urban Form:** Transit station is considered as a part of urban design and sky line. It is blended with different building forms, colors, materials, textures, forms, etc. Such areas should be integrated physically and visually with its surroundings having better access by foot, bicycles, bus, cars, metro, etc. Amenities that are stimulating, enjoyable and convenient should be offered to a wide range of possible users.

- **Transit in Landscape:** These areas should provide balance between the natural and the manmade environment and utilize each locations intrinsic resource – the climate, land form, landscape, and ecology – to maximize the experience. Design must put such built environment in proper landscape environment.

- **Design in Flexibility:** New development near and around transit station take very little time. New development needs to be flexible enough to respond to future change in use, lifestyle, and demographics. Flexibility must be evident in the use of property, public spaces, and infrastructure. Integration of any new modes with the station area requires more space for loading, unloading, transfer, integration, parking, traffic management, etc.

M_2OD is a capital intensive investment in transit area to cater needs of both present and future travel demands. Hence, such design and its outcome must be economically viable, well managed, and well maintained. Finally, M_2OD for transit area defines neighborhood character in design and provide multi modal mobility friendly environment.

13.0 Concluding Remarks

Good transit station demands transparent, functional simplicity and needs to be integrated well into the urban fabric. “Station that looks like Station” must have architectural expression, vertical articulation to understand space and use it more easily, sculptural qualities to express local climatic forces, steel-glass structure into urban forests and gardens, etc. *(Moffat 2004).* Multi modal transit station is an example of integration of ‘engineering art’ into ‘ building design’ to create iconic new forms which requires highly engineered environment to get accessed by human, mechanical and vehicular system. In Delhi, multi modal transit stations have been rediscovered as ‘ urban identity’ and ‘sense of place’ which demands a package of measures designed to improve the integration and attractiveness of public transport network, including better and quality focused accessibility to these services.

Accessibility to multi modal transit station can be improved by (i) develop a balance & successful multi modal access plan; (ii) reducing penalties of interchange through efficient operation; (iii) strategies to achieve seamless journey through better physical design; (iv) commercial exploitation opportunities at stations; (v) agreed minimum standards of passenger facilities; (vi) time competitive & cost effective transit feeder services; (vii)safety & security of both transit users and operators, etc. In fact, accessibility to multi modal transit station is a matter of infrastructure design, management, land use & development, fare policies & traffic restraint measures, environmental quality, local economical activities, etc. Hence, role and responsibilities of transit operators, facilitators’ and users are crucial to extend better accessibility. It is also worthy to adopt ‘learning experience’ and ‘art of modern transit station design’ in international perspective at city level for better and easy mobility and linkages among commuter buses, rail, high speed train service like metro, exclusive land based service like BRTS, other motorized and non-motorized transport at multi modal transit station.

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SUSTAINABILITY IN ROAD TRANSPORT: AN INTEGRATED LIFE CYCLE ANALYSIS FOR ESTIMATING EMISSIONS

M. H. Rahman[1*], H. C. Chin[2] and M. M. Haque[3]

[1]Research Scholar, Email: habibur@nus.edu.sg;
[2]Associate Professor, Email: cvechc@nus.edu.sg;
[3]Research Fellow, Email: mmh@nus.edu.sg
Department of Civil Engineering, National University of Singapore, Singapore-117576
[1,2&3]Telephone: +65-65162255; Fax: +65-67779592

Abstract: Despite of a significant contribution of transport sector in the global economy and society, it is one of the largest sources of global energy consumption, green house gas emissions and environmental pollutions. A complete look onto the whole life cycle environmental inventory of this sector will be helpful to generate a holistic understanding of contributory factors causing emissions. Previous studies were mainly based on segmental views which mostly compare environmental impacts of different modes of transport, but very few consider impacts other than the operational phase. Ignoring the impacts of non-operational phases, e.g., manufacture, construction, maintenance, may not accurately reflect total contributions on emissions. Moreover an integrated study for all motorized modes of road transport is also needed to achieve a holistic estimation. The objective of this study is to develop a component based life cycle inventory model which considers impacts of both operational and non-operational phases of the whole life as well as different transport modes. In particular, the whole life cycle of road transport has been segmented into vehicle, infrastructure, fuel and operational components and inventories have been conducted on each component. The inventory model has been demonstrated using the road transport of Singapore. Results show that total life cycle green house gas emissions from the road transport sector of Singapore is 7.8 million tons per year, among which operational phase and non-operational phases contribute about 55% and about 45%, respectively. Total amount of criteria air pollutants are 46, 8.5, 33.6, 13.6 and 2.6 thousand tons per year for carbon monoxide, sulfur dioxide, nitrogen oxides, volatile organic compounds and particulate matter, respectively. From the findings, it can be deduced that stringent government policies on emission control measures have a significant impact on reducing environmental pollutions. In combating global warming and environmental pollutions the promotion of public transport over private modes is an effective sustainable policy.

Keywords: Life Cycle Analysis, Environmental Pollution, Global Warming, Road Transport Emissions

1 Introduction

Global civil development is directly dependent on the performance of transport sector. Although mobility is the key parameter of economic, social as well as human development, its adverse impact on our ecological environment is also enormous. Road transport is one of the largest sources of energy consumption, green house gas emissions and environmental pollutions (Mayeres, 1996). To clearly understand and address the environmental impacts from this sector, it is important to quantify the impacts from the entire life cycle considering both operational and non-operational phases. An integrated approach is more appropriate in identifying significant contributors of emissions and thus will be helpful in designing guidelines and policies to combat negative environmental impacts from the road transport.

In the environmental inventory of transport sector, most of the studies focused on the operational phase of either passenger transport (e.g., Small, 1995, MacLean, 1998), public transport (e.g., Small, 1995) or freight transport (e.g., Stodolsky, 1998). Some studies have also been focused on individual components of non-operational phases. For example, Cohen et al. (2003) have conducted inventory on fuels used in transport and Lave (1977) has studied transport infrastructures. However, non-operational phases like manufacture, construction, and maintenance were not well addressed in estimating environmental impacts. Moreover, considerations on different modes of transport and

[1] Corresponding author

different life-cycle phases in a same life cycle analysis were also not given a fuller attention. Therefore an integrated study considering different phases of life cycle as well as different modes of transport is needed to achieve a holistic understanding on the contributing factors of emissions.

The fast-emerging global environmental concerns lead the global leaders to employ alternative technologies and options in reducing environmental impacts from the road transport sector. Successful evaluations of the alternatives require a holistic look on the inventory of the whole life-cycle of transport system. The objective of this study is to develop a component based life cycle inventory model for estimating emissions from the road transport sector. To achieve this, the whole life cycle of road transport has been segmented into vehicle, infrastructure, fuel and operational components and inventories have been conducted on each of these components for different modes of transport. The model has been demonstrated for the road transport sector of Singapore. Note that Singapore is a densely populated (4.8 million) and city state island country with an area of about 707 sq. kilometers, and about 3,325 km of road.

2 Methodology

2.1 Approaches in Life Cycle Assessment

The whole life cycle of the road transport sector has been segmented into four phases: vehicle, infrastructure, fuel, and operation. The vehicle, fuel, and infrastructure phases are complex with many processes as well as many resource inputs and environmental outputs. Life-cycle assessment (LCA) is the most comprehensive tool for dealing with these complexities and for quantifying environmental effects of these phases. The basic framework (ISO 14040, 1997) of the life cycle assessment and its applications is shown in figure 1.

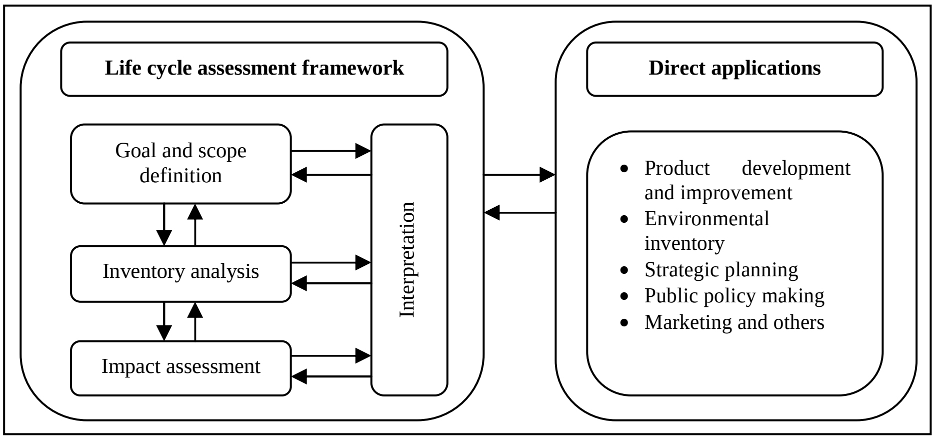


Figure 1 Framework and application of life cycle assessment

There are three approaches of LCA analysis: 1) process, 2) input-output, and 3) hybrid. The 'process' LCA approach identifies and quantifies resource inputs and environmental outputs at each life-cycle stage based on unit process modeling and mass balance calculations (Curran, 1996). Although 'process' LCA enables specific analyses more visual, it is usually time and cost intensive due to heavy data requirements, especially when primary, secondary, tertiary or higher level tiers of inputs are attempted to be included in the model. To overcome these limitations, an alternative LCA model has been emanated which is economic input-output based LCA (Leontief, 1936). The 'input-output' based LCA incorporates environmental impact data to economic flow databases. These databases are usually maintained by the statistical bodies of any nation or economy. This well-established econometric LCA model quantifies interdependencies among the different sectors by effectively mapping the economic interactions along the supply chain of any product or service in that particular economy. Emissions and associated impacts are then assigned to different sectors. In this study, the EIO-LCA software (EIO-LCA, 2008) has been used to calculate the environmental inventory of different products and processes. A hybrid LCA model combines the advantages of both process LCA and economic input-output LCA. In this study, the inventory from vehicle and fuel component is conducted using 'input-output' LCA approach whereas inventories from operation and infrastructure (except lighting) components have been computed following 'process' and 'hybrid' LCA approach, respectively. The inventory of the lighting sub-component in the infrastructure component is obtained by using 'process' approach. The estimation approaches will be briefly addressed in subsequent sections.

2.2 Proposed Model for Life Cycle Analysis of Road Transport

In order to assess the total environmental impact from road transport, the total transport life cycle has been divided into four phases: vehicle (which constitutes vehicle manufacture, tire production, vehicle maintenance and insurance), infrastructure (which includes construction and maintenance of road infrastructures, parking facilities, associated other infrastructure facilities and lighting operation), fuel (which includes fuel production) and operation (which includes the environmental inventory during the operational phase of vehicular travels). Due to uncertainties in the after-use dumping and recycling of vehicles, the end-use phase is not considered in this study. In addition, only operation phase of lighting has been considered due to insignificant contribution from non-operational phases of lighting facilities. The emission types considered in the inventory include both green house gases and criteria air pollutants. According to Land Transport Authority (LTA), vehicles are classified as motorcycles and scooters (MC), car and taxis, light goods vehicles (LGV) ($\leq$ 3.5 ton), heavy goods vehicles (HGV) (> 3.5 ton) and buses (LTA, 2008). The scope of the analysis in terms of object boundaries of LCA in road transport has been presented in figure 2. The phases for the inventory are presented in the dashed border.

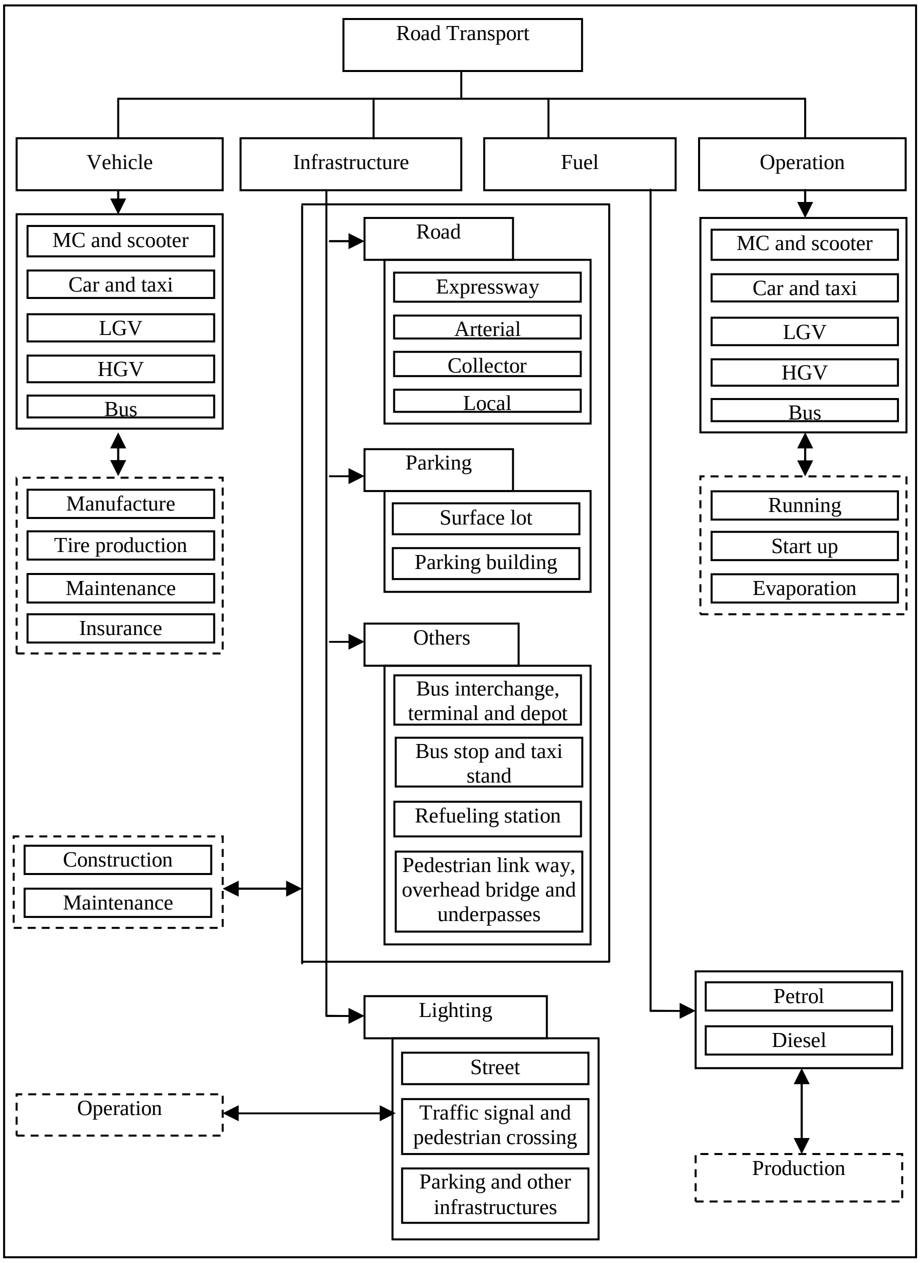


Figure 2 Component based LCA framework of road transport

2.3 Mathematical Equations for Estimating Emissions

To illustrate computation methodology of the total yearly inventory, let f_{ev} be the emission factor per VKT (vehicle kilometers travelled) of travel (summation of inventory from all life cycle components, obtained using methodologies described in section 2.4) for emission type e from vehicle type v. The total yearly amount of emission of emission type e from all vehicle types can be obtained from the following relationship:

$$\mathbf{X} = \mathbf{FT} \qquad (1)$$

where $\mathbf{X}$ is the $(E\times 1)$ emission impact matrix, and E is the number of emission types given by

$$\mathbf{X} = \begin{bmatrix} \vdots \\ x_{Y_e} \\ \vdots \end{bmatrix} \qquad (2)$$

The elements x_{Y_e} of $\mathbf{X}$ matrix represent the total yearly amount of emission type e generated from road transport. $\mathbf{F}$ is the $(E\times V)$ emission factor matrix; where E and V are the number of emission types and number of vehicle types, respectively. The elements of $\mathbf{F}$ matrix f_{ev} denote the per VKT emission factor in g/km of the emission type e from vehicle type v. $\mathbf{T}$ is the $(V\times 1)$ VKT matrix, whose elements t_v denote the total annual VKT for vehicle type v. $\mathbf{F}$ and $\mathbf{T}$ are given by

$$\mathbf{F} = \begin{bmatrix} \cdot & \cdot & \cdot \\ \cdot & \frac{f_{ev}}{10^6} & \cdot \\ \cdot & \cdot & \cdot \end{bmatrix} \text{ and } \mathbf{T} = \begin{bmatrix} \cdot \\ t_v \\ \cdot \end{bmatrix} \qquad (3a\text{-}b)$$

The matrix **T** is obtained from the following relationship:

$$\mathbf{T} = \mathbf{PK} \qquad (4)$$

where **P** is a diagonal matrix with diagonals p_v representing the population of vehicle type v and **K** is the $(V \times 1)$ average annual kilometers matrix with elements k_v representing the average annual kilometers travelled by vehicle category v.

2.4 Estimating Emission Factors for Life Cycle Components

The whole environmental inventory has been performed for data and economic year 2008. Emission factors have been obtained from the inventory of each life cycle component for all modes of road transport. Finally, these emission factors have been summed up to obtain the life cycle emission factor (f_{ev}). The subsequent sections discuss estimation of emission factors from vehicle, infrastructure, fuel, and operational components respectively.

2.4.1 Vehicle Component

For each category of vehicle the life cycle inventory has been performed by dividing it into four sub-components: manufacture, tire production, maintenance and insurance. Environmental inventory has been conducted for whole vehicle life. Afterwards based on life period of the vehicle the inventories have been normalized to per vehicle year. In order to reflect the vehicle usage policy of Singapore (VQS-CoE), the lifetime of each of motor cycle, car, LGV and HGV have been assumed to be 10 years and 12 years lifetime has been assumed for each bus. The vehicles used in Singapore are imported from different economies of the world. Therefore computation of the manufacturing inventory should be based on the economy where the vehicle was manufactured. The global market share for the motor vehicle production of different economies (OICA, 2009) shows that China shares the maximum of twenty three percent of global motor vehicle production followed by Japan, USA, Germany and other economies. Due to poor availability of the Japanese environmental data, the EIO-LCA database currently does not include Japanese economy. Therefore, China, USA and Germany have been considered in this study for the lifecycle environmental inventory of vehicle manufacture and weighted based on their respective global vehicle production share. In the EIO-LCA model, the accuracy of inventory from a particular economy increases with increase in number of sectors in input-output table. Therefore, the combined weight (based on global production share and number of sectors in EIO-LCA) of economies is used in computing inventory from vehicle manufacture. For each category of vehicle the economic cost of vehicle manufacture, tire production, maintenance and insurance based on economic year 2008 has been inputted into the EIO-LCA and the associated environmental inventories have been obtained. After obtaining the total life cycle (manufacture, tire production, maintenance and insurance) inventory of a particular vehicle type v, the emission factor, $f_{ev,vc}$ per VKT (from vehicle component) for emission type e and vehicle type v is computed as:

$$f_{ev,vc} = \frac{x_{ev,life}}{Life_v \times k_v} \qquad (5)$$

where $x_{ev,life}$ = total amount of emission type e from whole life of a vehicle type v, $Life_v =$ life of vehicle type v in years.

2.4.2 Infrastructure Component

Road transport infrastructure component has been divided into three sub-components: (1) road construction and maintenance, (2) construction and maintenance of parking and other facilities and (3) lighting operation.

2.4.2.1 Road Construction and Maintenance

The roads in Singapore are classified as expressway, arterial, collector and local. The length and width of each type of roads are obtained from Land Transport Authority (LTA, 2008). The life of pavement has been assumed as 45 years with maintenance interval after initial construction (years) as

15-10-10 (AASHTO, 1993; Huang, 2004). The flexible pavement has two layers: sub base layers (compacted soil and aggregate) and wearing course layer (asphalt). In the inventory analysis, maintenance represents the replacement of wearing course. The thickness of layers is obtained from AASHTO specifications for roadway design. The life cycle assessment tool for flexible pavement PaLATE (PaLATE, 2004) has been used in this analysis in which total inventory is a function of length, width and thickness of wearing course and sub-base layers and materials. Inventory per year from road construction and maintenance is obtained from life cycle inventory for construction and maintenance of roads by dividing with their life period.

2.4.2.2 Construction and Maintenance of Parking and Other Facilities

An indirect approach has been undertaken to estimate the area of parking and other infrastructural facilities (other than roads) for the road transport. This approach divides parking and other infrastructural facilities (other than roads) into two sub-categories: (1) Sub-category 1: Construction of parking buildings, passenger facilities at bus interchanges, pedestrian overhead bridges and underpasses, and refueling stations; and (2) Sub-category 2: Construction of surface lots for parking, bus and truck terminals, bus interchanges, bus depots, bus stops, taxi stands and pedestrian covered link ways. Infrastructures of sub-category 1 are assumed as similar to typical concrete structure buildings and life cycle environmental inventory has been performed on account of total floor area of these concrete buildings. Infrastructures of sub-category 2 are assumed to be similar to road pavements and life cycle environmental inventory is performed on account of total area of these facilities. At first, net vehicle footprints of each vehicle type *v* have been computed from its net dimensions. An increase of 80% has been made on net vehicle footprint (based on AASHTO, 2004): 30% increase to get the parking footprint from the net vehicle footprint and another 50% increase to facilitate vehicle maneuvering for parking. Assuming that, every motorcycle and private car will consume double parking daily (a night-time origin parking and a day-time trip destination parking), their parking area have been doubled. A further 30% increase for each category of vehicles has been assumed since the available parking area or actual parking capacity is 30% more than the current usage. Assumed parking location by vehicle type is: Motorcycles (surface lots), cars and taxis (50% at multi-storied car parks and 50% at surface lots), goods vehicles (surface lots and truck terminals), buses (bus terminals, bus interchanges and bus depots). Total area of bus stops, taxi stands and pedestrian covered link ways and total area of passenger facilities at the bus interchanges, pedestrian overhead bridges and underpasses is obtained from LTA (LTA, 2008). Thus, total pavement equivalent area and total multi-storied car-park building floor equivalent area is estimated. Life of multi-storied car-park buildings has been assumed to be 50 years (Guggemos, 2005) and assumed reconstruction interval of surface lots or pavement equivalent areas is 15 years (Huang, 2004). LCA of the multi-storied car-park building equivalent areas are computed as the concrete structure based on floor area estimates (Guggemos, 2005) using 'hybrid' approach and LCA of pavement equivalent areas of surface lots are calculated by PaLATE using 'process' approach. Finally, the life cycle inventory of these parking areas is normalized to per year inventory by dividing by their respective life.

2.4.2.3 Lighting Operation (Street, Traffic, Parking and Others)

Road transport lighting has been classified into three classes: (1) street lighting, (2) traffic signals and pedestrian crossing lighting and (3) lighting for parking and other facilities. The street lighting pattern, spacing and bulb type, total number of traffic and pedestrian crossing lights and parking lighting data is obtained from LTA (LTA, 2008). Street and parking lights are assumed to operate 12 hours daily whereas traffic and pedestrian crossing lights operate for 24 hours. Based on bulb wattage used in each lighting type, total yearly electricity consumption has been computed for each lighting type and then summed up. The yearly inventory from lighting is computed by multiplying total yearly electricity consumed (KWh) by lighting by the amount of inventory per KWh of electricity production (Deru, 2007).

2.4.2.4 Summary on Infrastructure Component

Total yearly inventory from infrastructure component is obtained by summing up the yearly inventories from all of its sub-components (road, parking and other facilities, lighting). In order to estimate the infrastructure inventory by vehicle category, (1) the inventory from road construction and maintenance have been distributed to vehicle categories by their respective damage shares, which is the cumulative effect of damage factors (Huang, 2004), total numbers of vehicles and average annual VKT by vehicle type; and (2) the inventory from road lighting, parking and other infrastructure facilities have been distributed to vehicle categories by their respective cumulative effect of vehicle footprint area, total numbers of vehicles and average annual VKT. Finally, emission factor, $f_{ev,ic}$ per VKT (from infrastructure component) for emission type *e* and vehicle type *v* is computed as:

$$f_{ev,ic} = \frac{x_{ev,Y,ic}}{p_v \times k_v} \quad (6)$$

where $x_{ev,Y,ic}$ = total amount of emission type *e* per year from infrastructure component for all vehicles of vehicle type *v*.

2.4.3 Fuel Component

The life cycle inventory from fuel component has been computed using input-output LCA. In Singapore, petrol is used as the primary fuel for motorcycles and cars and diesel is the primary fuel for LGV, HGV and buses. The production cost (EIA, 2010) of each fuel type is taken as input into the EIO-LCA model and corresponding environmental inventory has been obtained. Yearly inventory from fuel component by vehicle type *v* using fuel type *f* is calculated as:

$$x_{ev,Y,fc} = FE_{v,f} \times p_{v,f} \times k_{v,f} \times x_{e,f} \quad (7)$$

where $x_{ev,Y,fc}$ = total amount of emission type *e* per year from fuel component for all vehicles of vehicle type *v* using fuel type *f*; $FE_{v,f}$ = fuel efficiency of vehicle type *v* using fuel type *f* (liter/km); $p_{v,f}$ = population of vehicle type *v* using fuel type *f*; $k_{v,f}$ = average annual kilometers travelled by each of vehicle type *v* using fuel type *f*; $x_{e,f}$ = amount of emission type *e* per liter of production of fuel type *f* and *f* = fuel type (petrol and diesel). Emission factor per VKT (from fuel component) for emission type *e* and vehicle type *v* is calculated as:

$$f_{ev,fc} = \frac{x_{ev,Y,fc}}{p_{v,f} \times k_{vf}} = FE_{v,f} \times x_{e,f} \quad (8)$$

2.4.4 Operation Component

The regulations on vehicle emission control in Singapore are tabulated in table 1.

Table 1 Vehicle emission control regulations in Singapore

Emission control regulations	Effective from
Mandatory periodic inspection	na
Unleaded petrol and diesel	1 July, 1998
Sulfur content ≤ 0.05% or 500 ppm	1 March, 1999
Smoke emission test: Chassis dynamometer smoke test (CDST) instead of free acceleration smoke test	1 September, 2000
Euro II (1996) emission standard for all new vehicles	1 January, 2001
ULSD: Sulfur content ≤ 0.005% or 50 ppm	1 December, 2005
Euro IV (2005) emission standard for all new vehicles	3 September, 2006

Based on the emission standard followed in Singapore, the vehicle operational emission inventory model COPERT 4 (EEA, 2009) is considered as the most suitable and accurate model for Singapore context and therefore has been used in this study which estimates vehicle operational inventory based on vehicle emission standard, fuel standard, vehicle speed and weather data. The model inputs are: (1) weather data: minimum and maximum temperature: 77°F and 90°F respectively; specific humidity: 160 grains/lb (for average temperature of 85°F and an average relative humidity of 80%); (2) vehicle speed: average; (3) vehicle population by vehicle type, fuel type used, average annual VKT by vehicle type; (4) vehicle emission standard: Euro II; and (5) petrol and diesel standard: Euro II; fuel sulfur content: 50 ppm. Total yearly emission by vehicle and emission types is obtained as output. Emission factor $f_{ev,oc}$ per VKT (from operation component) for emission type *e* and vehicle type *v* is calculated as:

$$f_{ev,oc} = \frac{x_{ev,Y,oc}}{p_v \times k_v} \quad (9)$$

where $x_{evY,oc}$ = total amount of emission type *e* per year from operation component of all vehicles of vehicle type *v*.

2.4.5 Summary on Emission Factors

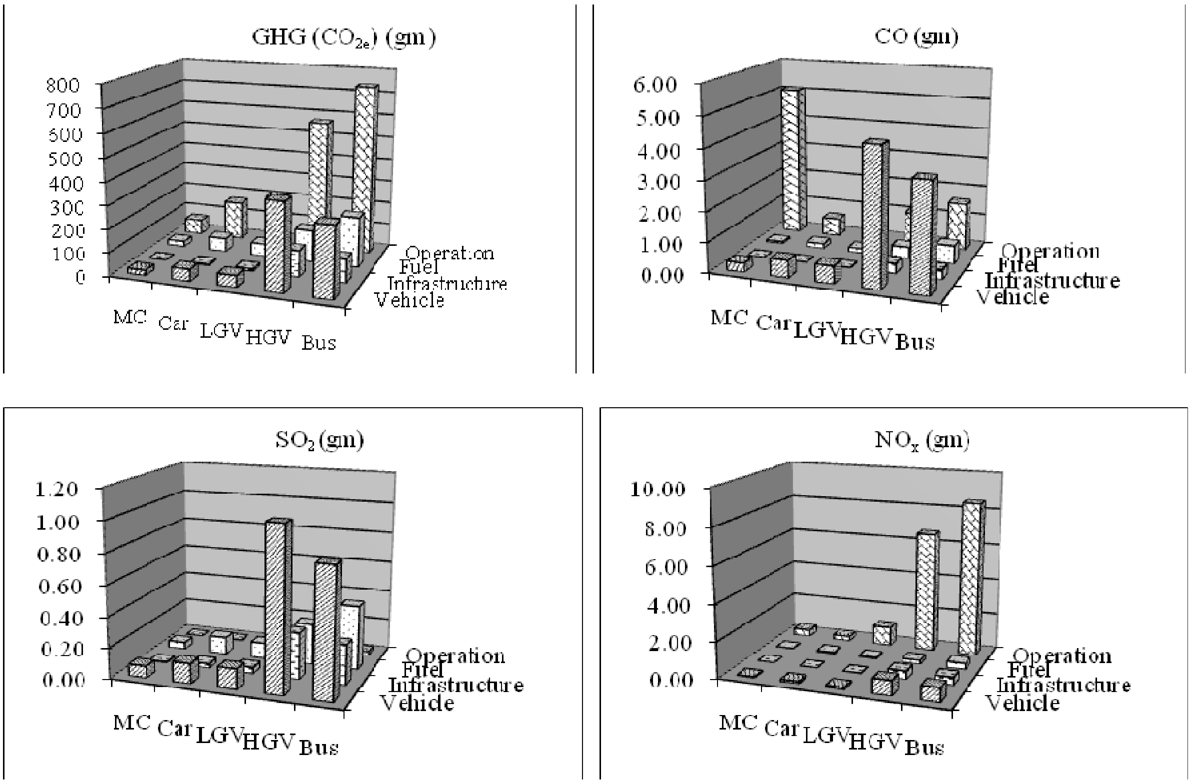
Total emission factor f_{ev} per VKT (from all life cycle phases) for emission type *e* and vehicle type *v* is calculated as:

$$f_{ev} = \sum_c f_{ev,c} \quad (10)$$

where $f_{ev,c}$= emission factor per VKT for emission type *e* and vehicle type *v* from life cycle component *c* and *c* = life cycle components (i.e., vehicle, infrastructure, fuel and operation). Finally, the total yearly amount of emission for each emission type *e* from all vehicle types *v* is obtained from equation (1).

3 Results and Discussion

Figure 3 presents the emission factor per VKT of travel for different vehicle types. Here carbon dioxide equivalent (CO_{2e}), which is obtained from weighted global warming factor of carbon dioxide (CO_2), methane (CH_4), ozone (O_3) and nitrous oxide (N_2O) is representative of all green house gases and carbon monoxide (CO), sulfur dioxide (SO_2), nitrogen oxides (NO_x), volatile organic compounds (VOC) and particulate matter (PM_{10}) are criteria air pollutants. For a particular vehicle type *v*, the emission factor of emission type *e* is presented separately for different life cycle components, the sum of which determines the total life cycle emission factor per VKT (f_{ev}). For example, in year 2008, each kilometer of car travel was associated with 287 gm of CO_{2e} emission, of which 168 gm was from operational phase and 52, 8 and 59 gm were from vehicle, infrastructure and fuel components, respectively.



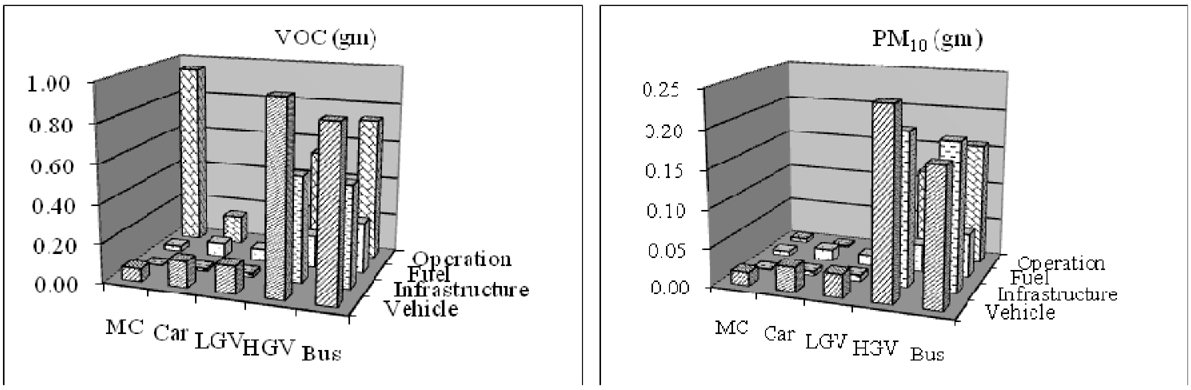


Figure 3 Pollutant emissions (by mode and LCA phase of road transport) per VKT

The vehicle component of the emission factor is determined by the amount of emission generating materials and processes involved in manufacture, maintenance etc. As noticed from figure 3, emissions from heavy vehicles are higher as more amounts of materials and processes are involved in their manufacture, tire production, maintenance and insurance. Green house gas (CO_{2e}) emission factors for vehicle component of MC, car, LGV, HGV and bus are 23, 52, 51, 379 and 305 gm, respectively. The emissions of air pollutants also exhibit variations in a similar fashion.
The emissions associated with infrastructure component are primarily led by damages of different vehicles on road infrastructures. Hence it is not surprising that heavy vehicles lead higher emission factors (figure 3). The CO_{2e} emission factors for this component are 2, 8, 10, 113 and 103 gm for MC, car, LGV, HGV and bus, respectively. PM_{10} emissions are higher for this component as more particulate matter in the form of dusts is associated with the construction and maintenance of roads and other infrastructure facilities.
For fuel component, the amount of emission is led by two factors: fuel type and vehicle fuel efficiency. In this component the emissions are generated in the processes and materials involved in the production of fuels. The CO_{2e} emission factors for this component for MC, car, LGV, HGV and bus are 24, 59, 54, 169 and 208 gm, respectively.
The emissions from the operational component is the most significant, as it is directly associated with the road environment. The principal factors affecting the emission from this component are vehicle fuel efficiency, fuel type and standard, vehicle emission standard, and emission control measures (e.g., inspection and maintenance, use of catalytic converters etc.). The CO_{2e} emission factors for the operational phase are 65, 167, 215, 567 and 737 gm for MC, car, LGV, HGV and bus, respectively. The CO_{2e} emission factors are mainly determined by fuel type (carbon content of the fuel) and vehicle fuel efficiency. For example, hybrid cars and those cars fuelled by natural gas produce CO_{2e} up to 25% less than cars running on petrol. However, the usage of alternative fuels in all modes of transport in Singapore is still at the initial stage. Recently, only 0.005% of buses are CNG-driven; about 1% of cars use alternative fuels (hybrid and bi-fuel CNG). However about 8% of taxis are now driven by bi-fuel CNG. Due to less hauling capacity, the usage of these alternative fuels in heavy goods vehicles is almost negligible. Singapore is looking forward to implement more energy efficient fuel and vehicle technologies in near future.

The use of catalytic converters significantly decreases the CO, NO_x and other hydrocarbon emissions. These emission factors were about 40% - 70% higher, when catalytic converters were not in place. Due to dimensional inconvenience, its usage on motorcycles is limited. This may lead in higher operational emission factors for CO, NO_x and VOC in case of motorcycles (figure 3). The improvement of vehicle emission standards has also significantly reduced the emissions from vehicles. For example, adoption of Euro II from Euro I standard has resulted in 70% and 40% reduction respective in CO and PM_{10} emissions and further stringent standard, i.e., Euro IV standard results in 50% and 60% reductions on those gases.

SO_2 emissions have drastically reduced due to stringent sulfur content regulations in both petrol and diesel fuels. For example, currently a bus using 50 ppm standard ULSD (ultra low sulfur diesel) emits only 0.02 gm SO_2 per km, while it was emitting approximately 10 times SO_2 when the fuel it used followed 500 ppm sulfur standard (before 2005).

Results imply that incorporation of continuous stringent emission control policies lead Singapore to reduce the emissions from the operational component significantly over past few years. The gradual implementation of stricter vehicle standards (see table 1) has reduced the operational emission factors.

The average car occupancy in Singapore is 1.7 while an average bus accommodates passengers in the range of 85 to 143 (Menon and Kuang, 2008), which is 50 to 85 times higher than that for an average car. An average bus emits only 4, 3, 4 and 5 times higher CO_{2e}, CO, SO_2 and VOC emissions, respectively, than that of an average car in the operational phase and these multipliers are 5, 4, 5 and 6, respectively when the whole life cycle emissions are considered (figure 3). However when emission per passenger km is considered, emission from buses are approximately 10 times lower than that of a private car. For a city state country like Singapore, good public transport is the most promising solution not only to combat the increasing problems of traffic congestion, but also contribute in lowering global green house gases and environmental pollution. Management policies are looking forward to a more sustainable transport system mainly by further promoting public transport usage in Singapore (Haque et al., 2010).

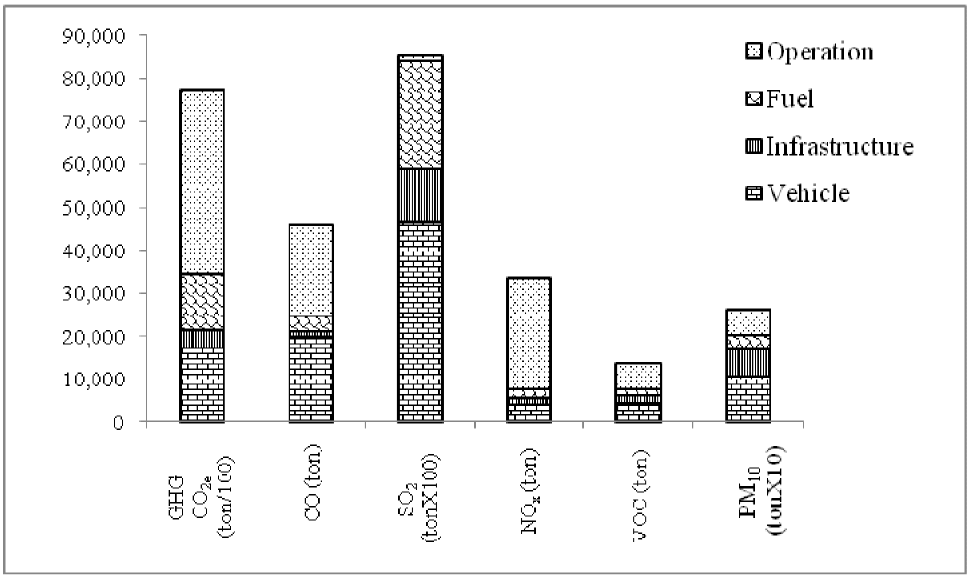


Figure 4 GHG and air pollutant emissions for year 2008

The total life cycle emissions from the road transport of Singapore in year 2008 are presented in figure 4. For year 2008, the total life cycle CO_{2e} emissions from the road transport sector of Singapore is 7.8 million tons, among which operational phase and non-operational phases contribute about 55% and about 45%, respectively. Total amount of criteria air pollutants are 46, 8.5, 33.6, 13.6 and 2.6 thousand tons for CO, SO_2, NO_x, VOC and PM_{10}, respectively. In almost every emission type (except SO_2 and PM_{10}) the operational component is the dominating contributor in emissions. The drastic reduction in the operational SO_2 emission (figure 4) has been obtained by stringent sulfur content regulation in fuels. It is notable that, although the whole life cycle inventory for road transport of Singapore has been conducted in this study, all pollutants and gases are not emitted in Singapore. The operational inventory is fully emitted from the road environment, whereas part of emissions from other non-operational life-cycles components is not directly emitted from this road environment. For example, for the road construction and maintenance there are both on-site emissions and off-site emissions, which is associated with the production and processes associated with materials used in road construction and maintenance.

4 Conclusions

This study has been aimed to develop an integrated life cycle inventory of road transport which considers both operational and non-operational phases as well as takes into account different modes of transport. The model has been demonstrated for the road transport sector of Singapore. It has been found that total life cycle green house gas emission from the road transport sector of Singapore is 7.8 million tons per year. Importantly non-operational phases contribute a significant 45% of those emissions. While CO, NO_x and VOC criteria air pollutants represent more pollution during operational phase, the corresponding proportions for SO_2 and PM_{10} are higher during non-operational phases. Stringent government policies and regulations on fuel and vehicle technologies and standards have been found to yield significant lower emissions from vehicles. The utilization and promotion of public transport modes is an effective policy in combating negative environmental impacts and will be helpful to achieve a sustainable transport system.

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About the Authors

M. H. RAHMAN, B. Engg. (Civil), BUET; Currently pursuing his Ph.D. at the Department of Civil Engineering at National University of Singapore. His research interests are sustainable transport, intelligent transport systems, and traffic safety analysis.

H. C. CHIN, B. Engg. and M. Engg. (Civil), NUS; Ph.D. (Transportation), Southampton; Currently associate professor of Civil Engineering at National University of Singapore. His research interests are in the areas of sustainable transport, urban sustainability, transport system modeling, traffic safety analysis, and public transport operations.

M. M. HAQUE, B. Engg. (Civil), BUET; Ph.D. (Civil), NUS; Currently research fellow at the Department of Civil Engineering at National University of Singapore. His research interests are in the areas of transport system modeling, traffic safety analysis, sustainable transport, and smart technologies.

SUSTAINABILITY OF NATIONAL HIGHWAY SYSTEM IN INDIA : LESSONS LEARNT

Sudeshna Mitra

Assistant Professor
Civil Engineering Department
Indian Institute to Technology Kharagpur
Kharagpur, West Bengal, India
Phone: +91-3222-283400
Fax: +91-3222-282254
Email: sudeshna@civil.iitkgp.ernet.in

ABSTRACT

India is going through a transitional phase of economic growth and development in recent years. The Government of India has invested a significant amount of fund for road development throughout India. A significant percentage of these roads are high speed roadways. While there is a false notion about the functional characteristics of these highways— classifying them as "uninterrupted flow facilities", often these highways are what is known as multilane highway elsewhere. Thus, the access controls of these highways are close to (or even worse than) multilane highways and in no way near to "true uninterrupted flow facilities". This has implications both on mobility and safety— the two very important indicators of sustainability. The situation is even more difficult since the needs of road users in *mixed* traffic are very different and often conflicting. The obvious result is that vulnerable road users (VRU) like bicycles and pedestrians are often at high risk due primarily to the absence of adequate facilities, or due to ill planning or design of such facilities. However, a recent case study conducted near Kharagpur, over a 100 km stretch of National Highway (NH) provided somewhat interesting insights to this problem. There are strong evidences that 3Es "engineering", "education" and "enforcement" have their distinct role in making the study stretch of NH to be unsafe and non-sustainable in its present form to such an extent that noble cause of road development is questioned. Some areas of improvements are identified with appropriate countermeasures so that concerned authorities adopt a plan for safe and sustainable NH.

Key words: Safety and sustainability, mixed traffic, vulnerable road users

1. INTRODUCTION AND BACKGROUND

India is going through a transitional phase of economic growth and development in recent years. The Government of India has allocated a significant amount of fund for road development throughout India through programs like National Highway Development Program (NHDP), Pradhan Mantri's Gram Sadak Yojona (PMGSY), which enhance connectivity with cities and villages. Ministry of Road Transport and Highways (MoRTH) is responsible for road development and maintenance in general and National Highway Authority of India (NHAI), an agency under the Government of India and an agency of MORTH, is responsible specifically for National Highway (NH) development and maintenance in India. The NHs are high speed facilities meant primarily for mobility and they connect major cities and states in India. India is building such high speed roads under NHDP since year 2000. While roads are being built for economic enhancement of the country, India is loosing more than 3% of National GDP each year from road traffic accidents (Sundar committee report, 2007). NHAI and MORTH recognize the importance of improving road safety along NHAI roads. A report by Asian Development Bank (World Bank, 1995) identified that the traffic accidents and deaths along National Highways in India have been high when compared to similar roads in the Asian countries. The numbers of traffic accidents and deaths have been growing in the recent years, with the increase in the number of vehicles on the roads. A report by MoRTH (Sundar Committee report, 2007) indicated a steady increase in road traffic accidents from 391449 in 2000 to 460920 in 2006. During the same time fatal accidents increased from 78911 in 2000 to 105749 in 2006. While the number of vehicle ownership has steadily increased during this time period, it is shocking to see a jump of total number of road accidents from 439255 in 2005 to 460920 in 2006, almost a 5% increase in a single year. During the same time, fatality increased by more than 11%, from 94968 in 2005 to 105749 in 2006. With many more new

NHs and four-laning of the major National roads, the number of traffic accidents and deaths are expected to further increase due to reasons such as lack of good engineering, improper enforcements and lack of education of road users. This has implication on sustainability of the NH since a sustainable transportation system must meet the mobility and accessibility needs by providing safe and environmentally friendly transportation. However, the elevated crash statistics on NHs question the sustainability of the system even though they are meant to enhance mobility of passenger and freight movement through road transportation throughout the country. A critical analysis of the current state of safety performance of NHs reveals some of the unique challenges that Indian National Highway transportation are facing and they need to be addressed before we expect any better safety performance. In the following sections these are described in detail with respect to a case study.

2. ROAD ENGINEERING

Under NHDP, India launched a massive program of highway upgrades, in which the main north-south (Kashmir to Kanyakumari) and east-west (Gujrat to Assam) connecting corridors and highways connecting the four metropolitan cities, i.e Delhi, Kolkata, Chennai, Mumbai have been fully paved and widened into 4-lane highways. Other NHs connecting major cities are mostly 2-lane highways. While a few busy stretches of NHs

are access controlled expressways, major portion of NHs can be classified as high speed multilane facilities with minimum or no control of access. These facilities often have intersections with state highways and other major roads. NHAI guidelines although specified the need for service lanes in built up areas and grade separated interchanges in urban areas for better management of access to NHs, most projects have often failed to include such facilities for reasons such as inadequate land acquisition and minimizing overall cost. Land acquisition for NHs in India would remain a major challenge since historically development of most built up areas were concentrated along the major highways and there was hardly any concept of clear zone in such areas. Consequently any lateral expansion at present day requires major political pressure and face opposition from stake holders. However, in a haste where the country is more interested in adding capacity and expanding its roadway network, the safety is often overlooked as it has been the case in almost all nations' history. Moreover, the problem is far more serious in India compared to other developed countries due to the mixed traffic condition with high percentage of vulnerable road users (VRUs) such as motorcycles, bicycles and pedestrians whose access must be controlled for safer operations.

2.1 Access management in mixed traffic

Access management/control is one of the critical elements of geometric design and is related to the management of the interference with through traffic (AASHTO). If access to a highway is managed, interference due to vehicles', pedestrians', bicyclists' entrance and exit could be minimized and they would get designated entrance and exit suitable for traffic and land use needs. As pointed out by AASHTO, the absence of access management encourages roadside businesses to develop haphazardly, which is happening in India in present time—thereby reducing capacity and increasing crash potential. Also, considerable portions of NHs traverse through suburban and rural locations, where a significant share of population is from low-income group. As a result, share of bicycle is very high since it is often an only form of private transportation other than walking a family can afford in rural and suburban regions. Rural population is also located in a scattered manner over a large geographical area. As a results bicycles are found not only on village roads (where motorized vehicles are not significant), but also on NHs, since they often provide the only direct connection between populated areas and there are no alternative roads connecting their origins and destination. Due to such reasons, infrastructure design that works for countries with homogeneous traffic mix may not be suitable for mixed traffic conditions with very large share of non-motorized traffic (NMT). Road design in India must meet the needs and requirements of motorized automobiles as well as NMTs for safer operations. This is undoubtedly a challenge since the needs of the two groups of users are very different and often conflicting. As rightly pointed out by Tiwari (1999) for safer movements, the NMTs need to be provided with a safe infrastructure, either physically segregated road space from motorized traffic, or the speed of the motorized traffic must be reduced. Now the second option is meaningless in case of NHs since NHs are meant to provide mobility, the only option left in the context of NHs is to provide separate facility for NMTs. The question might rise as to why NMTs like bicyclists and pedestrians would be allowed in NHs since their entry is often prohibited in similar facilities elsewhere. In this context, it is important to remind what was mentioned before— that in India, rural population constitutes about 70% of the country's population and a significant share of rural population is from low-income group. Also, absence of alternative route between points with high demands for pedestrian and bicycle force them to use NHs. As a result, the designer must consider their effects on safety and act accordingly. As mentioned earlier that even though NHAI specification include that local traffic in built up area shall be separated with provision of service roads in all sections of the Project Highway, the provision is there ONLY WITHIN the limits of municipal towns having continuous length of 200 m or more in non-municipal areas where dwellings / shops have been built on one or both sides of the Project Highway on at least 50 percent of the total length of each such section (NHAI). However, this service roads are also missing in most locations where they qualify for and it is practically impossible to develop such continuous service roads alongside all national highways primarily due to lack of financial resources. It is also practically impossible to restrict the use of bicycles on NHs due to poor access management. It is therefore necessary to come up with innovative solution which is cost effective but will safeguard vulnerable road users like bicyclists and pedestrians and improve safety and sustainability of these high speed facilities.

3. FINDINGS AND RECOMMENDATIONS BASED ON A CASE STUDY

A case study was conducted on 120 km stretch of NH-6 between Dankuni (Located near Kolkata) and Kharagpur which is a part of Golden Quadrilateral (GQ) connecting two major cities— Kolkata and Chennai. The study stretch is currently a four lane divided facility with service lanes only at two suburban locations. A considerable part of this road goes through rural set up where there is no alternative roadway connecting the localities alongside of the NH-6. As a result the share of the non motorized traffic is very high along those stretches of NH-6. Also, there is only one interchange along this stretch and the rest of the intersections are at-grade with no traffic control devices except for channelization islands. As a result the numbers of potential conflicts are high at these locations. Increased number of incidents and accidents forced authorities to place traffic police on NH who designate right of way to oncoming traffic. While NHAI authority plan to convert all of these at-grade intersections to interchanges during its up gradation to 6-laning, the problem will remain for NMTs since grade separated facilities are not designed for NMTs. As a result the question still remains as to how these NMTs will be handled along the NH corridors where there is no alternative roadways. The problem is quite serious along those stretches where busy sections of SH were up graded to 4-lane NH with poorly managed access to businesses alongside the highway. In designing these new interchanges design of access also needs to be good, otherwise even after building expensive structures, little benefit will be achieved and over time access from nearby business may cause problem in traffic operation. The study area is a perfect example of a roadway, where anyone can get access almost anywhere especially the NMTs and highway parking to access roadside business is a common occurrence. While enforcement has a strong role to mitigate some of these issues, the planning and engineering for future NHs should consider the lessons learnt from NH6 on GQ and keep provisions for non-motorized traffic into country's expanding road network by providing designated space and with better access control. This is no doubt unique and challenging, but at the same time very essential for safety and sustainability of NH system. A preliminary survey of the study area indicate some of the obvious measures that NHAI can adopt as part of NH building, even though current specifications and code of practice has no provision for such measures. However, it is anticipated that sooner or later pertinent authorities would appreciate the need for adopting such inclusive and forgiving designs not only for safety but also from the view point of equity of access of various road users. Some of the plausible recommendations that could be worthwhile to consider are:

1. Service lanes will definitely be a good measure to alleviate unsafe interaction of bicyclists and

other non-motorized road users. However, it is practically impossible to develop such continuous service roads alongside all national highways due to lack of financial resources. Hence, the planning and construction of service roads will ONLY be governed by currently available NHAI specification. However, the justification of inclusion of such VRU friendly facilities should NOT be assesses only from the view point of improved mobility and travel time savings but also be based on the benefit of savings from VRU crash reductions.

2. An alternative to expensive service road will be construction of low cost service roads along the highway with subways or on street crossings at intersections. These service roadways will be separated by safety barrier and cut/fill slope will start from the outer edge as shown in Figure 1. Materials that may be used for such roadways include but not limited to compacted earth, water bound macadam (WBM), wet mix macadam (WMM) or other low cost materials suitable for the topography and weather conditions. These roads need to strong enough to carry NMT and they will also provide strength and stability to the motorized carriageway.

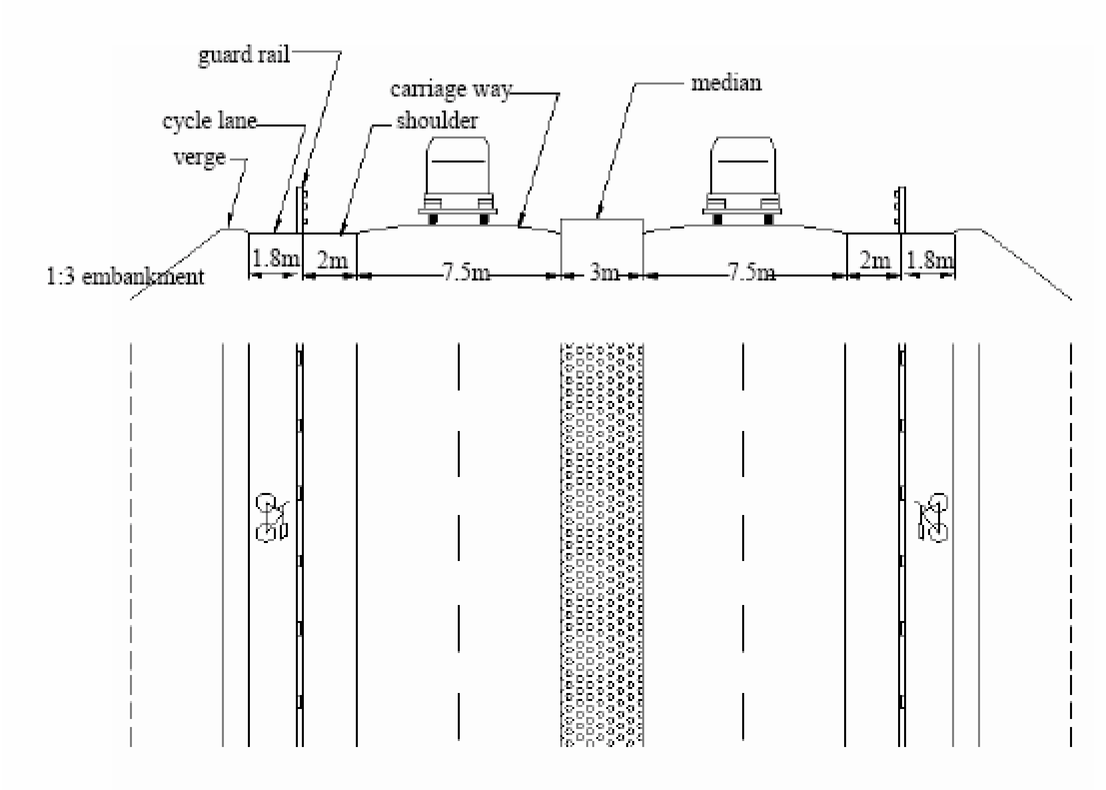

Figure 1. *Example of a 4-lane National Highway with cycle lane in both direction*

3. Another provision would be to construct wider shoulders with rumble strip separation (which will function very much like centerline rumble strip) between carriageway and shoulder. These rumble strips along the edge of the carriageway could be very effective in providing separation as well as cautioning inattentive drivers in case they shift towards outer shoulder. The design of these rumble strips can be very similar to center line rumble strip with a length between 10 and 12 inches (along the cross section) and a width of 6 inches along the direction of traveled way as shown in Figure 2. However, the only drawback this design may have is that it will not force NMT users outside the safety barrier thereby leaving some chance of their actually using the carriageway as they do presently due to lack of education and enforcement.

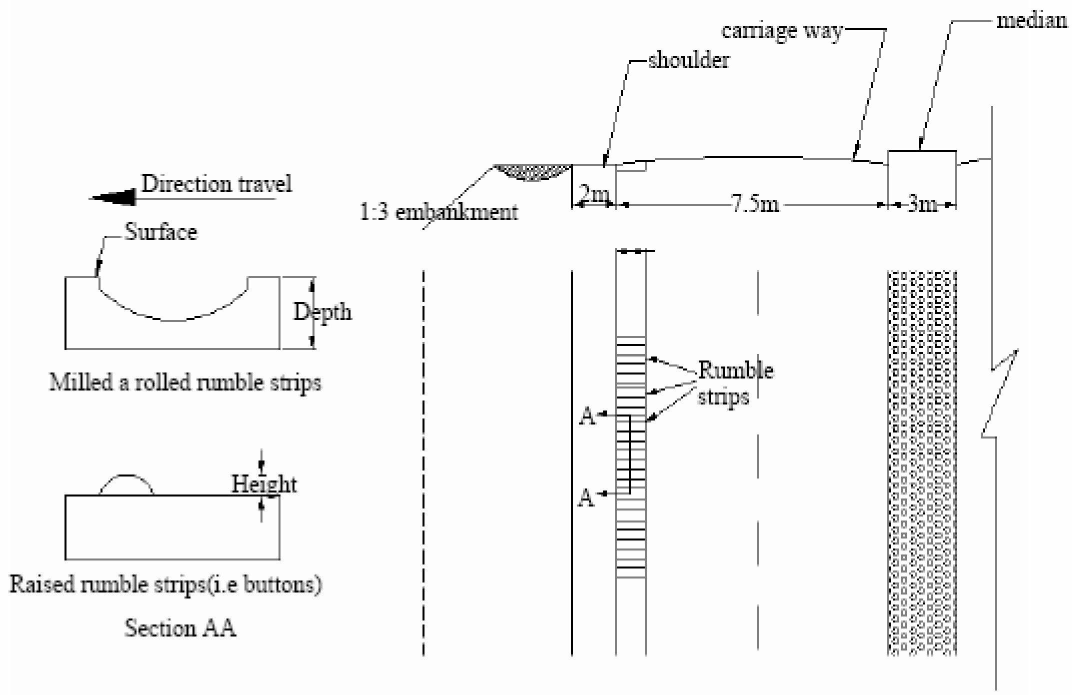

Figure 2. *Example of shoulder rumble strips along the border between carriageway and shoulder*

Finally the two other E's— "Education" and "Enforcement" need to be improved for the country. The current evidence from user survey done by students of IIT Kharagpur show that road users both drivers of motorized vehicles and NMT users are unaware of rules of roads primarily due to a lenient licensing process and lack of road safety education at school level. Also, a high percentage of rural populations have minimum or no literacy— making it even more difficult to educate them on road safety. There is also namesake presence of highway patrolling for NHs with little or no enforcement. There is no way to catch moving violators and standing on-spot enforcement is not always transparent. All these lacunas in the system add up to make road safety a serious challenge in India amidst its prosperous economic development.

4. CONCLUSION

Access management is one of the very important design elements for ensuring better mobility and safety. However, access management and control is often poor in National Highways leading to disproportionate number of deaths of vulnerable road users. In mixed traffic conditions where the share of bicycles and pedestrians are high, there needs to be provision for such users in general and particularly along stretches of NHs with no alternative road network. While high speed facilities like NHs do not provide access to NMTs elsewhere, situation is very different in India. Hence, it calls for revisiting the existing design norms for NHs and come up with innovative solutions to provide safety and mobility of NMTs on NHs. If such measures are not taken up in near future, NH system in India will be non-sustainable due to its poor safety performance and the noble cause of road development will not be fulfilled.

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DYNAMIC SIMULATION AND FULL-SCALE TESTING OF A PRE-FABRICATED STRAW-BALE HOUSE

Shea[a*], A., Beadle[b], K., Walker[c], P.

[a,b,c] BRE Centre for Innovative Construction Materials, Department of Architecture and Civil Engineering, University of Bath, BA2 7AY, United Kingdom.
[a*] corresponding author. Tel: +44(0)1225 386158, Fax: +44(0)1225 386691, email: a.shea@bath.ac.uk
[b] Tel: +44 (0) 1225 386621, email: k.beadle@bath.ac.uk
[c] Tel: +44 (0) 1225 386646, email: p.walker@bath.ac.uk

Abstract: Straw is an agricultural by-product and, after accounting for use in animal bedding, the mushroom sector, and Biomass power stations, there is an annual overall net straw surplus in Great Britain of approximately 5.5 million tonnes. The use of this non-food crop in construction is considered beneficial not least for the sequestration of carbon during production. The availability and inherent sustainability of this building material has led to a resurgence in its use over the last decade. As many countries strive towards low or zero carbon targets for building energy use, designers are exploring new methods of construction using natural materials such as straw, which, due to its insulating properties, has the potential to significantly reduce the energy required for space heating. However, there is a lack of scientific research into the thermal performance of straw-bale buildings, particularly in the UK. This paper presents the results of computer simulations and initial field-testing of the thermal performance of a BaleHaus constructed using ModCell pre-fabricated modular panels. A highly-instrumented two-storey BaleHaus has been built on the University of Bath (UK) campus. For this study, the internal environment was monitored by wireless Relative Humidity and Temperature (RHT) sensors at 12 locations whilst a weather station close to the building records the external environmental conditions.The results of co-heating and air-tightness tests are presented in addition to dynamic simulation modelling which predicts the annual energy use and CO_2 emissions.

Keywords: *Straw-bale, Co-heating, Simulation, Thermal transmittance.*

1. Introduction

Building with straw-bales appears to have originated in the Midwestern US but examples can be found across Europe and Australia. The oldest known straw-bale house still in existence, the Burke house in Nebraska, was constructed in 1903 (King *et al.*, 2006). Few viable construction materials existed in the vicinity and so straw-bales were used. Interest in straw as a construction material decreased as a result of the industrial revolution due, in part, to increased availability of modern, highly processed, building materials. In the UK, over the last 10 years straw and other natural fibre materials have become the focus of research and development aimed at producing modern innovative construction products with minimal environmental impact. Straw is a natural, renewable, and bio-degradable material that can be sourced locally and requires relatively little processing for use in construction. A co-product of the agricultural industry, straw has low embodied carbon and high thermal resistance helping to substantially reduce the whole-life carbon impact of buildings constructed of this natural material.

Although straw-bale construction has over a century of use, the development of a wholly prefabricated load-bearing panelised building system is a new concept. Modern buildings and their component parts must satisfy performance standards for load-bearing capacity, fire resistance, thermal transmittance, acoustic performance etc. Accordingly, in order to advance the use of straw-bales in construction innovative methods must be employed to create a product suitable for use in 21st century dwellings and non-domestic buildings. The development of ModCell (a portmanteau of modular and cellulose) panels forms one part of ongoing research at the University of Bath investigating the performance of these straw-bale-filled structural timber units. The BaleHaus @ Bath (Figure 1) is the first house constructed using this system for the entire building envelope. The BaleHaus is a full-size, two-storey building built on campus at the University of Bath, constructed over the summer of 2009 as part of a collaborative research project funded by the Technology Strategy Board (TSB).

Figure 1: *The BaleHaus@Bath*

2. Materials and methods

In order to reduce the environmental impact of transportation and to protect the un-rendered straw-bales from rain, the ModCell panels are assembled in a temporary ‘Flying Factory’ close to the construction site (Figure 2). In the case of the BaleHaus @ Bath a suitable agricultural building was found approximately six kilometres from the Bath campus.

Figure 2: *ModCell panel construction in a Flying Factory*

Various combinations of straw-bale, door, and window panel are produced. A standard ModCell straw-bale panel is constructed from a 100 mm thick pre-cut engineered glue-laminated timber frame measuring approximately 3190 mm wide, 2660 mm high and 490 mm deep. The frame is filled with straw-bales which are pinned together every other row with wooden stakes. The straw is pre-compressed vertically to prevent future settling of the straw and the frame is reinforced with stainless steel threaded bars in the corners and vertically to provide rigidity. Conduit for electrical cables is fitted and the panels are then spray rendered with formulated lime in three layers before being delivered to site.

The organic nature of straw makes it liable to decay when subjected to certain combinations of temperature and humidity (Goodhew *et al*., 2004) and, therefore, the conditions within the ModCell panels are monitored. A total of 66 wireless sensors recording relative humidity and temperature (RHT) are embedded within the panels with a further nine sensors monitoring conditions at the junctions of the panels. The results of within-panel monitoring will be presented in the near future. The focus of this paper is the energy performance of the building, which uses data gathered from 12 wireless RHT sensors located inside the house and referenced to the external conditions monitored on the roof of a nearby building. The 12 wireless sensors record RHT at three heights (0.45 m, 1.6 m, and 2.1 m from the finished floor level) and at two locations per floor within the BaleHaus. The roof-top weather station is located on the Department of Architecture and Civil Engineering building very close to the site of the BaleHaus. The Weather station measures global and diffuse radiation on the horizontal plane, wind speed and direction, rainfall, relative humidity, temperature, and barometric pressure. A data logger records all measurements at 10-minute intervals.

3. Laboratory and full-scale testing

3.1 Thermal transmittance testing, U-value

A thermal transmittance (U-value) test was undertaken on a single panel at the British Board of Agrément's (BBA) Thermal Laboratory, Garston, UK. The facility uses a Guarded Hot Box test method in accordance with BS EN ISO 8990 (1996).

3.2 Air permeability testing, q_{50}

Current Building Regulations in England and Wales (ODPM, 2006) require the air-permeability of the building envelope to be less than 10 $m^3/hr.m^2$ @ 50 Pa; often referred to as q_{50}. Air permeability tests were conducted in accordance with the ATTMA TS1 (2007) and BS EN 13829 (2001) test methodologies. The BaleHaus under test is presented in Figure 3.

Figure 3: *Blower-door fitted to the BaleHaus*

3.3 Co-heating test

A co-heating test determines the overall building heat loss coefficient through measurements of the heating power input (W) required to maintain a temperature difference (K) between the indoor and outdoor air. An electric resistance heater, rated at 3 kW, and two circulation fans were located on each floor of the BaleHaus. The heaters and fans were controlled by a thermostat. The interior space was heated to 25 °C and the external air temperature and solar radiation were monitored by the weather station. Electricity consumption for all fans and heaters was recorded by an energy meter with a pulse output connected to a data logger. All data were recorded at 10-minute intervals for the duration of the test. Data were recorded for approximately three days until unseasonably warm weather necessitated premature curtailment of the test. The contribution of 'free' heating energy from solar radiation was removed using regression analysis.

3.4 PassiveHaus Planning Package (PHPP) & Integrated Environmental Systems (IES) analysis

Dwellings constructed to PassivHaus standards have achieved space heating energy savings of more than 80% compared with existing building stock (Feist *et al*., 2001) and the design philosophy has had over 20 years' development. Accordingly, the German PassivHaus standard represents a proven standard enabling the development of ultra-low energy buildings. PHPP is a Microsoft Excel energy calculation tool specifically developed for certification of PassiveHaus dwellings. This design tool contains 16 core worksheets enabling calculation of building energy use plus a further 10 worksheets that permit calculation of non-standard items such as, for example, the contribution of energy from renewable sources. Integrated Environmental Systems (IES) is a suite of applications. The key modules used for analysis of the BaleHaus were ModelIT, an application for the input of 3D geometry used to describe the building, and ApacheSim, a dynamic simulation module driven by hourly weather data and used to simulate the thermal performance of the building envelope.

4. Results and inputs to models

Under laboratory tests at the BBA Thermal Laboratory a standard ModCell straw-bale panel with, nominally, 30 mm of lime render on inner and outer faces achieved a U-value of 0.19 W/(m^2K). Internal and external surface resistances were calculated to be 0.132 (m^2K)/W and 0.045 (m^2K)/W, respectively. The thermal conductivity of the straw-bale, as with most building materials, varies with density such that a lowering of density of the product results in a lowering of its conductivity; subject to conduction remaining the dominant heat transfer mechanism. The bales used in the construction of the test house and the specimen subjected to laboratory testing were approximately 115kg/m^3; the thermal conductivity of straw-bales across a range other densities has been tested as part of the BaleHaus project and these results will be presented in the near future. U-values for the remaining opaque elements of the BaleHaus @ Bath were calculated from conductivity data provided by their respective manufacturers and in accordance with BS EN ISO 6946 (1997); together with a typical window U-value and the measured wall U-value these are summarised in Table 1.

Table 1: *Summary of BaleHaus @ Bath U-values*

Element	BaleHaus U-value, W/(m^2K)	Regulatory limit[a], (ODPM, 2006)
External wall	0.19	0.35
Ground floor	0.23	0.25
Roof	0.16	0.25
Typical window	1.3	2.2

[a]UK Building Regulations in effect at the time of design & construction.

Using ATTMA TS1 (2007) Method B the BaleHaus @ Bath achieved an air permeability test result of 0.86 m^3/hr m^2 @ 50Pa, based on an external envelope of 247 m^2 and enclosed volume of 262 m^3, which represents an improvement of over 90% on current UK regulatory limits. The geometry, air permeability and fabric details of the BaleHaus were transferred to both PHPP and IES. In PHPP the q_{50} test result was converted to suit the PassivHaus calculation method. The corresponding value for the PassivHaus test methodology, which is based on a reduced internal volume discounting internal partitions of 254 m^3, is an air change rate at 50 Pa (n_{50}) of 0.84 ac/h. In both PHPP and IES, weather data for Manchester, UK, was chosen, in part, because this weather file is close to the UK population-weighted average but also because the on-site weather station is yet to record a full year of weather data; having only been operational since December 2009.

The first run of the PHPP and IES models was used to determine the heat loss coefficient of the test building for comparison with measurements from the co-heating test. Preliminary results from the co-heating tests are limited, for reasons outlined earlier, and additional, longer duration, tests are planned. The average whole-building heat loss coefficient for the duration of the test period was 50 W/K. The heat loss coefficient determined by PHPP and IES was 80 W/K and 78 W/K, respectively. The average for all dwellings currently in the UK housing stock is approximately 247 W/K (Utley and

Shorrock, 2008). Figure 4 presents the calculated and simulated results of PHPP and IES with the measured data from the co-heating test.

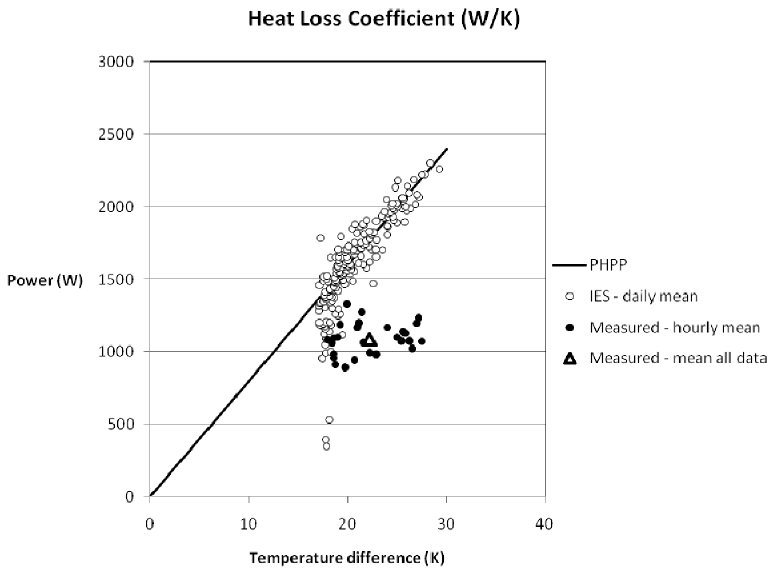
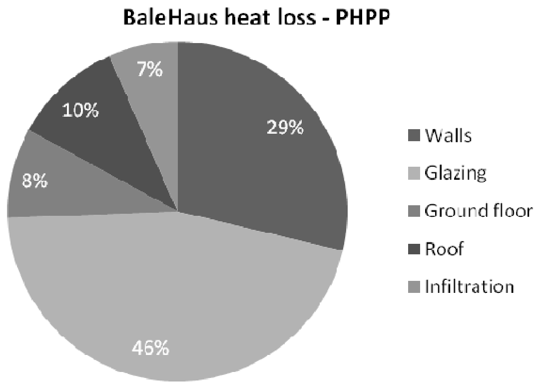
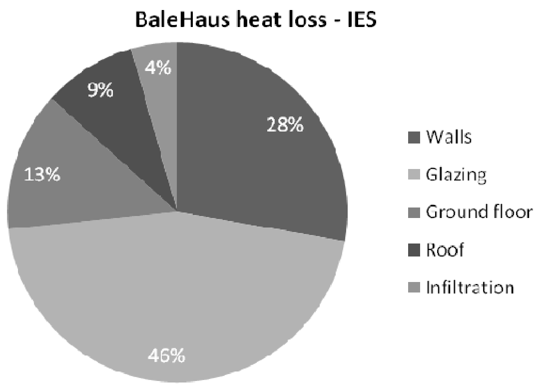


Figure 4: *BaleHaus Heat loss coefficient*

Figures 5a and 5b present the breakdown of heat losses as determined by PHPP and IES





Figures 5a and 5b: *Breakdown of BaleHaus heat loss.*

Presently the BaleHaus @ Bath has no mechanical services installed. In order to estimate the annual energy use of a 'real' BaleHaus the following heating, ventilation, and domestic hot water (DHW) services were added to the PHPP evaluation of the BaleHaus:

1. Gas condensing boiler – seasonal efficiency 88.2% (gross).

2. Mechanical ventilation system with heat recovery – heat recovery efficiency 78%.

3. Solar hot water system – 2.5 m^2 vacuum tube collector, facing south and tilted 45°.

Additionally, internal useful heat gains were specified at 0.25 W/m^2, in line with typical PassivHaus values. The space heating demand, a function of conduction and infiltration losses but offset by useful solar and internal gains, is presented for both PHPP and IES analyses in Figure 6.

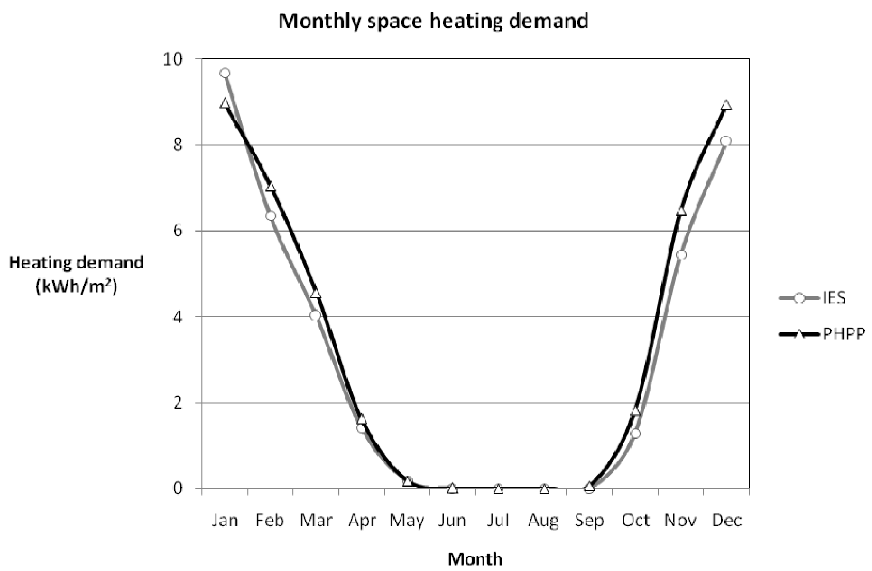


Figure 6: *Space heating energy demand.*

5. Discussion

Annual Space heating demand for the BaleHaus was calculated as 39.7 kWh/m^2 and 36.5 kWh/m^2 by PHPP and IES, respectively. The pressurisation test n_{50} air change rate was 0.84 ac/h and total primary energy demand, calculated in PHPP, using SAP 2009 (BRE, 2010) primary energy factors, was 108 kWh/(m^2 a), which includes a 14 kWh/(m^2 a) saving due to the contribution to DHW heating from the solar hot water system. It is evident from the breakdown of heat losses (Figures 5a and 5b) that a considerable amount of the space heating demand is due to heat loss through the glazing. The typical U-value for the glazing was 1.3 W/(m^2K), whilst better than UK building regulations it is significantly higher than the recommended maximum of 0.8 W/(m^2K) suggested for PassivHaus dwellings – this modification alone would result in an annual space heating demand reduction of almost 10 kWh/m^2 to 30 kWh/m^2. The PassivHaus standard presents strictly defined criteria that include three key elements, which are determined using the PHPP workbook:

1. The energy required for space heating must not exceed 15 kWh/(m^2 a).

2. A maximum air change rate pressurisation test result of 0.6 /hr @ 50 Pa.

3. A primary energy demand of not more than 120 kWh/(m^2 a).

For the BaleHaus, any further significant reduction in space heating demand, i.e. towards the PassivHaus standard, would require changes not only to the thermal performance of the glazing but also the area and orientation of glazing in combination with the provision of shading devices to ensure a suitable balance between useful, winter-time, solar gain and overheating in summer.

In the UK, primary energy use per unit floor area was an average of approximately 365 kWh/(m^2.a) in 1990 and 382 kWh/(m^2.a) in 2005 (TSB, 2009). Accordingly, 108 kWh/(m^2.a) represents a 70% reduction on the 1990 average for the UK stock. In carbon emissions terms this level of primary energy use equates to approximately 22 kgCO_2/(m^2.a); comparable to the CO_2 emissions level required by the Association for Environment Conscious Building's Silver Standard, and equalling a 70% reduction on the current UK stock average (AECB, 2007).

6. Conclusions

The results of full-scale field and laboratory tests in addition to dynamic and steady-state modelling results have been presented for the first ModCell straw-bale panel-constructed 'BaleHaus' located at the University of Bath, UK. These results indicate a high level of thermal performance is achieved from the use of straw-bales as part of an innovative pre-fabricated load-bearing panelised unit. Great Britain has a surplus of straw exceeding 5 million tonnes per annum (Copeland and Turley, 2008); sufficient resource for over a million BaleHaus dwellings. The use of local, natural, materials to construct dwellings capable of meeting the performance standards required by building codes and achieve a 70% reduction in primary energy use and CO_2 emissions represents a promising opportunity for designers to make a significant contribution to meeting national and international CO_2 emissions targets and other legislation aimed at improving building energy performance.

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CHARACTERISTICS OF MASONRY UNITS FROM IRON ORE TAILINGS

Ullas S N[1], B V Venkatarama Reddy[2] and K S Nanjunda Rao[3]

[1]Research Scholar, Centre for Sustainable Technologies, Indian Institute of Science, Bangalore, India.
[1]E-mail: ullas80@yahoo.com
[1]Telephone: +91-080-22932327

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

[3] Principal Research Scientist, Department of Civil Engineering, Indian Institute of Science, Bangalore.
[3]E-mail: ksn@civil.iisc.ernet.in

Abstract: This paper deals with an experimental study on masonry units made of iron ore tailings in compressed earth block. Compressed earth blocks (CEB) or stabilised mud blocks (SMB) are widely accepted as energy efficient alternatives to burnt clay bricks. Natural river sand is often used to obtain optimum soil gradation in the production of SMB. In order to reduce adverse impacts of indiscriminate mining of natural sand, iron ore tailings (IOT), which is a mine waste, is used as an alternate to the natural river sand. Based on the gradation of soil used for production of SMB, optimum mix proportion of soil, sand and cement was fixed and the sand fraction was replaced by IOT at 25%, 50% and 100%. The block characteristics like wet compressive strength, water absorption, initial rate of absorption and linear elongation were examined and discussed. From the experimental results it is found that considerable amount of sand can be replaced by IOT without compromising desirable characteristics of SMB used for masonry.

Keywords: Masonry units, bricks, mine wastes, stabilised bricks, iron-ore tailings

1.0 Introduction

Masonry is widely used to construct both small and large structures because of its structural versatility and attractive appearance [1]. Masonry is of considerable volume in most of the structures and masonry units are consumed in bulk quantities [2]. Compressive strength of masonry greatly depends on strength of the masonry units. In order to cater to the different needs of construction, various masonry units have been developed and used. Natural resources are indiscriminately extracted for construction needs. To reduce adverse impact on nature there is a large potential and scope for utilising industrial and mine solid wastes for the manufacture of construction products [3]. India generates huge volumes of mine wastes every year. In the present work iron ore tailings which is a waste generated after extraction of iron ore was investigated for production of stabilised mud bricks.

Stabilised mud bricks are energy efficient alternative to burnt clay bricks [3]. Stabilised mud blocks are manufactured by compacting a wetted mixture of soil, sand and stabiliser in a machine into a high density-block. Natural river sand is commonly used to achieve an optimum clay and sand content in the mix for production of good quality SMB. Crushed stone dust which is a waste from granite industry is also used as replacement to river sand in making SMB. These are cured for 28 days and can be used in the construction of load bearing masonry elements.

2.0 Methodology

The methodology adopted in the present investigation is discussed in the following section. The raw materials used were characterized first and mix proportion was fixed. Bricks were produced with decided mix proportion to study various parameter and results obtained are discussed.

2.1 Characteristics of raw materials

Locally available red loamy soil and natural river sand were used to make SMB with Ordinary Portland Cement of 43 Grade as stabiliser. Grain size distribution of natural sand and IOT were also obtained. The combined grain size distribution curves of soil, sand and tailings are presented in Fig. 1. The physical properties of materials used are given in Table - 1. The IOT is fine graded compared to natural sand used.

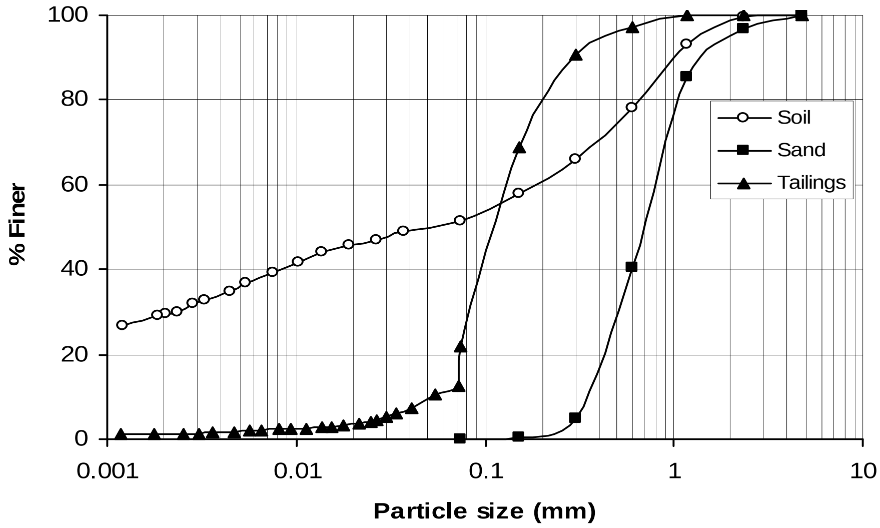

Fig. 1 – *Particle size distribution curves for soil, sand and iron ore tailings*

Table – 1: *Physical properties of raw materials used*

Material	Sand fraction (0.75-4.75mm) %	Silt fraction (0.75-0.002mm) %	Clay fraction (<0.002mm) %	Fineness modulus	Specific gravity
Soil	48.6	22.5	28.9	1.04	2.62
Sand	100	0	0	2.71	2.57
Tailings	78	20.7	1.3	0.4	2.77

2.2 Mix proportioning

Earlier research on SMB has examined in detail the role of clay and its optimum content in the soil for better performance of SMB both with respect to strength and durability [4]. These studies recommend optimum clay content to be in the range of 12% to 16%. Hence, in order to bring the clay content of the soil within desirable limits, ratio of soil to sand was kept as 1:1 by weight. It is also brought out in earlier research that about 7% of stabiliser is sufficient for three storey load bearing masonry of moderate span residential buildings. Hence 7% cement by weight of soil and sand was used to stabilise the soil. In general, water content of about 10% of dry materials by weight is commonly used to make SMB. However, as the river sand is being replaced by very fine IOT, demand for increased moulding water content was expected. In order to arrive at proper moulding water content, optimum moisture contents (OMC) was first determined for each mix with IOT content to replace sand. Also, it is found that for a given mix, moulding water content to the wet side of OMC gives better strength [5]. Hence, for all the mixes, moulding water content was kept 10% more than the respective OMC. The river sand fraction was replaced by IOT at 25%, 50% and 100% and compared with results for mix containing 100% river sand. The bricks made out of the mix with sand to IOT ratios of (1: 0), (0.75: 0.25), (0.5: 0.5) and (0: 1) are designated as A, B, C and D respectively. The target dry density was kept at 1.8 g/cc, which is again based on the recommendations by earlier research on SMB technology. Details of mix proportion and moulding water content are presented in Table – 2.

Table -2: *Details of mix proportion*

Mix type	**Mix proportion by weight**			**Cement content (%)**	**Moulding water content (%)**
	Soil (%)	Sand (%)	IOT (%)		
A	50	50	0	7	12.15
B	50	37.5	12.5	7	12.79
C	50	25	25	7	14
D	50	0	50	7	15

2.3 Brick production

Manually operated press was used to make bricks. Soil, sand and IOT were dry mixed first and then cement was added and re-mixed. Wet mix was prepared by adding water content equal to 10% towards wet side of OMC corresponding to different tailing contents. Weight of wet mix to be pressed to make bricks of 230 x 110 x 70 mm size was controlled to achieve dry density of 1.8g/cc. The bricks so produced were cured for 28 days under wet burlap. The process of brick production using manually operated press is shown in Fig. 2.

Fig. 2 – *Production of compacted SMB using manual press*

3.0 Characterisation of bricks

3.1 Wet compressive strength

Masonry is commonly used to take compressive loads. Hence compressive strength of masonry units is an important characteristic to be considered in selection of bricks for construction of load bearing masonry members. The bricks were tested as per IS: 3495 (Part 1) [6] guide lines in a compression testing machine as shown in Fig. 3. Satisfactory brick strength in wet condition ensures even better strength in dry condition. The results obtained are given in Table -3.

Fig. 3 – *Brick under wet compressive strength test*

3.2 Initial Rate of Absorption (IRA)

Initial rate of absorption gives an idea on what rate the bricks tend to absorb water. This information is useful in understanding the rate of moisture transport from mortar to the brick in masonry construction. As the brick or block-mortar interlocking is mainly due to locking of cement hydration products from mortar [7], brick has to absorb right amount of water along with binder in order to facilitate the bond development. This experiment was carried out as per the procedure given in ASTM C-67 [8] and the results are presented in Table – 3.

3.3 Water absorption

Water absorption was determined as per the procedure laid down in IS 3495 (Part 2):1992 [9]. The bricks were dried till they achieve constant weight and then soaked in water for 24 hours. The water absorption during this period was calculated as percentage increase in weight and the results are presented in Table – 3.

3.4 Linear expansion on saturation

Dimensional stability of SMB is an important issue. This can be measured by monitoring linear expansion on saturation. The variation in length of oven dry bricks was measured in a length comparator set up fabricated in the laboratory and a brick under observation is shown in Fig. 4. Initial length of dry bricks was measured with a digital vernier. The dry bricks were first placed in the length comparator and initial reading of dial guage was recorded. The bricks were then soaked for 24 hours and again placed in the length comparator to measure the dial guage reading of saturated brick. The percentage change in length of dry bricks upon saturation is given in Table – 3.

Fig. 4 – *Length comparator*

Table – 3: *Test results of bricks*

* Number of specimens tested in each case: 8; Standard deviation values are in parenthesis

Mix type	Wet compressive strength (MPa)	Water absorption (%)	IRA ($kg/m^2/min$)	Linear Expansion (%)	Achieved dry density (g/cc)
A	6.89 (0.42)	12.13 (0.36)	0.59 (0.09)	0.046 (0.02)	1.83 (0.01)
B	6.76 (0.52)	12.35 (0.53)	0.655 (0.18)	0.031 (0.008)	1.84 (0.01)
C	6.77 (0.36)	13.22 (0.83)	0.568 (0.12)	0.041 (0.014)	1.82 (0.01)
D	6.63 (0.27)	15.07 (1.3)	0.582 (0.14)	0.035 (0.02)	1.82 (0.01)

4.0 Results and discussion

4.1 Mix proportions

The details of mix proportions at different IOT contents to replace sand are given in Table -1. The reference mix (i.e. “A”) was first selected to have a typical standard mixture with optimum clay and sand contents. In this mix, the sand content was replaced by IOT at different percentage to get three more mix proportions. It was observed that higher the IOT content, higher was the water content to achieve required density. This is due to increase in surface area to be wetted due to finer IOT particles.

4.2 Water absorption, IRA, Linear expansion and Strength

Details of the test results of water absorption, IRA, linear expansion and wet compressive strength are given in Table -2. These results are average of 8 specimens. The water absorption of bricks increases with increase in IOT content to replace sand. The increase in water absorption is in the range of 12.13% to 15.07%. This may be because of increase in voids due to higher fine fraction in the mix. IRA values are also varying as IOT content to replace sand is varied. However, it is not possible to draw any strong conclusion as the variation is in a very narrow range (0.65 – 0.56 kg/m^2/min). Similarly, though there is variation, no adverse effect on linear expansion of bricks at different IOT content was observed. Expansion on saturation is within limits and the values fall within the upper limit of 0.1% suggested in earlier research work. This may be attributed to the fact that the clay content which is the main cause of volumetric change remains same in all the four mix proportions. From the results obtained, it is clearly seen that the wet compressive strength, which is the main parameter of concern shows negligible fall in average strength value of 0.25 MPa as the sand was completely replaced by IOT.

5.0 Conclusions

In the present study, sand content of a selected mix proportion for the production of SMB was replaced by iron ore tailings at different percentages. the results shows that it is possible to completely replace natural river sand by iron ore tailings without sacryfying on compressive strength of SMB. Water absorption increases with increase in iron ore tailings content but it is within limits. The results clearly demonstrate that iron ore tailings can be used as sand substitute in SMB production. The wet compressive strength of SMB is about 7MPa when 7% cement was used.

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About the Authors

ULLAS S N, M. Tech (Construction Technology) is a Research Scholar at the Centre for Sustainable Technologies, Indian Institute of Science.

B V VENKATARAMA REDDY is a Ph.D. in structural Engineering from Indian Institute of Science India. Currently working as a Professor at the Department of Civil Engineering and associate faculty at the Centre for Sustainable Technologies, Indian Institute of Science. His research interests include structural masonry, low carbon construction materials and technologies, energy in buildings and green buildings, and recycling non-organic solid wastes into building products.

K S NANJUNDA RAO, Ph.D in structural Engineering from Indian Institute of Science India. He is a Principal Research Scientist at the Department of Civil Engineering, Indian Institute of Science. His research interests are in the areas of earthquake resistant structures, plates & shells and reinforced concrete.

STRAW BONDED SOLID PANELS AS A WALLING MATERIAL – A TWO STOREY HOUSE WITH "DURA"

S. Jayakody[1], M. Kondadeniya[2], M. T. R Jayasinghe[3]

[1]Deputy Geneal Manager, International Construction Consortium, Kalubowila, Sri Lanka.
[1]E-mail: samitha@icc-construct.com
[1]Telephone: +94-11-4768600 (Ext: 8647); Fax: + 94-11-4768666

[2]Sales Manager, International Construction Consortium, Kalubowila, Sri Lanka.
[2]E-mail: madavi@icc-construct.com
[2]Telephone: +94-11-4768600 (Ext: 8647); Fax: + 94-11-4768666

[3]Professor, Department of Civil Engineering, University of Moratuwa, Moratuwa, Sri Lanka.
[3]E-mail: thishan@civil.mrt.ac.lk ; mtrjayasinghe@yahoo.com
[3]Telephone: +94-11-2650567-8 (Ext.2006 - 2001) ; Fax: + 94-11-2651216

Abstract: From the point of greater degree of sustainability, the use of renewable materials for at least part of a built environment can be appreciated. In this context, the straw bonded solid panels manufactured as boards with a thickness of 58 mm can offer an ideal solution to be used as loadbearing walls and floor slabs panel. This paper describes the uses of "Dura", a straw bonded solid panel and the various strategies that can be employed to ensure strength, durabilityand safety.

1. Introduction

The use of rapidly renewable building materials has been identified as an important concept that can reduce the depletion of finite raw materials and long-cycle renewable materials. In projects recognized by LEED, the use of rapidly renewable building materials and products is promoted for 2.5% of the total value of all building materials and products used in the project based on the cost [1]. The examples can be drawn from the use of agricultural fiber such as wheat in composite panels as a substitute for wood products. In Sri Lanka, straw obtained from paddy cultivation can be a good candidate for manufacturing of straw bonded solid panels. According to data available, a land extent of more than one million hectares would be cultivated each year leading to a significant quantity of straw as a by-product [2].

Figure 1: A building constructed with "Dura" panels and steel channels to be used as a site office

Straw bonded solid panels are manufactured by Dura building systems, a subsidiary of the International Construction Consortium, is known as “Dura” panels. They are manufactured to a width of 1.2 m. The length can be 2.4 m, 2.7 m, 3.0 m or 3.3 m. The thickness of the panel is 58 mm. The general applications have been internal partitions and semi – permanent detachable buildings such as site offices, store rooms, etc. Due to densely packed straw, the fire resistance of Dura is high [3]. Figure 1 shows a building where “Dura” panels have been used with a steel channel system to create a site office for the consultant team at a hotel project.

Figure 2 shows the roof arrangement used with Dura as the insulation material and while providing structural support to the roof cladding. Steel members have been used as the truss system. One of the main drawbacks of this truss system was the high cost involved in the steel components used with the panels to produce sufficient rigidity. The advantages have been the use of light weight panels promoting modular construction that can considerably reduce the construction duration. One of the key issues of this system of single storey buildings was the durability of the panels since they could be susceptible to water absorbed especially at plinth level. For this, many precautions have to be taken during the construction of the building shown in Figure 1.

Figure 2: Arrangement of the roof used with Dura as the insulation material and steel members as the truss system.

Tropical climatic conditions that prevail in countries located close to equator have few main features. One is high intensity of solar radiation in days without rainfall. During monsoon times, there could be very high rainfall. Thus, roof is one of the key elements that allow a significant heat gain during the daytime in days with clear sky. Roof will also need considerable strength and preferably weight to ensure cyclone resistance. A key advantage of Dura in tropical climates is the ability of Dura to fulfill structural requirement of the roof while providing adequate resistive insulation for thermal gains through roof. This can be considered as a major advantage since the structural rigidity of “Dura” would be able to provide a robust, cyclone resistant roof structural system with minimum amount of additional structural members. With the vast experiences gained by constructing these semi – permanent detachable buildings, an attempt was made to extend the applications of “Dura” to two storey houses where the straw panels are expected to act as one of the main structural system consisting of loadbearing walls, floor slabs and roof structure.

2. Objective

The main objective of this research was to develop a structurally stable, durable and cost effective two storey houses with straw boarded solid panels.

3. Methodology

The following methodology was adopted:

1. The research on load bearing characteristics of "Dura" panel obtained as part of product development was used to predict the structural adequacy when used as a structural material.
2. The structural requirements were identified with a number of typical two storey houses.
3. The structural solutions have been developed with several cycles of optimization.
4. A model house was constructed that can be used for long term research and development while paying sufficient attention for durability.

4. The loadbearing characteristics of Dura

Dura panels are manufactured with a thickness of 58 mm. The length of a panel can vary from 2.4 m to 3.3 m. The width of a panel is 1.2 m. Dura panels have been found to be a good material for partition walls. It has also been successfully used as a floor board when supported at 1.2 m intervals. However, its application as loadbearing walls in a two storey house needed special attention.
There were two key issues to be resolved. One is the tendency for buckling when carrying significant loads. The other is any local failures that could occur due to bearing failures under the timber joists used to support the floor boards. Both these issues have been successfully addressed by using a full scale model that was load tested to obtain a clear understanding on the behavior of Dura panels. Figure 3 shows the loading arrangement of the model with loads and the floor joists used for supporting the floor boards. The loads applied were well in excess of the maximum load of 2.0 kN/m^2 that can be expected in a two storey house [4].

Figure 3: The load testing carried out for Dura panels with full scale model
With this load testing, it was found that Dura panels could be successfully used for three applications:

a. *The loadbearing and partition walls*
b. *The floor boards*
c. *To create part of the structural system of the roof*

a. The load bearing and partition walls

The load bearing walls can be either external or internal. External walls would have sufficient number of openings for windows and doors. Since the thickness of the panel is only 58 mm, it has a tendency to buckle when subjected to heavy loads. Since, the failure by buckling of the panel is not a desirable mode to failure, this had to be avoided. The strategy adopted was the use of staggered panels side by side that completely eliminated the weakness that could occur at joints between panels while ensuring a wall of 58 mm x 2 = 116 mm thickness. Previous experience has indicated that there would be a tendency to form a crack on the finished surface when Dura panels are combined to form walls. Figure 4 indicates the installation of Dura panels to form an external wall. In the initial phases, an insitu-cast *wet joint* has been used to connect the panels. In order to ensure adequate rigidity while minimizing the tendency for shrinkage, a polymer modified cement grout has been used to form this *wet joint*. However, this *wet joint* was found to be labour intensive in the actual construction of the two storey house. Hence, for the upper floor, this *wet joint* was eliminated with mechanical joints effected using screws.

Figure 4: "Dura" Panel wall used for load bearing external walls along with anchors used to connect to the foundation

b. The floor system

One of the main advantages of straw based solid panels is its ability to be used as floor boards supported on a suitable structural system. Generally, it is sufficient to use Figure 5 indicates the ability of "Dura" to carry structural loads, even when they are of concentrated nature. For two storey houses, a floor structure consisting of timber beams that will act in composite with Dura panels could be used as shown in Figure 6. If a greater degree of rigidity is required for the floor, it is possible to use short intermediate beams between the timber beams that would enhance the ability of floor boards to withstand point loads behaving as a two way slab system thus effectively eliminating any localized deformations. Such localized deformations could give some kind of softness under the foot of the users. Hence, its elimination could be important.

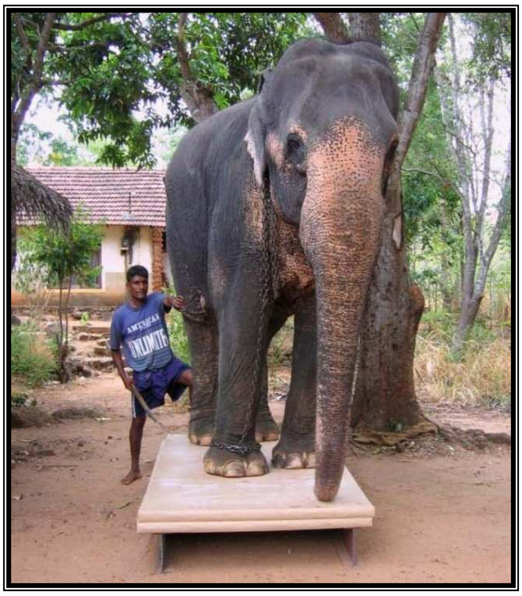

Figure 5: Dura panels were tested for larger concentrated loads

Figure 6 also indicates the additional timber framework that has been used to ensure proper connectivity of the timber floor beams to the Dura panels used for the upper floor. Such connectivity is important with respect to cyclone resistance. Since the two storey house is of light weight construction, it has to derive the cyclone resistance by mobilizing the rigidity provided by the panels locally coupled with the weight of the completed structure for overall stability. This two storey house has been checked as for BS 6399: Part 1: 1997 [5].

Figure 6: The beam arrangement for supporting floor slabs made by Dura panels

c. The roof system

One of the key uses of straw based panels for the roof is enhancing the cyclone resistance. A robust structural system consisting of timber beam and straw panel acting as a composite could be designed to withstand wind forces induced due to heavy wind loads generated by 33 ms^{-1} or 42 ms^{-1} basic wind speeds. With few additional precautions, even the wind loads due to 47 ms^{-1} could also be resisted. The roof covering material will be Zinc – Alum sheets.

5. The two storey house

A view of the completed two storey house is shown in Figure 7. The internal dimensions have been carefully selected to ensure functionality while enabling modular form of construction. The durability issues were handled in the following manner:

1. The house is located at an elevation with no possibility for flooding as shown in Figure 7.
2. The straw boards are started on a foundation with a plinth beam that would hinder the upward moisture movements while ensuring robustness required with respect to earthquake resistance.
3. The roof is provided with adequate eaves as shown in Figure 8 coupled with provision for the installation of a well detailed gutter system.
4. The balconies are tiled to ensure water tightness. The tiles have been laid on a Ferro-cement based system as shown in Figure 8 to ensure proper adherence of the tiles and effective water proofing.
5. Bathrooms have been tiled as shown in Figure 9. These tiles could also be combined with a thin layer of Ferro – Cement based structural system to further enhance the robustness and water – tightness.
6. The roof is provided with properly fixed fasteners connected to Dura panels to ensure a leak free roof that has sufficient cyclone resistance. An additional layer of polythene was also located as an additional protection underneath the roofing sheets as shown in Figure 10.
7. Whenever, additional water proofing is needed, it is possible to laminate the Dura panels with a fiber based boards of 3 mm thickness as shown in Figure 11.

Figure 7: The two storey house that has been constructed with Dura panels being used for ground and upper floor walls, floor slabs and the insulation in roof

Figure 8: The two storey house under construction

Figure 9: Bathrooms provided with a tiles to ensure proper water proofing

Figure 10: Roof with Dura boards supported on timber joists and also provided with a polythene sheet

Figure 11: Additional water-proofing provided with fiber cement boards

6. The cost aspects

The solid straw panels are manufactured with straw as the main ingredient. Therefore, the cost of panels could be contained at about Rs 500/= per square meter. For the two storey house shown in Figure 7, the cost of Dura panels could be maintained at about Rs 10,000/= per square meter of floor area. With the experience gained with this house, many new innovations have been introduced to reduce the cost of construction in future. One of the key improvements was the elimination of the polymer based wet joint shown in Figure 4. The strategy used was the adoption of a mechanical connection with staggered panels. With many such innovations, straw bonded solid panels could offer a product that can be effectively used as walls, floors and roofs of two storey detached houses.

7. Conclusion

The extensive use of natural resources as building materials has caused many environmental issues. Therefore, the development of alternative materials that can provide adequate strength and durability could be a significant step in improving the sustainability of built environment. In this context, innovative approach and extensive use of structural concepts have been successfully utilized to explore effective usage of straw bonded solid panels for two storey houses. This new application can have many benefits with respect to cost and sustainability related issues.

Acknowledgement

The authors are grateful to Dura Building Systems for providing financial assistance and “Dura” panels needed for this research.

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STRAW BONDED SOLID PANELS – THE CONSTRUCTABILITY, THERMAL PERFORMANCE AND STRUCTURAL BEHAVIOUR

K. Munasinghe(1), S. Jayakody(2), . M.T.R.Jayasinhe(3)

[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] Deputy General Manager, International Construction Consortium, Kohuwala, Sri Lanka.
[1]E-mail: samitha@icc-construct.com
[1]Telephone: +94-114-768600 (Ext: 8647); Fax: +94-114-768666

[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: Straw bonded solid panels manufactured as boards with a thickness of 58 mm can be used for many applications such as internal partitions, floor boards, acoustic panels, ceiling, etc. However, there have been many issues related to the use of straw panels as a load bearing material for walls. This paper describes the details and results of a full scale load test carried out with straw bonded solid panels to assess the constructability, structural adequacy with short term loads and sustained loads. It also indicates the trends that can be expected with thermal performance.

1. Introduction

In Sri Lanka, the total amount of rice produced is 2.79 million metric tons in year 2009 [1]. This could increase to about 3.83 million metric tons by year 2020. Every ton of rice would produce 0.4 tons of straw. This straw may have many uses. Straw is considered as a very good organic material for restoring the soil quality. It can be used for manufacturing of paper. Many countries uses the readily available wheat straw which is renewable annually, to produce ethanol fuel as a cheap alternative to imported crude oil fuel. The use of straw for manufacturing of solid panels is another application practiced universally. A solid panel is shown in Figure 1. Figure 2 shows the dense arrangement of straw within the panel. The straw is placed in the transverse direction. Therefore, the panel can derive a considerable flexural strength in that direction and hence, it is advisable to support the panel at 1.2 m when used as a structural member carrying loads that can cause flexture.

Figure 1: A DURRA straw panel of 1.2 m width x 2.4 m length

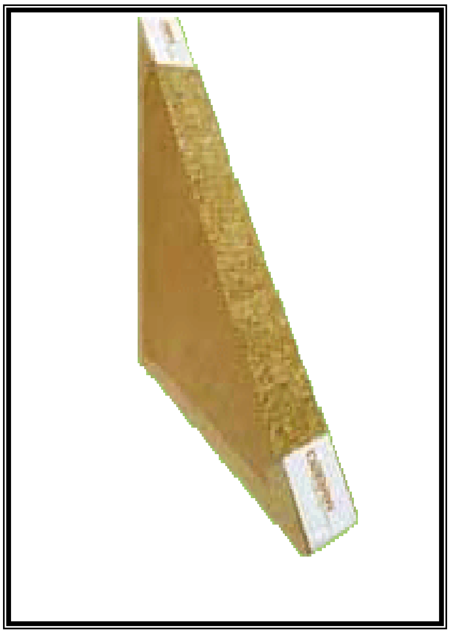

Figure 2: Cross section of a DURRA straw panel with straw arranged in the transverse direction

These panels show promise with cost competitiveness and robustness though it may have few durability related issues that should be addressed. If the data on structural performance is available, it would be possible to overcome durability related issues easily with suitable water proofing system.
Further the design trends of built environments slowly change in style, type, social acceptance, and principals that can be identified roughly based on decade by decade basis. In 70's it was energy efficient design and passive solar energy concepts. In early 80's with the higher cost of energy efficient built environment, it was more about modern designs. In 90's it was a re-interpret of successful older designs into more contemporary modern designs. In the first decade of the 21st century, it appears recycled or green built environment will be given priority (LEED's with U.S. Green Building Council USGBC) [2]. The designs that conserve energy and environmental may be appreciated. In this context, straw is sustainable, recyclable, and biodegradable.
This paper describes a full scale load test carried out using a model to assess the structural capacities and performance especially at ultimate limit state along with few specific issues that may need addressing with respect to constructability.

2. Objectives

The main objective of this research is to determine the structural capacity of straw bonded solid panels when used as load bearing wall panels and in floor systems while addressing the issues related to constructability and thermal performance.

3. Methodology

The following methodology was adopted:

1. The data available on the performance of solid straw panels determined using laboratory experiments were used to make an initial assessment of the performance.

2. A full scale model was created using solid straw panels consisting of load bearing walls and a floor system while addressing issues related to constructability.

3. This model was loaded with representative loading that can be expected in various practical applications to assess the performance.

4. The loads were sustained over longer periods to assess the long term creep induced deformation characteristics.

4. The straw bonded solid panels and the usage

These panels are manufactured with a thickness of 58 mm. The Durra technique combines extreme heat and compression in a unique dry extrusion process to form the solid panel core that is encapsulated with a high strength, water resistant, recyclable craft paper liner. No chemical binding agents, glues or resins are added during the panel core production. The width of the panel is generally maintained as (1187mm) 1.2 m. The length could be 2.4 m, 2.7 m, 3.0 m.
The weight of a panel of 2.4 m length and 1.2 m width is about 60 kg. This gives a density of 3.5 kN/ m^3. The surface density is 0.2 kN/m^2. Conventional masonry has a densityof about 20 kN/m^3. The panel shown in Figure 1 has a length of 2.4 m. In the panels, the straw is densely packed in the direction of width as shown in Figure 2. The production process is purely natural and does not use any chemical like formaldehyde. Hence, the panel poses sustainable characteristics such as low embodied energy levels per panel. It is about 5.0 MJ/m^2. It is also biodegradable after its productive life span. If two panels are used side by side, it would give a wall having an embedded energy of 10 MJ/m^2. This can be compared with the embodied energy of a typical brick wall with plaster. It is in the range of 500 - 700 MJ/m^2. Therefore a straw bonded wall with finishes could give much lower embedded energy which being a renewable resource. Another important property of straw bonded panels is the lower conductivity. In tropical climate conditions, the west facing wall could absorb considerable amount of heat especially in two storey houses. A straw bonded double panel of 116 mm thickness can give a U value of about 1.3 W/m^2K. Although U value may not be very significant in houses operated as free running, the significantly lower conductivity in the range of 0.1 W/mK [3] could transfer less heat inwards and hence facilitate much faster rate of structural cooling during the night time. The thermal conductivity of masonry would be about 0.8 W/mK [3]. Therefore, from the thermal performance point of view, walls formed with straw bonded panels could indicate a superior performance. For air conditioned built environments, it can have significant advantages.
Various desirable properties of straw bonded solid panels have been carried at as laboratory experiments (Department of Civil Engineering, University of Moratuwa Ref No CE/GA17/ST/2005/118, and National Building Research Organization NBRO/ENV/26201/2008/147(a) Industrial Technology Institute (ITI) report No CTS-7412, CS-11561). The extensive uses of these panels for many applications have given a significant level of exposure to its uses, strengths and weaknesses. Since, it indicates good promise as a load bearing walling material in addition to its ability to act as a floor panel, a full scale test was carried out to obtain further information on desirable structural characteristics and construction related issues.

5. The constructability

Straw bonded solid panels are manufactured to have a width of 1.2 m. Hence, when one panel is installed, it can cover an area of about 3 m^2. However, one of the key issues would be fixing two panels side by side to form a wall. Although it would be possible to create an effective joint at top and bottom, facilitating effective joint at the intermediate points over the height was a challenge. The generally adopted solution was the use of galvanized steel channels as shown in Figure 3. However, the cost of steel channels could be significant. Easthetically, it may indicate that the house would be different to conventional construction.
In order to improve the joint, two methods are available. One is to use a mechanical connection effected with timber pieces with a polymer modified cement grout. The polymer is expected to have shrinkage compensating characteristics. It can be considered as a wet joint. The other is to have a mechanical connection with staggered panels. This method was found to be economical than the wet joint. It also consumed less labour, less time and also facilitated a robust joint. The use of staggered panels ensured that one Dura panel was continuous when the other was having a joint. This effectively eliminated the chance of forming a crack after the completion of finishes. Thus, majority of the constructability issues could be addressed with the use of staggered double panels connected with mechanical connections.

Figure 3: Panels that have been fixed using galvanized steel channels in a site office

6. The structural behaviour

In order to determine the performance with respect to load bearing characteristics, a full scale model was constructed. The load test has been carried out up to a live load intensity of 5.0 kN/m^2.

6.1. The model

The model for load testing was created using straw based solid panels of 58 mm thickness that is adequately reinforced with another panel to prevent buckling induced failures. In order to apply loading, a floor has been created using straw panels where the timber joists of 125 mm depth x 75 mm width have been used at 1.2 m spacing to support the straw bonded solid panels. Special attention was placed on any local failure that could occur, especially due to the concentrated loads transferred through the timber joists on the straw panel used as a load bearing member. The model is shown in Figures 4 and 5. Figure 4 indicated the external view. Figure 5 indicates the floor system. It also indicates additional supports used prior to load testing to prevent any sudden collapse. Figure 6 indicates the loads applied on the model.

Figure 4: External view of the house model

Figure 5: Floor system of the house model

Figure 6: The model with loads

6.2. The application of loads

The loads were applied on to the floor slab in gradual increments shown Table 1 Since, the timber beams have already been designed for the expected loading, emphasis was placed on making as overall structural assessment for ultimate behavior rather than any serviceability failures. First the slab was loaded up to 2.5 kN/m^2. This is excess of 1.5 kN/mm^2 generally specified for houses [4]. This was to determine the behaviour with respect to adequacy of floor system. Then the load on the floor was increased up to 5 kN/m^2. This was intended to determine the performance of the walls when the upper floor is constructed. Under both these cases, the wall with a thickness of double boards withstood the loads with satisfactory behaviour.   After loading the floor successfully up to a live load of 5 kN/m^2, it was retained to determine the long term load carrying characteristics along with assessing any creep induced problems for 60 days.

Table 1: The Loading increments for the loads applied on the floor of the model

Sequence of loading and timing	Loads applied on top of the floor board	Structural deformation observation
24th May 2010 -10:15- 12.45 pm	1,545 Kg	Stable, no deformation
25th May 2010 - 3:24 - 4:43 pm	873 Kg	Stable, no deformation
26th May 2010 -10:00 -11:45 pm	566 Kg	Stable, no deformation
Total Loaded	2,984 Kg	

6.3. The results

The load test carried out on the full scale model clearly indicated the ability of solid straw panels to withstand significant loads either of short term nature or sustained nature. A minor deformation was observed under the concentrated loads transferred through the floor joists as shown in Figure 7. When the floor load was increased up to 5 kN/m^2. However, this local failure occurred at a load that is well in excess of that would occur in a two storey house. It also occurred at a location where only one panel has been used to support the timber beam. Hence, it failure would not have any significance for a two storey house with a properly planned layout.

Figure 7: The localized failure suffered by the straw bonded solid panel

The full scale testing was backed by structural design calculations. Thus, it gave adequate confidence to venture into a new application for two storey houses where walls, slabs and roof could be constructed with solid straw panels to create a very robust structural system that could safely withstand any expected loads. The plan of the two storey house is given in Figure 8.

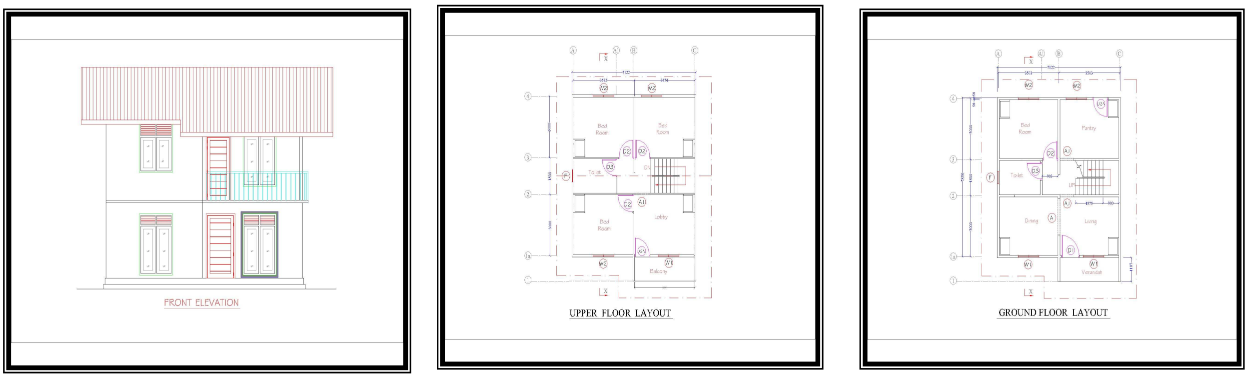

Figure 8: Plans of a two storey house

7. Conclusion

In order to determine the structural performance of a system of floors and load bearing walls constructed with solid straw panels, a load test was carried out on a full scale model. This load testing aimed primarily at determining the behavior at ultimate conditions shed light on many important issues that would need the attention of the structural engineer. With the confidence gained with this testing, the construction of a proto – type house has been undertaken and build as the project office for the engineers at the construction site of John Keels Chaaya Bay hotel at Beruwala Sri Lanka with the aim of gathering further information and experience on real time performance. Figures 9 and 10 are showing a proto –type building under construction at Chaaya Bay hotel at Beruwala.

Figure 9: Side view of the two storey house under construction at Chaaya Bay hotel at Beruwala

Figure 10: Front elevation of the two storey house under construction at Chaaya Bay hotel at Beruwala

Acknowledgement
The authors are grateful to DURRA BUILDING SYSTEMS for providing financial assistance and providing the space and location for the test cubical to be set up and for providing the materials including the DURRA boards for the test.

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K. MUNASINGHE, Dip in PSCM is the Business Promotional Manager in International Construction Consortium (Private) Limited. He has research interest on evaluation of life cycle cost of sustainable materials, sustainable buildings and eco –tourism

S. JAYAKODY, B.Sc(Eng) Hons, M.Eng(Structural) (Japan)C.MIE(SL) MJSCE (Japan),MBA (Colombo) is a Deputy General Manager in International Construction Consortium (Private) Limited. His research interests are in high – rise building construction, green buildings and innovative construction methods.

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.

INVESTIGATION OF COMPRESSIVE STRENGTH OF CONCRETE CONTAINING RICE-HUSK-ASH

R. B.Y. B. Uduweriya[1], C. Subash[1], M. M. A. Sulfy[1] and Sudhira De Silva[2]

[1]Graduate Student, Department of Civil and Environmental Engineering,
Faculty of Engineering, University of Ruhuna
Email: uduweriya@gmail.com
Tel: +94-(0)77-2666601

[2]Senior Lecturer, Department of Civil and Environmental Engineering,
Faculty of Engineering, University of Ruhuna, Hapugala, Galle, Sri Lanka
Email: sudhira@cee.ruh.ac.lk
Tel: +94-(0)91-2245765~7

ABSTRACT

The rice husk ash (RHA) is a pozzolanic material that can be blended with the Portland cement in concrete to obtain a better performance of normal concrete. This paper proclaims an experimental investigation on utilization of RHA for the concrete as it is a byproduct of brick-kilns in Sri Lanka. The results of three different replacement percentages of RHA in concrete (10%, 20% and 30% by mass of cement) were compared with the concrete that does not contain RHA. Those samples were tested for compressive strength, tensile strength, surface water absorption and the durability aspects. A comparative study on chemical composition and physical properties was carried out and the experimental results were discussed. A significant improvement on the compressive strength at early stage is identified and optimum strength was achieved at the 20% RHA replacement by mass of cement. The strength of the concrete with different sizes of RHA particles was compared to identify the effects of particle size of RHA. Around 50% of the RHA collected from the brick-kiln can be utilized for the concrete after proper grinding.

Keywords *–Rice husk ash, compressive strength, brick-kiln, surface water absorption*

1. INTRODUCTION

In the present situation, concrete is one of the most widely used construction material in the world. The most common form of concrete is Ordinary Portland Cement (OPC) concrete, which consists with coarse aggregate, fine aggregate, cement and water.

In most of the countries, different cementitious materials such as Fly-Ash, Ground Granulated Blast-furnace Slag (GGBS), Silica Fume and Rice Husk Ash (RHA) to achieve high performance, good quality and low cost concrete mixtures. In here, rice husk is a byproduct of de-husking operation of paddy rice that is produced in about 115 million of tons in all over the world annually. The total amount of rice produced in Sri Lanka in year 2002 is 2.79 million metric tons [1]. The total amount of rice needed in Sri Lanka over the year 2010 and 2020 are estimated at 3.46 and 3.83 million metric tons respectively [2]. Approximately 20 kg of rice husk are obtained for 100 kg of rice. Rice husks contain organic substances and 20% of inorganic material [3]. Therefore considerable amount of rice husk is wasted annually and it may increase with the time.

Rice husk ash (RHA) can be used as a highly reactive pozzolanic material to improve the microstructure of the interfacial transition zone (ITZ) between the cement paste and the aggregate in high-performance concrete. The utilization of rice husk ash as a pozzolanic material in cement and concrete provides several advantages, such as improved compressive strength and durability properties, reduced materials cost due to cement savings and environmental benefits related to the disposal of waste materials and reduced carbon dioxide emissions [4].

From 1980 to 1996, the world's annual consumption of Portland cements increased around 2 million tons to 1.3 billion tons. This was associated with major environmental problems since the cement manufacturing is the third largest CO_2 producer and for over 50% of all industrial CO_2 emissions (for every 1.0 ton of cement produced, 1.0 to 1.25 tons of CO_2 is released in the air) and 1.6 tons of natural resources is consumed to produce 1.0 ton of cement [5].

The reaction of the rice husk ash with the cement is dependent on several factors. Burning duration, burning temperature and the particle size of the ashes are the most significant. The optimum temperatures are around 600 ^{0}C for 2-3 hrs burning and 400 ^{0}C for 4 hrs burning. There is an optimum temperature for each burning time and burning at higher temperatures beyond a certain limit does not necessarily help to produce quality ash [6].

The grinding process would affect the water absorption capacity of concrete. Longer the grinding time, it may lower the water absorption rate [3]. With the proper grinding process, the RHA can be converted to the ultrafine particles. Therefore, of ultrafine RHA in to concrete to reduce permeability, water absorption of concrete and many beneficial effects on concrete performance [4].

This paper proclaims an experimental investigation on possibility of utilization of RHA from the byproduct of brick-kilns in Sri Lanka. In the present investigation, rice husk has been blended with Ordinary Portland Cement (OPC) at various percentages and experiments were carried out to investigate the mechanical properties. The results were compared with conventional OPC concrete.

2. MATERIALS AND METHODS

2.1 Rice Husk Ash

RHA for the experiment was obtained from the brick-kiln that used rise husk as a fuel. The burning temperature was within the range of 600 ^{0}C to 850 ^{0}C. The fully burnt ashes and the partially burnt ashes are separated by 10 minutes sieving. The ash passing 300 mm sieve size are considered as fully burnt ashes and it may be around 60% of the ashes collected from the brick-kiln. The particle size of the RHA is limited to 75 µm and fineness of the RHA was increased by the grinding process. The particle size distribution of 75 µm passing RHA is as shown in Fig 1. The average particle size is 18 µm and specific gravity is 2.16. Since the RHA is finer than cement it can be expected to have micro-filler effect as well as the pozzolanic reaction [7].

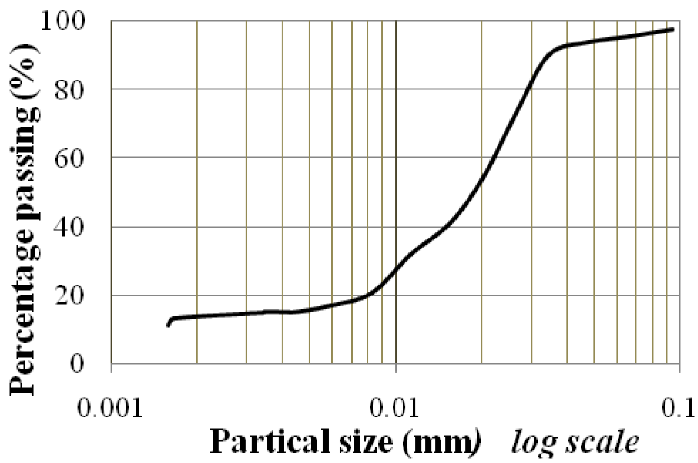

Fig 1: *Particle Size Distribution for RHA obtained from 75µm Passing*

2.2 Fine and coarse aggregates

The fine aggregates that used for the experiment were natural river sand passing from 4.75 mm sieve. The specific gravity of the sand was tested according to BS 812: Part 2: 1995. The specific gravity of fine aggregate was observed as 2.65.

The coarse aggregate that used for the experiment were crushed granite with maximum size of 20 mm. The specific gravity of the coarse aggregate was tested according to BS 812: Part 2: 1995. The specific gravity was observed as 2.65 for coarse aggregates.

Table 1 - *Water Absorption in Aggregates*

Type of aggregate	Water absorption
Fine aggregate	1.30 %
Coarse aggregate	0.31 %

2.3 Compressive strength of concrete

The Compressive strength of the concrete was measured by preparing cubes of 150x150x150 mm mould size. Concrete with 0.75 W/C ratios was used for the experiments. Mechanical vibrator was used to compact the concrete during casting. The moulds were removed after 24 hours and placed in the curing tank until the testing dates. The specimens were tested after 3, 7, 14, 28, 56, and 91 days with the loading rate of 0.3 N/mm^2/s. The average value of compressive strength was obtained by testing of three specimens.

2.4 Split tensile test

Split tensile test for the concrete was carried out according to the BS 1881:Part117:1983. Concrete cylinders of 150 mm diameter and 300 mm height were casted by using concrete with W/C ratio of 0.75. Mechanical vibrator was used to compact the concrete during casting. The moulds were removed after 24 hours and placed in curing tanks until the testing dates. The specimens were tested after 91 days with the loading rate of 0.03 N/mm^{2}s. The average value of tensile strength was obtained by testing of three specimens.

2.5 Surface water absorption test

The surface water absorption test was carried out with a concrete cube of 150x150x150 mm size. The W/C ratio of 0.75 was used for concrete mixture. The test was conducted after 28 days and the variation of the results were discussed as percentage deviation with reference to the control sample. All cubes were subjected to oven dry under 100 ^{0}C before testing. The surface water absorption of concrete was tested at 10, 30 and 60 min after the water head was released to the concrete surface.

3.0 RESULTS AND DISCUSSION

3.1 Compressive strength of concrete

The compressive strength of concrete cubes containing different amount of RHA (75µm passing) was tested according to BS 1881: Part 116:1983. The variation of compressive strength of concrete with days is shown in Fig. 2.

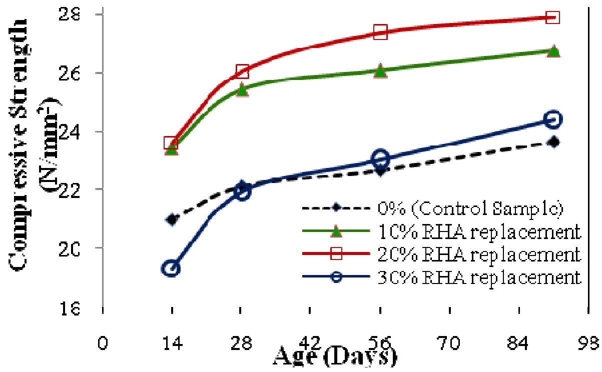

Fig 2: *Variation of Compressive Strength of Concrete*

Table 2: *Compressive Strength of Concrete with Difference Percentage of RHA*

Replacement	Average Compressive Strength (N/mm^2)			
	14 days	28 days	56 days	91 days
0%(Control)	21.00	22.10	22.69	23.63
10% of RHA	23.40	25.41	26.07	26.75
20% of RHA	23.60	26.04	27.33	27.87
30% of RHA	19.33	21.95	23.01	24.38

The maximum strength is achieved with 20% replacement of RHA for the cement in concrete. The maximum increment is around 17.9% than the control sample. Therefore 20% of RHA can be identified as the optimum percentage to replace the cement in concrete.
The effect of the particle size also tested during this experiment. The sieve sizes of 75 µm and 150 µm passing RHA mixed concrete specimens were compared with control sample. The variation of compressive strength of concrete containing different two sizes of particle sizes of RHA is shown in Fig. 3.

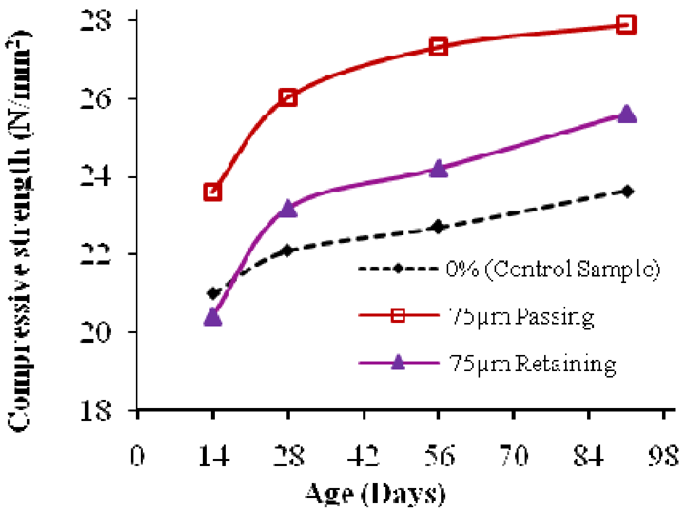

Fig 3: *Variation of Compressive Strength of Concrete*

The compressive strength development of concrete with 150 µm passing was lower than RHA passing 75 µm. However, the strength development rate is almost equal in both cases. Therefore, it can be concluded that when the fineness of the RHA increases the strength development of the RHA mixed concrete also increased. Similar results have been obtaining in previous studies as well. [7, 8]

3.2 Splitting tensile strength

The tensile strength of the concrete was tested according to the BS 1881: Part 117: 1983 by using the concrete cylinder. The experimented results are as follows.

Table 3: *Results of Splitting Tensile Strength*

Sample No.	Replacement (%)	Splitting Tensile Strength (N/mm^2)
1	0	1.86
2	10	2.60
3	20	3.01
4	30	2.56

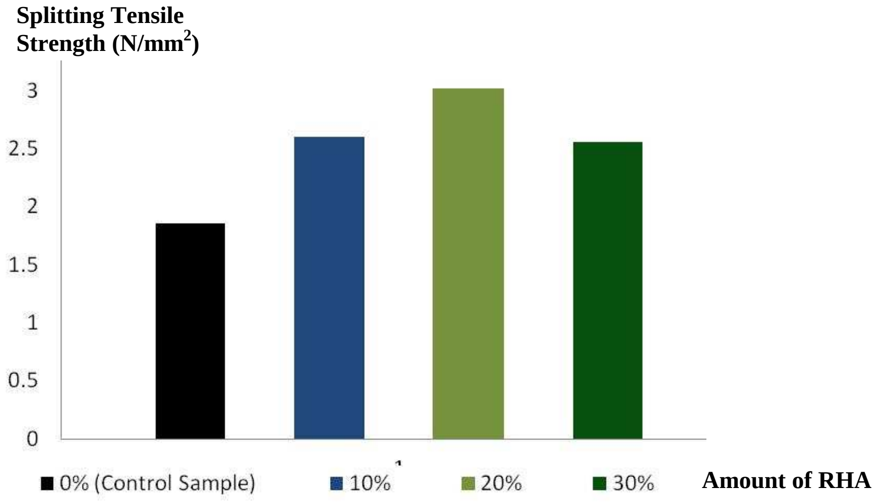

Fig 4: *Variation of Splitting Tensile Strength*

Fig 4 shows that significant increment in the tensile strength in concrete containing RHA. The maximum tensile strength is resulted with 20% replacement. Therefore tendency of cracking of concrete containing RHA can be considered as low compared to the normal concrete.

3.3 Surface water absorption test

The result of surface water absorption test is shown in Table 4. Around 15% reduction in water absorption was observed with the 20% replacement of fly ash. Around 11% reduction in water absorption with 20% replacement of RHA and around 10% reduction with combination of same amount of RHA and Fly Ash were observed.

Table 4: *Surface water absorption test results*

Time (min)	Control Sample	Percentage water absorption compared to control sample		
		Fly-Ash 10 and RHA10	RHA 20	Fly-Ash 20
10	100	88.24	91.18	86.76
30	100	91.11	88.89	86.67
60	100	88.57	85.71	82.86
Average	**100**	**89.31**	**88.59**	**85.43**

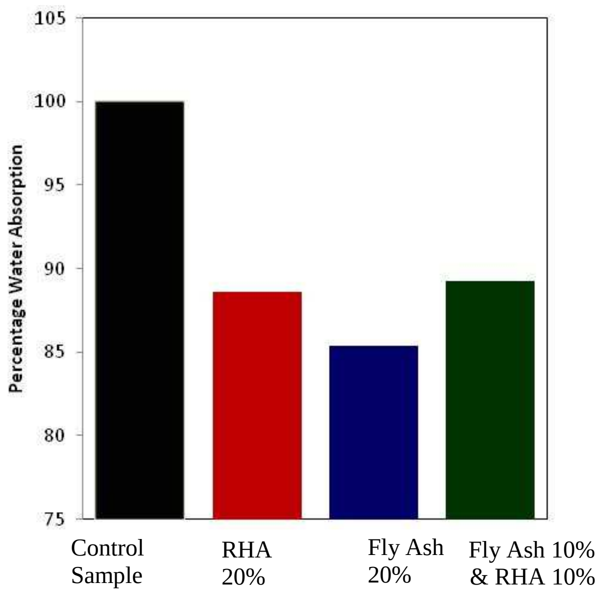

Fig 5: *Surface water absorption test results*

The concrete containing RHA has shown a considerable reduction in surface water absorption in concrete. Further addition of fly ash has given better result than the 20% RHA replacements. It may be due to higher fineness of fly ash than the RHA particles. However, according to the degree of replacement of RHA, the rate of water absorption can be different. Therefore, number of experiments consisting with different percentage of RHA is required to find out the optimum level of RHA to minimize the surface water absorption of concrete.

4.0 CONCLUSIONS

- It is possible to use the RHA that is produced as waste material from the land molded burnt clay brick production process in Sri Lanka to improve the properties of concrete.
- 20% replacement of RHA by mass of cement shows about 18% increment of compressive strength of concrete.
- There is considerable increment in splitting tensile strength with 20% replacement of cement by RHA.
- There is a significant reduction of workability in fresh concrete with the increase amount of RHA content in concrete. The Fly Ash can be used with the RHA to increase the workability of concrete.
- The performance of the concrete would depend on the fineness of the RHA. The finesses of RHA would increase the compressive strength of the concrete.
- The addition of RHA for the concrete decreases the water absorption of concrete. There was around 11% reduction in surface water absorption with the 20% of RHA replacement compared to control specimen.

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STRUCTURAL AND THERMAL PERFORMANCES OF RICE HUSK ASH (RHA) BASED SAND CEMENT BLOCK

[a]Nilantha B.G.P., [b]Jiffry I., [c]Kumara Y.S. and [d]Subashi G.H.M.J.

Department of Civil and Environmental Engineering, Faculty of Engineering, University of Ruhuna, Galle.
[d]E-mail : subashi@cee.ruh.ac.lk

Abstract: The sand cement blocks have been used in many countries of the world including Sri Lanka over a long period and it plays a major role in the building and construction industry. However, in order to make the indoor environment of the building as thermally comfortable as well as for achieving high structural performances, it is useful to use an alternative material to make blocks and bricks.

Rice Husk Ash (RHA) is a by-product obtained from the combustion of rice husk which consists of non-crystalline silicon dioxide RHA having pozzolanic properties would reduce the demand of Portland cement whose cost has risen in Sri Lanka. The paper focuses on the study of structural characteristics and thermal performance of Rice Husk Ash (RHA) based sand cement blocks through an experimental investigation.

Strength characteristics of the blocks were investigated by conducting laboratory experiments. Thermal behavior of the block was investigated by comparing the variation of indoor temperatures in two model houses constructed with the sand cement blocks and Rice husk Ash (RHA) based sand cement blocks. It was found that the optimum compressive strength of RHA based cement sand block is achieved at 5% replacement level. The RHA based sand cement block make indoor environment more thermally comfortable than the sand cement block.

1. INTRODUCTION

Different types of bricks and blocks are used in building construction in Sri Lanka and also in other countries. They are generally used for load bearing and non load bearing walls. Structural performances and thermal comfortable are mainly considered when masonry units are used for constructing walls. Blocks should absorb the heat in the day time and slowly releasing it in the night time, to ensure internal temperatures are consistent during the day time and the night time.

Block units absorb water due to their porous nature. The volume of water absorbed is an indication of the pore volume which depends on the interstitial arrangement of the particles of the constituent material at micro level. In using blocks for external wall in humid climate, the density and water resistant ability of the blocks must be considered in order to minimize penetration of moisture or rain water into the interior of the building. Similarly, when block work is to be constructed as canals for drainage, blocks to be used must have a very low value of water absorption coefficient and hence, highly impermeable. Damp penetration weakens the blocks and block work can collapse. [1]

Sri Lanka is a tropical country where warm humid climate conditions prevail and the occurrence of hot discomfort is felt within the built environment during the day time [2]. Many building designers in this region have ignored the climate in their wall construction probably because they adopt same old traditional methods in their house construction. The difference in indoor thermal comfort levels can significantly affect the running cost of the building. The use of fans and air conditioners, which lead to higher electrical consumption, can be reduced with higher indoor thermal comfort rice husk ash blocks comprise of natural sand, water and binder.

This research study has concentrated only the sand cement blocks production. Sand cement blocks containing mixture of sand, cement and water are used extensively in many countries of the world especially in Sri Lanka. As the cost of cement is increasing significantly in these days, this research is carried out to replace the cement with Rice Husk Ash (RHA) by getting a satisfactory strength level and thermally comfortable for sand cement blocks. Since the Sri Lanka is a developing country, the block which is made with RHA is better to use in low cost building construction. Also the open air burning of RHA makes pollution to the environment. It is easy to get RHA for rice producing countries. Therefore replacing cement with RHA could be economically counterproductive for local sand cement block manufactures at the same time; the unit cost of making block can be reduced. Well graded fine materials of RHA will fill the voids and high compressive strength can be expected than currently using sand cement block as well as with the low water absorption properties.

In practice, the type of ash varies considerably according to the burning technique. The silica in the ash undergoes structural transformations depending on the conditions (time, temperature etc) of combustion. At 550°C – 800°C amorphous ash is formed and at temperatures greater than this, crystalline ash is formed. These types of silica have different properties and it is important to produce ash of the correct specification for the particular end use [3].

Open burning and oven burning can be used for producing the rice husk ash (RHA) and it cannot be suitable temperature for producing rice husk ash in an open burning, also lot of unburnt carbon can be exist in the rice husk ash. Even though, temperature can be controlled in oven burning less amount of rice husk ash can be produced with long time consuming. Hence it also not economical for producing the rice husks ash. Local brick burning place waste lot of amount of rice husk ash than above methods. Temperature can't be controlled and long hour burning is occurred. It may be economical and advantage if they use for manufacturing sand cement block with suitable compressive strength. Therefore this study investigates effect of partially replacing cement with the RHA mainly on the structural and thermal performances in the manufacture of masonry blocks.

2. OBJECTIVES

This research was carried out with the following main objectives;

- To investigate the utilization of locally produced Rice Husk Ash (RHA)
- To investigate an optimum proportion of the Rice Husk Ash (RHA) with satisfactory level of compressive strength.
- To find the Water absorption properties of the RHA based cement-sand block.
- To find the thermal performances of the block.

3. METHODOLOGY

3.1 Materials

For the purpose of this research Rice Husk Ash (RHA) was selected from the Kiln and sieve analysis was carried out so as to determine the particle size distribution. Particle size of 150µm RHA retained (Sample 1), 75µm RHA retained (Sample 2) and 75µm passing (Sample 3) were selected for the study (Figures 3.1 and 3.2).

3.2 Manufacturing of Blocks

First, the RHA was mixed with the cement and then the mortar was prepared as usual. Solid masonry block having the size of 360mm×100mmx170mm were cast with the mix proportion of 1:6 cement-sand by using local block manufacturing machine. The water-cement ratio was controlled to 0.64.

Characteristics of RHA based sand cement blocks having six different RHA contents (5%, 10%, 15%, 20%, 25% and 30%) have been examined so as to investigate RHA used as partial replacement of cement. The RHA based sand cement block was manufactured with Samples 1, 2 and 3 to investigate the compressive strength of blocks (Figure 3.3 and 3.4).

3.3 Compressive Strength

The compressive strength was investigated with the laboratory experiment by using crushing machine as shown in Figure 3.5. Three samples were tested for each replacement level at 28 days and averaged. The strength characteristics of RHA based sand cement blocks were compared with the 0% of RHA content cement sand bock (Control block). The block with replacement level which gives optimum compressive strength was selected for investigation of thermal performance.

3.4 Water Absorption

Water absorption test was carried out to identify and specify the water absorption properties of developed RHA based sand cement block (Figure 3.6). The blocks manufactured with Samples 1and 2 only used to test the water absorption.

Three samples of both sand cement blocks and developed RHA based sand cement blocks were used for water absorption test. First, the blocks were kept in an oven at a temperature of 100-105 °C, for the time period of 24 hours and the dry weights of the blocks were measured. Then the same blocks were immersed in water for the time period of 24 hours and the wet weights were measured. Water absorption of individual sample blocks was determined and the average values were computed.

3.5 Thermal Performances

To ensure the thermal property of RHA, two model houses were constructed, one from sand cement blocks and the other from the blocks manufactured with the Rice Husk Ash (RHA) (Figures 3.7(a) and (b)). All four walls of model houses were constructed with those blocks to explore the thermal properties. The size of the both model houses is 1m x 1m x 1m. For both model houses, cement-corrugated sheets were used for roofing and the floors were cemented. Identical conditions (i.e. Building sizes, roof and floor materials, Building orientation and size and orientation of openings) were provided for both model houses except the wall material. The mortar mix having 1:5 cement: sand was used to construct the walls of the model houses. Opening size is 500 mm x 900mm and the plywood sheets were used for doors.

Temperatures were measured throughout a day (24 Hours) in June to monitor the thermal behavior of both sand cement block and RHA based sand cement block. The weather condition on this particular day was sunny day. Outdoor air temperature, Indoor air temperature, outdoor block surface temperature and Indoor block surface temperature were measured for 15 minutes interval. Air temperature was measured by using a hygrometer and the block surface temperature was measured by using a digital thermometer

4. RESULTS

4.1 Chemical Composition of RHA

The Figure 4.1 shows the chemical composition of rice husk ash which collected from the kiln. The total percentage composition of iron oxide (Fe_2O_3=1.56%), Silicon dioxide (SiO_2=91.75%) and Aluminium Oxide (Al_2O_3=2.07%) was found to be 95.38%.

4.2 Compressive Strength

The average 28 days compressive strength of blocks manufactured with different RHA (Sample 2) replacement level is shown in Table 4.1

The variation of average 28 days compressive strength of blocks with RHA content for blocks manufactured with Sample 2 is shown in Figure 4.2. The optimum compressive strength was obtained at 5% RHA replacement level and it complies with the minimum standard value of 2.8 N/mm^2 according to BS 6073: Part 2: 1981 [4]. And also the variation of compressive strength with % of RHA for Sample 1, Sample 2 and Sample 3 at 28 days is shown in Figure 4.3.

4.3 Water Absorption

The average water absorption of blocks manufactured with different RHA (Sample 2) replacement level is shown in Table 4.2

Table 4.1: *Compressive strength of blocks manufactured with Sample 2*

Block Type (Percentage of RHA)	Average Compressive Strength (N/mm^2)
0% RHA	2.486
5% RHA	**2.928**
10% RHA	1.776
15% RHA	1.933
20% RHA	1.979
25% RHA	1.491
30% RHA	1.323

Table 4.2: *The Water absorption of Blocks*

Type of Blocks (Percentage of RHA)	Average Water Absorption (%)
0% RHA	14.255
5% RHA	**16.583**
10% RHA	17.499
15% RHA	21.788

The results show that the water absorption increases with the percentage of RHA content for blocks developed with Sample 2. But the acceptable value is 12% for masonry blocks according to BS 5628: Part 1: 2005 [5]. And the value obtained is greater than the acceptable value. Also the variation of water absorption with % of RHA for Sample 1and sample 2 is shown in Figure 4.4.

4.4 Thermal Performances

The variation of indoor air temperature with time for both model houses is shown in Figure 4.5. The results clearly show that, the indoor temperature of model house which was constructed with manufactured RHA (at 5% replacement level) based sand cement block is lower than that model house which was constructed with sand cement block for the time period of 9 a.m. to 12 a.m. and in the night time from 1 a.m. to 8 a.m. For the rest of the period of the day, the indoor temperature of model house which was constructed with manufactured RHA (at 5% replacement level) based sand cement block is almost same as the model house which was constructed with sand cement block.

The difference in outdoor surface temperature and indoor surface temperature for both model houses were compared in Figure 4.6. The results show that between 10.30 am to 2.00 pm the (outside- inside) surface temperature is greater for RHA based sand cement block than the sand cement block. And for the remaining time period of the day the fluctuations reduce for both model houses and almost the same temperature was observed.

5. DISCUSSION

5.1 Structural Performances

Compressive strength of RHA based sand cement block increase at 5% RHA (Table 4.1). It is the 17% development of the compressive strength comparing 0% RHA. This may be due to pozzolanic reaction of RHA. Hydration of cement increases the P^H value of the water. Under high P^H value SiO_2 in the mix dissolves. The hydrous silica reacts with Ca^{2+} and produce insoluble compounds (CSH) called secondary cementitous products. With the curing Insoluble compounds produce harden for the mixture. This may contribute to increase the compressive strength of the RHA based sand cement block at 5% RHA content. Further addition of RHA causes decrease in compressive strength.

The chemical reaction involves fixing of $Ca(OH)_2$ in liquid phase from the hydrating cement with the silica in the pozzolana. For lower percentage replacement in such as 5% the silica from the pozzolana is in required amount. This aids the hydration process producing with high compressive strength. For higher replacement level such as 10%, 15%, 20%, 25% and 30%, the amount of Rice Husk Ash in the mix is higher than required to combine with the liberated calcium hydroxide in the course of the hydration. The excess silica substitute part of the cementitious materials and consequently causing a reduction in strength.

However the water absorption results show the considerable water absorbent behavior has to the rice husk ash and also it is expected that the porosity increases with addition of RHA. Also as the RHA collected from the Brick Kiln, it may consist of the burnt clay particle and it would also absorb some amount of water.

5.2 Thermal Performances

The temperature variation shows (Figure 4.5) that the RHA based sand cement blocks have a significant thermal performance than the sand cement block. This shows that for a high thermal mass material, it should have the ability to absorb the heat at day time and release the absorbed heat at night time. Also the results showed that the maximum indoor temperature difference between two model houses is 2 °C and it was experienced at 1.00pm.

At the beginning of the late night, it is seems that the RHA based sand cement blocks started to release the heat, which was stored at day time, assuring the high thermal mass performance. This behaviour causes to maintain an approximately constant temperature, around the optimum, in the indoor environment, while sand cement block maintain a slightly lower temperature at night time and higher temperature in day time compared to the optimum.

6. CONCLUSIONS

The rice husk ash wasted from brick burning place is pozzolonic and therefore is suitable for use in manufacturing masonry blocks. Also the RHA based sand cement blocks can make indoor environments more thermally comfortable than the sand cement blocks with the satisfactory level of strength and the RHA based sand cement block is most suitable for internal wall as it has considerable water absorbent behavior.

ACKNOWLEDGEMENTS

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Figure 3.1 Brick Kiln

Figure 3.2 Collected RHA Sample

Figure 3.3 RHA based cement-sand blocks

Figure 3.4 Local block manufacturing machine

Figure 3.5 Compressive Strength Testing

Figure 3.6 Weighing Balance

Figure 3.7(a) Model House

Figure 3.7(b) Model House

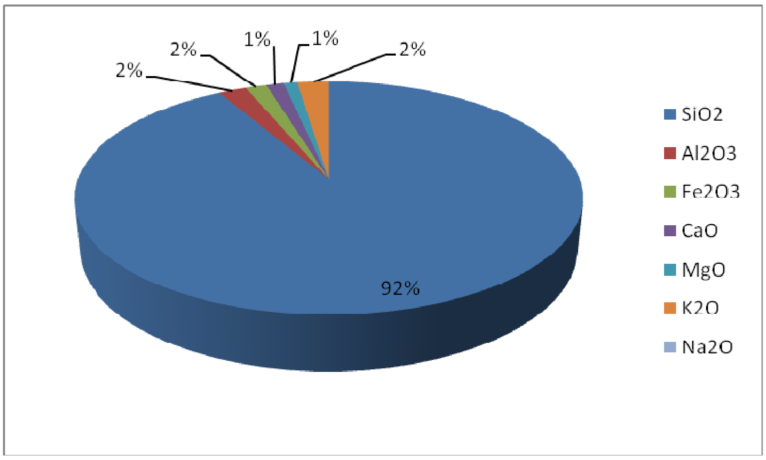

Figure 4.1: The Chemical Composition of RHA

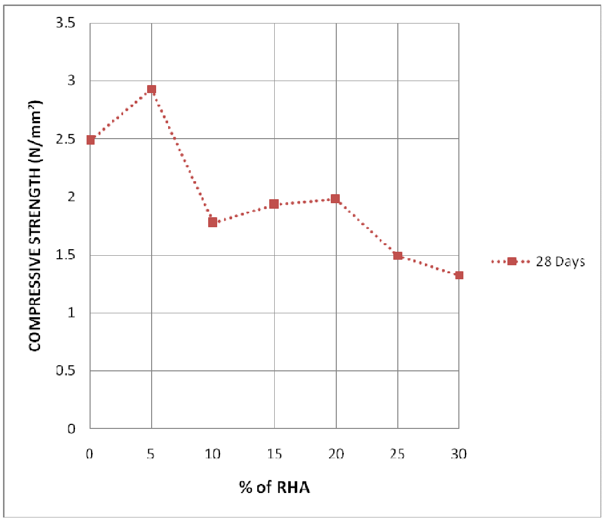

Figure 4.2: The Variation of Compressive Strength with % of RHA (Sample 2) content for with the replacement of cement

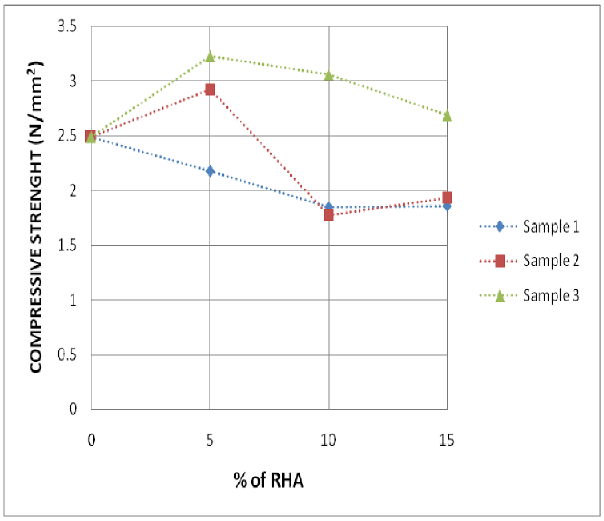

Figure 4.3: The Variation of Compressive Strength with % of RHA for various particle sizes of RHA at 28 days

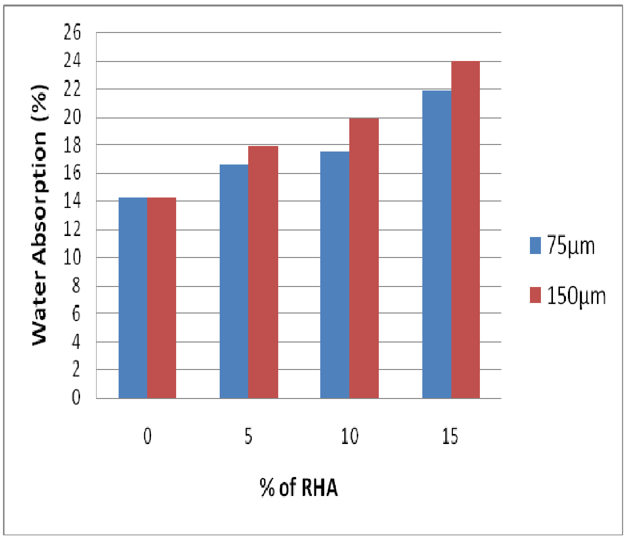

Figure 4.4: The Variation of water Absorption

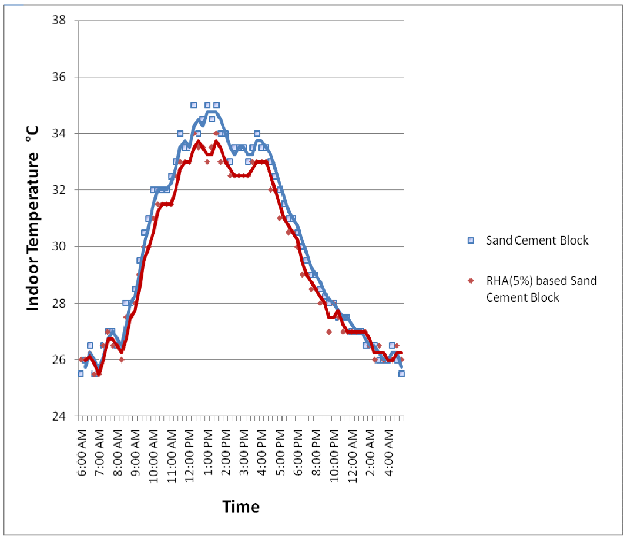

Figure 4.5: The Variation of Indoor Temperature with Time for both Model Houses

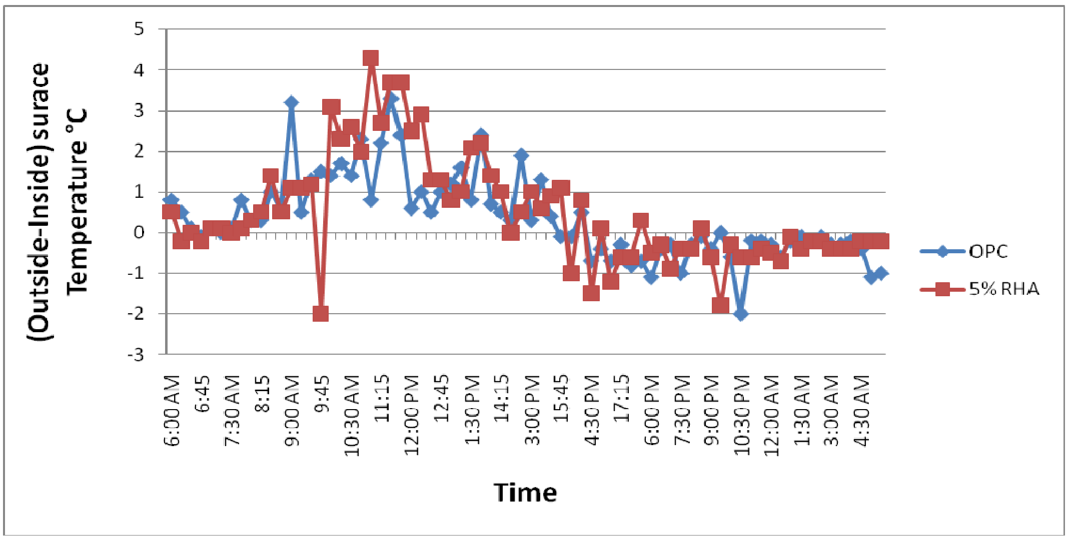

Figure 4.6: The Variation of (Outside-Inside) Surface Temperature with Time for both Model Houses

PHYTOCAPPING AS A COST-EFFECTIVE AND SUSTAINABLE COVER OPTION FOR WASTE DISPOSAL SITES IN DEVELOPING COUNTRIES

S. T. S Yuen[1]*, R.N. Michael[1], M. Salt[2], M.B. Jaksa[2], J. Sun[1]

[1]Department of Civil and Environmental Engineering, University of Melbourne, VIC 3010, Australia
[2]School of Civil, Environmental and Mining Engineering, University of Adelaide, SA 5005, Australia

*Corresponding author: Dr Samuel T. S. Yuen,
Department of Civil and Environmental Engineering, University of Melbourne, Vic. 3010, Australia.
E-mail: stsy@unimelb.edu.au; Phone: +61 3 8344 6716; Fax: +61 3 8344 4616

Abstract

Few waste disposal sites in developing countries are designed and operated as engineered sanitary landfills due to common technical and financial constraints. Phytocapping presents a natural soil-plant alternative to the conventional engineered landfill cover design. It requires less engineering input and has a lower cost than conventional impermeable covers as it only utilizes local recourses. It also offers the advantage of oxidating methane to reduce landfill greenhouse emissions. This type of covers has the potential to make a significant difference in the way that developing countries are capping their waste sites. This paper introduces the phytocap concept as well as discusses its relevance and advantages for developing countries.

Keywords – *evapotranspiration cover; A-ACAP; landfill; methane oxidation; greenhouse emission*

1. Solid waste management in developing countries

In many developing countries, much of the basic infrastructure for water supply, wastewater treatment, and solid waste management is limited (Johannessen and Boyer 1999). The cities in developing countries generate nearly 40% of the world's solid waste, which is approximately 500 million tons (Hoornweg and Thomas 1999). Rapid population growth and uncontrolled industrial development in the cities have severely affected urban environments, and inadequate management and improper disposal of solid waste is an obvious cause for the degradation of the environment in those countries (Schertenleib and Meyer 1992). It is not unusual to see developing countries spending 20-50% of their municipal operating budget on waste management, but still without satisfactory results (Hogland and Marques 2000). The most common waste management method is land disposal, mainly open dumping. Other waste management methods such as composting, incineration, recycling, anaerobic digestion, conversion to resource-derived fuel, are only sparingly used (von Einsiedel 2001).

Technical and financial constraints are two significant obstacles that have hindered waste management improvements in developing countries (Schertenleib and Meyer 1992; Ogawa 1996). In most developing countries, there is a serious lack of technical expertise as well as engineering infrastructure preventing the transition of open dumps to sanitary landfills. It is not uncommon to see inappropriate technologies that are not considered affordable and sustainable but directly funded and imported from high-income countries. Also given the low priority allocated to waste management, very limited funds are provided to the solid waste management sector by the governments. The funds are often not sufficient to achieve the level of protection required for public health and the environment.

2. Phytocaps compared to conventional landfill covers

One of the essential components of a well-engineered landfill is its final cap installed over the landfill after closure. The purpose is to control percolation of water into the waste, promote surface runoff, minimise erosion, control odour and prevent the occurrence of disease vectors. The cap is also important for landfill gas containment and capture.

The criteria of most interest to environmental regulators for measuring the performance of a landfill cap is the quantity of water draining through the cap into the buried waste. Conventionally, the materials considered to be most suitable for the construction of landfill caps have been impermeable barriers commonly constructed of compacted clay layers. However, there is a growing body of evidence to suggest that compacted clay barrier caps deteriorate within a short time frame (e.g. Albrecht & Benson, 2001; Dwyer, 2001; Albright et al., 2006)). For example, Albright et al. (2006) measured the performance of compacted clay barrier covers for a number of sites and concluded that large increases in the hydraulic conductivity of clay barriers with time are not uncommon, as compacted clay layers are subjected to cracking under cycles of repeated drying and wetting. Plant root activities can also have impact on the integrity of clay barriers.

Phytocapping presents a natural soil-plant alternative to the conventional compacted clay barrier cover design. Instead of providing a “rain-coat” barrier, it relies on the capacity of a porous substrate (usually of locally available soil) to store water together with the natural processes of surface evaporation and plant transpiration to remove the stored water as a means of controlling water ingress into a landfill, as shown in Figure 1.

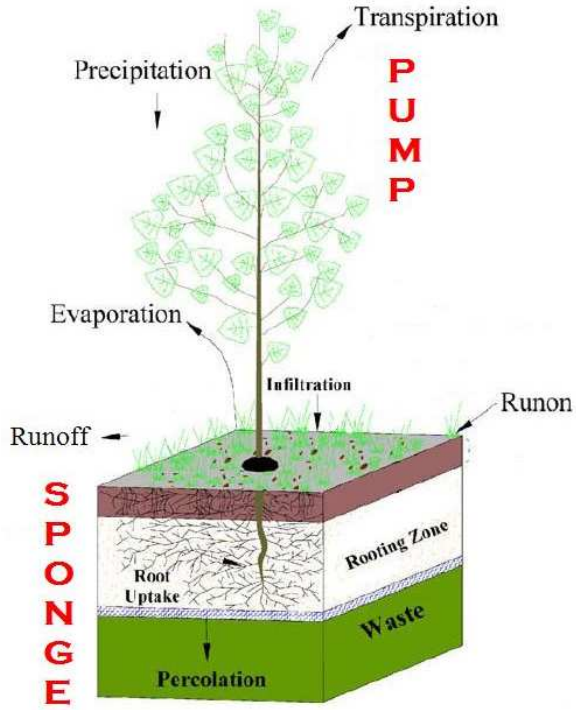

Figure 1 – *Schematic cross-section of a phytocap (adapted from Licht & Isebrands 2005).*

Phytocaps are often appropriately called evapotranspiration (ET) covers, soil-plant covers, store-and-release covers or monolithic covers as they rely on the capacity of the layer of soil to “absorb” water and the plant community acting as biological “pumps” to remove the stored water. The term phytocap is in predominant use in Australia due to its inclusion of phyto (the New Latin prefix for plant) which emphasizes the importance of the plant-based element of the system. While vegetation is incorporated as part of a barrier cap, it is primarily employed to prevent erosion and to improve the aesthetics of the site. On the contrary, vegetation plays an essential role in the phytocap function.

In terms of hydrological performance (i.e. minimising water percolation), evidence has been obtained from field studies to support that supposition that phytocaps can perform at least as well as, and in some cases better than, compacted clay layers (e.g. Dwyer, 2001; Albright et al., 2004). In contrast to compacted clay barriers, the performance of phytocaps is expected to improve over time as the vegetation community establishes and the soil profile develops. This expected advantage, alongside the potential for phytocaps to enhance the ecological value of a site gives phytocaps the potential for greater long term performance and sustainability.

As conventional barrier covers commonly include drainage layers aiming to reduce the hydraulic head acting on barriers to minimise percolation, their design is therefore inherently more complex and costly. The construction cost of phytocaps has been found to be lower, typically at only 35 to 72% of conventional covers (Hauser et al., 2001). Given the simplicity of phytocaps, their maintenance and repair costs can also be expected to be lower.

3. Phytocaps as biotic systems to mitigate landfill greenhouse emissions

Methane is the second most important greenhouse gas after carbon dioxide. Landfill gas typically consists of 40-60% methane and has thus been implicated in global climate change scenarios. Methane emissions from the waste sector account for about 18% of the global anthropogenic methane emission worldwide (Bogner et al. 2007).

In developed countries, landfill gas extraction and plant utilization are commonly mandatory for new waste disposal sites. Recent research has focused on the development of low-cost technologies that minimize methane emissions from existing landfills where gas collection systems have not been implemented or are not economically feasible (Scheutz et al. 2009). It has been demonstrated that porous biotic cover systems can mitigate landfill gas emissions by creating favourable aerobic environments to promote microbial methane oxidation in soil covers (Huber-Humer et al., 2008). While landfill gases may significantly affect root growth in cover soils, vegetation can also influence the biomass and activity of methanotrophs. It has been reported that plant cover could significantly improve soil methane oxidation potential (Stralis-Pavese et al. 2006). Wang et al. (2008) found that methanotrophic bacteria in landfill cover soils were stimulated by both plant growth and additional landfill gas supply.

The methane oxidation potential of phytocaps can be considered as a type of biotic cover where microbial activity is enhanced by plant roots. As active landfill gas collection is uncommon in developing countries, using phytocaps to oxidate methane and reduce greenhouse emissions would provide another major advantage over conventional impermeable caps.

4. Phytocap Design Approach

Phytocap functionality relies on the inherent properties and interaction between the local climate, the substrate (soil) and the selected plant community. Due to the reliance on local site characteristics, the design of phytocaps is necessarily specific to each landfill. When designing a phytocap, it is therefore important to transfer the phytocap design methodology rather than a site-specific design.

Shifting large volumes of earthen materials is an expensive undertaking, even within close proximity, and in order for a phytocap to be more cost effective than a conventional barrier cap, it is often essential for landfill operators to work with the soils that are readily at hand. The ideal phytocap substrate is one of high water storage capacity with properties that promote vital and sustained growth of the phytocap plant community. However, as the choice of substrate is often limited, the thickness of the soil can be manipulated to provide the required critical storage capacity during dry seasons. This requires analysis of the local historical meteorological conditions and the inherent water storage capacity of the soil. The selected plants must be able to exploit water from the full depth of the cover profile and their transpirative capabilities must be such that, together with evaporation, sufficient stored water is removed from the cover to prevent percolation into the underlying waste. The ideal plant community will maximise the number of days which transpiration occurs across the seasons.

A successful vegetation selection is even more critical when designing systems outside of the semi-arid and arid climatic zones, where there is a greater reliance on plant performance (Albright et al. 2004; Gross, 2004). The selection of plant species relies on the species' compatibility with the available soil substrate, local climate and long-term establishment on the site. Site assessment would involve defining broad climatic characteristics from historical data and investigating properties of the native plant communities with the endemic soils.

Observations of adjacent land and available literature can be used to determine the original vegetation communities of a study site including their species composition, structure, eco-hydrology and conservation status. Plants species should be selected for tolerance of limiting conditions rather than modifying or augmenting the substrate. This approach is considered better aligned with creating an economically viable and self-sustaining native phytocap plant community. Another core phytocap plant selection criterion is the inclusion of biodiversity to ensure the resilience of the plant community.

5. The Australian Alternative Covers Assessment Program (A-ACAP)

A-ACAP is an on-going field and laboratory research program (2006 to 2011) co-funded by the Australian Research Council and the Waste Management Association of Australia to investigate phytocover alternatives to conventional landfill caps in the Australian context. The program has established five full-scale test facilities across Australia to investigate the effects of a wide range of climatic conditions as well as site-specific phytocover designs. From tropical in the north to arid in the interior to temperate in the south, these test facilities are located across all 5 mainland states in Australia – Victoria, South Australia, Western Australia, Queensland and New South Wales, representing an excellent climatic diversity.

The major goals of A-ACAP are to demonstrate that phytocovers can perform to the satisfaction of regulators and to develop guidelines for their application, design and construction. The guidelines will address (1) control of percolation of water into the waste; (2) reduction of greenhouse gas emissions (with particular reference to methane oxidation); (3) sustainability of vegetative covers comprising a mixed flora of native species.

Central to the project’s experimental approach is the use of side-by-side comparisons of both conventional covers and candidate phytocovers. Large scale lysimeters together with other instrumentation are used to assess their hydrological performance. As an important improvement to similar studies conducted in the past, all test facilities are placed directly on top of active landfills. This arrangement is to allow realistic landfill interactions such as the effects of temperature and gas fluxes on cap performance. The inclusion of additional unlined test sections (i.e. without lysimeters) also allows the field experiment to investigate the methane oxidation potential of phytocaps in reducing landfill greenhouse emissions.

The field program is supplemented by laboratory and glasshouse experiments to investigate native plant performance as well as landfill gas transport related to methane oxidation (Sun et al. 2009). A detailed description of the A-ACAP program was provided by Wong et al. (2007).

The A-ACAP trial sites are located in a diverse range of climates and have utilised locally available soil materials and native vegetation species and associations, as shown in Figure 2. The trial sites were established between 2007 and 2008, commencing with Lyndhurst (near Melbourne) and McLaren Vale (near Adelaide).

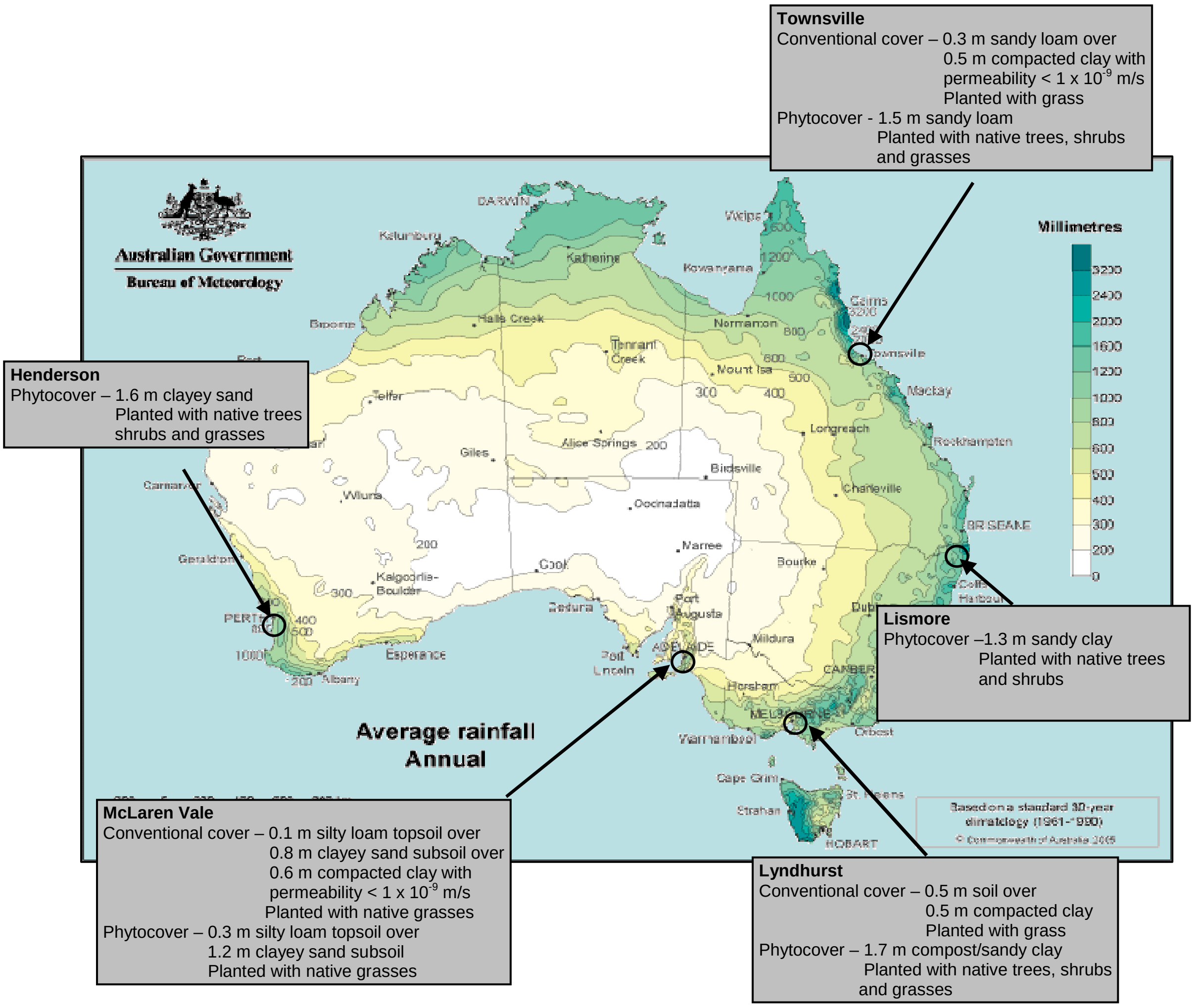

Figure 2 – *A-ACAP Trial Site Summary*

Data recorded at all sites includes rainfall, ambient temperature, relative humidity, wind speed and direction, soil volumetric moisture and soil temperature. Surface collection channels and pan lysimeters have been constructed to enable measurement of runoff and vertical drainage respectively from a defined area (trial plot).

Rainfall has varied at all sites over the monitoring period, as one would expect given their climatic diversity. Presenting the "Rainfall Year" as starting in April, i.e. at the commencement of the dry season for the northern sites or commencement of the wet season for the southern sites, Table 1 shows that rainfall has been below average in Lyndhurst and above average in Lismore and Townsville. Some data gaps exist in the drainage data presented in Table 1, however, based on the rainfall received during these periods, these inaccuracies are considered to have minimal impact on the data trends.

Table 1 – *Measured Rainfall and Drainage from Phytocovers at A-ACAP Trial Sites*

Site	Climate	Avg Rainfall (mm)	Apr 2007 to Mar 2008		Apr 2008 – Mar 2009		Apr 2009 to Mar 2010	
			Rainfall (mm)	Drainage (mm)	Rainfall (mm)	Drainage (mm)	Rainfall (mm)	Drainage (mm)
Lyndhurst, VIC	Cool temperate	810	585	43.1 (7%)	622	3.7 (1%)	749	0.3 (<1%)
McLaren Vale, SA	Mediterranean/ Semi-arid	520	230.4	0.0 (0%)	361	0.0 (0%)	654*	25.9 (4%)

Henderson, WA	Mediterranean	790	Limited data	778	277.0 (36%)	521	85.4 (16%)
Townsville, QLD	Tropical	990	Not yet constructed	2365	53.4 (2%)	1742	6.5 (<1%)
Lismore, NSW	Warm temperate	1340	Not yet constructed	1405	12.6 (<1%)	1490	5.7 (<1%)

* This rainfall amount included 84mm irrigation applied in August 2009 (see text)

The most evident trend is the decrease in drainage over time, with the exception of the McLaren Vale site. Comparing the drainage over time as a percentage of the total rainfall shows that at Lyndhurst, Henderson and Townsville, the proportion of drainage decreased over time, regardless of whether the rainfall increased or decreased.

When considering the drainage data at the McLaren Vale site, it is important to note that about 84 mm of irrigation was applied in August 2009 to simulate rainfall with an aim to investigate a stressed drainage response from the site. This additional application resulted in 26 mm of drainage. It should be noted that the plants on the lysimeter were dormant when this irrigation was applied and hence little transpiration was occurring.

Also of note is the extremely high drainage from the Henderson site, particularly when compared with the much higher rainfall sites of Lismore and Townsville. The main differences between these sites are:

- The soil used at the Henderson site was sand, with very little clay, while the soil used in Lismore and Townsville contained > 35% clay fraction;
- Rainfall is winter dominant in Henderson but summer dominant in Townsville, with Lismore receiving rainfall throughout the year;
- The plants at the Henderson site were slow to establish, remaining < 0.5 m high for the first few years, while plants at both Lismore and Townsville established quickly and > 1 m high after 1 year.

McLaren Vale has a similar climate to Henderson and was only planted with grasses. However no drainage (except in response to irrigation) has been measured from the site. The silt content in the soil used at McLaren Vale was > 20% and the native grasses established quickly over the entire site.

The experience from the A-ACAP trial suggests that phytocaps may be used in a range of climate types but careful selection of soil material and plant communities is required to minimise drainage. Increasing the soil profile depth may not be as effective by itself at controlling drainage. Finally, as plants establish, the drainage is likely to decrease.

6. Conclusions

Based on the above discussions, phytocapping could provide a cost-effective and sustainable cover option than the conventional barrier approach for landfills in developing countries. The obvious advantages are their lower costs, utilizing available recourses (i.e. use only local soils and native plants), and requiring less technical skills and engineering infrastructure to construct and maintain. While the phytocap concept was originated and has been trialled mainly in developed countries, this type of cover has the potential to make a significant improvement in the way that developing countries are capping waste disposal sites given their technical and financial constraints discussed earlier.

As phytocap functionality relies on the inherent properties and interaction between the local climate, the substrate and the selected plant community, the design of phytocaps is necessarily specific to each landfill. However, there is significant potential for the knowledge and design methodology learned elsewhere, such as the guidelines to be produced by the A-ACAP program, to be transferred and applied in developing countries.

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STRUCTURAL SYSTEM IDENTIFICATION FROM AMBIENT AND FORCED VIBRATION TESTING

N. Haritos

Associate Professor and Reader, Department of Civil & Environmental Engineering,
The University of Melbourne, Grattan Street, Parkville, Victoria, 3010, Australia

E-mail: nharitos@unimelb.edu.au
Telephone: + 61-03-83446829; Fax: +61 -03-83444616

Abstract: For over two decades now the author and his team have accumulated wide-ranging experience in the dynamic testing of structural systems (such as flooring systems in buildings and stadia, pedestrian and road bridges and other structural forms such as gantry frames) for the purposes of their condition and performance assessment. Identification of the in-service state of a structure is important:

- for determining its basic articulation and the nature of the conditions at its supports;
- for determining its structural performance characteristics whilst in service;
- as a precursor to performing an assessment of its load carrying capacity;
- for gauging the effect of a retrofit that may have been introduced on it;
- for performing general condition monitoring and assessment to gauge the effects of any degradation over a period of time or following a specific possibly damaging event, (eg an earthquake, storm or accidental load). This paper considers several examples of application of dynamic testing drawn from this experience to illustrate the utility of the approaches adopted concentrating on (but not restricted to) bridge engineering applications

Keywords: dynamic testing; experimental modal analysis; vibration modes; structural health monitoring

1. Introduction

Vibration-based assessment techniques for estimating the structural “health” or in-situ condition or damage of bridges and other structures has been receiving increasing attention in the engineering scientific community in recent years, [1, 2, 3]. The basis behind the techniques adopted for doing this is the simultaneous recording of vibratory response, resultant from some sort of forcing stimulus, taken over a pre-designed grid of measurement points on the structure. When the forcing stimulus is able to also be contemporaneously measured and recorded with the vibratory response, then it is possible to perform traditional frequency-domain based Experimental Modal Analysis (EMA) on the data capture, [4], whereas if this is not possible, then alternative time-domain based methods or approximate frequency-domain based methods can be used, [5, 6] for performing the EMA. Ambient vibration is the term coined for where the excitation stems from the normal operating conditions of the structure of interest eg, wind action on a tall building; wave action on an offshore structure; traffic excitation on a road bridge and/or ground motion induced by traffic on roadways underneath a flyover bridge; pedestrian excitation of a footbridge or floor system, etc. Forced vibration is the term coined for where the excitation is purpose introduced through a shaker system (eg electromagnetic/ hydraulic shaker) or an impact hammer/device. In the case of forced excitation the input forcing function and its characteristics are user controlled/specified and able to be measured whereas for ambient excitation, it is not normally possible to measure the excitation forcing function and its characteristics.

The primary function of EMA is to essentially produce a set of mode shapes and their associated natural frequencies of vibration and damping levels from the original contemporaneously measured data capture after suitable conditioning and transformation. Information gleaned from the modes of vibration themselves or from the transformed original data can then be used to perform a “health assessment” of the structural system to which the data capture corresponds. This could be in the form of a direct comparison of the modes themselves with those predicted by a Finite Element Analysis (FEA) model in which differences between these can be used to perform FEA model updating of modelling parameters. Alternatively, a number of approaches that operate on the measured response characteristics to determine a Damage Index can also be exercised in an attempt to extend the condition assessment to the point of identifying the damage location(s) and degree of severity.

1.1 Overview of approaches for condition identification

Traditional Experimental Modal Analysis (EMA) is able to be performed when controlled forced excitation takes place simultaneously with response measurement over a sufficiently detailed grid of points over the structure to enable mode shapes, mode frequencies and associated damping levels to be determined using specialist software that operates on Frequency Response Functions (FRFs) in the frequency domain, [6]. It is also possible to estimate mode shapes, modal frequencies and associated damping levels from response only measurements, as would be the case with ambient vibration testing, where the modeling takes place in the time domain, eg ARMA models, [5], or a simplified modal analysis via Operational Deflection Shapes (ODS) in the frequency domain, [7], or through the use of wavelet analysis (combined/hybrid approach in the time and frequency domains), [8]. In any of these cases, the experimentally obtained structural dynamic response characteristics as exemplified by the identified modal properties (or various derivatives therefrom that distinguish the style of "damage detection algorithm") can then be compared with those obtained from previous past testing or from Finite Element Analysis (FEA) models of a "healthy" structure to ascertain whether degradation/damage has taken place and if so the location(s) and/or degree of severity, [9, 10, 11].

Figure 1 provides a diagrammatic representation of the so-called *model updating* approach of structural system identification that is representative of one of these classes of structural condition assessment, by way of illustration of such a technique. The model parameters that are "updated" would include the degree of fixity at the abutment supports and the effective EI and GJ value of the bridge deck for the bridge example depicted in the case of Fig. 1. Matching criteria would include frequency matching and "goodness of fit" of the mode shapes as described by the Modal Assurance Criterion (MAC_{ij}) between the modeled mode shape $\{\phi_i\}$ and the experimentally observed mode shape $\{\psi_j\}$ given by:

$$MAC_{ij} = \frac{\left|\{\phi_i\}^T\{\psi_j\}\right|^2}{\{\phi_i\}^T\{\phi_i\}\{\psi_j\}^T\{\psi_j\}} \tag{1}$$

MAC values greater than 0.9 reflect a high degree of correlation (with 1.0 being a "perfect" fit) and values less than 0.1 associated with uncorrelated (virtually orthogonally disposed) modes.

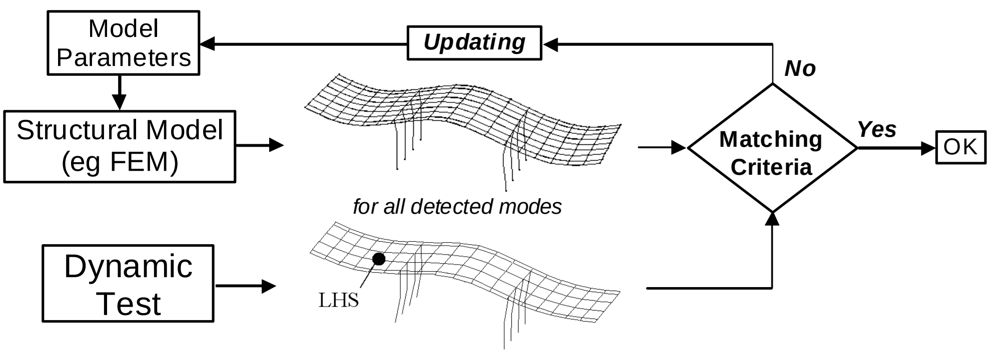

Figure 1: *Model updating approach for structural system identification*

1.2 Requirements for implementation

Ideally, the "hardware" necessary for performing structural system identification via vibration response measurements principally consists of:

i. A set of transducers suitable for measuring vibration response (eg accelerometers often used)
ii. An excitation source (impact device, shaker system (such as the Linear Hydraulic Shaker (LHS) or electromagnetic type) or healdrop excitation by a person (in the case of a floor system)

iii. A Data Acquisition System (DAS) with anti-aliasing filters, simultaneous sample and hold, modules for transducer signal-conditioning and excitation, and data storage/processing features

In addition to the above hardware set, suitable software to analyse the data capture for the key dynamic characteristics of the system under test, would also be necessary. The degree of sophistication of the hardware used for the data capture and the algorithms adopted in the analysis software is dependent upon the nature of the structural system under investigation and the degree to which the information content in the data capture is to be explored by the analyst.

Possible investigation scenarios would include:

i. Analysis of just the primary modal characteristics (natural frequency and associated damping) from response measurement records at a single point, eg from footfall excitation using simple curve-fitting of the response decay record itself (in the case of a floor system) or from performance of a Randec analysis [12], on multiple repeat record sets, (in situations where response only measurements have been performed). For this situation a simple inexpensive tri-axial accelerometer, such as a GCDC X6-2, [13], can be used, as it is compact and incorporates on-board storage of the data capture onto an SD card, which is easily accessible via USB connection directly onto the accelerometer unit itself.
ii. Performance of Experimental Modal Analysis (EMA) from ensemble-averaged FRF data determined from multiple repeat measurement of both the vibration response (over a grid of points) and the single point excitation force responsible for the vibration, [4, 6]. (This analysis determines the modal characteristics – mode shapes and corresponding natural frequencies and damping levels, of all participating modes in the test frequency range).
iii. Performance of a Simplified Experimental Modal Analysis (SEMA) from ensemble-averaged *Relative* Response Function (RRF) data evaluated from multiple repeat record sets of the vibration response (again taken over a grid of points) *relative* to a chosen reference response measurement point for when the excitation source is not measured, such as is the case with ambient vibration – eg pedestrian-induced vibration of footbridges and floor systems, traffic induced vibration of road bridges, [7], wind-induced vibration of building structures and trees, [14]. It is normally possible to determine the mode shapes and natural frequencies of only those modes that are sufficiently separated in frequency reasonably accurately using this simplified approach with estimation of associated damping being less reliable.
iv. Evaluation of modal characteristics via a Data Dependent Systems (DDS) approach using Auto-Regressive Moving Average Vectorized (ARMAV) modelling from time-domain records of response when excitation force measurement not possible, [5, 15]. Alternatively, commercial specialist software for dealing with data of this type that have built on and improved upon the DDS approach, such as ARTeMIS, [16], can be used.

In essence, the choice of analysis technique is dependent upon the particular conditions at hand. The experience gained with performing structural system identification via vibration response measurements of bridge, beam/frame and floor systems, and even trees, by The University of Melbourne, has embraced the full range of possibilities outlined above, [17, 18]. Some examples drawn from this experience, featuring key results, are presented in the sections that follow.

2. Structural system identification of road bridges from forced excitation

A 10-tonne Linear Hydraulic Shaker (LHS) system in combination with a 16-channel DAS involving 15 “roving” accelerometers has been used on a number of dynamic testing exercises on road bridges in country Victoria, Australia, for the purpose of performing structural system identification and gauging the in-service condition of the bridges so tested.

2.1 Application to typical simply supported span of multi-span RC deck on steel girder road bridge

A typical nominally simply supported span of McCoy's Bridge over the Goulburn River was dynamically tested so as to identify its in-service condition and verify the integrity of the composite action of the RSJ girders imbedded in the RC deck slab. Figure 2 depicts some of the features of this field test experience with a view of the hydraulic shaker mounted on the bridge deck, the 16 second long traces of vibration measurement taken contemporaneously with the force trace (a type of Swept Sine Wave forcing) and the associated ensemble averaged FRF function for this accelerometer (from 16 repeat test records) typical of an internal point from the 7 x 7 measurement grid adopted. Figure 3 depicts a photo of the bridge with its multiple simply supported spans, and the first three modes identified from DSMA which compare favourably with FEA predictions for a "healthy" bridge deck. Structural system identification here has verified the integrity of the composite action between the steel girders and the bridge deck, despite the age of the bridge at the time, being over 60 years.

2.2 Application to typical continuous span slab on beam bridge over Concongella Creek

Figure 4(a) depicts the Concongella Creek Bridge near Stawell, Victoria. This RC deck-on-beam bridge is over 60 years old and consists of three continuous spans and is meant to have been constructed en-castre with its abutment ends. Dynamic testing using the LHS and EMA via DSMA software enabled identification of several modes of vibration, the first three of which are depicted in Fig 4(b). It is noted that DSMA was capable of distinguishing Modes#2 and #3 despite these being so close in frequency. In addition, it is also clear from the observed modes that the fixity at the abutment ends has deteriorated to now act virtually as pins. The enhanced torsional stiffness due to aggregate interlock is reflected as a slightly higher modal frequency in Mode#2 than is predicted by FEA, whereas the observed and predicted modal frequencies of the flexural modes are in closer agreement.

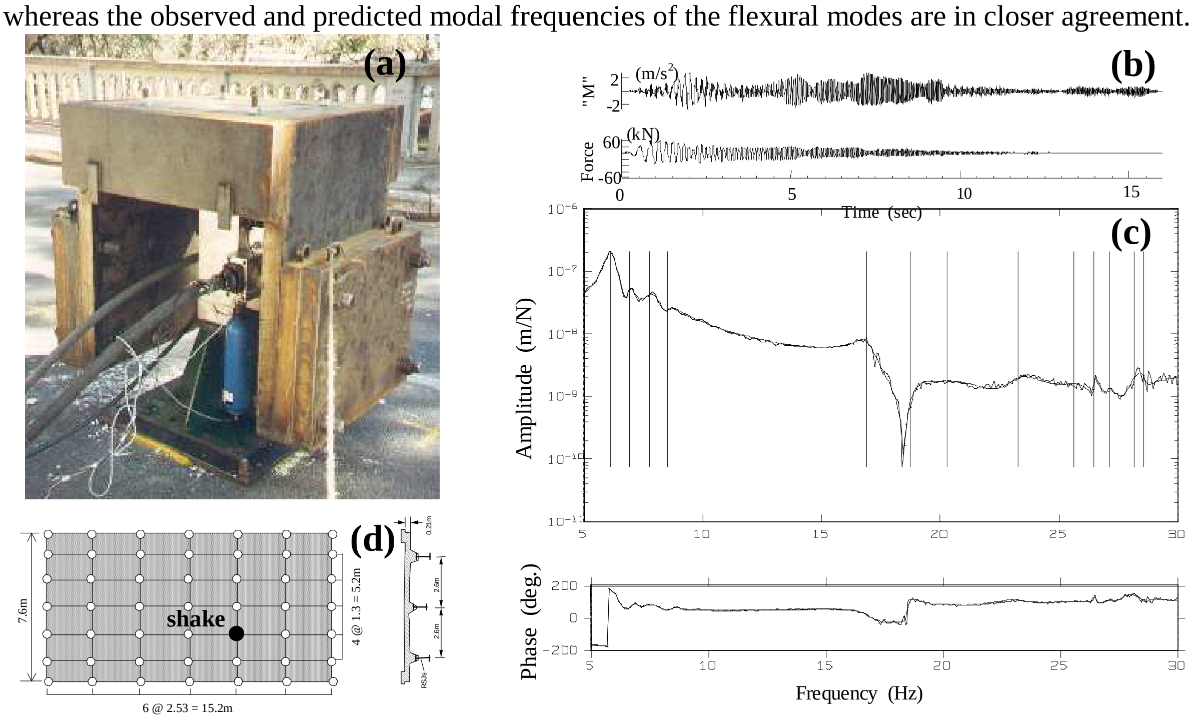

Figure 2: (a) *Hydraulic Shaker unit* (b) *Typical accelerometer and Forcing traces* (c) *FRF details for a selected accelerometer on a test span of McCoy's Bridge (d) Grid of accelerometer locations*

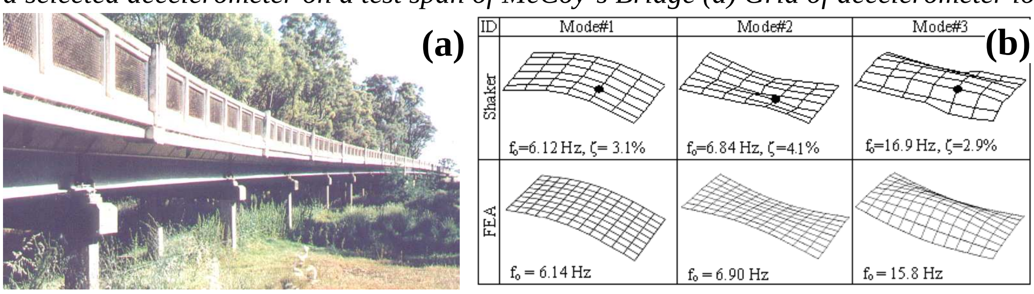

Figure 3: (a) *View of McCoy's Bridge* (b) *Comparison of EMA and FEA modal predictions*

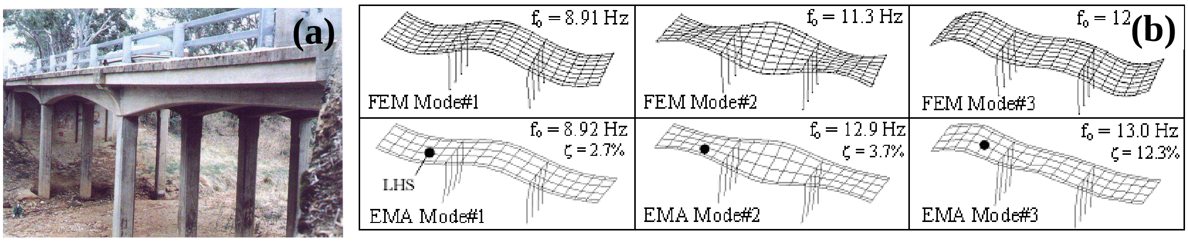

Figure 4: (a) *View of Concongella Creek Bridge* (b) *Comparison of EMA and FEA modal predictions*

3. Structural system identification from impulsive or ambient excitation

Simplified EMA, (SEMA), would suggest that the operational deflected shapes at or close to a natural frequency of vibration of a structure, can yield quite a good approximation to the corresponding mode shape of the structure at that frequency. Vibration response measurements taken contemporaneously over a grid of points on a structure, in the absence of an ability to measure the excitation responsible for the response, would be invaluable to such a modal identification exercise. Specialist software, such as ARTeMIS, can improve beyond the approximate capabilities of SEMA to provide better estimates of the modal parameters to include estimates of damping levels as well as modal frequencies and corresponding mode shapes. A couple of examples of SEMA drawn from our experience are provided in this section.

3.1 Application to heritage listed Swing Bridge at Sale, Victoria

The Swing Bridge at Sale, constructed in 1883 principally of wrought iron trusses with timber decking, and a balanced single swing span of 45.7m is located over the Latrobe River near Longford, close to the confluence of the Latrobe and Thomson Rivers, (see Fig. 5(a)). SEMA was performed on the bridge with excitation from a drop-weight device prior to and just after restoration works to verify the stability and integrity of the central pile group about which the bridge swings and the effect of replacement of the timber deck which was deemed to be in poor condition, [19, 20].

Dynamic response measurements were performed over a grid of 16 measurement points on one cantilever span before remedial work took place and repeated on the opposite balanced span when the bridge was part open with cantilever ends free, (first test series). These measurements were repeated at a later date, post installation of the refurbished timber deck, for the part open and fully closed conditions, (second test series). Model updating of an FEA model suggested that the effective soil stiffness determining the stiffness of the central pier and influencing the modal characteristics of the bridge as a whole was virtually mid range to the values inferred from soil tests at these piers, [19]. In addition, the effect of replacing the timber deck essentially improved the torsional stiffness of the cantilever sections leading to a higher torsional mode frequency compared to the original deck where timber planks were rather loosely fitting on the deck. Figure 5(b) provides insight into the modal results of the tuned/updated FEA model by providing a comparison with those observed from SEMA.

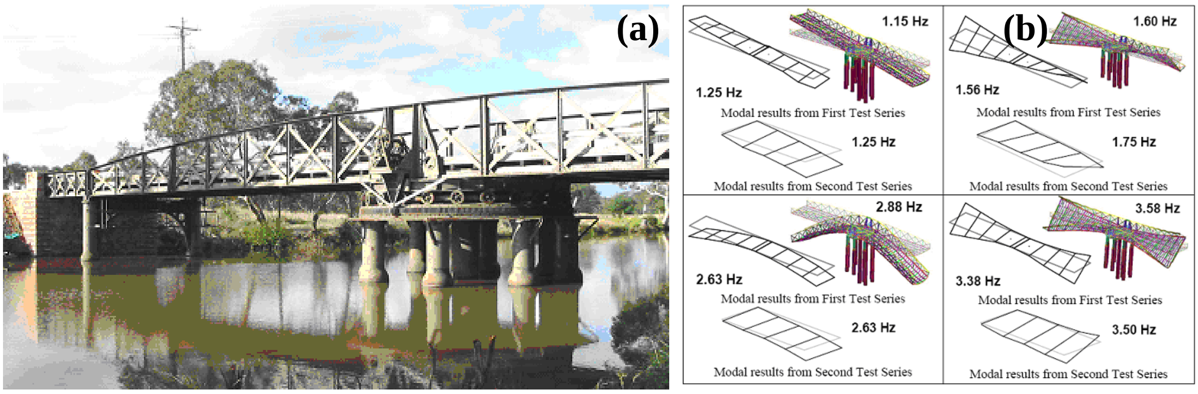

Figure 5: (a) *Sale Swing Bridge* (b) *Comparison of SEMA results pre and post restoration works*

3.2 Application to MCG Great Southern Stand

The design of the new Great Southern Stand at Melbourne Cricket Ground (MCG) involved the use of deep tapered steel cantilevers supporting pre-cast sectioned concrete units onto which seating could be bolted, once these were fixed in position. The steel cantilevers act as the primary structural support and as such were found to be significantly under-stressed under normal operating conditions (fully seated audience). There were deemed significant potential savings to be realised by reducing the plate thickness of these cantilevers, whilst satisfying strength requirements. However, these savings would only be possible if the primary mode frequency of the stand (and integer multiples thereof) remained clear of the frequency bands potentially able to be excited by crowd behaviour. Figure 6(a) & (b) depict a view of a portion (two adjacent sectors) of the stand being dynamically tested for its primary mode characteristics when construction was in progress for the remaining sectors, whilst Fig. 6(c) & (d) depict the response spectrum of a typical accelerometer and the results of SEMA taken over the relatively course grid of accelerometer measurements for excitation from an impact hammer. The spectrum depicts a cluster of closely spaced modes essentially of the same shape. The frequency conditions for the first mode cluster were deemed not to warrant re-design of the cantilever units to reduce plate thickness and overall material, fabrication and construction costs.

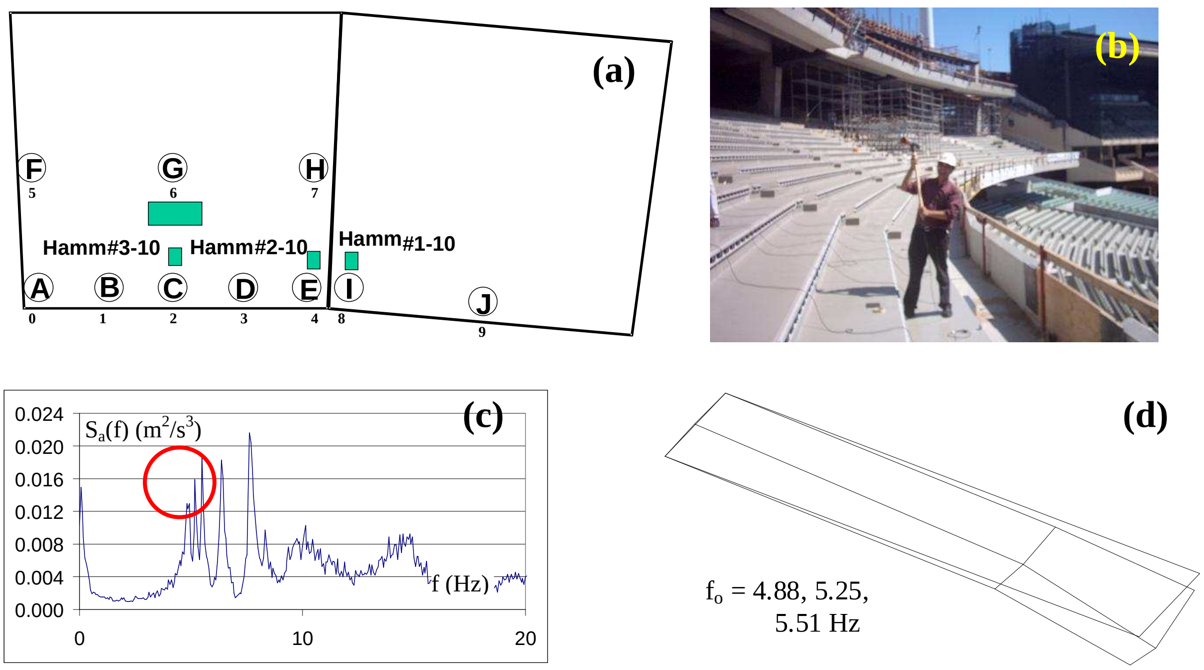

Figure 6: (a) *Accelerometer grid* (b) *Impact hammer testing in progress*
(c) *Primary mode "cluster" (accelerometer C response spectrum)* (d) *SEMA mode shape*

3.3 Application to gantry frames over Monash freeway

A much simpler approach towards ascertaining the primary mode frequencies of a selection of the fifteen newly constructed gantry frames over the Monash freeway was adopted recently, [21], with the advent on the market of the GCDC X6-1A and X6-2 model compact tri-axial accelerometer units with self-contained data logging and recording capabilities, [13]. The rather slender design (spans ranging between 40.3 and 53.6m with uniform box girder beam sections 1.2m wide and depth ranging from 0.8 to 1.2m, depending on the particular gantry frame considered) and the sharp features of the box edges of these gantry frames, prompted some concern over the possibility of these being susceptible to vortex induced excitation by wind and other associated aero-elastic instability phenomena such as galloping excitation. It was therefore decided to ascertain the in-service primary modal properties (frequencies and damping values) for the longitudinal and the horizontal and vertical transverse directions relative to the axis of the box girder beam section, of a selection of these gantries, using single point accelerometer response measurements to ambient excitation from traffic flowing on the freeway below the frames and from the surrounding wind.

Figure 7(a) and Figure 7(b) depict a photograph of a typical gantry frame and the acceleration response spectra for the X (longitudinal), Y (vertically transverse) and Z (horizontally transverse) directions for Gantry frame "G3", respectively. The primary mode frequencies in the three mutually orthogonal response directions X, Y and Z directions of 1.18 Hz, 1.47 Hz and 1.24 Hz are clearly distinguished as rather sharp peaks, reflecting the very low associated damping values of 0.4%, 0.3% and 0.8% critical, respectively. These conditions would suggest that a mean wind speed of 13.6 m/s incident at right angles to the plane of gantry frame G3 would correspond to a Strouhal number of 0.13 that would be associated with the possibility of vortex shedding with the potential of causing resonant vertical vibration on this gantry frame. The signage and other local attachments on the frame would likely disrupt the formation of any regular vortex street, but this contention remains to be further investigated.

4. Concluding remarks

This paper has considered a range of dynamic testing methods of varying degrees of sophistication aimed at performing some sort of structural system identification. Model updating techniques for tuning FEA model parameters to obtain high correlation of predicted modal characteristics to those observed from EMA or Simplified EMA (SEMA) have been overviewed and examples presented drawn from the author's experience of application to road bridges and the MCG grandstand. The versatility of newly developed compact tri-axial accelerometers with on board data-logging capabilities has been exemplified in this paper through reporting of their recent successful use for investigating the primary mode characteristics of slender gantry frames over the Monash freeway.

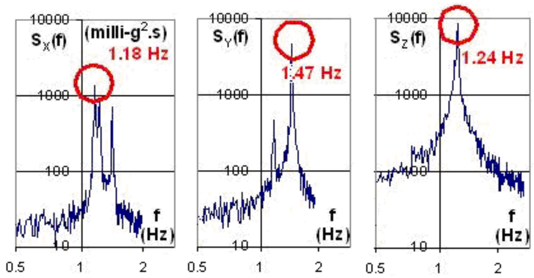

Figure 7: (a) *View of typical gantry frame* (b) *Ambient response spectra (X, Y, Z directions)*

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About the Author

N. HARITOS, B.Eng, M Eng Sc.., Ph.D. The University of Melbourne., FIE(Aust), CPEng. is an Associate Professor and Reader in the Department of Civil and Environmental Engineering, The University of Melbourne. He has over 30 years of post-graduate experience in research, academia and specialist consulting. His research interests include: structural dynamics of both land-based and offshore structures; hydrodynamics; condition/capacity assessment of bridge structures; experimental modal analysis testing and computers in modeling, simulation and control.

SEISMIC PERFORMANCE OF SUPER TALL BUILDINGS

Nilupa Herath, Priyan Mendis, Tuan Ngo, Nicholas Haritos

The Department of Civil and Environmental Engineering
University of Melbourne
Vic 3010, Australia
Email: rherath@civenv.unimelb.edu.au

Abstract:
With the rapid population growth and dynamic economic developments, the demand for residential, mixed use and commercial buildings has been increasing significantly all around the world. Due to the excessive increase in height of buildings in this era, there is a significant impact on the methods used to analyze and design of tall buildings. There is a clear acceptance within the engineering community that the specifications given in codes of practice are not suitable for very tall buildings.
General concepts, current methods of analysis and seismic performance of super tall buildings are reviewed in this paper. Further the effect of higher modes on the performance of super tall buildings is also discussed.

Key Words: *Super tall buildings, Seismic loads, Higher mode effects*

1. Introduction

During the last few decades, the application of new techniques and of new mechanical means was witnessed in virtually every human activity. It became clear in time that the innovation in architecture would come from those who grasps the possibilities of the new materials and techniques. The present day, construction of super tall buildings have become a trend and the impetus behind the surge of these tall buildings has been the need to satisfy the growing demand for office and apartment space. The convenience of having all of the services one needs in a single building is now becoming a reality with mixed-use buildings; some of these buildings may also bring the prospect of being able to live and work without leaving the building. Further the value of time and the high cost of gasoline may be part of the economic drivers that have sparked renewed interest in urban living and a return to the central city or downtown areas of many cities, which is a reverse trend from living in the suburbs as in the past. Due to this new technology towards super tall buildings, engineering judgment has to be made carefully.

As buildings are built to greater heights, the aspect ratio becomes larger, leading to excessive deflection problems due to the lateral load acting on the building. To meet the challenge for limiting deflections due to wind or seismic loads, innovative structural schemes are being continuously developed. Various wind bracing concepts have been developed and outrigger braced structural system is one of the most popular systems among them. There are well recognized analytical methods available for the analysis of buildings under wind or seismic loads. But due to these extreme heights in the super tall buildings, some general methods of analysis had to be reviewed and modified accordingly. In this paper, the behavior of super tall buildings under seismic loads will be addressed.

2. Earthquake ground motions for the design of tall buildings in high-seismic areas

In general, tall buildings respond to seismic motion differently compared to low rise buildings. A tall building tends to be more flexible than a low rise building, and in general would experience accelerations much less than a low rise building. On the other hand, a very flexible tall structure subjected to motion for a prolonged time may experience much larger forces if its natural period is near that of the ground waves. During the first few seconds of the earthquake, the acceleration of the ground reaches a peak value and is associated with relatively short period components, which have little influence on the fundamental response of the building. On the other hand the long period components that occur at the tail end of the earthquakes, and have periods closer to the fundamental period of the building have a profound influence on its behavior. Further tall buildings tend to experience greater structural damage when they are located on soils having a long period of motion because of the resonance effect that develops between the structure and the under laying soils.

It is very apparent that the specification of the earthquake ground motions is important in the process of designing a tall building. The development of the design criteria for performance-based seismic design may include provisions to ensure that a tall building will not collapse under a very rare event such as the maximum credible earthquake ground motion (usually defined as those ground motions having a 2% probability of being exceeded in 50 years, or having a return period of about 2400 years). This ground motion is usually represented in the form of response spectra that considers all of the seismic sources in the surrounding region of the site, given the estimated activity of the sources and the site characteristics for that given level of risk. (Lew 2007)

As this design spectra are estimated for a very long return period, the uncertainty included in the analysis is large. There is an uncertainty coming from the reality that there are very few recorded ground motions at close distances for large-magnitude events; thus, there is little guidance to constrain ground motion attenuation relations that attempt to model ground motions at these distances and magnitudes. Further, the long return period results in more uncertainty in the events, and the dispersion of the results increases the ground motion estimates. In addition to the above, most current ground motion attenuation relations do not extend to more than 2–5 s. As many of the new proposed tall buildings are well over 70 storeys, the fundamental period may be as high as 8–10 s. Thus, there is even a greater uncertainty as to the characteristics of the spectral ordinates of the ground motion spectra at these long periods.

3. Analysis and design of tall buildings for seismic loads

With the advent of computers and greater understanding of the earthquake phenomenon, the structural engineering profession has gradually moved towards more exact approaches in the seismic design of tall building. Relatively simple methods based on equivalent static loads are no longer satisfactory. The current design practice often requires more precise determination of local seismicity and critical ground motion characteristics and application of advanced dynamic analysis techniques using sophisticated computer programs. For most buildings, an inelastic response can be expected to occur during a major earthquake. Performance based design is one of the most popular design methods put into practice to incorporate inelastic behaviour. Some well recognized methods available in the design of building for earthquake forces are discussed in detail in the following sections.

4. Modal Analysis

One of the most commonly adopted means of designing structures for earthquakes in line with code legislation is to use response-spectrum analysis, also referred to as multi-modal analysis, in order to obtain estimates of structural response both in terms of design forces and displacements. The basis of the multi-modal analysis procedure is illustrated in Fig. 1.

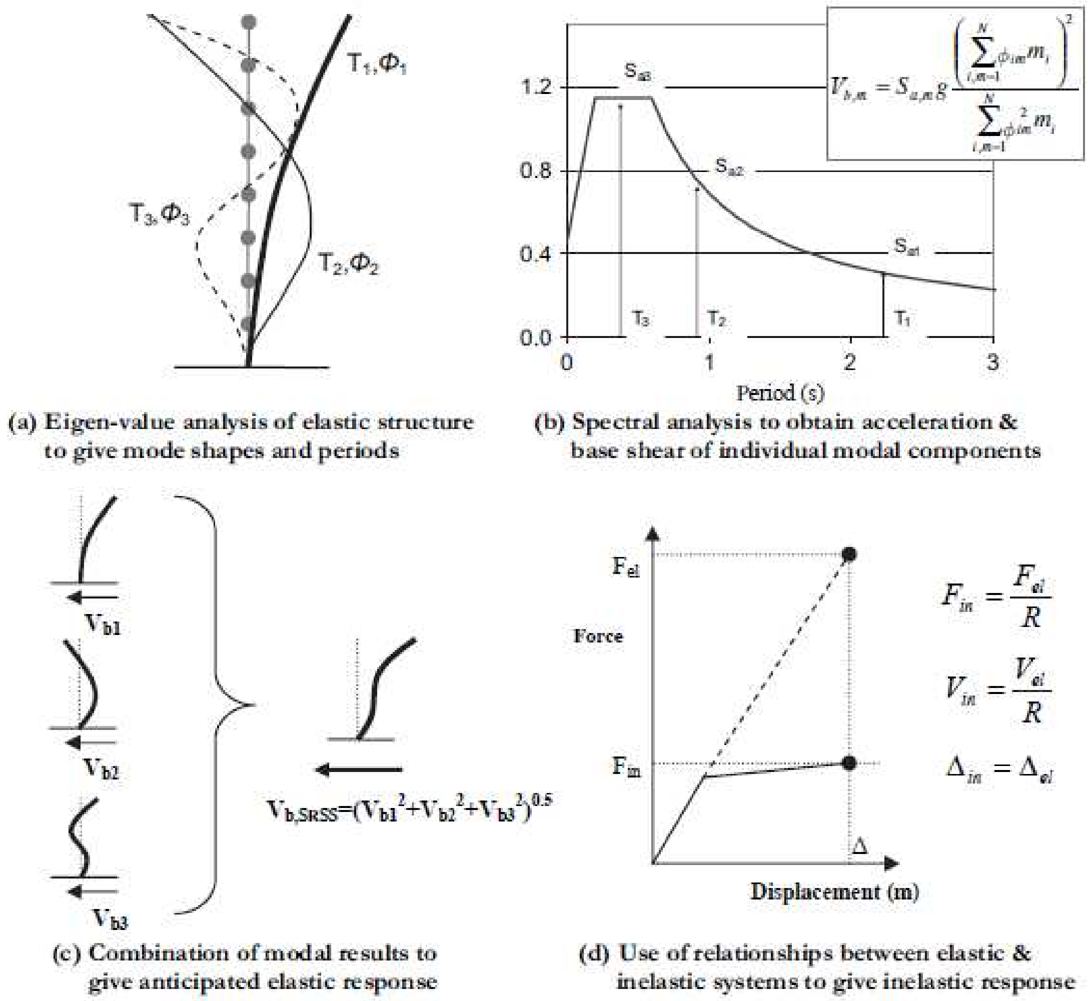

Fig 1 Modal analysis -Source: (Sullivan 2008)

The first step of the procedure, shown in Fig. 1(a), is to perform Eigen-value analysis of the structure with a given mass and elastic stiffness in order to identify its modal characteristics. The characteristics of particular importance are the modal periods and modal shapes. The modal periods are used together with the design acceleration spectrum to read off acceleration coefficients for each mode, as shown in Fig. 1(b). The mode shapes furnish the mass excited by each mode, which is then multiplied by the acceleration coefficient to give individual modal base shears. By distributing the base shear for each mode up the height of the structure as a set of equivalent lateral forces (proportional to the mode shape and mass distribution), the elastic-response is obtained for each mode. These components are then combined in accordance with established modal combination rules, such as SRSS or CQC (Chopra 2000) to provide design forces and displacements associated with elastic response, as indicated in Fig. 1(c). Finally, given that the actual response of the structure will be inelastic, a set of behavior factors, R, are then used to scale the building's elastic response to provide inelastic design actions. The assumed relationship between the elastic and inelastic response shown in Fig. 1(d) is representative of the equal-displacement rule.

5. Capacity Spectrum method

The capacity spectrum method was developed by Freeman (2004). By means of a graphical procedure, it compares the capacity of a structure with the demands of earthquake ground motion on the structure (Figure 2). The graphical presentation makes possible a visual evaluation of how the structure will perform when subjected to earthquake ground motion. This method is easy to understand. The capacity of the structure is represented by a force displacement curve, obtained by non-linear static (pushover) analysis. The base shear forces and roof displacements are converted to the spectral accelerations and spectral displacements of an equivalent Single-Degree-Of-Freedom (SDOF) system, respectively. These spectral values define the capacity spectrum. The demands of the earthquake ground motion are defined by highly damped elastic spectra. The Acceleration Displacement Response Spectrum (ADRS) format is used, in which spectral accelerations are plotted against spectral displacements, with the periods represented by radial lines. The intersection of the capacity spectrum and the demand spectrum provides an estimate of the inelastic acceleration (strength) and displacement demand.

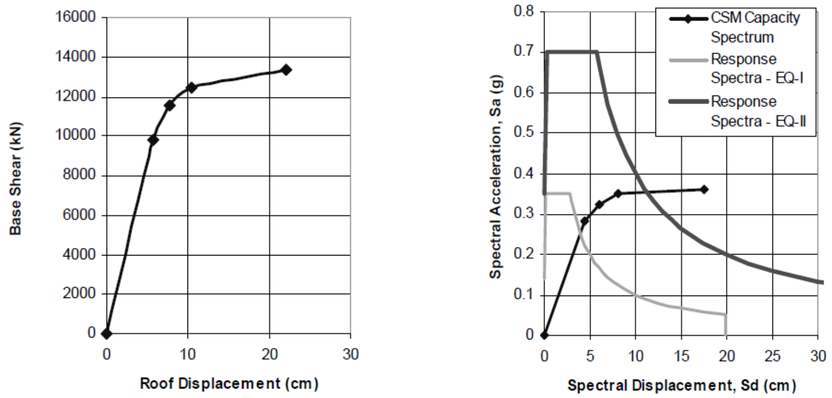

Fig 2: Demand and capacity spectra

6. N2 method

The N2 method is, in fact, a variant of the capacity spectrum method based on inelastic spectra. Inelastic demand spectra are determined from a typical smooth elastic design spectrum. Reduction factors, that relate inelastic spectra to the basic elastic spectrum, are consistent with the elastic spectrum. A simple transformation from a Multi-Degree-Of-Freedom (MDOF) to an equivalent SDOF system is used. Therefore the some deficiencies of the original version of capacity spectrum were overcome by introduction of the N2 method. The graphical representation of N2 method is shown in Fig 3.(Fajfar 1999)

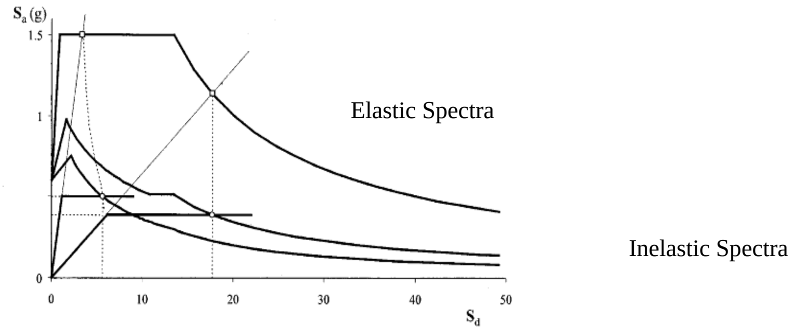

Fig 3: Demand and capacity spectra for inelastic analysis -Source: (Fajfar 1999)

7. Direct displacement-based design (DDBD)

The aim of the direct displacement-based design (DDBD) procedure is to develop an equivalent SDOF representation of the MDOF structure. This is achieved by assigning strength proportions and subsequently using the moment profile in the walls to set a design displaced shape before any analysis has taken place. Knowledge of the displacement profile and recommendations for the combination of frame and wall damping components enable representation of the structure as an equivalent single-degree of freedom system. Then the required effective period and the stiffness are determined using the substitute structure approach. The design base shear is obtained through multiplication of the necessary effective stiffness by the design displacement and the strength of individual structural elements is set taking care to ensure that initial strength assignments are maintained. A flow chart, which explains the procedure briefly, is presented in Fig 4 (Sullivan 2005; Priestley 2007).

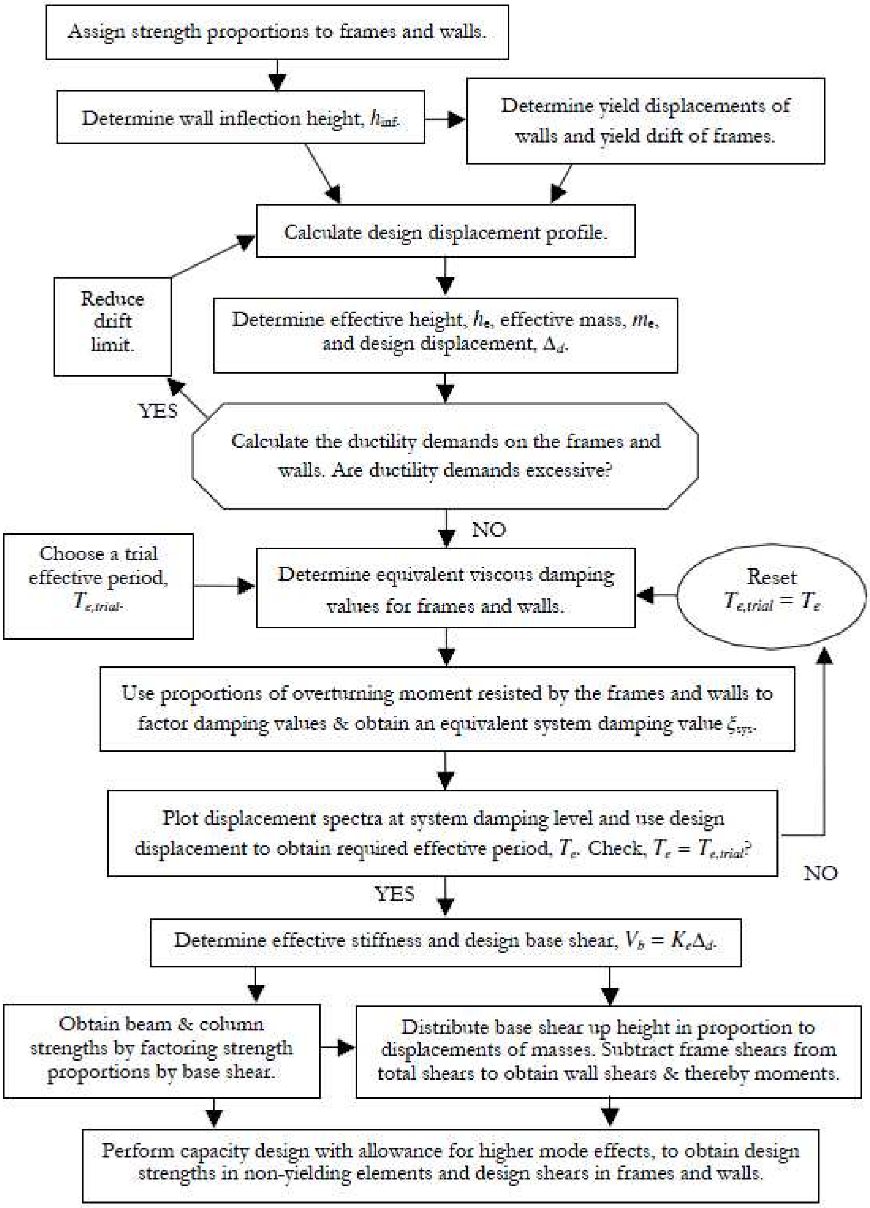

Fig 4 : Flow chart of Design methodology. Source :(Sullivan 2008)

8. Higher mode response of super tall buildings

As mentioned earlier, there is a clear acceptance within the engineering community that higher modes of vibration play an important role in the seismic response of structures. The fundamental period of super tall buildings may be high as 8-10s and the second mode period may be 2-3s. Therefore the second mode period also lies within the constant displacement portion of the response spectrum and this is different to the medium or low rise buildings. As it is already known that the higher modes play an important role in the analysis of super tall buildings, it is uneconomical to analyze these types of buildings always assuming the first mode governs the behavior of the structure. Most of the analysis methods derived so far to analyze the buildings for seismic loads apply the characteristics of the first mode. But when it comes to super tall buildings, as the higher mode participation is important, the methods derived so far has to be modified accordingly (Sullivan et al. 2008).

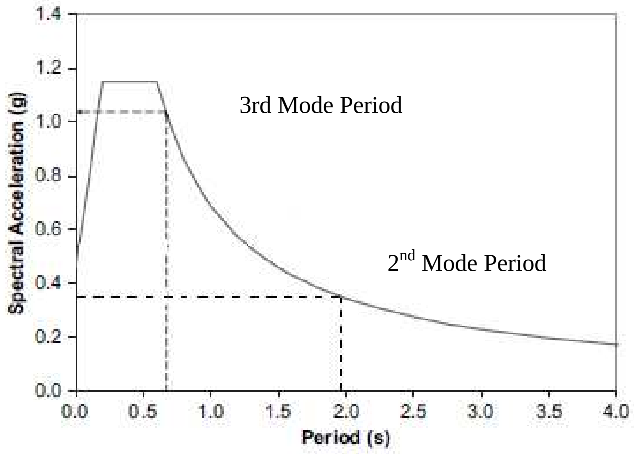

Fig 5 General Response Spectrum

Sullivan et al. (2008) have introduced a new concept of transitory inelastic modes and it has been shown that the characteristics of transitory inelastic modes for RC frame-wall structures may be approximated in a simple manner by performing Eigen-value analysis of a structure in which plastic hinge locations are assigned a low post-yield tangent stiffness. A new modal superposition approach that utilizes these transitory inelastic modes of vibration has been shown to provide much improved prediction of peak base shears than traditional modal superposition methods. The results of this work suggest that there may be real benefits in using transitory inelastic modal characteristics instead of elastic modal characteristics for the capacity design of structures. But this study was conducted only for low to medium rise buildings and this concept is explored for super tall buildings at the University of Melbourne.

9. Study conducted on the proposed method

An outrigger braced reinforced concrete residential building with a total height 300m and horizontal floor dimensions of 36mx36m was considered for the study. The core was assumed to be 12mx12m in square with 500mm thick walls on average. Floor to floor height was considered to be 3.5m and the depth of outriggers was 7m. Two outrigger locations were considered and one outrigger was placed at 58th level and the other outrigger was placed at 27th level. The optimum outrigger positions for wind was found by Chung (2009) and the same outrigger positions were considered for this study.
In order to investigate the behavior of the structure, the core, outrigger and outrigger braced columns were transformed to an equivalent frame with an equivalent stiffness and the analytical model was subjected to nonlinear time-history analyses in ANSYS 12 using the earthquake records listed in Table 2. These records were selected to be compatible with the code design spectrum. In modeling the structure in ANSYS 12, elastic properties were assigned to elements that were not intended to yield and Beam 4 elements were used for such elements. All structural components (beams and columns) are considered to have rigid connections and floors were modeled as rigid. A constant 5% damping was considered for the structure.

Record Name	Earthquake Name	Year	Record length (s)	Time Step (s)	Station	Unscaled GPA	Scale Factor
EQ 1	Parkfield	1966	43.8	0.02	Cholame - Shandon Array #13	0.50	1.8
EQ2	Tabas	1978	49.9	0.04	Tabas	0.81	1.5
EQ3	San Fernando	1971	41.7	0.02	Pacoima Dam	1.16	1.0
EQ4	Chile	1985	116.4	0.01	LLOLLEO, D.I.C	0.72	2.0
EQ5	Northridge	1994	60.0	0.02	New Hall	0.20	2.0

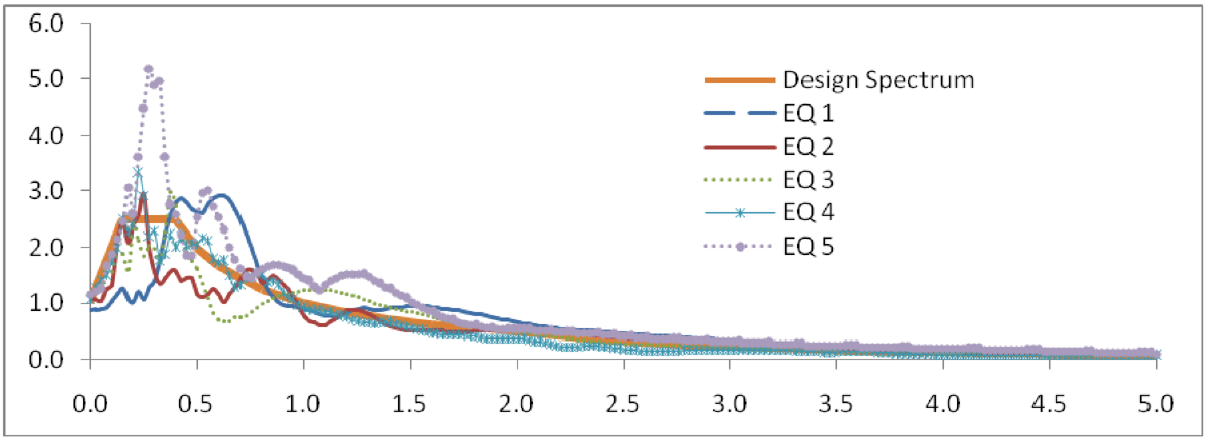

Fig 6 Acceleration response spectra at 5% damping for the five spectrum-compatible earthquake records

Fig. 5 indicates that the periods of the higher modes in outrigger braced building will generate larger spectral accelerations. Fourier analysis was used to study the dynamic response of the building and the frequencies of each mode were compared with the elastic periods of the structure. For this process, base shear variation with time was taken and subjected to a Fourier analysis using the program Seismosignal to identify the frequencies that most influence the base shear response. The peak Fourier amplitude is an indicative of a dominant structural frequency and it can be seen from Fig 7 that, the average peak Fourier amplitude for all the earthquake records is considerably lower than the elastic frequencies for second and third mode frequencies as shown in dotted lines. Therefore it can be concluded that even for tall buildings with outriggers, a period lengthening for the higher modes can be expected. Due to this, there will be a significant effect on the magnitude of the higher mode forces and this change of forces has to be accounted in the Response spectrum analysis. For shorter structures, both the elastic and the lengthened higher-mode periods may lie within the constant acceleration plateau, implying that the spectral accelerations of the higher modes will not be affected by the phenomenon of period lengthening. However, for longer-period structures such as the model structure used in this study, when the development of inelasticity shifts the periods along the descending branch of the acceleration spectrum, significant reductions in higher-mode response could be expected as explained in Fig 5.

From Fig.7, it is clear that the second and third mode periods obtained from Fourier analysis is comparatively less than the elastic mode periods. Therefore the transient inelastic mode method introduced by Sullivan et al. (2008) will be adopted for the calculation of base shear of the building. This study is continuing at the University of Melbourne

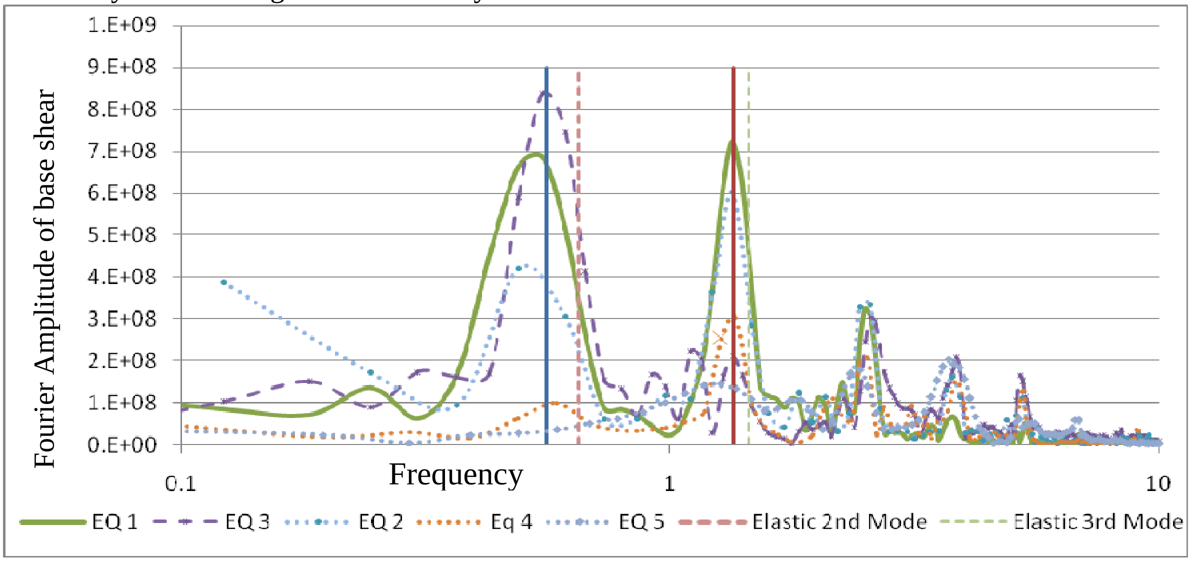

Fig 7: Results of the Fourier analysis of Base shear

10. Concluding Remarks

General concepts and widely used seismic analysis methods (Displacement based method, N2 method, Capacity Spectrum Method) are briefly reviewed in this paper. It is shown that these methods may not be directly applicable to super tall buildings due to the higher mode effects. The phenomenon of higher-mode period lengthening has been illustrated through the non linear dynamic analysis of a 300m tall building. This work is continuing at the University of Melbourne.

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SEISMIC ASSESEMENT OF MASONRY INFILL RC FRAMED BUILDING WITH SOFT GROUND FLOOR

[1]G V MULGUND, [2]D M PATIL, [3]P B MURNAL, [4]A B KULKARNI

[1]Phone: +919527003357 ; Fax: +912328223507 ; E-mail: gvmulgund@gmail.com
[2]Phone: +919975703454 ; E-mail: patil.dhanaraj@gmail.com
[3]Phone: +919970700044 ; E-mail: pmurnal@yahoo.com
[4]Phone: +919423267474

Abstract: Construction of multistoried buildings with open ground floor is a common trend of urbanization of cities of many parts of many countries. Social and functional need to provide parking space at ground level outweighs the seismic vulnerability of such buildings. Generally these buildings are designed as RC framed structures without regards to structural action of masonry infill walls present in the upper floors. In the present paper an investigation has been made to study the behavior of RC frames with various arrangement of infill when subjected to dynamic earthquake loading. Result of bare and infill frame are compared and some conclusions are made in view of *IS* -1893(2002) code.

Key words: *Masonry infill, RC frames, Soft storey*

1. Introduction

Reinforced concrete frames with masonry in-fills are a popular form of construction in high-rise buildings. Social and functional needs for vehicle parking, shops, reception, etc. are compelling to provide an open first storey in high rise building. Parking floor has become an unavoidable feature for the most of urban multistoried buildings. Though multistoried buildings with a parking floor (soft storey) are vulnerable to collapse due to earthquake loads, their construction is still widespread. These buildings are generally designed as framed structures without regard to structural action of the masonry infill walls. They are considered as non structural elements. Due to this, in a seismic action RC frames purely acts as moment resisting frames leading to variation in expected structural response. The effect of infill panels on the response of R/C frames subjected to seismic action is widely recognized and has been a subject of numerous experimental and analytical investigations. In the current practice of structural design in India, masonry infill panels are treated as nonstructural elements and their strength and stiffness contributions are neglected. In reality, the presence of infill wall changes the behavior of frame action into truss action thus changing the lateral load transfer mechanism.

In the present study, seismic performance of various configurations of infill panels in RC frames are compared with bare frame model using nonlinear analysis. The main objectives of this study were to investigate the behavior of multistory, multi-bay soft storey infilled frames and to evaluate their performance levels when subjected to earthquake loading.

2. Description of Structural Model

Significant experimental and analytical research is reported in the literature, which attempts to understand the behavior of infilled frames. Different types of analytical models based on the physical understanding of the overall behavior of an infill panel were developed over the years to mimic the behavior of infilled frames. The single strut model is the most widely used as it is simple and evidently the most suitable for large structures (Das and Murthy, 2004). Thus RC frames with unreinforced masonry walls can be modeled as equivalent braced frames with infill walls replaced by equivalent diagonal strut which can be used in rigorous nonlinear pushover analysis. Using the theory of beams on elastic foundations (Smith and Carter, 1969) suggested a non dimensional parameter to determine the width and relative stiffness of diagonal strut. Mainstone suggested another model representing the brick infill panel by equivalent diagonal strut. The strut area, A_e, was given by following expression:

$$A_e = w_e t \quad (1)$$

where,

$$w_e = 0.175\,(\lambda\, h)^{-0.4}\, w \quad (2)$$

$$\lambda = \sqrt[4]{\frac{E_i\, t \sin(2\theta)}{4 E_f\, I_c\, h'}} \quad (3)$$

where,

E_i = the modulus of elasticity of the infill material
E_f= the modulus of elasticity of the frame material
I_c = the moment of inertia of column
t = the thickness of infill
h = the centre line height of frame
h'= the height of infill
w'= the diagonal length of infill panel
θ= the slope of infill diagonal to the horizontal.

In this study, five different models of an eight storey building, symmetrical in the plan are considered. Usually in a building 40% to 60% of masonry in-fills (MI) are effective as the remaining portion of the Masonry Infills (MI) are meant for functional purpose such as doors and windows openings (Pauley and Priestley, 1992). In this study the buildings are modeled using Masonry Infills (MI) but arranging them in different manner as shown in the Figure 1. The building has four bays in North-South and East-West directions with the plan dimension 20 m × 16 m and a storey height of 3.0m each in all the floors. Further inputs include unit weight of the concrete is 25 kN/m^3, unit weight of masonry is 20 kN/m^3, Elastic modulus of steel is 2×l0^8 kN/m^2, Elastic Modulus of concrete is 22.36×l0^6 kN/m^2, Strength of concrete is 20 N/mm^2 (M20), Yield strength of steel is 415 N/mm^2 (Fe-415) and Live-load is 3.5 kN/m^2. The modulus of brick masonry and strut width is obtained using FEMA (306, 1998) recommendations i.e. $E_m = 550 \times f_m$=2035 N/mm^2. Window openings are assumed tiny relative to the overall wall area thus not included in the as they have no appreciable bearing on the general behavior of the structure (Jain, *et al.,* 1997).

Following five different models are investigated in the study.

1. Model I : Bare frame
2. Model II : Masonry infill are arranged in outer periphery
3. Model III: Masonry infill are arranged in outer periphery with soft storey
4. Model IV: Masonry infill are arranged as inner core with soft storey
5. Model V : Masonry infill are arranged as (+) cross in plan with soft storey

3. Nonlinear Analysis

Nonlinear analysis is the method used for determining the earthquake response of the structural systems. This method varies in methodology as nonlinear static pushover analysis and nonlinear dynamic time history analysis. In this study, nonlinear static pushover analysis is used to determine earthquake response of the structure using ETABS 9.5 (Computers and Structures) software.

Typical pushover analysis was achieved using displacement control strategy, where in the whole structure was pushed to evaluate the seismic performance of the buildings using preselected lateral load pattern until the roof displacement reaches the target value. The lateral load pattern was distributed along the height of the structure in such a way that each floor is subjected to a concentrated force. Two invariant load patterns were utilized to represent the likely distribution of inertia forces imposed on the building during the earthquakes. The invariant load patterns used are the following:

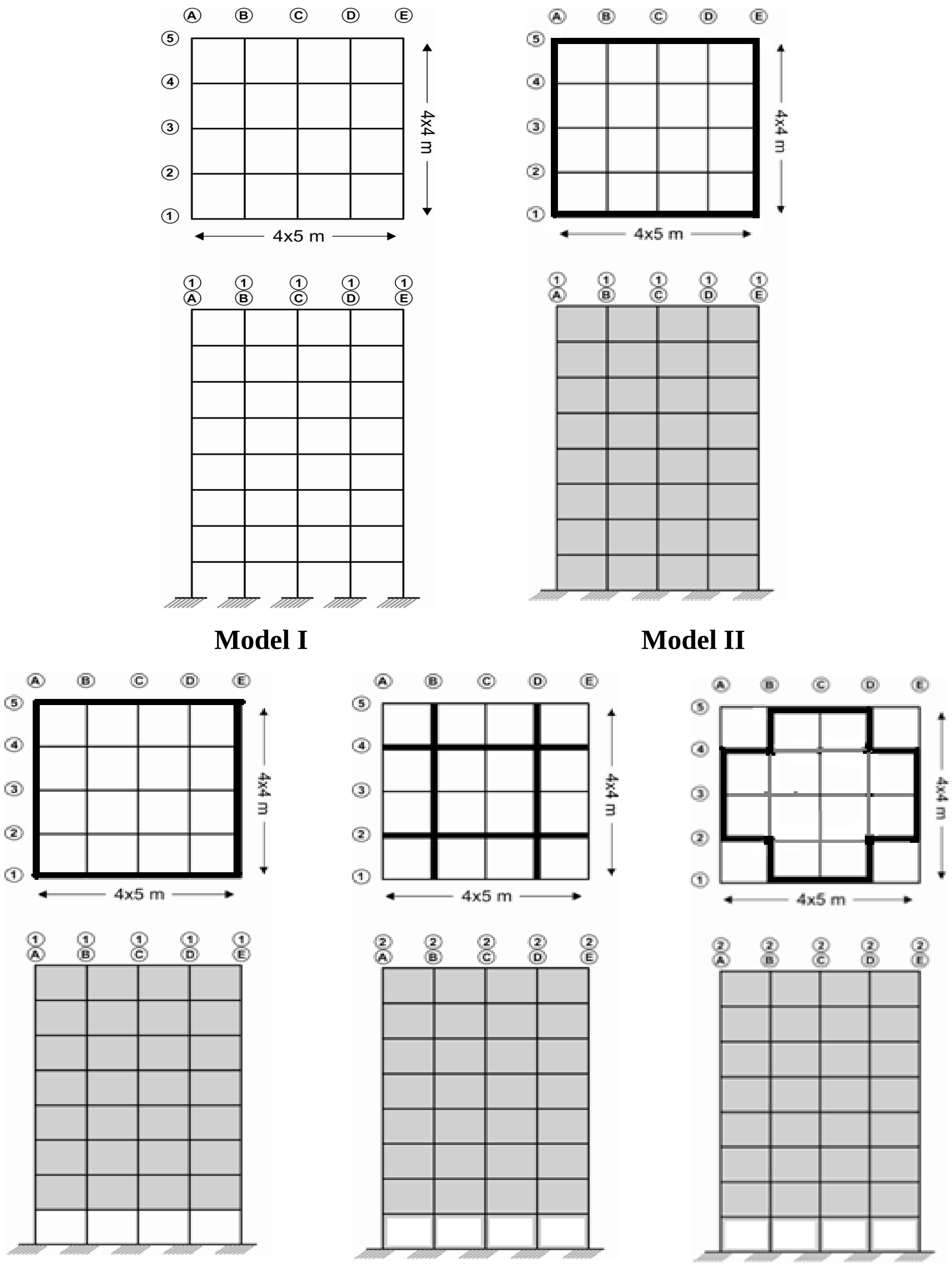


Figure 1: *Plan and Elevation of Eight Storeys Reinforced Concrete Building*

- **Elastic First mode Lateral Load Pattern** :

The first mode load pattern is related to the first displacement mode shape (Φ) of vibration. The lateral force of any storey is proportional to the product of the amplitude of the elastic first mode and mass (m_i) at that storey i.e.

$$F_i = m_i\Phi_i / \sum m_i\Phi_i \qquad (4)$$

where,

Φ_i = Amplitude of the elastic first mode of the storey.

- **Codal Lateral Load Pattern**:

This method uses the equivalent lateral forces due to fundamental period of vibrations. The code lateral load shape represents the forces obtained from the predominant mode of the vibration and uses the parabolic distribution of lateral forces along the height of the building. The following expression has been used to calculate the load pattern as per IS 1893 (Part-I): 2002.

$$V_B = A_h W \quad (5)$$

$$Q_i = V_B \frac{W_i h_i^2}{\sum_{i=1}^{n} W_i h_i^2} \quad (6)$$

Where,

V_B = Design Base Shear as per IS 1893(Part-I): 2002
Q_i = Lateral Force at Floor ***i*,**
W_i = Seismic weight of floor ***i*,**
h_i = Height of floor ***i*** measured from base and
n = Number of storey in the building.

In addition to these lateral loadings, the structures are subjected to dead loads and live loads. The displacement control method of pushover analysis was utilized with the target displacement 4% of total height of the building (ATC 40, 1996). The results were presented in the form of base shear vs. top displacement (Pushover Curves). The results of various models were discussed separately to have proper comparison between various load patterns and with that of the bare frame model. FEMA and ATC provide the frame work for performance based seismic design (FEMA 356, 2000, ATC 40, 1996). Prescribed performance levels in the FEMA-356 are the discrete damage states that the buildings can experience during the earthquake. In this study, inter storey drift capacity corresponding to the desired performance levels and two intermediate structural performance ranges were used. The discrete structural performance levels are Immediate Occupancy **(IO),** Life Safety (**LS**) and Collapse Prevention **(CP**).

3.1 Interstorey Drift

The inter storey drift is one of the commonly used damage parameter. The inter storey drift is defined as

$$SDi = \frac{\delta_i - \delta_{i-1}}{h_i} \quad (7)$$

Where, $\delta_i - \delta_{i-1}$ is the relative displacement between successive storey and h_i is the storey height. Acceptable limits of storey drift for various structural systems, associated with different performance levels were mentioned in section 3.2.

3.2 Results and discussions

As per FEMA-356, drift criteria for RC moment frames are **1%, 2% and 4%** for Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) performance levels, respectively. The drift criteria for unreinforced masonry infilled frames are **0.1%, 0.2% and 0.6%** for IO, LS and CP performance levels, respectively. Capacity curves along with Performance levels of building models for various load patterns are shown in Figure 2 (a-e). Fundamental natural time period as per IS 1893-2002 and as per analysis using ETABS software of various models are tabulated in Table 1. Base shear and top displacement at performance levels are tabulated in the Table 2 and Table 3 respectively for the First mode load pattern and Codal load pattern.

Table 1: *Fundamental Natural Time period (sec.) of Various Structural systems*

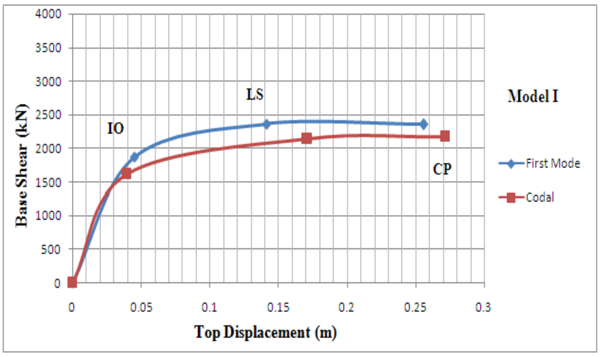
Systems	Model I	Model II	Model III	Model IV	Model V
As per IS 1893:2002	0.8130	0.4830	0.4830	0.4830	0.4830
As per Etabs analysis	1.0941	0.8673	0.8958	0.8954	0.9006

Table 2: *Base shear (kN) and Top displacement (m) at Performance levels for the First Mode Load Pattern*

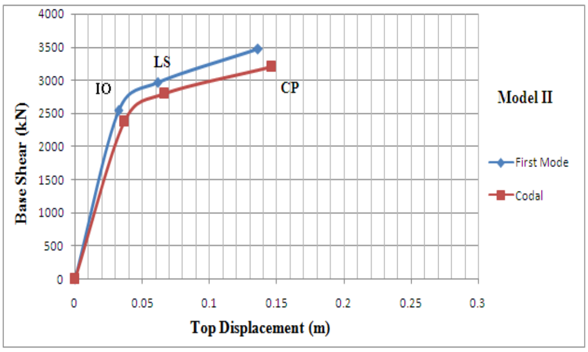
Systems	IO		LS		CP	
	Base Shear	Top Displacement	Base Shear	Top Displacement	Base Shear	Top Displacement
Model I	1868.34	0.0448	2367.21	0.1414	2352.12	0.2557
Model II	2551.74	0.0325	2970.63	0.0616	3474.98	0.1301
Model III	2494.09	0.0327	3153.58	0.0844	3269.43	0.1324
Model IV	2504.95	0.0331	3164.12	0.0860	3275.20	0.1333
Model V	2487.11	0.0327	3160.29	0.0863	3272.21	0.1342

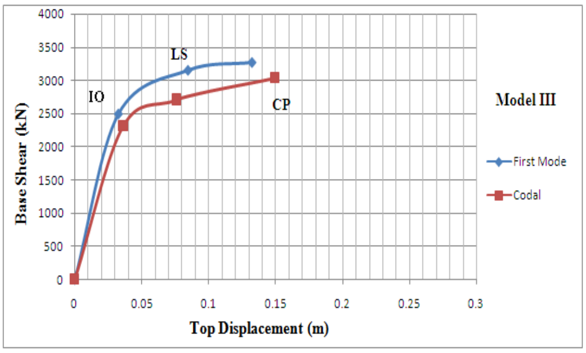
Table 3: *Base shear (kN) and Top displacement (m) at performance levels for the Codal Load Pattern*

Systems	IO		LS		CP	
	Base Shear	Top Displacement	Base Shear	Top Displacement	Base Shear	Top Displacement
Model I	1615.48	0.0393	2146.94	0.1708	2174.74	0.2718
Model II	2380.11	0.0366	2796.46	0.0664	3209.57	0.1463
Model III	2307.82	0.0364	2704.41	0.0760	3031.15	0.1499
Model IV	2319.93	0.0371	2721.02	0.0728	3028.85	0.1479
Model V	2329.79	0.0376	2730.02	0.0773	3032.99	0.1511

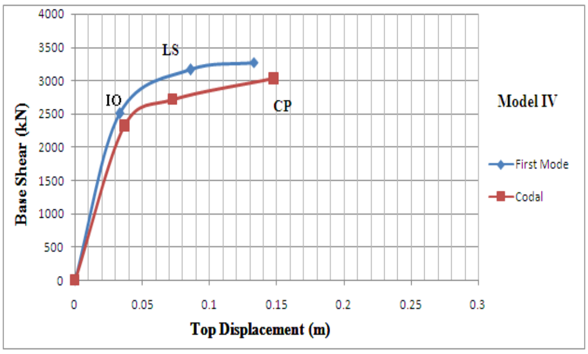


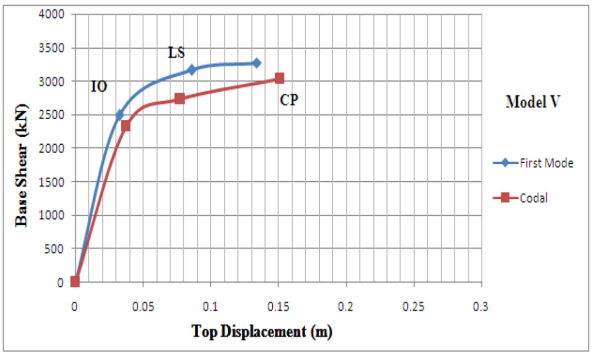
(a) Model I





(b) Model II (c) Model III





(d) Model IV (e) Model IV

Figure 2: *Pushover Curves Representing Performance Levels*

3.3 Fundamental Natural time period:

The codal (IS: 1893-2002) and analytical (ETABS) natural period of the various models are shown in Table 1. It is observed from Table 1 that the analytical natural period do not tally with the natural periods obtained from the empirical expression of the code. Introduction of infill panels in the RC frame reduces the time period of bare frames and also enhances the stiffness of the structure. Bare frame idealization leads to overestimation of natural periods and under estimation of the design lateral forces. It has been found that in the outer infill configuration (Model II) there was 25% reduction in time period compared to the bare frame (Model I). In all other soft storey models (Model III to V) 20 % reduction in natural period was observed compared to bare frame model (Model I).

3.4 Storey Displacement:

The top storey displacement profiles of models under consideration in Figure 2 show that introduction of infill panels in the RC frame reduces the lateral displacement considerably. From the study it was observed that First mode Lateral load pattern dominates the structures response. From Figure 2 and Table 2 it was observed that for the First Mode lateral load pattern the decrease in the top displacement in the Model II compared to the Bare frame Model (Model I) was nearly 50% and nearly 48% in Model III, IV and V respectively at the collapse prevention performance level. It was also observed that for the Codal load pattern the decrease in the top displacement in Model I compared to the Bare frame Model was nearly 46% for the Model II and nearly 44% in Model III, IV and V respectively at the collapse prevention performance level .On the similar line response of structure was seen at Life safety and immediate occupancy performance level for both lateral load patterns. It has been observed from above result that introduction of infill controls the lateral displacement and storey drift. However, in case of soft storey Models (Model III, IV and V) there was an increase in the top storey displacement by around 5 % compared to outer infill panel frame (Model II) at the Collapse prevention performance level. On the similar line lateral displacements of models were seen at the life safety and the immediate occupancy performance levels.

3.5 Base Shear:

Performance evaluation using the First Mode lateral load pattern resulted in higher base shear than the Codal load pattern. From the results in Table 2 and Table 3 it was observed that for the First mode load pattern the increase in the base shear in Model II was nearly 48% compared to the Bare frame model and was nearly 40% in soft storey models (Model III to V) compared to the Bare frame (Model I) at collapse prevention performance level. Similar to Elastic First mode pattern, the Codal load pattern also governed the structural response. On the similar line response of structure was seen at Life safety and immediate occupancy performance level for both lateral load patterns.

4.0 Conclusions

In this research, the effects of various configurations of masonry infills in the seismic response of gravity load designed RC frame buildings have been discussed. It has been found that the IS code provisions do not provide any guidelines for the analysis and design of RC frames with infill panels. It has been found that calculation of earthquake forces by treating RC frames as ordinary frames without regards to infill results in underestimation of base shear. Therefore it is essential for the structural systems selected to be thoroughly investigated and well understood for catering to soft ground floor, as the presence of masonry infill panels in the frame substantially reduce the overall damage. The performance of fully masonry infill panels was significantly superior to that of bare frame and soft storey frames. The present study also demonstrates use of nonlinear displacement based analysis methods for predicting performance based seismic evaluation.

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SUSTAINABLE BUILT ENVIRONMENT IN HIGH SEISMIC ZONE: CASE STUDY OF A MODERN TOWNSHIP IN THE NORTH EASTERN REGION OF INDIA

[1]Prof. (Dr.) Thiruvengadam V., Department of Building Engineering and Management.
[1]Email: thiruspa@rediffmail.com

[2]Prof. (Dr.) Mehta Rommel, Department of Landscape Architecture.

[3]Wason J.C., Visiting Faculty and Structural Engineer.
School of Planning and Architecture, I.P.Estate, New Delhi-110002 India.

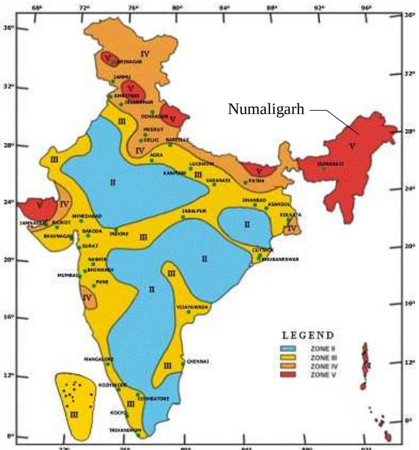
Abstract

The paper presents the details of sustainable aspects related to earthquake safety measures adopted in a recently constructed modern township in the high seismic prone north east region of India. This medium sized township constructed for the employees of Numaligarh Refinery Ltd is situated near Numaligarh, a small town in the Golaghat district in the state of Assam. The site is ecologically sensitive, densely vegetated and with steep undulating topography with deep valleys and background moderate hillocks. The site falls under severe seismic zone in the seismic map of India.
A seismic and sustainability conscious approach has been adopted in the planning, design and construction of the township. Site and landscape planning, building Architectural configurations, Structural and foundation systems and Infrastructure services are done to ensure safe and sustainable built environment with specific emphasis on earthquake safety and maintaining the natural landscape and ecology of the site.

Keywords: *Sustainability, built environment, seismic region, earthquake safety.*

1. Introduction

This paper brings out the planning and design concepts adopted for the establishment of a modern medium sized township located in an ecologically sensitive and very severe seismic zone in the north east region of India. The township is situated on the bank of the river Kalyani near Numaligarh, a small town in the Golaghat district of Assam. The site falls under severe seismic zone in the seismic map of India. Comprehensive architectural services including site and landscape planning, structural engineering, infrastructure services, construction specifications and cost estimates were carried out by the School of Planning and Architecture, (SPA) New Delhi, a premier educational institute in the field of Planning and Architecture.



Seismic Map of India

The Original Site Condition Still Retained in Areas within the Site

The main aspects covered in this paper are the planning, design and detailing for earthquake safety and various measures adopted for ensuring minimum impact on the environment and protecting the natural landscape of the area to achieve a safe built environment in an earthquake disaster prone region. The environmental protection was secured by strict adherence to damage prevention measures for existing landscape during execution and by ensuring that the proposed landscape is low maintenance.

2. Built Facilities and Infrastructure Services

The township is built to cater the residential requirements with associated facilities for the staff of the Numaligarh Refinery Ltd Assam. The refinery complex is located at about 4 km from the township location. The gross area of the township is about 250 acres including the non buildable hill slopes, natural valleys and drainage channels. The built facilities comprise of about 600 residential units in five categories of built-up areas, guest houses, recreational facilities in the form of club houses with swimming pools and play fields, primary and secondary school, hospital, community centre and convenient shopping, township park. The civil infrastructure services provided are; road network, water supply storage and distribution, sewerage collection and treatment, electrical substations and power distributions, street and area lighting and close circuit cable TV system.

View from the site towards the rive

NRLTownship Site

3. Site Characteristics

3.1. Site Topography

The eastern part of the township overlooks the scenic and meandering Kalyani river flowing adjacent to the eastern boundary of the site. The northern and western sides are hills with few identifiable low peaks. There is a general slope of the site towards the south. The valleys originate from the central ridge line draining out towards all the sides. The edges of the valley are at times having greater than 80% gradient but are well protected by dense natural vegetation against erosion. The undulating contoured land profile consists of ridges and natural valleys with dense vegetation.

The valleys divide the land into plateaus of undulating topography generally used for tea plantations. The elevation variation within the site ranges from 83 to 94 meters. The topographic survey was a difficult task due to steep slopes and with dense moist evergreen natural vegetation and tea gardens covering the entire site. The site rises abruptly above the flood plains of the river and proposals for reclaiming of some low areas adjoining the flood plains were not recommended for building activities.

3.2. Geotechnical Parameters: Subsurface Soil Conditions

The geotechnical technical investigation of the subsurface conditions showed varying soil conditions at different pockets of the township site. The soil conditions are generally medium type silty clay or clay with medium to high plasticity. The groundwater table is encountered at 2 to 5 m below the natural ground and the chemical analysis of subsoil water showed higher iron content. The allowable bearing pressure of soil is governed by settlement considerations and varied from 75 kN/Sqm to 120 kN/Sqm determined based on field and laboratory testing of the soils. The soil characteristics not being saturated loose sand have no potential for soil liquefaction under earthquake vibrations

3.3. Site Seismicity and Climatic Conditions

The township site located in the north east region of India which falls under seismic zone V, a very severe seismic zone category in the seismic map of India. The expected seismic intensity is IX on MSK 64 scale with potential for heavy damage. The seismic parameters used in the design are as per IS 1893-2002 (1). This Assam region has experienced two major earthquakes in the past, one in the year 1897 with a magnitude of 8.7 and another in the year 1950 with a magnitude of 8.50 indicating the severe seismic vulnerability of the region. The region experiences heavy rainfall from south west monsoon and falls under tropical hot and humid weather prevailing most of the summer and monsoon months. Total average annual rainfall is 1300 mm. and the heaviest rainfall in 24hours is 167mm. Maximum precipitation occurs in June and July. Maximum temperature is 38.0°C in June and minimum temperature is 10.0°C in December. The region falls within the ‘Humid Subtropical’ climatic zone of India.

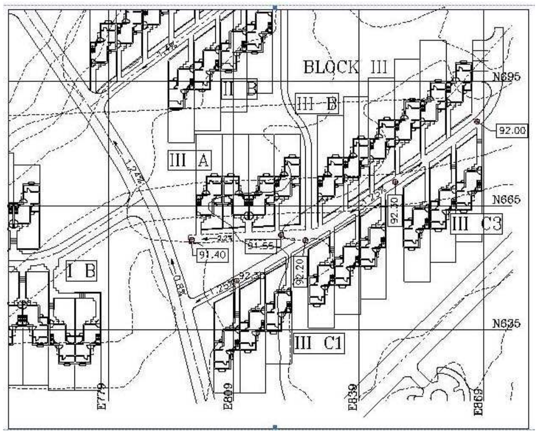
4. Site Planning and Landscape Design

The design programme emanated from the relationship amongst existing landscape, proposed design and sustainability. The basic goal of the design program was to protect and maintain the ecosystem of the site and ensuring least disturbance to the regional environment. Four components of the design were: environmental protection and perpetuation of bio-geo-chemical components, protection of existing vegetation, maintaining the prevailing hydrological systems – particularly the surface drainage discharge rates and routes, prevention of environmental damage during execution and maintaining the visual characteristics of the unique site. All this was achieved even after installation of the infrastructure services, the construction of buildings, and the vehicular and pedestrian circulation systems. The direction taken in realizing these program goals was a direct product of the alignment of design philosophy and approach of the client and the consultant. An integrated relationship, of pedestrian and vehicular circulation system with the sites storm water functions, has been achieved. Natural systems of existing plant communities and tea plantations were retained to a large extent to improve site sustainability. Substantial parts of site, on completion of project, were retained as low maintenance landscape zones.

A number of sustainable features are incorporated in the site and landscape planning aspects of the township. Buildings are avoided on hill slopes and near to the edges of the deep valleys. The existing land profiles of the plateau were left practically undisturbed without resorting to large scale cutting and filling and land reclamations. Minimum site grading surrounding the buildings are done to enable the location of the services and smooth flow of the storm water to the nearest valleys. The natural drainage paths of the site are left undisturbed and the existing landscape with dense vegetations is maintained. The alignments of the roads generally restricted to ridge lines are carefully planned and geometry suitably designed to achieve minimum earthwork. Crossing of the deep valleys are avoided eliminating the need for heavy bridge constructions but managing the storm drainage with few culverts. Unstable slope conditions are protected with low height breast walls with proper drainage arrangements. During construction operations restrictions were imposed not to dispose construction wastes in to the natural valleys.

5. Building Typology

The building typology adopted is low rise two storeyed constructions with light roofs. The form, height and roof lines are designed without dwarfing the tree heights and naturally vegetated landscape. For better utilization of buildable areas and community living residential units are designed as medium sized clusters scattered on the plateaus surrounded by ravines and drainage valleys. Topography determined the placement of the clusters resulting into aesthetically pleasing built environment. Due to contoured ground profile, varying plinth levels are adopted within the same clusters to avoid heavy site grading. However such situations complicated the structural aspects and roof forms which are resolved through careful design and detailing. Individual clusters are provided with small garden space in the front and with vehicular approach.



Type B Cluster Plan

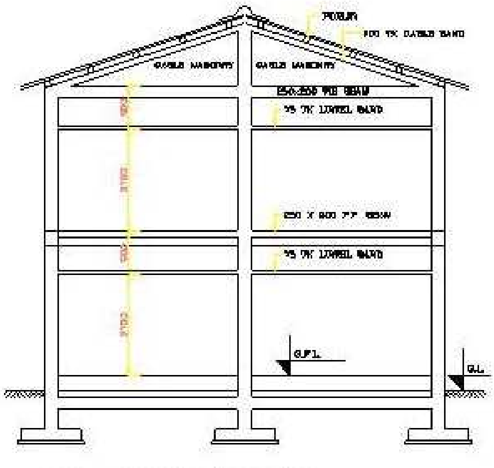
Type B Cluster

6. Building Architectural Configurations

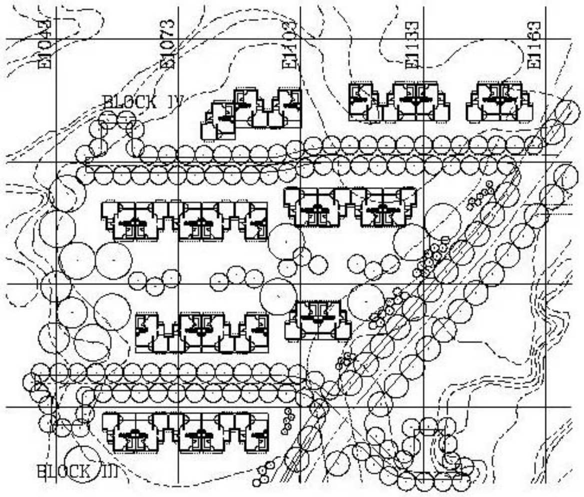
Compact building forms in terms of size, proportion and mass provided seismic resistant architectural configurations. Long building clusters for residential units are divided into compact lengths with the provision of seismic joints which also serve as expansion joints. Non residential buildings with unsymmetrical plans due to functional planning and site constraints are converted into simple and compact rectangular blocks with the provision of the seismic joints at appropriate locations. Buildings with open ground storeys are completely avoided. Due to high rainfall conditions, sloping roofs are adopted with light corrugated galvanized iron sheets in tune with the local construction practices. The light roofing systems provided the advantage of reducing the building mass that would result into reduced inertial forces during earthquakes. Buildings are mostly oriented in North-South directions from the considerations of better ventilation and thermal comfort.

Type C Unit

Type C clusters integrated with existing vegetation



Typical framing arrangement



Type B Cluster Plan

7. Building Structural Systems

Besides adopting suitable overall built forms and individual building configurations for reducing the vulnerability for earthquake damages, efficient structural system that could safely withstand severe earthquake forces are used. Special moment resisting reinforced concrete frames with ductility detailing are adopted to ensure the essential requirement of ductile behavior to avoid major damages and collapse situations under severe seismic conditions (2). The ductility detailing essentially provides adequate beam-column sizes for improved joint rigidity, adoption of closely spaced stirrups near as well as within the beam-column junctions as confining reinforcement, anchorage of beam bars into column members, 135^0 hooks for the effective anchorage of stirrups, adequate compression reinforcement and richer concrete mix. Although the brick masonry filler walls are treated as non structural elements, they would participate in the earthquake resistance as shear panels. The lintels over the doors/windows openings are made continuous between the columns and cast over the masonry filler walls and act as their horizontal stiffeners for improved seismic behavior. The top ends of the columns in the upper storey at in the roof tie levels are also connected with tie beams for the complete frame action.

As stated earlier, the material for the roofing system is light corrugated galvanized iron sheets fixed over steel tubular purlins supported over steel tubular trusses properly anchored with anchor bolts on columns/roof tie beams. The geometrical shapes of the roofs are as per the plan configuration of the buildings and are generally hipped roof system avoiding masonry gable ends which are susceptible for earthquake damage as observed in the past earthquake damage studies. However, in situations where gable ends are provided, such free standing walls are properly confined with reinforced concrete bands which are tied to the main structural frame. The trusses are provided with diagonal bracing to improve the diaphragm action at the roof tie level. For the external shading elements over the window openings, reinforced concrete chajjas are avoided due to the problem of rebar corrosion of these exposed elements in such moist high rainfall conditions. Instead, shading elements with corrucated galvanized iron sheets over steel tubular members are provided which also enhanced the aesthetic appearance of the building elevations. From durability considerations the thickness of galvanizing coating was specified as 0.80mm.

Senior Secondary School *School Hostels*

Both Represent an Architectural Expression of Sustainability

8. Foundation System

With the adoption of two storeyed constructions with light roofing system in moderate soil conditions, the foundation system with simple reinforced concrete isolated footings under the individual columns are adopted. For the integral action of the footings under earthquake motions and also to take care of possible differential settlements, grade beams connecting the individual footings in mutually perpendicular directions are provided. These grade beams, generally provided at 150 mm below the final ground level, also support the internal partitions and external enclosure brick masonry walls of the buildings. Due to undulating topography, placement of the footings of the adjoining columns at different levels are improved by taking the upper footing at a deeper depth so that the level difference between the founding levels of the footings is not more than half the distance between the edges of the footings. The buildings are carefully located at site so as to keep minimum distances between the foundations and edges of valleys.

9. Infrastructure Services

9.1. Water Supply Storage and Distribution System

The source of water to the township is from the river Kalyani and raw water is treated for the removal of suspended solids and other impurities in a water treatment plant located near the refinery site. The treated water is received in the township site in an underground storage tank of 16 lakh liters capacity catering about one and half days storage of the township requirement. The underground storage tank is located carefully on firm ground away from the valleys. The tank projects above the ground about 900 mm and merged with the landscape. The pumping station and chlorination equipments are kept as part of the tank below the ground so as to provide positive suction head to the pumps. The structural design of the tank has taken into consideration the soil characteristics and earthquake parameters. The water proofing of the tank is made as external tanking method with integral water proofing system.

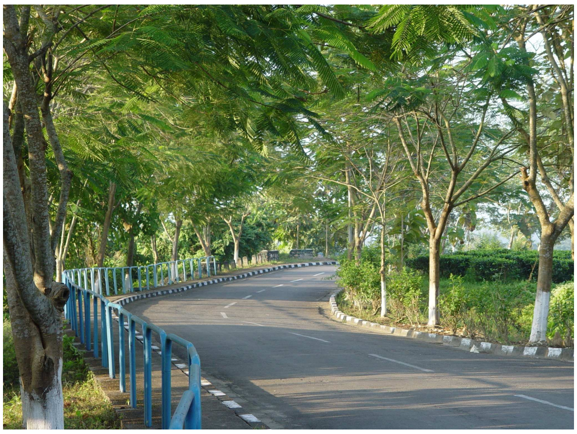
The water is pumped to an overhead tank of 5 lakh liters capacity from which water supply distribution is made through gravity flow to the individual buildings. During the conceptual design of the township, a conscious decision was made not to construct any tall conventional overhead tank in conflict with the subdued and low rise constructions. The presence of the hillocks on the northern boundary of the township enabled to construct a ground supported tank on the hill top without any staging structure which provided the required elevation for the hydraulic gradient to ensure for the gravity flow. The location of the tank was carefully selected to ensure stability and avoid cutting of slopes.

The water supply distribution network using cast iron and galvanized iron piping materials was designed taking into consideration the varying plinth levels of the buildings due to undulating site topography. All the piping networks are made with closed loops for better pressure distribution. Special care was taken to support the piping system over the valleys and depressions. The locations of the air relief valves, sluice valves and scour valves positions are carefully decided as per the site topography. For fire fighting operations, fire hydrants are provided at appropriate locations. In order to conserve the treated potable water, separate unfiltered water supply is being considered for nondomestic usages. An effective water conservation and management system with alternate source of water in emergent situations are under consideration.

9.2. Road Networks

The environmental uniqueness of the site led to the design of curvilinear alignment of vehicular roads. This is based on sound engineering practices and relates well to the topography of the site. This resulted in aesthetically pleasing and non environment safe circulation system. The curves and lack of formal geometry in the layout of roads and the resulting alignments, which evolved from the site itself, has contributed substantially to sustainability. The three dimensional aspects of all elements were suitably resolved; including various combinations of horizontal and vertical curves, cuts merging smoothly with fills, and side slopes blending well with the terrain.

The main spine road runs along the ridge lines almost in the middle of the township. Alternate routes were examined on site. These included considerations of avoiding deep valley crossings with major bridge constructions and heavy retaining structures. The road geometry is carefully designed to avoid large scale cutting and filling and ensuring proper storm drainage. An aesthetically pleasing alignment merging with the natural landscape of the site with minimum disturbance to the site topography could be achieved although this has marginally increased the length of the roads. Any conventional solutions in such an undulating site would have involved extensive earthwork, bridges and retaining walls resulting increased construction cost and time and destructions to the natural features of the site. Such situations were carefully avoided. The width of the main, sub main and cluster roads are 7.3 m, 5.5 m and 4.5 m respectively with foot paths on one side of the main and sub main roads. Each of the cluster roads serving the building units are carefully designed in relation to the plinth levels of the buildings to enable smoother approaches.



Curvilinear Alignment of Roads Relates to Site Topography

9.3. Storm Water Drainage

Although the site falls under heavy rainfall area, the storm water drainage did not pose problems due to significant topographic advantage with self draining nature of the site with the presence of natural channels and valleys around the plateau of the built-up areas. Storm water could ultimately find its path to the Kalyani River. However with careful planning with nominal site grading as a part of landscape design was carried out to direct the storm water to the nearest valley as sheet flow without the possibility of top soil erosion with the presence of dense vegetation.

9.4. Sewerage Collection and Treatment

The township site being a virgin land with contoured topography, it has been a challenging exercise to design and implement the sewerage network and treatment system for the cluster of buildings with different plinth levels. The main objectives of the design were to ensure gravity flow with minimal pumping, minimum disturbance to the natural landscape by avoiding deep sewer lines and manholes, avoidance of conventional septic tanks that would pollute the subsoil and vegetation, sewer treated to acceptable pollution control standards and the treated effluent is safe to discharge into the natural streams or used for gardening/horticulture purposes. Several alternative routes were examined at the site and an optimum alignment was chosen that involved only one pumping station in the entire township avoiding deep sewers. The sewerage treatment in done through extended aeration process and the effluent is subjected to filtration and chlorination to achieve the biochemical oxygen demand content to a level of 10 to 15 mg per liter and suspended solids less than 20 mg per liter. The location of the sewerage treatment plant is kept at the lower end of the township keeping in view the prevailing wind directions.

10. Township Plantations

The plantation systems are broadly categorized into road side planting, boundary planting, plantation in residential and public areas and plantations for protection of ridges and valley slopes. The plants selected are entirely from the existing regional indigenous plant species in consultation with the local horticulturists. Native flora is used to avoid the import of exotic species. In parts of the township the original tea plantations are also maintained as a pleasant reminiscence of the original tea gardens.

11. Summary

The case study presented in this paper brings out the various measures adopted in the planning and design of this medium sized township project in an ecologically sensitive and high seismic prone area. These efforts have led to the smoother functioning of the township and serve as an example of a safer and sustainable built environment. These measures are broadly summarized as under:

(i). Maintaining the natural undulating topography and landscape elements of the site
(ii). Avoiding construction activities on the hill slopes and protecting the vulnerable land profiles.
(iii). Natural drainage systems maintained without any kind of obstructions on realization of their environmental significance.
(iv). Existing vegetations, dense and prominent in the valleys, was protected during construction and construction wastes were carefully disposed at predetermined locations preventing the same from entering the valleys. Large scale additional plantation schemes were implemented.
(v). Adoption of low rise two storied construction with light roofing system suitable for high seismic zones.
(vi). Seismic conscious architectural and structural design and detailing of buildings and infrastructure services to provide earthquake safety.

The experiences and data gained from the long and comprehensive involvement in the project has proved to be very useful in teaching and research. This included the multi disciplinary technical as well as the contract related management and administrative experiences. All this has, over the years, been disseminated in the academic programs.

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DRASTIC IMPACTS OF 8TH OCTOBER EARTHQUAKE IN KASHMIR AND ROLE OF SUSTAINABLE DEVELOPMENT

Naeem Ejaz[1], Elina Dace[2] , Usman Ali Naeem[3], M Fiaz Tahir[4]

[1], PhD Scholar, Assistant Professor, Civil Engineering Department
University of Engineering & Technology, Taxila – Pakistan
naeem.ejaz@uettaxila.edu.pk

[2] PhD Scholar, Institute of Energy Systems and Environment
Riga Technical University- Latvia

[3,4] PhD Scholar, Assistant Professor, Civil Engineering Department
University of Engineering & Technology, Taxila – Pakistan

Abstract:
The main environmental loses from the earthquake to the environment were the heavy landslides, destruction of residential and commercial structures, destruction and rerouting of water bodies, etc. Due to the lack of sustainable planning and awareness, debris, building waste materials and misuse of other materials are seriously spoiling the sustainability of the area. Detail survey of the ruined areas of Kashmir due to 8th October earthquake after five years has been performed during this research. Present conditions of the study area are presenting the worst picture. Improper management of dumped construction waste is one of the serious issues. Environmental and sustainability conditions before and after earthquake are compared in this research work. It is concluded that infrastructure development and building construction during last five years after the earthquake are not fulfilling the sustainability requirement. Proper waste handing, recycling of materials, promotion of indigenous building materials and awareness among the local dwellers may help to improve the sustainability and environmental conditions in the study area.

Keywords: Kashmir, 8th October earthquake, sustainable development, infrastructure damages

1. Introduction

In October 2005, the Kashmir and northern regions of Pakistan were hit by a very drastic earthquake in the history of the country. These regions were quiet and prosperous tourism zones that were beginning to emerge as a perfect tourist destination for Pakistan. These regions are famous for their pleasant summer and attractive winter with a lot of snow falls. Kashmir region was becoming the ideal location for a stay at most beautiful valleys and green peaks. At 8th October 2005 early in the morning, these beautiful valleys were hit by a severe earthquake (RM:>7.5) that devastated the most of the parts of this region, rocked the countryside and killed thousands of the peoples. The epicentre of the 8th October earthquake is shown in Figure 1.

Figure 1: *Epicenter of the 8th October Earthquake*

The prevailing losses from the earthquake mostly occurred to humans and structures; this also results in drastic impact on the environment and sustainability [1]. The most visible consequences seen throughout earthquake hit areas of Kashmir are enormous amount of the debris and rubble resulting from damaged and destroyed structures. Disposal of rubble in rural areas, where structures are constructed of mud and stones, presented a different challenge than in urban areas, given that the amount of rubble is much less than in urban areas and it is spread out, making the task more complex [1-2]. Considering that there will be large quantities of material that can be salvaged for reconstruction, the disposal problem is far less significant that in urban areas [3-5]. There was evidence of haphazard dumping of debris and rubble in rivers adjacent to the cities in destructed areas of Kashmir. This has serious environmental penalty to water quality with potential downstream flooding impacts. In addition, there were signs of debris/ rubble being disposed of alongside roads, in fields, in open drains, in ditches and in forested areas. There is a practice of uncoordinated and unauthorized dumping which should be discouraged and a more systematic and planned approach be adopted.

2. History of earthquakes in Pakistan

The earthquakes are very frequently hitting the northern areas of Pakistan in the past. The Indo-Australian and Eurasian plate boundaries are present in northern and north-western regions of Pakistan. Due to this reason, a number of faults have been identified in these areas. The most of the earthquakes occurring in the northern areas are attributed to the energy release at the interface of these two boundaries. Only in a limited period from November 2002 to March 2003 there have been five major shocks measuring 5.3 to 6.5 on Richter scale. On 14th February 2004 an earthquake measuring 5.7 on the Richter scale again struck the northern areas damaging mostly Battagram and Mansehra districts and claiming 24 lives. An earthquake measuring 7.6 on the Richter scale struck Mansehra, Muzaffarabad, Garhi Habib Ullah, Balakot, Islamabad, Lahore, etc. on 8th October, 2005.

The epicentre of the earthquake as shown in Figure 1 was located at latitude of 34.402 and longitude of 73.560. The earthquake has by now claimed over 22,288 lives and left 50,575 injured and caused heavy damage to many buildings in that area. The heaviest damage was observed in Muzaffarabad area where entire villages were destroyed. Buildings were also collapsed in Gujranwala, Gujrat, Islamabad and Lahore. The earthquake was also felt in Chakwal, Faisalabad, Jhang, Sargodha and even up to Quetta.

3. Structural damages caused by the 2005 earthquake

So many residential, commercial and institutional structures were damaged by this severe earthquake. Hazara and Muzafarabad universities were mostly effected institutions in the region. Hazara University is located on the famous and historical Silk Route now known as Korakaram Highway (KKH) in Mansehra Hazara Division. It is about 40 km towards North from Abbottabad, 13 km from Manseehra and only 3 hours drive from the capital. In the East of Hazara University is Naran Kagan Valley and Azad Jammu & Kashmir while in the West is Oghi black mountains. Unfortunately on 8th October 2005, this campus was badly shaken and damaged by the most intensive and devastating earthquake of Pakistan History measuring 7.6 on Richter Scale. The old stone masonry structures could not withstand the severity of the earthquake and almost all the Department buildings suffered from severe structural damages. This research encompasses the effect of the earthquake on the structure and provides the recommendation for their future fate while considering the sustainability. Some of the structural damages at Balakot city are shown in Figure 2, 3 and 4

Figure 2: *Major destruction due earthquake at Balakot City*

Figure 3: *Destruction due earthquake at Islamabad and Abbottabad*

Figure 2: *Structural damages due earthquake*

4. Methodology & analysis

During this research work, the earthquake affected areas were visited to assess the role of sustainable development in rebuilding. The data was collected by using questionnaire, personal survey and oral discussions with the indigenous peoples during the field visits. This study helped to identify the role of sustainable development during the reconstruction phases.

The survey consisted of five demographic questions (sex, age, religion, birthplace, education level), four questions used to determine socio-economic status (do you own a television, car or phone; do you smoke), and two questions designed to assess general earthquake historical knowledge and role of sustainability in new construction (when was the last large earthquake; if and when will another quake occur in Kashmir, do you agree with the idea of sustainability in new construction)

5. Results & discussion

Some of the results from more than 200 respondents after successful interviews and surveys were predictable and are presented in Table 1.

The most notable results in this preliminary survey were found in the differing perceptions between education, technology, gender, and age, in addition to the general lack of belief in the use of seismic forecasting. This was not divulged in the simple responses but in the correlations between demographics and scaled responses. Especially interesting and statistically significant were the relationships found between education levels attained, age, television ownerships, and sex and many of the scaled response questions.

Table 1: *Questionnaire Response*

Sex:	Male: 89%	Female: 11%			
Age:	<20yrs: 10%	20-29yrs: 40%	30-39yrs: 16%	40-49yrs: 30%	50-70yrs: 4%
Birthplace:	city: 24%	Village: 76%			
Education:	none: 34%	Primary: 35%	secondary: 16%	baccalaur: 8%	4yr college:7 %
Smoke?	yes: 27%	no: 73%			
Telephone?	yes: 27%	no: 73%			
Television?	yes: 83%	no: 17%			
Car?	yes: 2%	no: 98%			
Last quake?	<10yrs: 18%	20-30yrs: 13%	40-60yrs: 34%	>100yrs: 1%	exact date: 34%
Quakes frighten you?	no: 32%	a little: 16%	somewhat: 6%	alot: 8%	yes a lot: 38%
Kashmir structures safe?	yes very: 21%	yes a bit: 16%	somewhat: 7%	not much: 20%	not at all: 35%
Is your house safe?	yes very: 24%	yes a bit: 15%	somewhat: 2%	not much: 20%	not at all: 39%
Kashmir earthquakes dangerous?	no: 0%	a little: 0%	somewhat: 2%	yes some: 5%	yes a lot: 93%
Role of sustainability in new constructions?	No idea: 23%	a little: 7%	somewhat: 20%	yes some: 16%	yes a lot: 34%

6. Conclusion and recommendations

During the field visit it was observed that most of the buildings were deteriorated due to old age, weathering effect and lack of maintenance. Wooden structures in the study area were severely deteriorated due to lack of maintenance and termite attack. The stone masonry structure behaviour is always brittle and it does not possess sufficient ductility which caused major damage in the study area.

The general direction of Hazara-Kashmir earthquake in the region is from North to South. The direction of earthquake has very significant effect on the performance of any structure. So, considering the sustainability factor of the structures should be oriented properly in future. The disposed of building materials are creating environmental hazards in the region. For the environmental sustainable point of view it is recommended to reuse the same stone with stronger mortar. For any future construction symmetrical and simple plan should be made to minimize the effect of any future earthquake. In case of new construction, the structures with more ductility and having good seismic performance should be considered. For the purpose it is recommended to consider the provision of reinforced masonry / stone structures having better ductile behavior.

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PROBABILISTIC SEISMIC HAZARD APPROACH FOR LOW SEISMIC REGIONS, VISAKHAPATNAM, ANDHRA PRADESH, INDIA

B.V.K Lalith Kumar[1], G.V Rama Rao[2] and K Srinivasa Rao[3]

[1]Research Scholar, Department of Civil Engineering, Andhra University College of Engineering, Visakhapatnam, AP, India, Tel +91-99634 28887; correspondence bvklalitkumar@rediffmail.com

[2]Professor, Department of Civil Engineering, Andhra University College of Engineering, Visakhapatnam, AP, India, ramaraogv_2000@yahoo.com

[3]Assocaite Professor, Department of Civil Engineering, Andhra University College of Engineering, Visakhapatnam, AP, India, killamsetti@yahoo.com

Abstract

This paper presents seismic hazard analysis of Visakhapatnam using probabilistic approach. Visakhapatnam is a rapid growing coastal city in India and considered as stable region with low intensities. In this paper previous earthquake history of region was considered to generate earthquake recurrence relation. The mean annual rate of exceedence is generated against peak ground acceleration considering study area site conditions. From the present investigation the values of peak ground acceleration varies from 0.004 g to 0.02 g with rate of exceedance 50% and 0.05 g to 0.12g for 10% rate of exceedance.

Keywords: Probabilistic seismic hazard method, peak ground acceleration, Earthquakes.

1. Introduction

Earthquakes are natural disasters and result in huge loss to mankind and assets. In India, large numbers of earthquakes took place with low to high magnitudes. Some areas earlier considered stable have experienced severe damages caused by earthquakes. Noticeable earthquakes happened in India in various places such as Latur in Maharashtra, Bhuj in Gujarat and Jabalpur in Chhattisgarh.

In this paper probabilistic seismic hazard approach was used to find out peak ground acceleration values with various return periods. Probabilistic seismic hazard analysis (PSHA) has gained popularity ever since it was formulated by Cornell (1968). It is considered as proven tool to estimate hazard analysis considering uncertainties like site, time and period. This tool is widely accepted in regions with poor earthquake data to analyze. Various steps involved in probabilistic seismic hazard approach shown in Figure 4.

2. Objectives and Methodology

Objective of present study is to find out peak ground accelerations of study area against probability rate of exceedance 50%, 10% and 2%. This process involves collecting geological features of study area such as faults lineaments and collection of previous earthquake catalogue. In the present study, a catalogue of past earthquake history was collected from United States geological survey (USGS) web site and the previous earthquake sources were identified. The radial distance search was used to obtain catalogue data; for present study earthquake catalogue around Visakhapatnam was obtained with a radial search of 350 km.

2.1 Geology of study area

The study area considered here is Visakhapatnam situated 800 km north east side of Chennai. Major folds are traced in the direction of north east and south west of Eastern Ghat hilly region. The Eastern Ghats are traversed by number of faults. Faults are existing in Sileru River which is 50 km away and situated in western border of Eastern Ghats. Kalinga Konda is 8 km away from Srikakulam and 90 km away from Visakhapatnam has significant faults. The solidified zone west of Endada hill near Visakhapatnam is also considered as fault zone.Major villages around Visakhapatnam are shown in Figure 2 and major lineaments are shown in Figure 3. The lineaments are of various lengths and close to main city Visakhapatnam.

2.2 Location of study area

Visakhapatnam is located along the east coast of India between 17 ^{0} 28 $^{'}$45” to 18 ^{0} 1 ’min latitude and 83^{0} 59’ to 83^{0} 35’ east longitude.

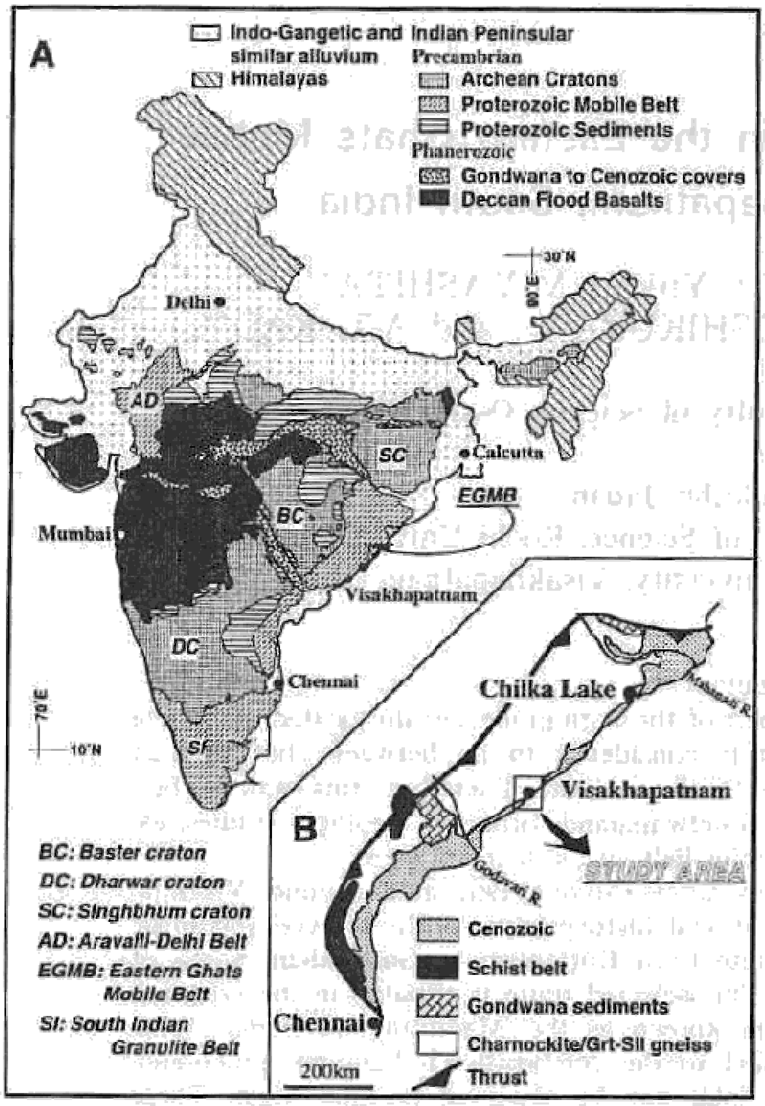

Figure 1: Study Area- Visakhapatnam

2.3 Earthquake History of Study Area

The seismicity of study area is addressed by Kaila et al 1972; Umesh Chandra 1977; as per IS 1893 (2002) seismic intensity is under zone II. The earthquake events were collected with United States Geological survey and presented in Table 1.The past records show earthquake magnitudes of 3 to 5 are available. Higher values 7 occurred in Srikakulam, Bhadrachalam regions. The summary list presented decade wise occurrence shown in Table 2. Based on the list histogram (Figure 4) is prepared and used for further calculations.

3. The Proposed Model

Seismic hazard assessment has been studied by many researchers. In general, the seismic hazard evaluation can be done in two ways first method is deterministic seismic hazard approach and second method is probabilistic seismic hazard approach.

The first method, the deterministic seismic hazard approach can be performed using earthquake epicentral locations, magnitude, rate of occurrence and intensities. The results give highly conservative estimates of seismic hazard.

The second method, the probabilistic seismic hazard approach which takes into account the uncertainties in the level of magnitude of earthquake, its epicentral location, its recurrence relation and its attenuation. This approach gives more realistic values for earthquake hazard parameters.

3.1 Steps in probabilistic model

The steps involved in probabilistic seismic hazard can be summarized as follows: identification of sources; establishment of recurrence relationships, magnitude distribution and average rate of occurrence for each source; selection of attenuation relationship; and finally, the computation of site hazard curve.

3.1.1 Regional recurrence relation

The recurrence relation is the relationship between the cumulative frequency of occurrence of earthquake and its magnitude. Gutenberg-Richter (1944) suggested following relation

$$\log N = a - b\,M \tag{1}$$

where N is the number of earthquakes magnitude greater than M

M is earthquake magnitude

a and b are constants depending upon source area and can be determined by least square method. The constants 'a' and 'b' have great physical meaning. The 'a' value indicates earthquake magnitude above zero and it depends upon source events. The b value is the measure of seismic severity of source region. A higher value of b indicated smaller fraction of a total earthquake count when lower value of b indicates higher earthquake count (Kobe 1994).Various investigators established values of a and b depending upon region specific and some of the equations related to India by Kaila (1971), Sitharam and Anbazhagan (2007) Vipin K.S (2009). Jaiswal and Sinha (2006) have suggested value of b is 0.88 plus or minus 0.7, as per Ram and Rathore (1970) a=4.58 and b= 0.891.

Since there were no significant earthquake magnitudes recorded in Visakhapatnam during the period 1959-72. Gutenberg-Richter relations cannot be used directly. Hence available values of earthquake magnitude are increased from 4 to 4.5 and 5 to 5.5. The output results obtained from PSHA software and results are shown in Figures 6, 7 and 8.

The standard Gutenberg-Richter (GR) equation (i) can be re-written as below

$$N_m = 10^{(a-bm)} = \exp(\alpha - \beta\, m) \tag{2}$$

Where α =2.303 a and β=2.303 b

Equation (ii) can be rewritten eliminating lower earthquake magnitudes

$$N_m = \nu \exp\left(-\beta(m - m_0)\right) \; m > m_0 \tag{3}$$

where $\nu = \exp(\alpha - \beta m_0)$ (4)

Cumulative distribution function CDF is as given below

$$F_M(m) = P[M < m \mid M > m_0] = \frac{N_{m_0} - N_m}{N_{m_0}} \tag{5}$$

$$\frac{N_{m_0} - N_m}{N_{m_0}} = 1 - e^{-\beta(m-m_0)} \tag{6}$$

And probability density function PDF is given below

$$f_M(m) = \frac{d}{dm} F_M(m) = \beta\, e^{-\beta(m-m_0)} \tag{7}$$

The mean annual rate of exceedance expressed below (McGuire and Arabasz, 1990)

$$N_m = \nu \frac{\exp[-\beta(m-m_0)] - \exp[-\beta(m_{max}-m_0)]}{1-\exp[-\beta(m_{max}-m_0)]} \tag{8}$$

$$m_0 < m < m_{max}$$

The CDF and PDF for Gutenberg-Richter law with upper and lower bound expressed as

$$F_M(m) = P[M < m | m_0 \le m \le m_{max}] = \frac{1 - \exp[-\beta(m - m_0)]}{1 - \exp[-\beta(m_{max} - m_0)]} \quad (9)$$

$$f_M(m) = \beta \frac{\exp[-\beta(m - m_0)]}{1 - \exp[m_{max} - m_0]} \quad (10)$$

where m is earthquake magnitude and m_o and m_{max} are minimum and maximum earthquake magnitudes.

3.1.2 Predictive relationships

Considering an earthquake influence due to source at a distance, the probability that a particular ground motion Y exceeds certain value y* for an earthquake magnitude, m, occurring at a distance r is given in below

$$P[Y > y^* | m, r] = 1 - F_y(y^*) \quad (11)$$

Where Fy(y) is the value of CDF of y at m and r. The value if Fy (y) depends on probability distribution used to represent Y.

The standard normal variation is given below equation

$$Z = \frac{\ln PHA - \ln \overline{PHA}}{\sum \ln PHA} \quad (12)$$

where PHA is peak horizontal acceleration

3.1.3 Poisson Model

The occurrence of an earthquake is seismic source is assumed to follow Poisson distribution. The probability of ground motion parameter at a given size, Z, will exceed a specified level, z, during specified time period T by the expression (S.L Krammer).

$$P(Z > z) = 1 - e^{-v(z)t} \quad (13)$$

Where v(z) is the mean annual rate of exceedance. v(z) depends upon time, size and location of future earthquakes.

3.2 Attenuation relation for present study

For the present study ground accelerations are based on attenuation equations given by Donovan (1973) and Joyner Boore (1981).

$$\log PGA = \frac{190}{R^{1.83}} \quad (14)$$

Joyner Boore suggested following equations for western US and worldwide

$$\log PGA = -1.02 + 0.249M - \log\sqrt{R^2 + 7.3^2} - 0.00255 \log\sqrt{R^2 + 7.3^2} \quad (15)$$

$$\log PGA = 0.49 + 0.23(M - 6) - \log\sqrt{R^2 + 8^2} - 0.0027\sqrt{R^2 + 8^2} \quad (16)$$

Where M is earthquake magnitude and R is the closest distance to the fault rupture in km

4. Results and Discussions

The probabilistic seismic hazard analysis has been performed using three significant sources having recorded earthquake magnitude. The results show regions close to site shows more hazard than far areas. The computer program PSHA has been developed based on Cornell and Krammer (1996) equations. The peak ground acceleration with 50% probability of exceedance varies from 0.023g to 0.027g and 10% rate exceedance 0.114 g to 0.119 g and 2% exceedance is 0.45g. Figure 9, 10 and 11 show rate of exceedance versus peak ground acceleration with sources 1,2 and 3.For source 1 the values of PGA for 50%,10% and 2% rate of exceedance are 0.004 g,0.061 g and 0.57 g and source 3 which is 250 km away the PGA values for 50%, 10% and 2% rate of exceedance are 0.004 g, 0.05 g and 0.4 g.

5. Conclusions

In this paper probabilistic seismic hazard analysis of Visakhapatnam with local conditions has been presented. The curves of mean rate of exceedance for peak ground acceleration generated at rock level considering local site conditions. The source of occurrence is considered as fault region. Since faults are known as weak zones during earthquakes. The obtained peak ground acceleration is 0.114g with 5% damping for Visakhapatnam for 10% probability of exceedance with return rate of 50 years. The peak ground accelerations generated using local conditions is 0.33g considering one dimensional linear analysis (P.S.N Raju and Lalith kumar 2010).The other significant parameter which is used by designer is spectral acceleration which has not been considered here left for future work.

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About the Authors

1. Research scholar, Department of Civil Engineering, Andhra University College of Engineering, Visakhapatnam. bvklalitkumar@rediffmail.com
2. Professor, G.V Rama Rao, Department of Civil Engineering, Andhra University College of Engineering, Visakhapatnam.
3. Associate Professor, Dr. K. Srinivasa Rao, Department of Civil Engineering, Andhra University College of Engineering, Visakhapatnam.

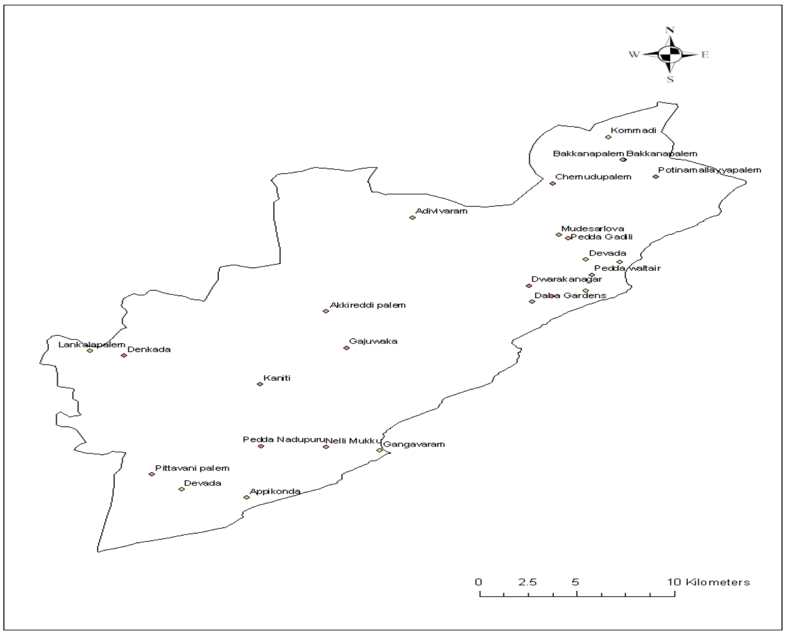


Figure 2: Satellite villages around Visakhapatnam

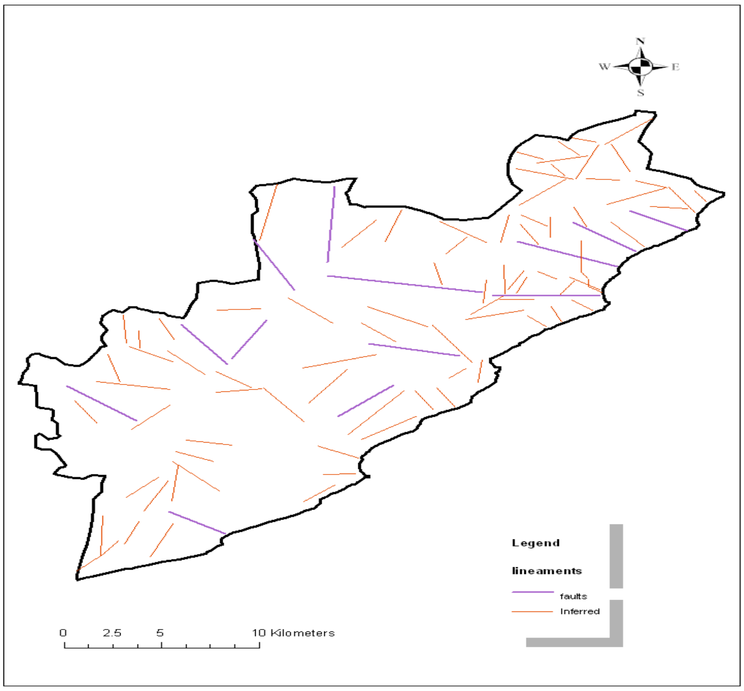


Figure 3: Lineaments and faults in and around Visakhapatnam

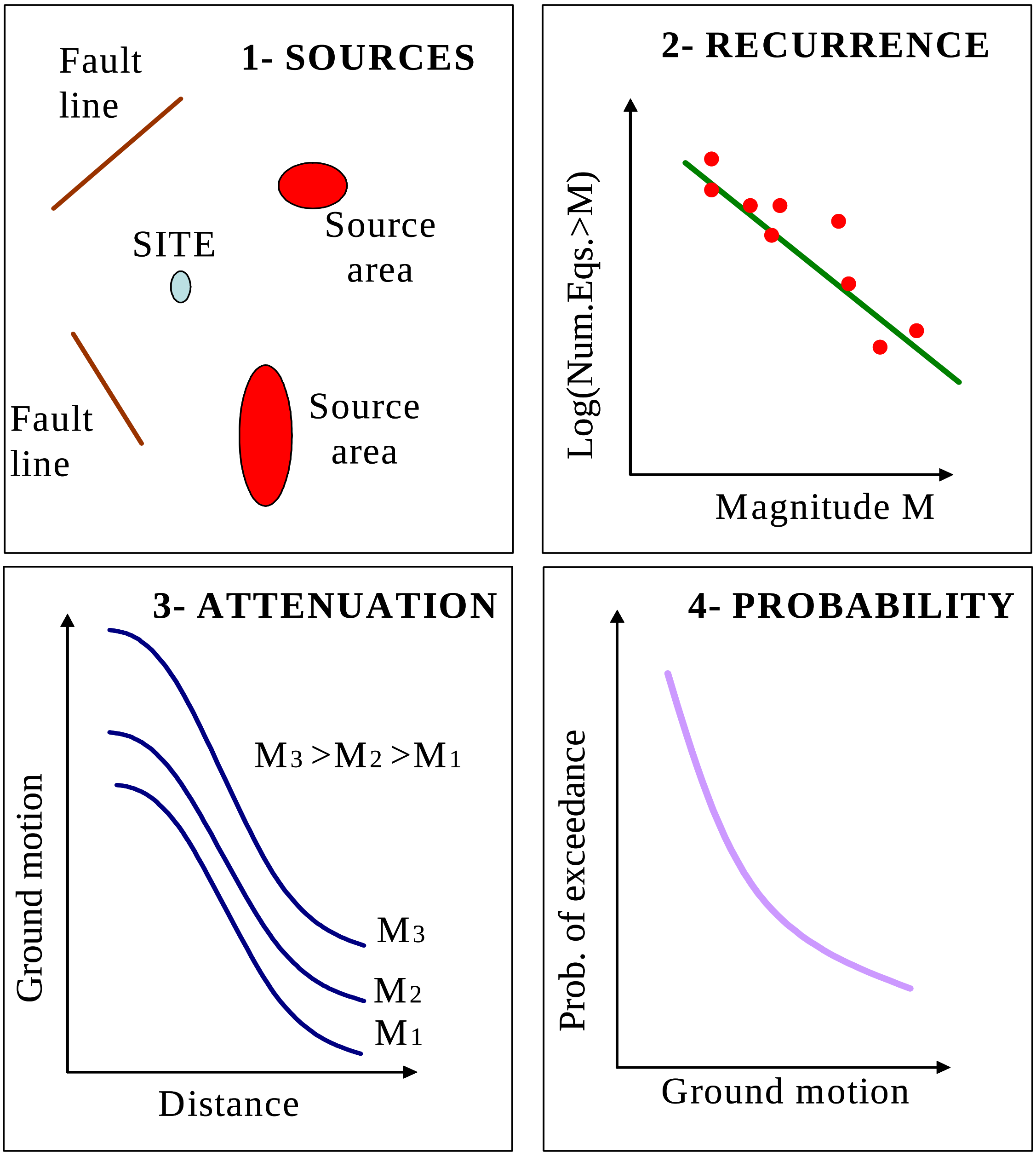
Table 1: Recorded earthquake events, from USGS web site

Number	Year	Month	Day	Latitude N (degrees in decimal)	Longitude E (degrees in decimal)	M_s	M_b	M_w
1	1827	1	6	17.70	83.40	4.8	5.2	5
1	1837	6	15	19.50	85.10	6	5.8	6
1	1853	2	21	17.70	83.40	3.1	4.2	4
1	1858	8	24	17.80	83.40	3.1	4.2	4
1	1858	10	3	19.50	85.10	3.1	4.2	4
1	1858	10	12	18.30	84.00	6	5.8	6
1	1859	7	21	16.29	80.50	6	5.8	6
1	1859	8	2	16.29	80.50	4.8	5.2	5
1	1859	8	9	16.29	80.50	4.8	5.2	5
1	1859	8	24	18.10	83.50	4.8	5.2	5
1	1860	2	25	19.40	84.90	4.8	5.2	5
1	1861	11	13	18.12	83.50	4	2.85	3
1	1869	12	19	17.90	82.30	4.8	5.2	5
1	1870	12	19	17.90	82.30	4.8	5.2	5
1	1871	9	27	18.30	83.90	4	2.85	3
1	1872	11	22	18.86	80.02	3.1	4.2	4
1	1885	7	22	20.06	85.37	4	2.85	3
1	1885	9	1	20.06	85.37	1	2	2
1	1886	5	2	20.06	85.37	4	2.85	3
1	1897	6	12	18.53	83.48	1	2	2
1	1897	6	22	19.00	84.90	7.6	6.6	7
1	1898	6	1	16.98	82.33	4	2.85	3
1	1917	4	17	18.00	81.30	7.6	6.6	7
1	1927	1	1	18.10	83.50	4.8	5.2	5
1	1954	1	5	17.30	80.10	4.8	5.2	5
1	1959	8	9	17.60	80.80	3.1	4.2	4
1	1959	12	23	17.60	80.80	4.8	5.2	5
1	1963	12	5	17.90	80.60	3.1	4.2	4
1	1968	7	27	17.60	80.80	4	2.85	3
1	1968	7	29	17.60	80.80	4	2.85	3
1	1969	4	13	17.90	80.80	7.6	6.6	7
1	1972	6	11	17.60	80.20	4	2.85	3
1	1975	4	24	18.70	80.70	4	2.85	3
1	1981	12	8	16.30	80.50	4	2.85	3

Sources considered in present study are source 1: Lat. 17.7 N, Long. 83.4 E source 2: Lat. 18.1N, Long. 83.50 E and source 3 Lat 19.50N, Long. 85.10 E (http://earthqukes.usgs.gov/regional/neic)

Table 2: Data of Earthquake events based on ten year period

Earthquake Magnitude M						
Duration	1<M<1.9	2<M2.9	3<M<3.9	4<M<4.9	M>5	Total No of Earthquakes
1828-1837					2	2
1838-1847						0
1848-1857				1		1
1858-1867			1	2	3	6
1868-1877			1		2	3
1878-1887						0
1888-1897		1			1	2
1898-1907			1			1
1908-1917					1	1
1918-1927					1	1
1928-1937						0
1938-1947						0
1948-1957					1	1
1958-1967				1	1	2
1968-1977					1	1
1978-1987				1		1
	0	1	3	5	13	



Steps of probabilistic seismic hazard analysis for a given site: (1) definition of earthquake sources, (2) earthquake recurrence characteristics for each source, (3) attenuation of ground motions with magnitude and distance, and (4) ground motions for specified probability of exceedance levels (calculated by summing probabilities over all the sources, magnitudes, and distances).

Figure 4: Procedure of probabilistic seismic hazard analysis

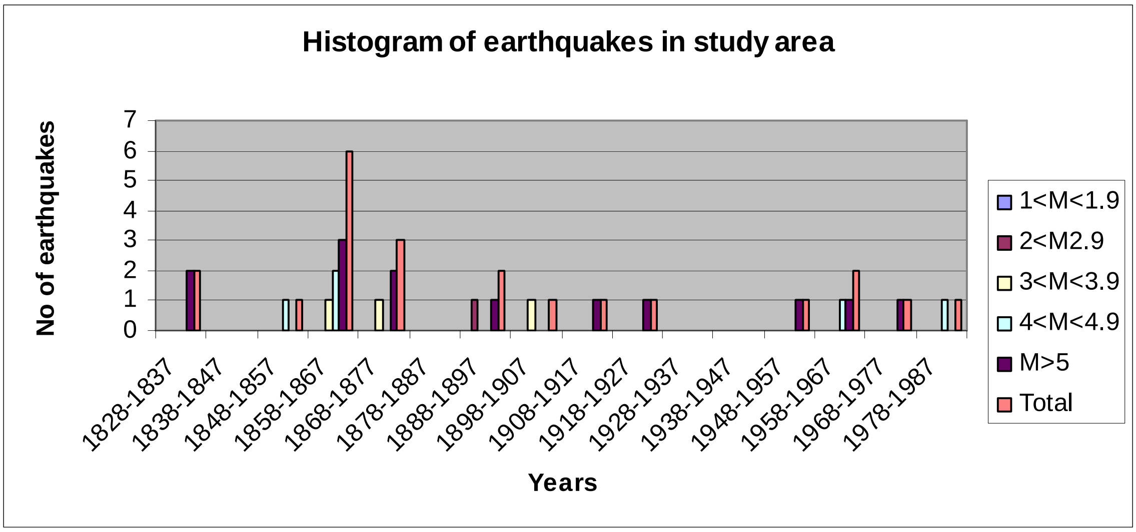


Figure 5: Histogram of earthquake history in study area

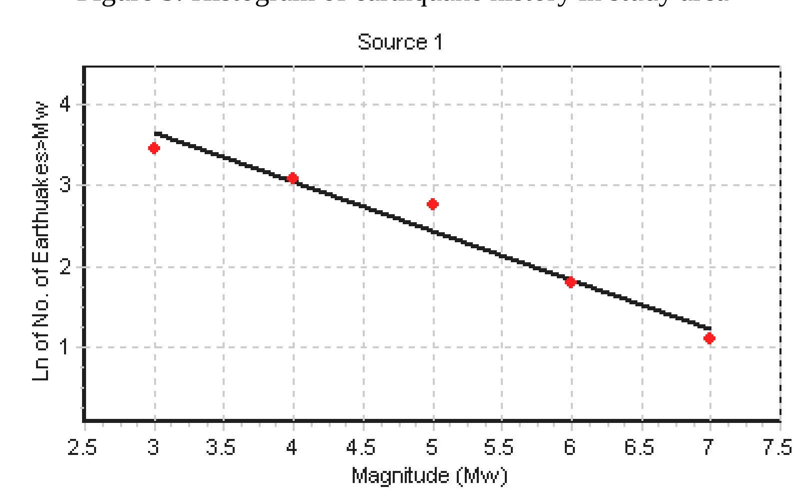


Figure 6: Lognormal events at source 1

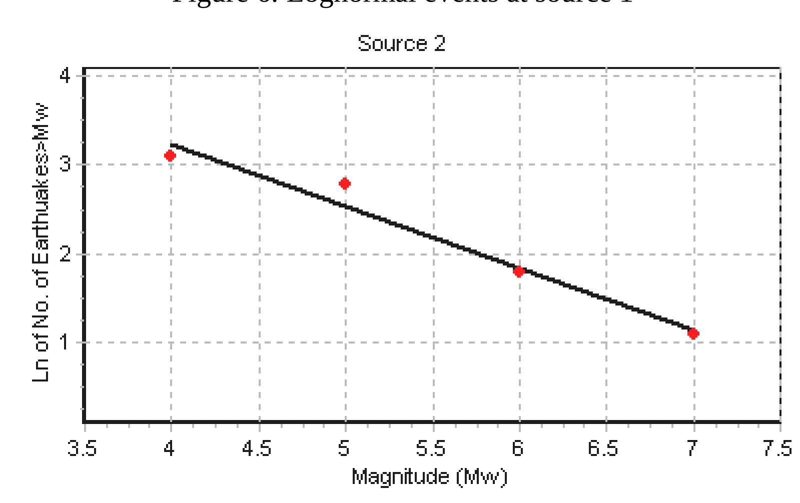


Figure 7: Lognormal events source 2

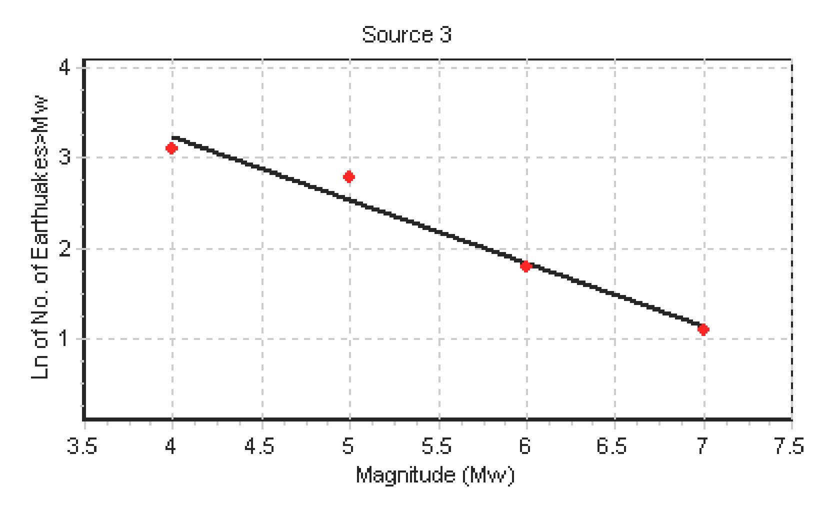


Figure 8: Lognormal events source 3

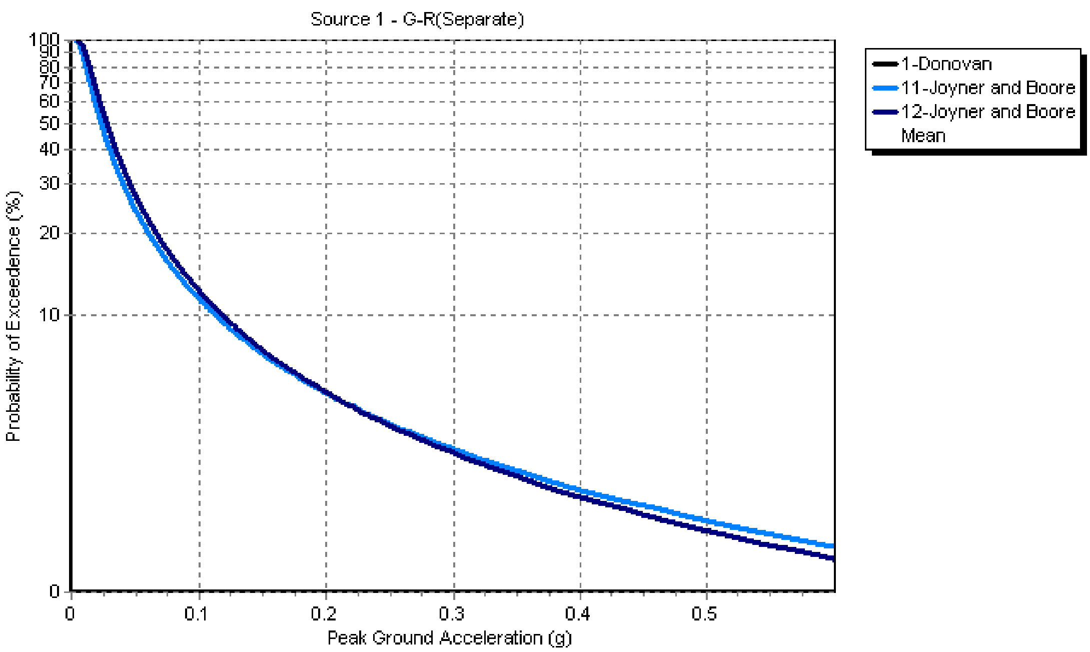


Figure 9: Probability of exceedance vs. peak ground acceleration for source 1

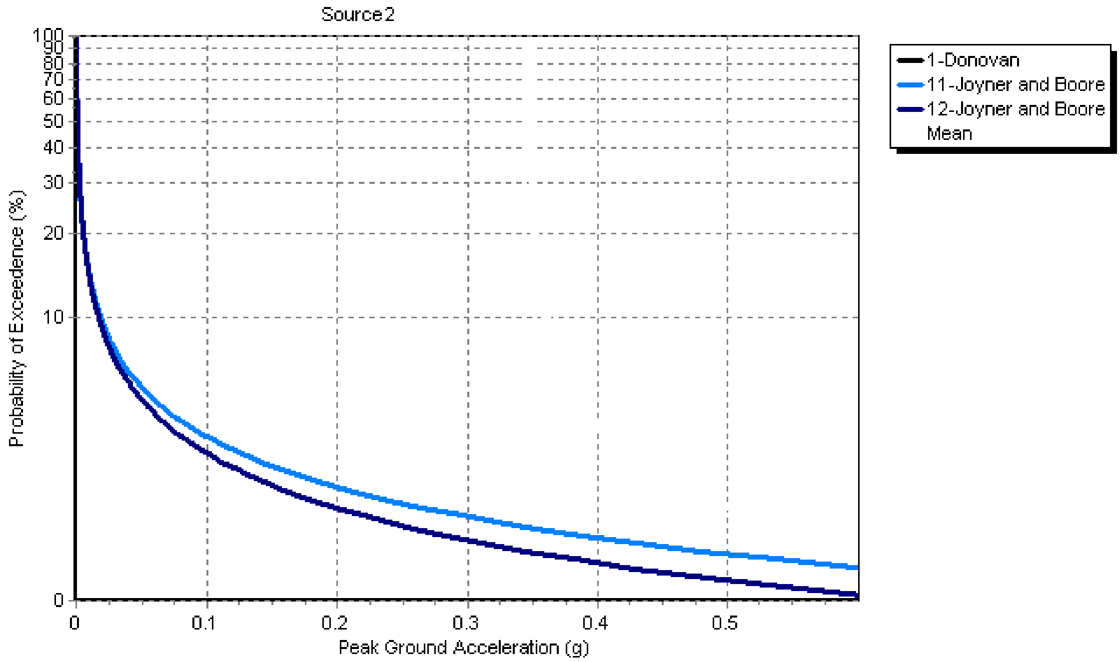


Figure 10: Probability of exceedance vs. peak ground acceleration source 2

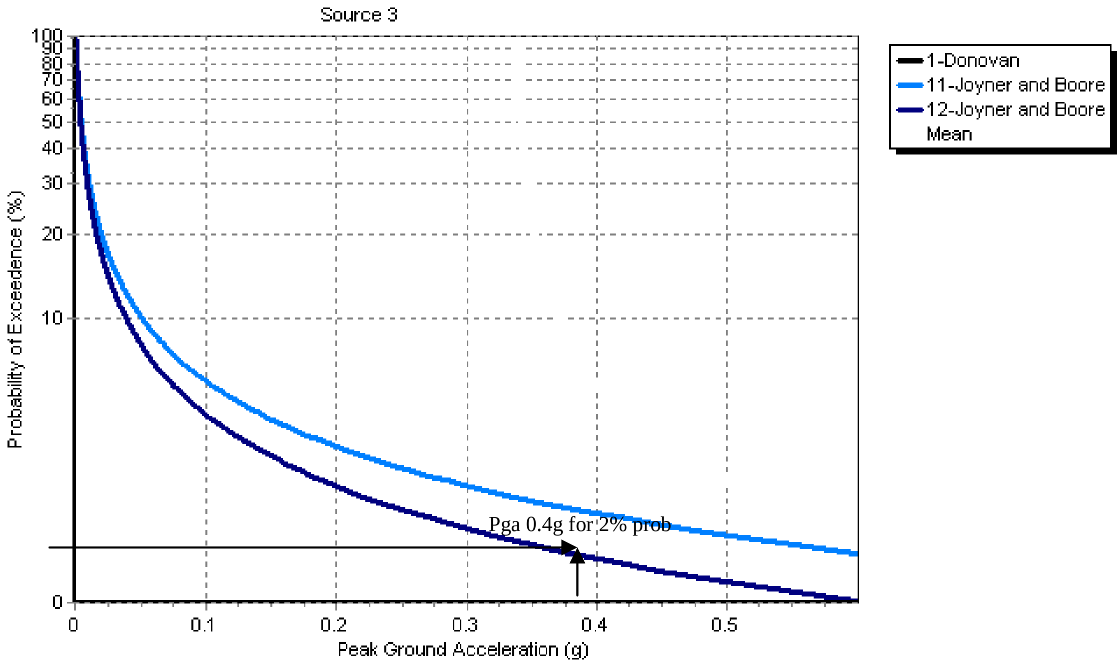


Figure 11: Probability of exceedance vs. peak ground acceleration source 3

SEISMIC STRENGTHENING OF EXISTING TYPICAL JAPANESE WOOD HOMES USING GFRP SYSTEMS

Jamaji R.[1] and Abdullah B. PhD.[2]

[1] Sr. Vice President, FYFE Asia Pte Ltd, 8 Boon Lay Way,
#10-03 Singapore 609964
Tel: +65 68985248, Fax: +65 68985181,
Email: rustom@fyfeasia.com (Corresponding author)

[2] Acting M.D., FYFE Japan Co., Ltd., the SOHO 1210, 2-7-4 Aoumi,
Koutou-Ku, Tokyo 135-0064, Japan
Tel: +81 3 64571910, Fax: +81 3 6457 1911,
Email: basem@fyfejapan.com

Abstract
It is estimated that out of existing 47 millions homes in Japan, approximately 11.5 million need urgent strengthening. These homes do not meet current earthquake resistant standards and would face severe damage in the event of a "Shindo" 7 quake as it is know on the Japan Meteorological Agency's seismic intensity scale.

Wooden houses in Japan are typically built by wood post-and-beam methods over a concrete strip foundation. Due to the constant mild seismic activity many of the existing homes have been further weakened with evidence of cracks in the foundation and in the wood-mortar walls. The currently available seismic strengthening systems involve a massive amount of intrusive work to the existing homes and are beyond the budget of an ordinary Japanese family.

This paper serves as a case study, from Concept to Implementation, into how an affordable minimally intrusive seismic strengthening system was developed to strengthen typical Japanese wooden houses for earthquakes resistance by using Glass Fibre Reinforced Polymers (GFRP) materials. It will highlight the concept for the concrete strip foundation and the concept for the wood-mortar wall strengthening with GFRP as well look into specific details of four years of research and development with the participation of Kyushu University in Japan, Nanyang Technological University in Singapore and Oita University.

Keywords: *Seismic Strengthening, Homes, Wood framed walls GFRP*

1. Introduction

A report by the Japanese Ministry of Transportation, Land and Infrastructure showed that approximately 41% of all Japanese homes need some sort of urgent seismic strengthening [1]. The strengthening systems currently available are far beyond the budget of an average Japanese household as it is required to change the structure of the house, which includes dismantling of walls and floors to add new structural members (such as wooden diagonal posts and bracings…etc.), then reassemble them and finally reapplying the architectural finishing.

This paper introduces a new method of strengthening typical Japanese wooden houses against seismic activity by using GFRP materials. This is achieved by reinforcing existing standardized cracked concrete strip foundations with GFRP to increase its flexural capacity as well as by increasing the shear capacity of the standardized wood framed mortar walls by applying a diagonal GFRP bracing system. The system can be installed directly onto the external walls of the house right over the architectural finish and is able to dramatically enhance foundation's flexural capacity as well as the wall's shear capacity.

The goal of this program was to test and develop and reliable, cost effective method to retrofit typical Japanese homes using minimally invasive methods as not to disrupt the lives of its occupants.

2. Concept for Strip Foundation Strengthening with GFRP

Fiber reinforced polymers (FRP) have been proven to enhance the flexural capacity of flexural members such as reinforced concrete beams and slabs by bonding the composite to the extreme tension surfaces of the member.

This practice of external bonding of FRP is an accepted practice with various technical reports; recommendations and codes of practice available worldwide. The concrete strip foundation of a typical Japanese home resembles and inverted T-beam, however due to site constraints, the accepted method of bonding FRP to the extreme tensile concrete surface of the strip foundation for enhancement of flexural capacity is not physically possible. In a majority of the homes only the external vertical surface of the strip foundation is accessible. With this major constraint in mind, a concept was put forth to bond GFRP to one side of the typical strip foundation as close to the tensile region as possible to enhance its flexural capacity as seen in Figure 1 in Appendix A.

2.1 Summary of Testing of GFRP strengthened Strip Foundation

In 2006 an in depth experimental study was conducted at the Graduate School of Civil Engineering, Kyushu University, Japan. Only the salient points of this study are pointed out in the sections that follow to determine the most effective scheme to strengthen the concrete strip foundation of a typical Japanese home.

2.1.1 Specimen dimensions and material details

Six full scale reinforced concrete T-shaped strip foundation specimens with dimensions and reinforcement typically found in the majority of homes were tested to investigate the possibility of increasing their flexural capacities by applying GFRP in various configurations based on the typical site constraints. All 3000mm length T-shaped specimens were typical full-scale cross-sections. All concrete specimens were cast with Grade 45MPa concrete, reinforced with 10mm diameter (fy= 364MPa) main bars and 13mm diameter at 100mm centers (fy=499MPa) single leg stirrups. Each specimen had a 100mm wide web by 420mm depth, with a 500mm wide flange of 120mm depth as shown in Figure 2 [2].

2.1.2 GFRP Material Selection

The uni-directional GFRP used in the research had composite values as follows: Ultimate tensile strength of 576 Pa, tensile modulus of 2.61 GPa, and elongation at break of 2.2% and a total composite thickness of 1.3mm per layer [3].

2.1.3 Strengthening Scheme

One control specimen (Type A) and five GFRP wrapped specimens with different wrapping configurations (Types B to F) were tested after pre-cracking and epoxy injecting the specimens. Pre Cracking and epoxy injection of the specimens was carried out to replicate the on-site condition of existing foundations prior to wrapping. The specimen wrapping configurations as seen in Figure 3, had GFRP running horizontally for the full length of the specimens as follows: Type B: 1-layer on one entire face, Type C: 2-layers (one full height and one partial height) on one face, Type D: 2-layers (one full height and one partial height) on both faces, Type E and F: 2-layers (both partial height) on one face with the introduction of 10mm diameter GFRP fiber anchors at 450 mm on center.

2.1.4 Test Setup and Loading

After the full curing of the composite, two-point static loading with a constant bending moment zone of 500 mm was applied gradually up to failure of the control specimen (Type A), the strengthened specimens without fibre anchors (Type B through Type D) and the one strengthened specimen with fibre anchors (Type E). The test setup is shown in Figure 4. The strains in the steel reinforcing bars, concrete and GFRP were measured while applying the load. The deflection of the specimens at mid-span were also recorded along with the corresponding load. For Specimen Type F, cyclic loading was introduced by controlling the deflection of the specimen, taking the deflection (δy) associated with yield load of identical specimen Type E as a base, which was tested earlier under static loading. The load increment of each further loading step was equal to $2\delta y$ and cyclic loading was applied three times in each loading step.

2.2 Test Results

The GFRP strengthening system was able to tremendously enhance the flexural capacity and ductility of the concrete strip foundation when compared the control specimen Type A. The comparison of the ultimate load capacities (Pu) of the specimens as well as the maximum strains developed in the GFRP can be seen in Table 1. Upon comparing the Pu of each GFRP strengthened specimen against the ultimate load of the control specimen (Pua), specimens Type B, Type C, Type D, Type E and Type F showed a 2.01, 2.46, 3.76, 3.44 and 3.11 times increase in ultimate load capacity respectively as shown in Figure 5.

A comparison of the ductility factor of specimen Type A ($\mu a = 7.6$) and the ductility factors of all other specimens is shown in Figure 6. The ductility factor is expressed as $\mu = \delta u / \delta y$ where δu is the deflection at mid- span at the time of ultimate load, and δy is the deflection at midspan at the time of yield loa [2]. The specimen with the maximum ductility factor equal to 22.3 was specimen Type E. This was 2.93 times higher than that of Type A. The ductility factor of cyclically loaded Type F was 16.6 and was 2.18 times higher than Type A.

It was observed that the primary failure mode of all the GFRP strengthened specimens was debonding and delamination of the composite from the concrete substrate. However the introduction of anchors in specimen Type E and Type F had a significant effect in delaying the onset of debonding and propagation of the delamination when compared to similar specimen Type C. The anchors allowed for more strain to be developed within the composite prior to debonding thus making the strengthening effect of the composite more efficient. This in turn contributed to the increase the ultimate load as well as the ductility of the anchored specimens. The strain values in the GFRP at Pu can be seen in Table 1in Appendix B.

The results of the load versus deflection relationship between specimens Type A to Type E can be seen in Figure 7. It was seen that Type C failed prematurely by delamination at a load of 70.2 kN with deflection of 17mm whereas Type E failed at a load of 98.1kN (39.7% higher than Type C) with deflection of 28.5mm. Cyclic loading seemed to have had an effect as Type F failed at a load of 88.7 kN and a corresponding deflection of 22 mm as seen in Figure 8. This failure load however was still 26.3% higher than that of specimen Type C which had no anchors.

At the conclusion of this study to test the concept of bonding GFRP to one side of the concrete strip foundation for flexural and ductility enhancement, it was determined that the most effective method to retrofit the typical strip foundation after taking into account the site constraints was to follow the strengthening scheme of specimen Type E and F. This resulted in an average flexural enhancement of 327.5% and an average increase in ductility of 255.5% over the un-strengthened strip foundation specimen.

3. Concept for Wood-Mortar Wall Strengthening with GFRP

The typical Japanese home is built on a concrete strip foundation using a wood post and beam construction method and finished with a 15mm cement mortar layer over the light wood framed walls as seen in the cross- section in Figure 9 [5]. This construction method results in an extremely poor resistance to lateral loads where even mild seismic activity results in cracking and spalling of the architectural mortar due to large deflection of the structure. To exasperate the problem further, most of these dwellings are topped off with heavy decorated ceramic of cement tiled roofs and in the event of a sizable earthquake these homes would suffer serious to catastrophic damage as these top heavy structures would deflect significantly until collapse. There are approximately ten million homes all over Japan that fit into this category [1]. In 2006 a strengthening promotion law came in to effect that stated that by 2015, 90% of Japanese wooden houses should be upgraded to handle any seismic activity up to a magnitude of 7 on the Japanese Richter Scale [5].

A unique concept was devised for strengthening Japanese wooden houses against seismic activity using GFRP materials. The concept was based on increasing the shear capacity of the typical wood-mortar framed walls by applying a diagonal GFRP bracing system. The system would be installed directly onto the external walls of the house right over the existing architectural finishes to enhance the wall shear capacity and the wall performance. The GFRP would then be covered over with a thin coat of architectural mortar making almost unnoticeable.

3.1 Testing of Wood-Mortar Walls Strengthened with GFRP

To investigate the effectiveness of this concept, a test program was initiated in September 2007 at the Nanyang Technological University in Singapore and followed with more in-depth testing at Oita University in Japan. Seven full scale standardized typical wooden wall specimens with mortar finishing were prepared and tested under in-plane cyclic loading up to failure following the instruction of the Japanese Building Standard Act, Enforcement order article 46, clauses 4 table 1-8 [6].

3.1.1 Wood Material Selection

All wood frame members were made of Japanese cedar, but only the upper beams were American pine with properties as shown in Table 2.

3.1.2 GFRP Material Selection

The GFRP composite system selected as the bracing system for six of the test walls was the same uni-directional high-strength GFRP with continuous E-glass fiber orientated parallel to longitudinal axis of the fabric that was used in the strip foundation strengthening. The same GFRP was used in order to try and keep the type of material uniform throughout the test program as well as under any future commercial environment as this would help keep costs down. Fiber anchors were once again incorporated to improve the end details and force transfer of the GFRP into the wood frame. For one specimen a custom, bi-directional GFRP with continuous E-glass oriented in the $\pm 45^{o}$ direction was used due to the specific configuration of this test wall specimen as it included a window opening. Details of the GFRP mechanical properties are listed in Table 3.

3.1.3 Specimen Details and GFRP Strengthening Scheme

The typical full scale timber wall specimen was composed of two horizontal beams (top beam and bottom sill) three vertical columns, two vertical internal studs and twenty-four lath boards as shown in Figure 10. The distance between two vertical columns is 910 mm, while the distance between the horizontal beams is equal to 2730 mm. All specimens have a total width of 2520 mm and height of 2857 mm.

The fabrication of the wooden wall section followed the same process as used in actual construction of a wooden house in Japan. The beams and columns were connected using a mortise and tenon joint with single dowel pin. The horizontal lath boards were attached to the wooden frame with nails. A waterproofing tar paper was then attached to the lath boards with staples at 300mm c/c both ways followed by a metal mesh (chicken wire mesh) stapled to the lath boards with staples 300mm c/c both ways over the waterproofing paper. A 15mm cement mortar finish was then applied to the surface of the wall over the steel mesh and waterproofing paper. The complete wall specimen configuration with lath boards and mortar finishing is shown in Figure 10.

In the experimental program, seven specimen types were tested. Specimen Type 1 (control specimen) was built to mimic a typical Japanese timber wall shown in Figure 10. Strengthened specimen Type 2 was similar to Type 1, but a one layer diagonal bracing of 300mm wide by 1.3mm thick GFRP was bonded over the mortar finishing and then fixed onto the four corners of the walls using introduced 300mm x 300mm plywood anchor plates (anchor boards) and fiber anchors as shown in Figure 11. To investigate the roll of the external finishing mortar, strengthened specimen Type 3 was prepared similar to Type 2 but without the 15mm finishing mortar as shown in Figure 12. Strengthened specimen Type 4 was similar to Type 2, but the GFRP diagonal sheets were extended and anchored into a reinforced concrete strip foundation to test the entire system as an assembly as seen in Figure 13. The role of plywood anchor plates was investigated in specimen Type 5 which was similar to Type 2 but with no anchorage plates as shown in Figure 14. This was done to determine if anchorage plates were really necessary in any future commercial environment as they involved more cost and labor. Specimen Type 6 (shown in Figure 15) followed the same strengthening system as Type 4 but represented a wall with a door-like opening. Finally specimen Type 7 used the +/- 45^{o} GFRP as this wooden wall specimen mimicked a wall with a window- like opening as shown in Figure 16. A summary of the details and GFRP strengthening scheme of all the wall specimens are listed in Table 4.

3.1.4 Test Setup

A hydraulic jack with 600 mm stroke was used to apply cyclic loading to the upper beam of the wall. The left end of the hydraulic jack was fixed to the steel frame which was used as a horizontal reaction wall. A load cell with 100kN capacity was connected to the end of the jack to measure the magnitude of the load. The wall specimens were fixed to a steel I-beam with three D16 anchor bolts and the steel I-beam was anchored to the ground as a rigid sill. The horizontal movement was restrained by the metal supports at both ends of the bottom of the wall. Four linear variant displacement transducers (LVDT) were employed to measure the displacements. The first one (H1) was used to measure the horizontal displacement at the top of the wall. The second (H2) was used to measure the bottom horizontal displacement. The last two (V3 and V4) were used to measure vertical displacements at the left and right column sides, respectively. The test setup can be seen in Figure 10.

3.1.5 Loading Procedure

To simulate the seismic loading conditions on real timber structures, cyclic loads with gradually increased amplitude are applied to the upper beam. Totally seven cyclic loading steps are applied. Three cycles push and pull are performed in each step. Loading steps is controlled by the observed Shear Transformation Angle (STA).

The details of the loading steps are shown in Figure 17. The vertical distance between the upper and lower horizontal LVDTs (H1 and H2) was equal to L= 2730 mm. The relative movement of the wall is $\delta 1$ = H1-H2. After the twenty-one cycles of cyclic loading, the specimens are loaded under static loading up to failure.

3.2 Structural Performance of the Wall

Based on the load versus STA data, the hysteretic performance envelope curve of the tested walls were drawn. The maximum load (Pmax), yield strength (Py), ultimate strength (Pu), allowable shear strength (Pa) and the ductility factor (μ) of the framed wall specimens were derived and calculated based on a bilinear model that follows the instruction of Japanese Building Standard [6]. Figure 18 shows details of the bilinear model. Line (I) is a straight line between 0.1Pmax and 0.4Pmax of the envelop curve. Line (II) is a straight line between 0.4Pmax and 0.9Pmax of the envelop curve. Line (III) is a line parallel to II and tangent to the envelop curve. The value of the yield strength (Py) can be determined by the intersection point of line (I) and line (III). The wall stiffness (K) can be obtained by dividing the value of yield strength (Py) by the value of the yield STA (Dy). The ultimate STA (Du) is equal to the value of the STA when the applied load is equal to 0.8Pmax. Line (V) is a line between the (0,0) point and the (Dy, Py) point. The ultimate strength of the wall

(Pu) is determined, so the area under the lines (V, VI and VII) is equal to the area under the envelop curve.
The Japanese Building Standard specifies that the value of the allowable shear strength (Pa) of framed wooden wall is taken as the smallest of the following values [6]:

1. Shear capacity when shear transformation angle is equal to 1/120 rad. (P120).

2. Yield capacity (Py).

3. 2/3rd the value of maximum load (Pmax).

4. The value of (0.2Pu) $\sqrt{(2\mu - 1)}$
   where: (μ=Du/Dy) is the ductility factor.

3.3 Summary of Experimental Results

All the strengthened specimens showed a remarkable increase in maximum load carrying capacity over the control wall specimen ranging from a 568% increase up to a 1036% increase depending on the type of specimen and GFRP wrapping configuration. A summary of the calculated test results can be found in Table 5. Note that the units for all the load values are based on a per meter running length of the wall (kN/m) as compared to the raw data in the hysteresis curves that show the loads in Kilo Newtons (kN). The Following five subsections compare and summarise the salient differences between the various types of wall specimens tested.

3.3.1 Type 1 vs. Type 2
The hysteretic performance of specimens Type 1 (control specimen mortar wall no GFRP) and Type 2 (with mortar wall and GFRP anchored in top wood beam and bottom wooden sill) are compared by the curves shown in Figure 11. For the control specimen without GFRP, the load capacity didn’t increase after the third load step of the cyclic loading. After the fifth step, the load dropped sharply. For specimen Type 2, the loads kept increasing throughout all the loading steps. When the applied load reached 33kN, initial failure in the wood sill ground beam occurred and the load carrying capacity of the wall started to drop gradually as the STA values increased. The allowable shear strength (Pa) of specimens Type 1 and Type 2 were 1.59kN/m and 8.63 kN/m respectively representing a 543% increase in Pa of Type 2 over Type 1. The wall stiffness also increased by
261% from 1.70 MN/rad to 4.44 MN/rad

3.3.2 Type 2 vs. Type 3
Figure 12 shows a comparison between the behaviour of specimen Type 2 (with mortar wall and GFRP anchored in top wood beam and bottom wooden sill by anchorage plates) and Type 3 (wall with no mortar but with GFRP anchored in top wood beam and bottom wooden sill with anchorage plates) under cyclic loading. Bonding the GFRP sheets directly over the mortar surface in wall specimen Type 2 seemed to change the role of the wall mortar from an architectural finish to a structural element by increasing the stiffness of the wall. The increased stiffness of the wall was 2.9 times over that of specimen Type 3 which had no mortar. The failure of Type 3 was due to horizontal longitudinal cracking in the wood sill ground beam. The Pa of Type 3 was 5.58 kN/m as compared to 8.63 kN/m for Type 2.

3.3.3 Type 2 vs. Type 4
Figure 13 shows a comparison between the behaviour of specimen Type 2 (with mortar wall and GFRP anchored in top wooden beam and bottom wooden sill by anchorage plates) and Type 4 (with mortar wall on concrete strip foundation assembly and GFRP anchored in the top wooden beam with anchorage plates and extended and anchored into the concrete strip foundation) under cyclic loading. The value of Pa for Type 4 was 1.72 times more than the value of the GFRP

strengthened specimen Type 2. The ductility factor and the initial stiffness were 1.93 and 1.4 times greater than that of Type 2. By extending the GFRP into the concrete strip foundation as in specimen Type 4, failure of the wood sill ground beam that sits on the foundation was averted as that seemed to be the weakest point in the system thus far. Specimen Type 4 also showed a remarkable increase in Pa of 938% over the control wall specimen Type 1 and a maximum load (Pmax) carrying capacity increase of 1036% over the control.

3.3.4 Type 2 vs. Type 5

Figure 14 shows a comparison between Type 2 (with mortar wall and GFRP anchored in the top wooden beam and bottom wooden sill by anchorage plates) and Type 5 (with mortar wall and GFRP anchored directly in the top wooden beam and bottom wooden sill without anchorage plates). Even though the Pmax of Type 5 was smaller than Type 2 by 11.4%, the Pa of Type 5 (8.14 kN/m) represented only a 7.8% drop over the Pa of Type 2 however this was still an 512% increase in Pa over the control specimen. What could be inferred with this result was that one could do away with the wooden anchor plates at each of the 4 corners of the wall specimens and directly insert to fibre anchor into the wooden posts and beams at the GFRP strip termination ends. This would be a cost and time saving under a commercial environment.

3.3.5 Type 6 and Type 7

The allowable shear carrying capacity for specimen Type 6 (mortar wall with a door like opening) after strengthening with GFRP is equal to 9kN/m, while the Pa for Type 7 (mortar wall with a window like opening) is equal to 3.8kN/m. Since there was no direct control to compare these type specimens, one can’t directly compare the increase in Pa. However, common sense dictates that control specimen Type 1 without a door or window opening would actually have a higher Pa than if there were control specimens of Type 6 and Type 7 with a door and window opening respectively. In using this assumption, the Pa of Type 6 and Type 7 would show a significant increase over their controls.

4. Conclusion

A three year study at three different universities was undertaken to systematically test, develop, patent and obtain performance based approvals for a method to strengthen wooden homes in Japan using GFRP.

Testing of standardized typical concrete strip foundations that are found in the majority of old homes strengthened with strips of GFRP bonded and anchored to only one side of the specimen showed a great increase in moment capacity and ductility over the un-strengthened control specimen. An average flexural enhancement of 327.5% and an average increase in ductility of 255.5% over the un-strengthened strip foundation specimen was obtained.

Testing of standardized typical wood-mortar walls that are found in the majority of wood construction homes in Japan for enhancement of shear capacity by bonding and anchoring a diagonal GFRP bracing system on the external mortar surface of the wall showed a significant increase in the allowable shear capacity of the strengthened wall specimens over the un-strengthened control wall. The increase in allowable wall shear capacity on the wood-mortar wall specimens ranged from 512% to 543% over the un-strengthened control wall specimen. When the diagonal GFRP bracing system was extended further down onto the concrete strip foundation a 938% enhancement of allowable shear capacity was observed.

At the conclusion of the wall study, it was clear that the existing weak wood-mortar walls of a typical Japanese home could be strengthen substantially with the use of GFRP strips applied in a cross brace pattern to the external surface of the walls and anchored into the already strengthened concrete strip foundation that was tested in the first phase of this research and development project. This minimally invasive, light weight system could be installed over the external walls of Japanese houses without disturbing the daily life of the residents as a majority of the work could be executed from the outside of the home.

This development program has resulted in a new, unique, effective and simple method to retrofit the concrete strip foundations and wood-mortar walls of a typical Japanese wood construction home for seismic resistance using of-the-shelf, readily available GFRP materials so as to meet the strengthening requirement as prescribed by the Japanese government. The system has subsequently been patented and approved by the local authorities and been on the market in Japan since March 2009 and has been installed on a number of homes throughout Japan.

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[1] White Paper on Land, Infrastructure and Transport in Japan - 2005, Ministry of Land, Infrastructure, Transport and Tourism, pp. 27 (2005).

[2] B. Abdullah, Dr. S. Hino, R. Jamaji, K. Kobayashi, Proceedings of the 8th International Symposium FRPRCS, Greece, Paper Id# 3-18, 3, pp. 1-9 (2007).

[3] Data Sheet: Tyfo® SEH51-A Composite using Tyfo® S Epoxy, Fyfe Co. LLC, California (2007). [4] Data Sheet: Tyfo® BC Composite using Tyfo® S Epoxy, Fyfe Co. LLC, California (2007).

[5] House Defender®-Seismic Strengthening for wood Residential Structures, Honebuto Jutaku, Japan (2009).

[6] B. Abdullah, R. Jamaji and S.P. Chiew, Proceedings of the 4th International Conference of FRP Composites in Civil Engineering, Switzerland, Paper Id# 2.D.4, pp. 2, 5 (2008).

[7] Bureau of Urban Development, Tokyo Metropolitan Government, Japan http://www.toshiseibi.metro.tokyo.jp/kenchiku/taisin/taisin_anzen-pnf.pdf , pp. 13 (2009).

Appendix A – Figures

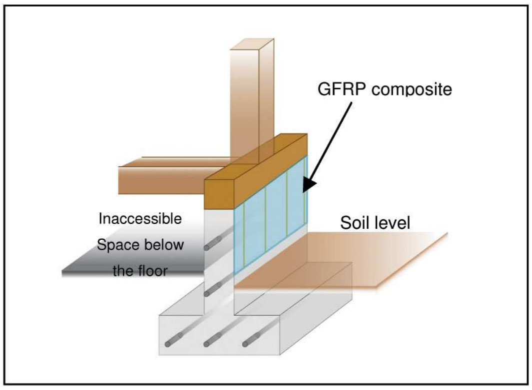


Figure 1: Proposed bonding of GFRP to External Face of strip Foundation

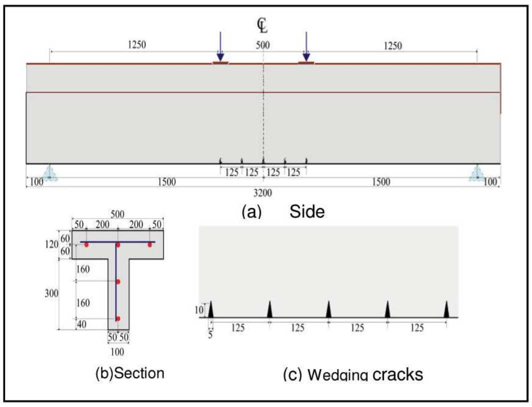


Figure 2: Specimen Dimensions and Details

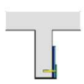
Type	Section	Description
A		Control Specimen – No GFRP
B		One layer of GFRP (3200mm x 300mm) bonded to one side of the web.
C		Two layers of GFRP: First layer (3200mm x300 mm) and second layer (3200mm x 150mm) bonded to one side of the web.
D		Two layers of GFRP: First layer (3200mm x 300mm) and second layer (3200mm x 150mm) bonded to both sides of the web.
E & F		Two layers of GFRP: Both layers (3200mm x 200mm) bonded to one side of the web. Six GFRP fibre anchors installed at 450mm c/c over the 2 GFRP layers.

Figure 3: GFRP Wrapping Scheme of Specimens Type A to Type F

Figure 4: Test Rig Setup of Strip Foundation

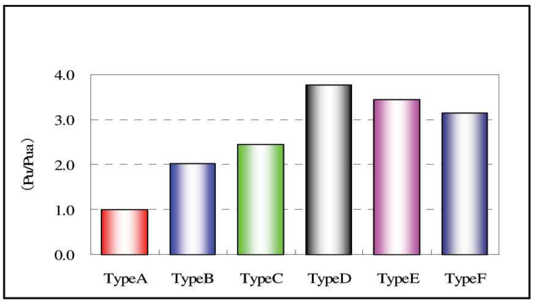


Figure 5: Ultimate load Comparison of Strip Foundation Specimens

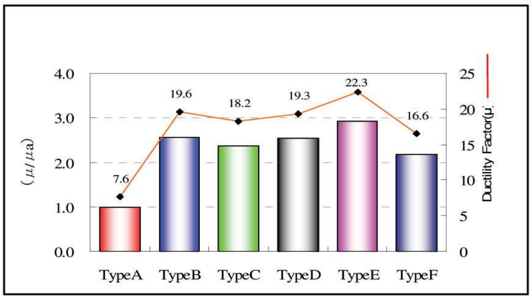


Figure 6: Comparison of Strip Foundation Specimen Ductility

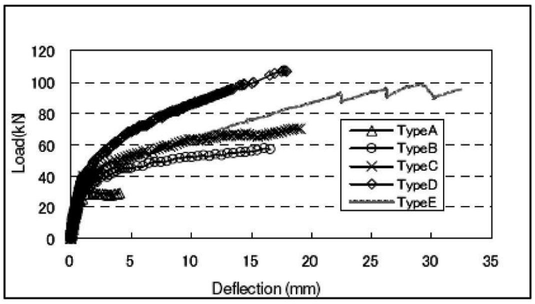


Figure 7: Load-Deflection Relationship of Specimens Type A to Type E

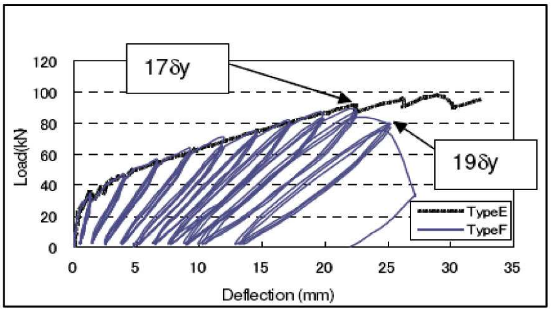


Figure 8: Load-Deflection Relationship of Specimens Type E and Type F

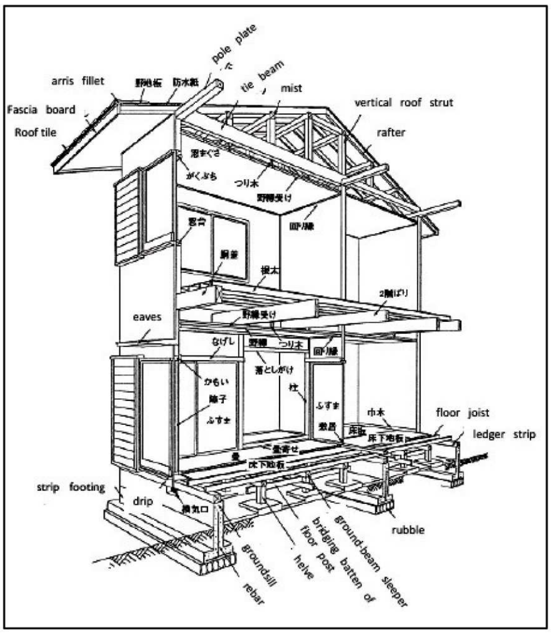


Figure 9: Cross Section of a Typical Japanese Home with Standardized Concrete Foundation and External Walls [5]

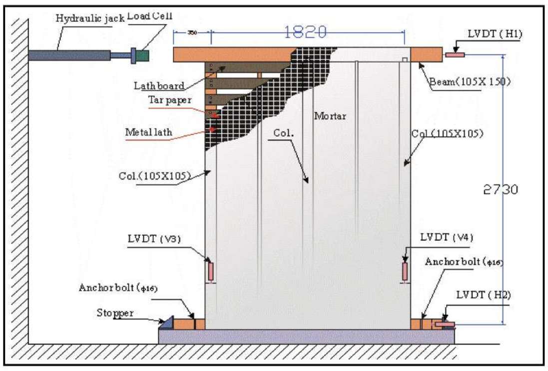


Figure 10: Test Setup and Cut Away Details of Typical Wall Specimen Type 1 (Control)

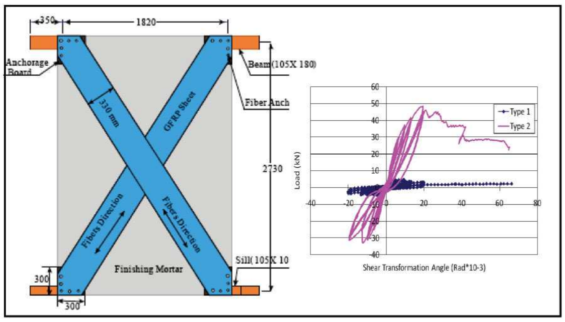


Figure 11: Wall Type 2 – Layout and Results

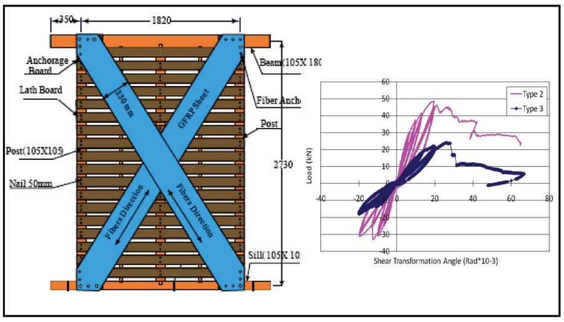


Figure 12: Wall Type 3 – Layout and Results

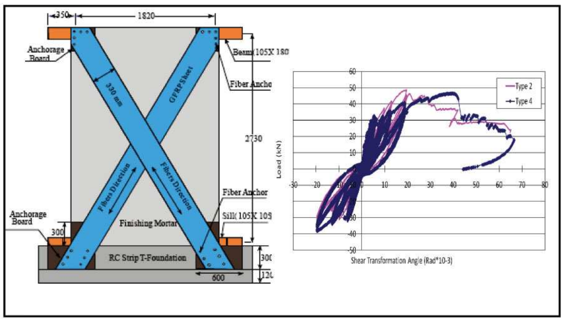


Figure 13: Wall Type 4 – Layout and Results

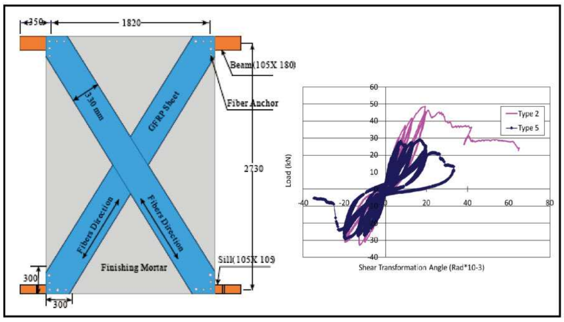


Figure 14: Wall Type 5 – Layout and Results

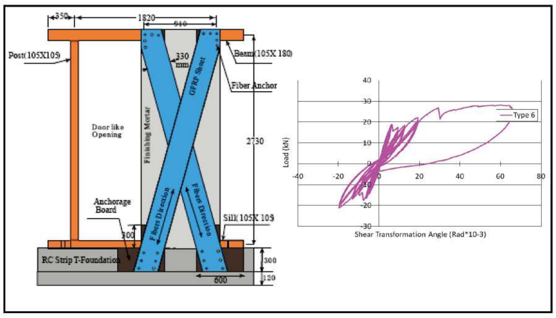


Figure 15: Wall Type 6 – Layout and Results

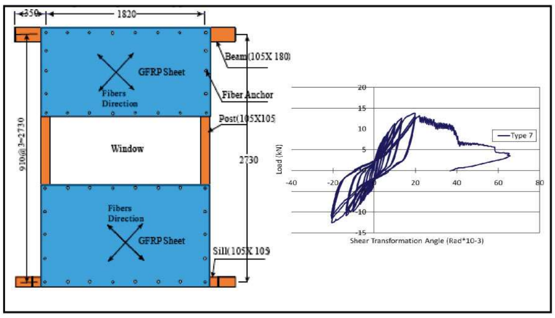


Figure 16: Wall Type 7 – Layout and Results

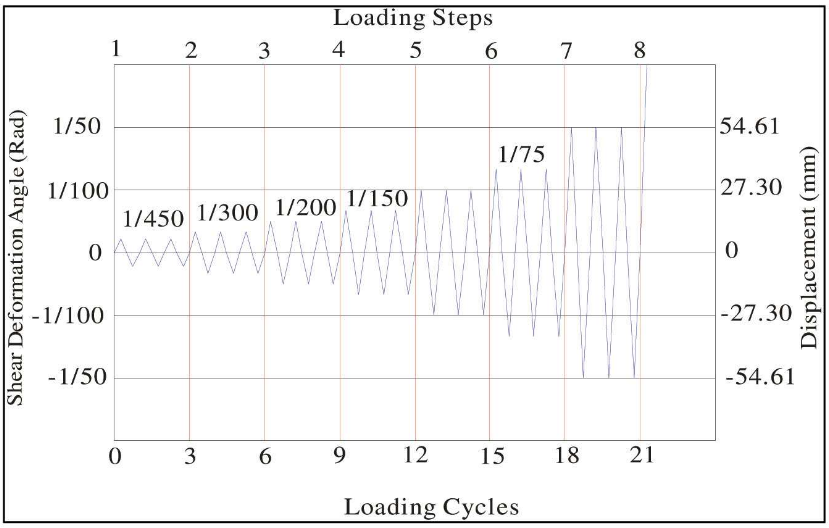


Figure 17: Load Step Procedure for Wall Tests

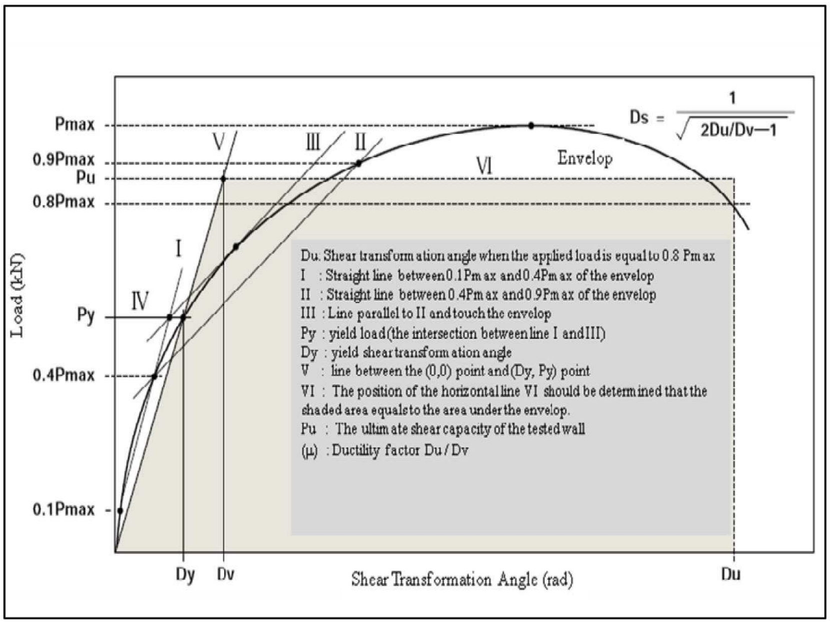


Figure 18: Details of Calculating Yield Strength, Ultimate Strength, Allowable Shear Strength and Ductility Factor of the Framed Wall Specimens using the Bilinear Model [6]

Appendix B: Tables

Table 1: Summarized Experimental Results for Concrete Strip Foundation

Property / Specimen Type	A	B	C	D	E	F
Crack Load, Pcr (kN)	23.0	20.5	23.5	23.7	20.5	20.5
Yield Load, Psy (kN)	20.6	29.2	36.0	36.6	28.3	28.3
Ultimate Load, Pu (kN)	28.5	57.3	70.2	107.3	98.1	88.7
Pu / Pua	1.0	2.01	2.46	3.76	3.44	3.11
GFRP Strain at Pu (%)	---	0.8	0.5	1.2	1.35	1.2

Table 2: Wooden Members Mechanical Properties

Members	Wood Type	E-Modulus (GPa)	Comp. Strength (MPa)	Tensile Strength (MPa)
Post	J. Cedar	6.86	17.7	22.2
Sill-Ground	J. Cedar	6.86	17.7	22.2
Beam	A. Pine	9.80	22.2	28.2
Lath Boards	J. Cedar	6.86	17.7	22.2

Table 3: GFRP Mechanical Properties [2], [3], [4]

GFRP	E-Modulus (GPa)	Ultimate Tensile Strength (MPa)	Ultimate Strain	Laminate Thickness per Layer (mm)
Unidirectional GFRP	26.1	575	2.2%	1.300
+/- 45^o GFRP	18.6	279	1.5%	0.864
Fiber Anchor	26.1	575	2.2%	10 mm Diameter

Table 4: Wall Specimen Details

Specimen Type	With Cement Mortar	Anchorage Plate	With RC Foundation	GFRP
Type 1	Yes	No	No	No
Type 2	Yes	Yes	No	Unidirectional
Type 3	No	Yes	No	Unidirectional
Type 4	Yes	Yes	Yes	Unidirectional
Type 5	Yes	No	No	Unidirectional
Type 6	Yes	Yes	Yes	Unidirectional
Type 7	Yes	No	No	+/- 45^o Bidirectional

Table 5: Wall Experimental Testing Results

Specimen Type	Wall Stiffness, K (MN/rad)	Incr. in K vs. Ctrl	Pmax*	Incr. in Pmax vs. Ctrl	Py*	Incr. in Py vs. Ctrl	2/3Pmax*	0.2Pu√(2μ-1)*	Pa*	Incr. in Pa vs. Ctrl
Type 1 (Ctrl)	1.70	---	2.47	---	2.20	---	1.65	1.59	1.59	---
Type 2	4.44	261%	18.13	734%	15.38	699%	10.99	8.63	8.63	543%
Type 3	1.53	-10%	14.04	568%	9.14	415%	8.03	5.58	5.58	351%
Type 4	6.17	363%	25.59	1036%	15.99	727%	17.10	14.91	14.91	938%
Type 5	6.73	395%	16.06	650%	12.93	588%	8.26	8.14	8.14	512%
Type 6	3.22	---	15.53	---	9.01	---	10.02	10.98	9.01	---
Type 7	1.85	---	7.64	---	5.63	---	3.77	4.18	3.77	---

* Units in kN/m

PRESTRESSED FIBRE REINFORCED POLYMER (FRP) LAMINATES FOR THE STRENGTHENING OF REINFORCED CONCRETE STRUCTURES

M.S.Mohamed Ali[1],D.J.Oehlers [1], D.Callary[1] and D.Knight[1]

[1]School of Civil, Environmental and Mining Engineering, University of Adelaide, Australia.

Abstract. Recent development in the field of strengthening has seen the application of prestressing of FRP laminate prior to bonding in order to exploit its high tensile strength. The method of prestressing the laminate induces an initial tensile strain in the concrete beams upmost fibre, thus reducing the deflection of the beam throughout the design loads. This alteration of the beams structural characteristics provides advantages in beam serviceability requirements. Structurally the beam can withstand greater ultimate loads, while yielding of internal reinforcement and cracking moments are delayed substantially, compared to unlaminated beams. Extensive experimental investigations have been undertaken by many researchers with variables ranging from anchorage type, number of laminates applied to beam, tensile reinforcement ratio and the initial prestress level of laminates before bonding. Despite the large amount of experimental data in the field, current analytical models generally employ elementary procedures in predicting beam behaviour and as a result the analytical results exhibit poor correlation with the experimental results. This implies the necessity for the development of a generic model that can accurately predict beam behaviour that will be the basis of the present study. The focus of this paper is the development of a new analytical model that can accurately predict the behaviour of an RC beam strengthened with an externally bonded (EB) prestressed fibre reinforced polymer (FRP) laminate. The model will be critically compared to an experimental database for calibration purposes then applied in a parametric study.

1. Introduction

As many reinforced concrete (RC) structures meet the end of their service life due to deterioration or the need to increase service loads, the development of alternatives to replacement are been found. Such an alternative is that of retrofitting the concrete structure with externally bonded fibre reinforced polymer (FRP) laminates. This technique suffers from many shortcomings including premature debonding; primarily intermediate crack debonding which affects flexural strength and ductility of the beam (Oehlers and Sercacino 2004). As the laminate debonds prematurely the full tensile strength of the laminate, in excess of 3700MPa is not achieved and therefore the method becomes inefficient. Recent development in this field has seen the application of prestressing of FRP laminate prior to bonding in order to exploit its high tensile strength. The method of prestressing the laminate induces an initial tensile strain in the concrete beams upmost fibre, thus reducing the deflection of the beam throughout the design loads (Yu et al. 2008). This alteration of the beams structural characteristics provides advantages in beam serviceability requirements. Structurally the beam can withstand greater ultimate loads, while yielding of internal reinforcement and cracking moments are delayed substantially, compared to unlaminated beams (Yang et al. 2008).

Extensive experimental investigations have been undertaken by many researchers with variables ranging from anchorage type, number of laminates applied to beam, tensile reinforcement ratio and the initial prestress level of laminates before bonding (Xue et al. 2008; Yang et al. 2008). Despite the large amount of experimental data in the field, current analytical models generally employ elementary procedures in predicting beam behaviour and as a result the analytical results exhibit poor correlation with the experimental results (Yu et al. 2008). This implies the necessity for the development of a generic model that can accurately predict beam behaviour; that will be the basis of the present study.

2. Moment-Rotation model

After cracking occurs in an RC beam the full interaction(i.e. there is no slip occurs between the reinforcement and surrounding concrete) approach becomes inaccurate and therefore using a partial interaction, moment- rotation approach is necessary. This approach has been developed at the University of Adelaide in the past few years and is most recently presented in Mohamed Ali et al.

(2009) and Haskett et al. (2009). Both approaches assume a single crack at mid-span about which the beam rotates, refer to Fig 1. This rigid body rotation can be idealised, as shown in Fig 3 (Haskett et al. 2009).

Concrete compression zone: In the analysis of the RC beam the concrete compression zone can be split into two regions; the concrete softening region and the concrete ascending region, refer to Fig 2. Equations have been derived to determine the force in each of these regions and are presented in Haskett et al. (2009) and Mohamed Ali et al. (2009), shown in Eqs.1&2. When developing the new model in this research, the two different cases will need to be taken into account for the compression region as presented in Mohamed Ali et al. (2009). Initially in the case where the maximum strain, $\varepsilon_{c,max}$, in the concrete is less than the peak strain, ε_{pk}, (Fig 2) then there will be no concrete softening i.e. no wedge forms. In this case a linear strain profile is assumed in the compression zone and therefore the force in the concrete can be found using the stress-strain relationship for concrete under uniaxial compression, as shown in Fig. 2 and Eq. 2. The second case is when the compressive strain reaches peak strain and therefore there will be a softening region with depth, d_{soft}, and length, L_{soft}, (Fig 1). Shear friction theory has been used to find the force in the softening region and this force can be found through knowing the stress at softening, σ_{soft}, which is dependent on the lateral confinement σ_{lat} of the concrete and the shear friction material properties, m and c, refer Fig 2 and Eq 2.

For $\varepsilon_{c,max} < \varepsilon_{pk}$ $\quad P_{cc} = \dfrac{f_c b \varepsilon_{c,max}}{\varepsilon_{pk}}\left[1 - \dfrac{\varepsilon_{c,max}}{3\varepsilon_{pk}}\right]\left[\dfrac{\varepsilon_{c,max} d_{asc}}{(\varepsilon_{c,max}+\varepsilon_r)}\right]$ (1)

For $\varepsilon_{c,max} \geq \varepsilon_{pk}$ $\quad P_{cc} = P_{asc} + P_{soft},\quad P_{asc} = \frac{2}{3} f_c b d_{asc}$ (2)

$$\& \quad P_{soft} = w_b\, d_{soft}\left[\frac{c + \sigma_{lat}\cos\alpha(\sin\alpha + m\cos\alpha)}{\sin\alpha(\cos\alpha - m\sin\alpha)}\right]$$

Where: f_c is the concrete compressive strength; b is the width of the beam; d_{asc} is the depth of the ascending region; ε_r is the strain in the reinforcement; d_{soft} is the depth of the softening region and α is the angle of inclination of the concrete wedge.

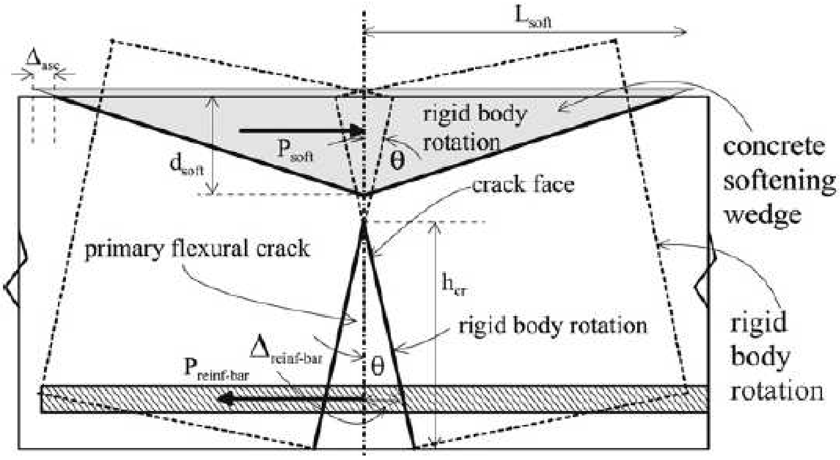


Fig.1 Idealised rigid body rotation

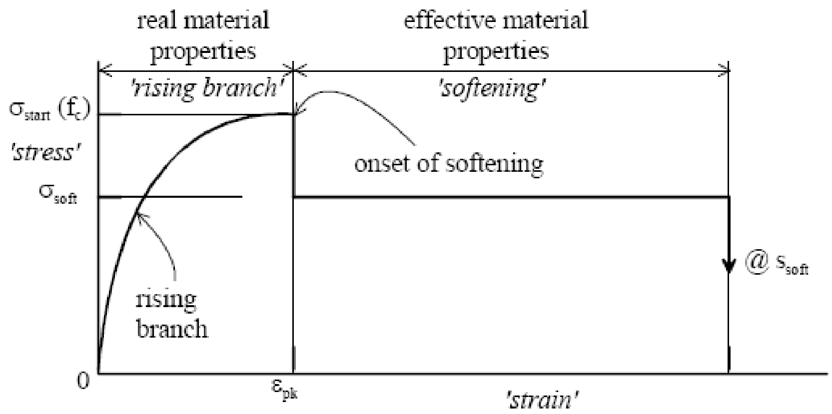


Fig.2 Stress strain relationship of concrete under compression

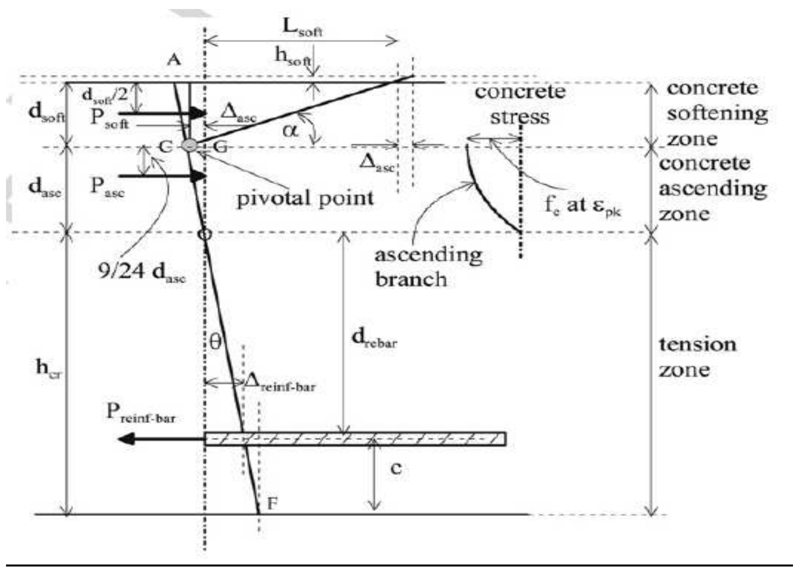


Fig.3 Moment-rotation analysis

Reinforcement force- slip relationship: The reinforcement force – slip relationship is critical in this approach as for a given slip (Fig 3), the corresponding crack width (twice the slip) and the force in the steel reinforcement can be found using derived equations. This force - slip relationship is dependent on the bond-slip characteristics between the reinforcement and the concrete. These characteristics, control the interface slip and hence the crack width and the rotation. Four typical idealized bond slip relationships are shown in Fig.4. For the model to be developed in this study, the uni-linear-descending relationship, line A-C, will be used as Eqs. 3-7, have been developed by Haskett et al. (2009) and Mohamed Ali et al. (2009), based on this relationship. These equations are presented below and will be crucial when developing our model. Two equations are shown for the force in the steel reinforcement as the force will vary depending on when the steel yields. For our model the equation for the force in the FRP reinforcement will need to be modified to incorporate the additional force due to the FRP laminate being prestressed. This additional force can be calculated through knowing the additional strain in the laminate due to prestressing as presented in Xue et al. (2009). The slip at yield can be determined by:

$$\Delta_{yield} = \delta_{max}\left(1 - \cos(\lambda_{el}\alpha_{el})\right) \qquad (3)$$

Where
$$\alpha_{el} = \frac{\arcsin\left[\frac{A f_y \lambda_{el}}{L_{per}\tau_{max}}\right]}{\lambda_{el}} \qquad (4)$$

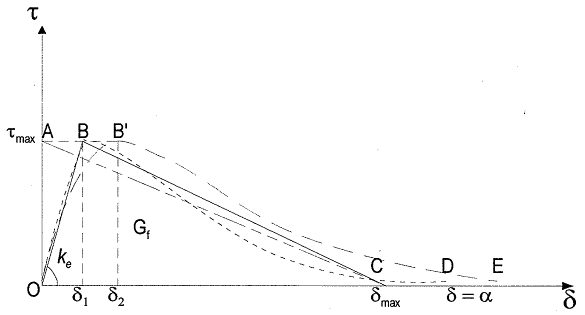


Fig.4 various interface bond characteristics

Equations for the force in reinforcing laminate and in reinforcing bar prior to yielding are:

$$P_{reinf-plate} = \frac{\tau_{max} L_{per}}{\lambda_{el}} sin\left\{arccos\left\{\frac{\Delta_{reinf-plate}}{\delta_{max}}\right\}\right\} \quad (5)$$

$$P_{reinf-bar} = \frac{\tau_{max} L_{per}}{\lambda_{el}} sin\left\{arccos\left\{\frac{\Delta_{reinf-bar}}{\delta_{max}}\right\}\right\} \quad (6)$$

$$P_{reinf-bar} = \frac{\tau_{max} L_{per}}{\lambda_{sh}} sin\left\{arccos\left\{\frac{\Delta_{reinf-bar} - \Delta_{yield}}{\delta_{max}}\right\}\right\} + A_r \sigma_y \quad (7)$$

Where $$\lambda_{el} = \sqrt{\frac{L_{per} \tau_{max}}{\delta_{max} E_s A}} \text{ and } \lambda_{sh} = \sqrt{\frac{L_{per} \tau_{max}}{\delta_{max} E_{sh} A}}$$

Where: τ_{max} is the interface bond-slip shear capacity (Fig 4); L_{per} and A are the perimeter and cross sectional area of the reinforcement respectively; δ_{max} is the bond slip capacity of the reinforcing ; E_s is the elastic modulus of the reinforcing material; E_{sh} is the strain hardening modulus of steel and $\Delta_{reinf-plate}$, $\Delta_{reinf-bar}$, Δ_{yield} are the slips at the reinforcing laminate, the reinforcing bar and at bar yield, respectively.

Through using the methods discussed above for finding the forces in the beam, an iterative approach can be used to allow for a moment-rotation relationship to be derived. Many pivoting points may be used when using an iterative approach; for the model in this research for ease of use, a depth of the softening region, d_{soft} will be initially assumed. Through knowing d_{soft} the slip of wedge can be found geometrically, which can then be used as the pivotal point in the analysis, i.e. change the rotation whilst keeping the slip in the wedge constant, until the force equilibrium is satisfied. Once the equilibrium is satisfied the moment for the given rotation can be found. This partial interaction, moment-rotation approach will be used for the analysis throughout loading. At every stage of the analysis, the limits on rotation will need to be checked. These limits are reinforcement debonding or fracture and concrete wedge sliding. The slip to cause FRP fracture, $\Delta_{fracture}$ (mm), and debonding, $\Delta_{debonding}$ (mm), are given as:

$$\Delta_{fracture} = \delta_{max}(1 - \cos(\lambda_{el} a_{el})) \quad (8)$$

Where $$a_{el} = \frac{\arcsin\left[\frac{A f_{fract} \lambda_{el}}{L_{per} \tau_{max}}\right]}{\lambda_{el}}$$

$$\Delta_{debonding} = \delta_{max} + \frac{\varepsilon_{\delta max} d}{2} \quad (9)$$

Where $\varepsilon_{\delta_{max}}$ is the strain in the FRP laminate at debonding and *d* is the effective depth of the flexural member. The slip at which sliding failure, s_{slide} (mm), occurs is given by Haskett et al. (2009) as:

$$s_{slide} = 2.51\frac{\sigma_{lat}}{f_c} + 0.42 \qquad (10)$$

Where σ_{lat} is the confinement provided by stirrups, therefore if there are no stirrups then $\sigma_{lat} = 0$.

3. Typical results

A parametric study has been undertaken on beam to find the changes in strength at serviceability due to the increase of application of a prestressed FRP laminate with varying prestress level using the above model. The prestress level in this paper is referred to as the percentage axial strain applied compared to that of the ultimate axial strain of the FRP laminate. A small size beam is used with prestress level varying from 25 – 50% of the axial strength of the FRP laminate and is compared to that of a virgin beam with the same material properties. Fig.5 shows an increase in ultimate load as much as 230% is achieved with the application of a 50% prestressed laminate on the RC beam, in comparison with the unplated beam. Furthermore a decrease in ductility of 46% is achieved with the application of the 50% prestressed laminate. Despite the increase in ultimate load of over 2 times the virgin beam, it should be noted that the act of increasing the prestress level from 25% to 35% and 35% to 50% only increases the ultimate load by 10% and 11% respectively, which poses the question if the extra amount of prestressing is worth the gain in ultimate beam strength.

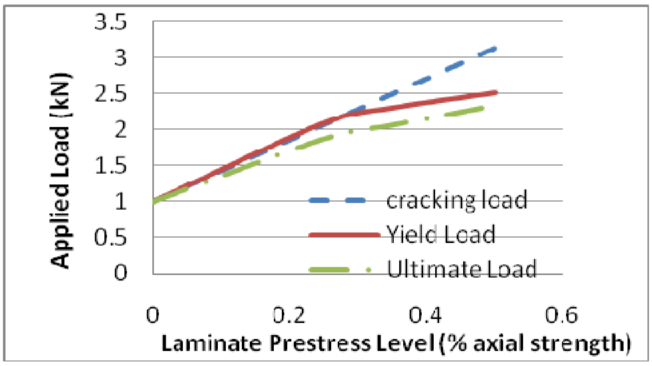


Fig.5 *Normalized-Load Milestones Vs. Laminate prestress levels*

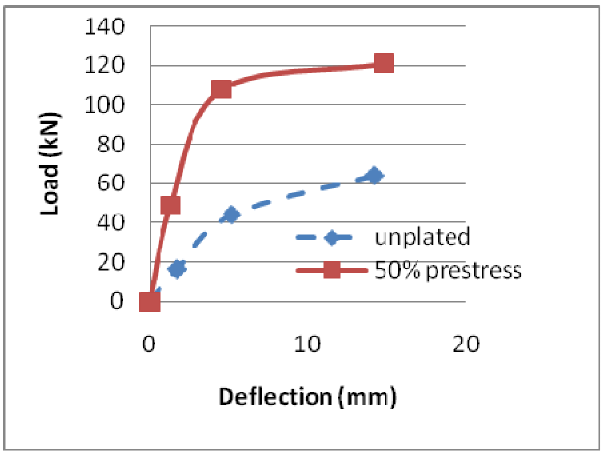


Fig.6 *Load Vs beam deflection*

The affects of cracking load on the beam due to prestress is similar to that of the changes in ultimate strength with an increase of over 300% been achieved due to the application of a 50% prestressed

FRP laminate. An increase in prestress level from 25% to 35% and 35% to 50% results in and increase in cracking load of 17% and 20%, respectively. These results and other findings suggest the preliminary conclusion that the ‘optimal’ prestress level lies somewhere between 25-45% of the axial strength of the laminate. From a serviceability perspective, the application of a prestressed FRP laminate greatly reduces the deflection of the beam throughout the design loads and up until ultimate failure. In reference to Fig.6**,** it can be seen that the presence of a prestressed laminate shifts the load-deflection curve in a vertical manner. The 3-points represented on from left to right are cracking, yield and ultimate, respectively. For example it can be seen that the load at 5mm deflection is increased by in excess of 200%.It can then be concluded that if the optimal prestress level does lie within the 25-45% region, then increases in prestress level of the laminate do not increase the ultimate load

ABSTRACT: The roof slab of a portion of a government public utility building situated in Colombo, Sri Lanka, comprises of precast – prestressed ‘T’ beams. There was a huge fire in this building and during the subsequent refurbishment of the building it was noted that the majority of the ‘T’ beams at this area were exhibiting distresses in the form of cracks. A thorough investigation conducted by the concerned authorities in conjunction with the University of Moratuwa concluded that the structural distress was due to event overloading onto the roof slab during the fire fighting operations. It was also concluded that a requirement of certain remediation and retrofit measures to the distressed elements were necessary to bring the ‘T’ beams back to their intended service performance level. This paper describes in detail the site case study of the repairs and retrofit that was carried out to achieve the desired design objective. Repairs to cracks were carried out using multi-port, low pressure resin injection, the retrofit was carried out using the Carbon Fibre Reinforced Polymer (CFRP) system, thus resulting in a quick, discreet and cost effective way to meet the design objective.

withstood, rather it will decrease the beams deflection.

4. Concluding Remarks

In this paper, a novel analytical model for estimating the rotation and deflection in RC beams bonded with prestressed FRP plate based on the rigid body displacement model at the University of Adelaide and the study is under progress and the model shows promise in estimating the optimal prestressing level for a given RC beam and FRP parameters.

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STRUCTURAL REMEDIATION & RETROFIT OF DISTRESSES PRECAST-PRESTRESSED 'T' BEAMS OF A PUBLIC UTILITY BUILDING, IN COLOMBO, SRI LANKA. – A SITE CASE STUDY.

[1]Dhammika Wimalratna, [2]Sandeep Savardekar

[1]Principal Consultant, Navcorp Engineering Consultants, Colombo, Sri Lanka.
[1]navcorpec@gmail.com

[2]Sr. Technical Manager, FYFE Asia Pte. Ltd., Singapore.

ABSTRACT: The roof slab of a portion of a government public utility building situated in Colombo, Sri Lanka, comprises of precast – prestressed 'T' beams. There was a huge fire in this building and during the subsequent refurbishment of the building it was noted that the majority of the 'T' beams at this area were exhibiting distresses in the form of cracks. A thorough investigation conducted by the concerned authorities in conjunction with the University of Moratuwa concluded that the structural distress was due to event overloading onto the roof slab during the fire fighting operations. It was also concluded that a requirement of certain remediation and retrofit measures to the distressed elements were necessary to bring the 'T' beams back to their intended service performance level. This paper describes in detail the site case study of the repairs and retrofit that was carried out to achieve the desired design objective. Repairs to cracks were carried out using multi-port, low pressure resin injection, the retrofit was carried out using the Carbon Fibre Reinforced Polymer (CFRP) system, thus resulting in a quick, discreet and cost effective way to meet the design objective.

1. INTRODUCTION

Project description

A prominent and critical public utility building of a government department situated in the central business district of Colombo, Sri Lanka comprises of the front tower portion of basement + ground + 14 upper floors and a rear portion rising 4 floors high (Figure 1). The building is a typical reinforced concrete column beam primary framed structure with precast-prestressed type 'T' beams acting as the secondary grid infill. The building was subject to a huge fire on the night of 20th February 2009, after an airborne incursion was shot down and crashed into it. Fire fighting operations were carried out to bring the blaze under control. Subsequently, a major refurbishment effort was initiated to bring the building back to functionality.

Figure 1. : *Rear façade view of the public utility building.*

During this refurbishment exercise, it was noticed that a majority of the 4th floor (roof floor) 'T' beams of the rear portion of the building were exhibiting structural distress in the form of cracks.

Need for structural remediation

The distresses to the prestressed-precast beams in the form of cracks were jointly investigated by the concerned authorities and academicians of University of Moratuwa. The cracks observed were typical flexural crack patterns as shown in Figure 2. The details of the structural investigation and repair retrofit design are not in the scope of this paper and hence are not elaborated upon herein. Briefly, the structural investigation concluded that the distresses in the form of cracks to the beams was due to the impact and event overloading caused by the evacuation of various bulky office equipment from the upper floors of the tower portion onto the roof of the lower rear portion, during the firefighting

Figure 2. : *View of beams showing typical flexural crack patterns.*

operation. The investigations also deemed it necessary to carry out appropriate repair and retrofit measures to bring the distressed beams back to a satisfactory service level.

2. STRUCTURAL REPAIR & RETROFIT DETAILS

Structural repair & retrofit objective

To appropriately address this particular issue at hand, the objective was first determined. The objective was two pronged viz. restoring the structural integrity of the cracked beam elements i.e. repair and thereafter bring about enhancement in the flexural capacity of the repaired beams i.e. retrofit. Out of the various available repair and retrofit options available the most feasible options were explored and short listed to the following based on various criteria:

- The repair / retrofit technique and its feasibility to the current scenario.
- Level of intrusion of a particular technique onto already distressed elements.
- Performance of the repair / retrofit technique with respect to the current scenario.
- Time and cost aspects vis-à-vis other methods.

In view of the above criteria it was imperative that the repair and retrofit system to be put in practice should be non-intrusive, quick, discreet and one which does not impose any additional loads onto the elements. The use of conventional techniques such as concrete jacketing or steel plate bonding for retrofit had been ruled out because of their invasive, time consuming process and additional dead loads. Thus, the repair technique recommended was surface mounted, multi-port low viscosity epoxy resin injection into the cracks, whilst the retrofit technique recommended was carbon fibre reinforced polymer (CFRP) wrapping.

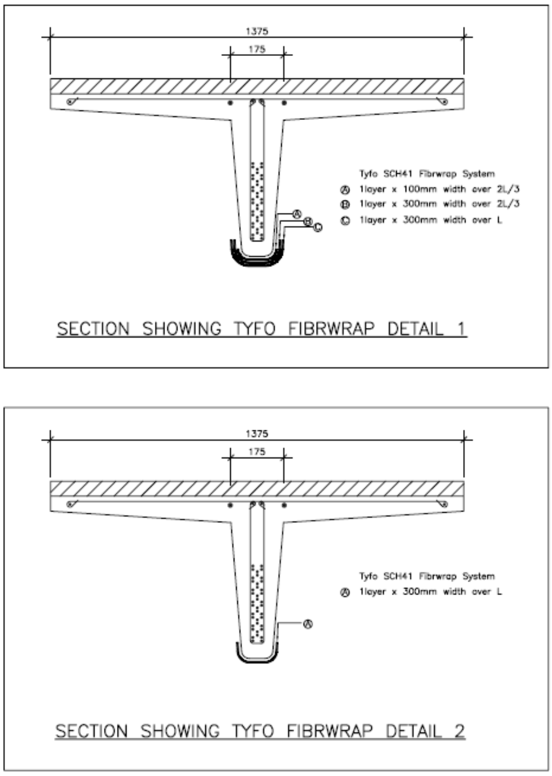

Figure 3. : *CFRP wrapping details*

Repair & retrofit basis

Out of the thirty five beams, four outer beams on each side of the building were found to be without any cracks. These beams were considered to be structurally sound and needing no remedial measures. The central eleven beams exhibited the most severe structural cracks whereas the remaining beams were having minor cracks. The repair process required injection to all such beams exhibiting cracking. Two separate CFRP retrofit schemes / details were designed for the central twelve beams categorized as severe cracking and the other remaining beams categorized as minor cracking. CFRP wrap details are given in Figure 3.

Crack injection resin properties

An appropriate crack injection resin was selected based on the following physical and mechanical criteria;

- Viscosity low enough to have good penetrability into hair line cracks.
- Pot life long enough to facilitate the proposed low pressure, surface mounted, multi port injection technique.
- Good mechanical properties such as bond strength, tensile strength, slant shear strength.

The salient properties of the injection resin used for carrying out the crack repair works are as shown in Table 1.

Table 1. *Properties of injection resin used for crack repair.*

Property	Test Method	Typical Value*
Mix viscosity		200cps @ 25^{o}C
Pot Life		85 minutes @25^{o}C.
		21 minutes @40^{o}C.
Tensile strength	ASTM D-638	62.1 MPa
Wet Slant shear strength	AASHT0 T-237	Cement mortar failure.
Compressive strength	ASTM D-695	110.3 MPa.

* Values reported by injection resin manufacturer in technical datasheet.

CFRP system properties

The CFRP system selected for use was a proprietary system from a reputed CFRP system manufacturer. This system selection was based on the criteria of material characterization, system performance and environmental durability considerations. The CFRP system comprised of a unidirectional carbon fibre fabric as the reinforcement and a compatible two component epoxy resin as the matrix. The key properties of the CFRP system used for carrying out the structural retrofit are as shown in Table 2.

Table 2. *Properties of CFRP system used for retrofit.*

Property	Test Method	Typical Test Value*
Ultimate tensile strength	ASTM D-3039	986.0 MPa.
Elongation at break	ASTM D-3039	1.0%
Tensile modulus	ASTM D-3039	95.8 GPa.
Laminate thickness		1.0 mm

* Values reported by CFRP system manufacturer based on gross laminate properties.

3. STRUCTURAL REPAIR & REMEDIATION PROCESS

Repair & retrofit sequence

The repair and retrofit sequence adopted was as follows:

- Identifying on site and demarcating the various categories of beams viz. beams requiring no intervention, beams requiring crack repairs & detail 1 CFRP wrapping and beams requiring crack repairs & detail 2 CFRP wrapping.
- Carrying out repairs to cracks by low pressure, low viscosity multi port epoxy resin injection.
- Carrying out CFRP wrap installation onto beams.

Repair with injection of cracks

The crack injection works comprised the following activities:

- The crack lines on the beams were identified and demarcated.
- Paint, surface finishes and other deleterious materials were removed by means of wire brushing or grinding to clean the crack line.
- Injection port locations along the crack line at appropriate spacing were then marked.
- Surface mounted injection ports were then fixed along marked locations with epoxy putty.
- The rest of the crack line was sealed with cementitionus non-shrink grout.
- The two parts of epoxy resin were mixed in small batches according to the mix ratio as specified by manufacturer.
- The low pressure auto injector syringes were then filled with the mixed resin and fixed onto the installed port for auto injection.
- The auto injector syringes were replenished with fresh resin mix as and when they were depleted.
- Injection for a crack line continued until all the ports no longer accepted any more resin.
- Injected resin was allowed to cure and the ports were then removed.

Figure 4 shows the crack injection process.

Retrofit with CFRP – the preparatory work

The preparatory works for retrofit process comprised of the following activities:

- Beams requiring the appropriate type of wrap detail were first identified and demarcated on site.
- The areas of wrapping (beam soffit and some portion of the side faces) were then ground off using diamond cup grinders to remove paint and make the surface free of protrusions.
- The ground surface was then air cleaned and localised punning with polymer modified cementitious skim coat was carried our to fill up small pin holes, undulations and surface defects to make the surface fairly smooth.
- After adequate water curing, the surfaces were then sanded using sand paper to smoothen out any fine protrusions thereby making the surface ready to receive CFRP application.

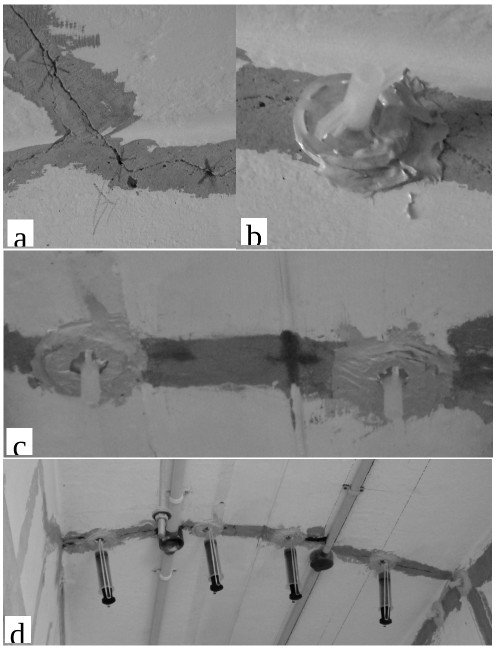

Figure 4: *a) marking port locations. b) & c) fixing surface ports with epoxy putty. d) low pressure, multi port epoxy resin injection.*

Retrofit - CFRP Wrap Installation

The installation of the CFRP was wet wrapping type process as recommended by the system manufacturer. The installation process comprised of the following activities;

- The carbon fibre fabric was sized and cut to the required dimensions that were obtained from site measurements.
- The two components of the saturant resin matrix were mixed together in the ratio as per manufacturer's specifications.
- Sized fabric was then manually saturated from both sides with the mixed resin matrix.
- Saturation was carried out manually using fabric rollers.
- The saturated fabric was then rolled onto spools and taken to site for installation.
- Prior to installation of the CFRP wraps, the prepared surface was first wet primed with a coat of saturant resin.
- The saturated fabric was then installed onto the primed surface by evenly rolling out the spool along the primed surface.
- The CFRP strips were adhered onto the surface using uniform hand pressure along the main fibres, thus ensuring removal of any entrapped air voids behind the CFRP wrap.
- Additional layers were installed on wet strips to build up to the required number of layers as specified.

Figure 5 shows the CFRP installation process.

4. CONCLUSION

The use of CFRP for structural retrofit of the distressed beams of the building was found to be an appropriate technique to achieve the desired objective and at the same time being minimal invasive, quick and discreet as compared to conventional strengthening techniques. The adoption of this technique led to saving of beam that would otherwise needed demolition.

Figure 5. *A) sizing carbon fibre fabric. B) instaling saturated fabric onto element.*

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REHABILITATION OF BLAST DAMAGED PRE-STRESSED CONCRETE BEAMS WITH FIBER REINFORCED POLYMERS (FRP)

C S Lewangamage[1], M T R Jayasinghe[2]

[1]Senior Lecturer, Department of Civil Engineering, University of Moratuwa, Sri Lanka,
[1]E-mail: sujeewal@civil.mrt.ac.lk

[2]Professor, Department of Civil Engineering, University of Moratuwa, Sri Lanka,
[2]E-mail: thishan@civil.mrt.ac.lk

Abstract: Damage to buildings due to blasts is a somewhat common occurrence in today's context. When such damages occur, one option is to demolish the structure and then to reconstruct. However, a better approach from sustainability point of view is to rehabilitate the damaged structure with suitable means so that a lot of materials and time can be saved while reducing the creation of heaps of construction waste. Since the damages due to blasts are not so common, it is worth reporting sustainability approaches employed in every such occurrence. This paper will present one such rehabilitation work that has been carried out for a complete floor consisting of pre-stressed concrete beams with pre-tensioned wires that suffered heavily due to extremely high loads caused by a blast that occurred directly above it when a small plane carrying explosives was shot down.

Key words: *Blast damage**,** Retrofitting, Fiber Reinforced Polymer (FRP), Reinforced concrete beams*

1. Introduction

When the buildings are damaged due to blasts, an option that can be appreciated more from the point of view of sustainability is repair and reuse. However, this is a challenging task due to the need for proper assessment of the effects of blast on short and long term performances. Therefore, the studies carried out using actual situation could be important as the laboratory based experiments. This paper presents one such rehabilitation work that has been carried out for a complete floor consisting of pre-stressed concrete beams with pre-tensioned wires that suffered heavily due to extremely high loads caused by a blast that occurred directly above it when a small plane carrying explosives was shot down.

The blast that is reported here was of very severe nature. It caused extensive cracking in T shaped pre-tensioned beams at both tension face (at the bottom) and also in the flanges that are supposed to be in compression [1]. At the outset, a simple solution for rehabilitation was reconstruction after demolishing the damaged beams. However, due to various other constraints such as time, congested site, difficult access, etc., an alternative was sought though it was a very challenging task for the structural engineer who was supposed to give a solution that can carry a guarantee with respect to durability and strength. The solution was first developed by approaching the solution with analysis based on fundamentals and basic properties of materials such as concrete and pre-stressing wires. The understanding on the behaviour of beams was then extended to cover the behaviour with Fiber Reinforced Plastic system that could be implemented economically. The theoretical model developed can be considered as a good approach that can be adopted not only to the beams damaged by blasts, but also to the structural members subjected to flexure that have suffered some damage due to corrosion of pre-stressing strands and reinforcement. This research paper describes the experimental study on FRP composites for flexural strengthening and also the practical applications of FRP.

2. Role of FRP in sustainable, built environment

Use of fiber reinforced polymer (FRP) composites for construction of new structures and rehabilitation of existing structures has increased significantly over past decades. FRP composites are lightweight, noncorrosive, exhibit high specific strength and specific stiffness, are easily constructed, and can be tailored to satisfy performance requirements [2]. For structural applications, FRP composites are typically fabricated using polymer matrix, such as epoxy, vinylester, or polyester, and reinforced with various grades of carbon, glass and/or aramid fibers. Due to its advantageous characteristics, FRP composites have been included in construction and rehabilitation of structures through its use as reinforcement in concrete, bride decks, modular structures, external reinforcement for strengthening and seismic upgrade. While mechanical advantages of using FRP composites are widely reported in literature, questions remain in regards to the feasibility of FRP composites within the framework of a sustainable environment[2].

The fabrication of constituent materials for FRP composites, namely matrix of fiber, could be areas of concern especially when considering that the primary resources from which polymers are produced; they could be crude oil, natural gas, chlorine and nitrogen [3]. The most commonly used fiber reinforcement in structural applications, can be identified as glass and carbon fibers. When considering only energy and materials resources it appears that the argument for FRP composites in a sustainable built environment is questionable. However, such a conclusion needs to be evaluated in terms of potential advantages present in the use of FRP composites related to consideration such as: high strength, light weight, high performance, longer durability, ability of rehabilitating existing structures and extending their life, seismic upgrades. They may also have applications in defense systems unique requirements, space systems and ocean environments.

It is important to note that the best way to minimize use of resources is to not rebuild in the first place. In this regard, the primary benefit of FRP composites will be its role in the solutions that seek to extend the service life of existing structures and to develop new structures that achieve superior service life with minimal maintenance. Essentially, it involves efficiently maximizing the benefit of potentially limited nonrenewable resources and avoiding the environmental, social and economic impacts associated with replacement and new construction. The benefits of FRP composites can be realized from its physical characteristics and their potential in developing structural systems with service life exceeding traditional materials [2]. The light weight of the composite can result in lower construction costs and increased speed of construction resulting in reduced environmental impacts. FRP composite material’s high strength and stiffness characteristics can require less material to achieve similar performance as traditional materials resulting in minimizing resources use and waste production. In general, the promise of FRP composites is its potential to extent the service life of existing structures and to develop new structures that are far more resistant to effects of aging, wreathing, and degradation in severe environments.

3. Performance of FRP composites with reinforced concrete beams subjected to flexure

An experimental study on behavior of FRP materials which has been used for the flexural strengthening of the beams were carried out. Three numbers of beams were casted and one beam was kept as the control specimen and other two beams were bonded with FRP composites and tested to find out the increment of flexural capacity. The deflection pattern and failure modes have also been checked. Beams were designed to avoid the shear failure with the increment of the loading after bonding with FRP.

3.1 Specimen details

2000 mm long × 150 mm wide × 200 mm deep beams were constructed. Cross section with the reinforcement details of the beams is given in Figure 1

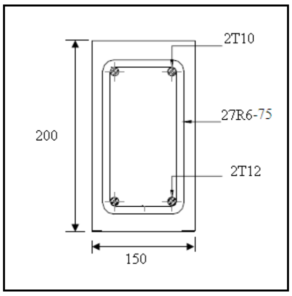

Figure 1: Cross section of the beam with R/F details

Grade of concrete used for beam casting was 30 and the properties of the FRP composites are given in Table 1.

Table 1: Properties of FRP composite

Thickness	**1mm/one layer**
Ultimate Tensile stress	834 N/mm^2
Rupture strain	85%
Modulus of Elasticity	82 kN/mm^2

Figures 2 and 3 show the cross section and the longitudinal section of the beam bonded with FRP, respectively.

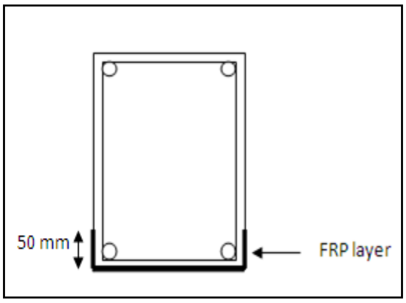

Figure 2: Cross section after bonded with FRP where FRP extends 50 mm on each side.

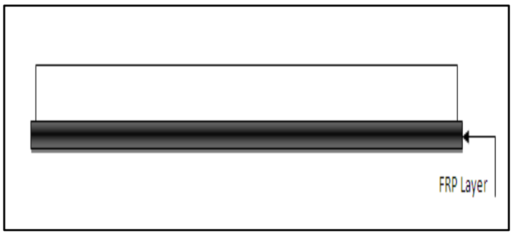

Figure 3: Longitudinal section after bonded with FRP

3.2 Test procedure

The beams were tested in four point bending, being simply supported on a pivot bearing on each end, over a span of 2.0 m. Identical bearing pads were placed at the loading points on top of the beams. A spreader I-beam resting on top of these provided a system for load distribution. Load was applied, by the increments of 5.0 kN throughout the tests. At each load increment, observations of crack development and the deflection of the mid span on the concrete beams were noted. Figure 4 shows the loading arrangement of the beams.

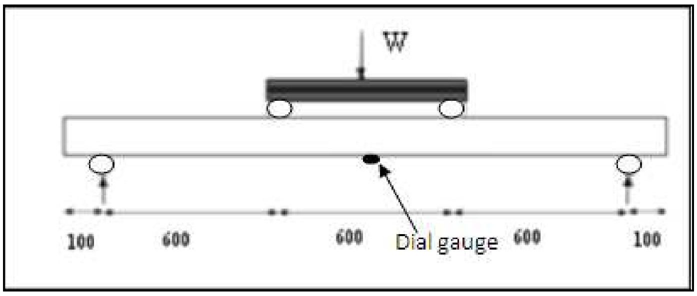

Figure 4: Loading arrangement

The bonding stresses of FRP-concrete interface are mainly shear and normal stresses. FRP on bottom of the beam that is subjected to flexural strengthening carries tensile stresses transferred through interface shear stresses and improves the bending capacity of the beam. The interface normal stresses also have influence on strengthened beam behavior [4]. At the end part of FRP where there is a truncation of FRP, stress concentration occurred and could initiate the FRP de-bonding [5].

Figure 5 shows the flexural cracks were propagated on the control specimen and Figure 6 shows the failure mode of the FRP bonded specimens and finally it can be concluded that the all the beams have failed due to flexure.

Figure 5: Flexural cracks of the control specimen

Figure 6: Concrete crushing and de-bonding of FRP bonded beam

3.3 Results

Deflection pattern of the FRP strengthened beams were almost the same and failure load was doubled compare to the control specimen. Figure 7 indicates the Load Vs Deflection of the mid span.

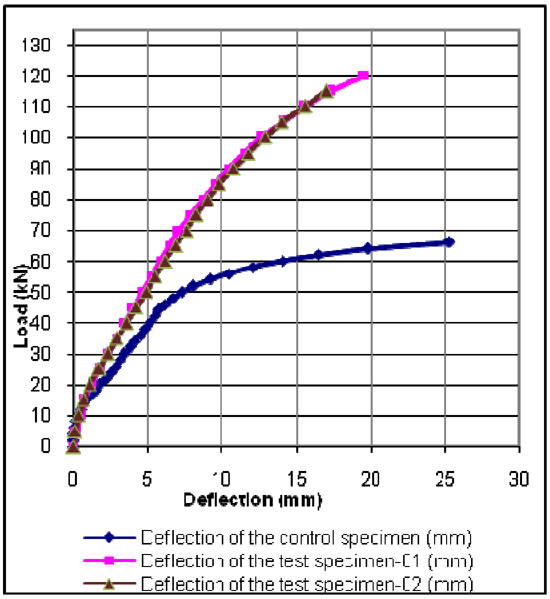

Figure 7: Load Vs deflection curve of the mid span of the beams

Failure load increment was nearly 84%. Summary of the failure loads and the failure modes are given in Table 4.

Table 4: Failure loads and failure modes of beams

Specimen	**Failure Load (kN)**	**Failure mode**
Control Specimen	66	Flexural failure
FRP bonded specimen-01	120	FRP de-bonding Flexural failure
FRP bonded specimen-02	123	FRP de-bonding Flexural failure

It is thus evident that FRP contributes to the beam's flexural capacity by restricting the opening of vertical cracks in the constant bending region. When the load and deformation are further increased, the developed interfacial shear stress at the concrete-FRP interface exceeds its capacity and then separation of the FRP plate could occur.

4. Rehabilitation of blast damaged pre-stressed concrete beams using FRP composites.

This work was part of the refurbishment and reconstruction of Inland Revenue head office building after the blast that occurred in the plane hit by ground fire. It was observed that about 27 number of precast pre-tensioned concrete T- beams (Figure 8) have hair line cracks close to their mid spans. The span of simply supported beams is 10. 6 m. These beams are located at roof terrace which is located at the 4th floor level. The main building has 12 stories.

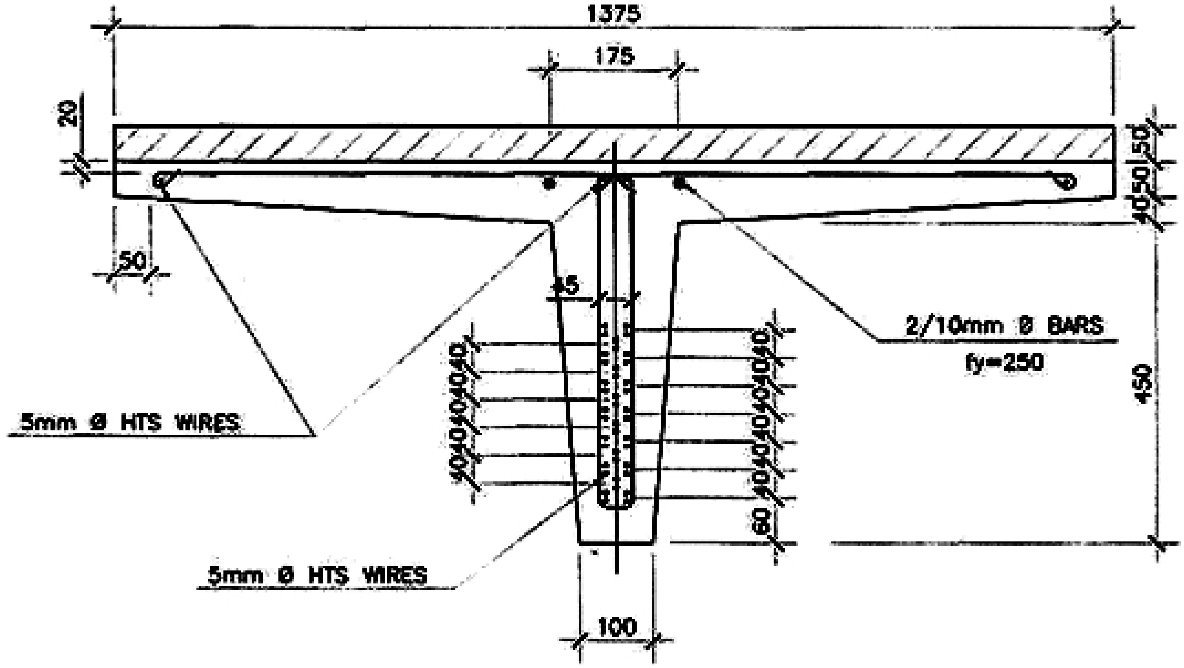

Figure 8: A section of the typical T-beam

During the blast that occurred, the building was hit by the many pieces of a small plane. The blast occurred just before the plane hit the building and hence could have occurred right above the roof slab. Some pieces and the engine of the plane ended up within the building. The location of blast was about 20 m above the roof terrace floor. This blast occurred at about 9.00 pm and a subsequent fire spared within the building. During the period of fire fighting and later, furniture, equipments and debris due to fire have been thrown away from the upper floors of the building on to this roof terrace. The day following the incident, it was observed that roof terrace floor was stacked with debris with over 2.0 m height. The debris consisted of equipments, computers, furniture, etc. which would have exerted significant loads. It was also reported that no such cracks have been observed in floor beams (floor beams in other location are also pre-stressed concrete T-beams) in other locations. Figures 9 and 10 show the details of cracked T-beams. The cracks appeared near the mid span of the beams are hair line cracks with maximum cracks width was about 0.1 mm.

Figure 9: Hair line cracks on T-beams close to mid spans

Figure 10: Cracks at T-beams flanges

The beams have suffered two types of loading that could be of significant nature. The blast that occurred over the 4th floor roof slab as could be seen with images captured by cameras with night vision would have applied a very significant impact load in the downward direction. The other load was due to throwing of various goods during fire fighting and cleaning operations that followed afterward by the security forces. It should be noted that for a heavy slab supported on beams, a gradual built up of debris could primarily over-load the slab and such overloading is most likely to cause few flexural cracks at the centre. However, there could also be few localized damages.

It should be noted that the crack width are as small as 0.1 mm with only few isolated ones reaching 0.2 mm. This is an indication that the steel wires would have suffered some overstress and would have been stressed beyond the 0.2% proof stress, thus inducing some permanent elongation. However, there is one property of steel that can still allow such over-stressed steel to be used further in structures for supporting loads of normal magnitudes. That is if a steel section does not break due to over stressing that induce work hardening, the steel can still be used if it is unstressed to a appreciably lower stress.

In this particular case, the steel tendons are partially unstressed and the actual stress would be as low as 750 – 800 N/mm^2 where as the characteristic tensile strength of original steel was much higher and in the range of 1560 N/mm^2.

5. Remedial measures

It was recommended to strengthening all the beams with cracks to enhance the flexural capacity using carbon fiber reinforcements so that some extra load could be independently carried by the carbon fiber based system. This position can be supported by the following facts. The systems of beams are pre-tensioned. Thus, the energy stored in the system allowed the beams to survive the blast though they could have suffered excessive deflections that led to high strain in steel wires and also some cracks at the very highly reinforced flanges. However, system recovered after the blast and indicated the possibility carrying self weight without noticeable deflections. A load test carried out has also indicated the possibility of carrying some loads without noticeable deflections. Here, bonded system was considered as a secondary system that would give a guarantee on load carrying capacity, both short and long term. Hence, only the additional loads that could induce further deflections were considered for determining the FRP system. The loading considered were the followings: (a).Allowance of 1.0 kN/m^2 is desirable to take account of finishes. (b). Live load on the slab can be considered as 5.0 kN/m^2 The corresponding design bending moments will be about 27 kNm for permanent loads (with a partial factor of safety for loading of 1.4) and 154 kNm for live loads (with a partial factor of safety for loading of 1.6) under simply supported conditions.

When carbon fibers are used, it is possible to determine the flexural capacity with triangular stress block even for ultimate loads. This will be a better approach since inducing high strains in carbon fiber is not prudent from the point of view of brittleness indicated by carbon fibers. Thus, it would be better to rely on a triangular stress block in concrete even at ultimate for the additional loads. This will results in slightly higher use of carbon fibers, but will ensure that there would be a very remote chance for exceeding the strain capacities of carbon fiber even when the beams are overloaded.

The corresponding resisting tensile force due to above loading is 322 kN. Hence required area of carbon fiber reinforcements is 386 mm^2 (corresponding to design tensile strength of carbon fiber and epoxy composite is 834 N/mm^2).

The stresses that will occur in this beam under service conditions have also been determined. The stresses under working condition will be; (a) at the top of the screed is 5.1 N/mm^2, (b) at the top fiber of precast section is 5.06 N/mm^2 and (c) at the bottom fiber of the precast section is -0.93 N/mm^2

It can be stated that the stresses in concrete are within the allowable limits for this 10. 6 m long beam. The allowable value for compressive stress will be 0.33 x f_{cu} = 0.33 x 40 = 13.2 N/mm^2. The allowable tensile stress for a class 2 structure is about 2.0 N/mm^2. Figures 11 and 12 show the repair work of cracks in beams and floor and the beams completed with FRP work.

Figure 11: Repairs to non-structural cracks

Figure 12: The beams completed with FRP work

6. Conclusion

The beams have suffered two types of loading, the blast loading and the loads due to throwing of various goods during fire fighting and cleaning operations. The gradual built up of debris could have over-loaded the slab which would have already suffered heavy overload due to the blast. Hence, it is most likely that the cracks near mid span of the beams would be of flexural nature, and are existing because steel tendons have suffered strain beyond 0.2% proof stress during the blast.

The blast damaged pre tensioned concrete T-beams were successfully strengthened using FRP composites with a carefully selected system that can ensure acceptable behaviour and a safe structure with respect to both short term and long term performance. This could be a solution that can be appreciated very much from sustainability point of view.

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RENOVATION AND RESTORATION OF OLD BUILDINGS IN SRI LANKA

Eng. Nandana Abeysuriya B.Sc. Eng (Hons), M.Eng.,C.Eng.,FIE(SL), Int PE(SL)

1A, Ananda Road, Melder Place, Nugegoda, Sri Lanka.
Tel/Fax: (+94) 011 280 9880 ; E-mail: ncdcon@sltnet.lk

Abstract

Sri Lanka has a great history about man- made structures. Most of the prestigious structures such as the palaces and buildings for Maha sanga were built with stone, clay bricks and timber. The dwellings of average people may have built using perishable material such as clay and timber leaving no traces.

Portuguese established in 1505 and settled in Colombo after their second visit in 1517. It was only after peace was established on the south western coast in 1594 and north western coast in 1618 that substantial Portuguese buildings worthy of the name of Architecture were built. They were military forts and churches of Catholic missionaries. The Dutch (1656- 1796) who took over in 1656 converted a lot of Portuguese buildings for their uses and in the later part of their period they built with their characters. The British occupied in 1796 but the true British styled buildings were erected after the first decade of 19th century. This will be discussed in detail under the history of buildings.

The nineteenth century was a great period for building construction. The industrial revolution influenced the building construction by introducing new materials and technology.

Most of the old buildings which were constructed in Sri Lanka after the 19th century, have built with new materials such as rolled steel and reinforced concrete.

Restoration process of ancient buildings should be approached after studying the design concepts, building characters, structural formations, materials used, durability of elements of the building …etc.

The structural drawings are not available in those buildings and renovations or restoration has to be done without un-stabilizing the supporting systems.

The experience gained by doing renovation and restoration works in different parts of Sri Lanka such as Hanthana (central hills), Galle fort, Kekanadura (, down south) and in Colombo, will be discussed in this article. The technology, materials used ,..etc were not similar in those buildings due to topography, availability of space and construction materials.

1. INTRODUCTION

Renovation is the act of improving by renewing and restoring to a previous or better condition. It is a sustainable process as sustainability could also be defined as living and working in ways that do not jeopardize our current and future social, environmental and economic resources.

The value of a building can be elevated to a great height by renovating in professional manner.

The characters of a properly built building could witness about the construction period, richness of the owner and his social status, economic situation in the country, level of foreign influence that the country faced during the construction period, construction standards and materials availability and many more. The comfort has been a prime factor considered in old houses and the environment friendly systems have been used without depending on electricity, air conditioners and other artificial ventilation systems. So it is very important to study these systems and propose renovations without damaging the unique characters of the building.

In restoration process, it is necessary to complete the building as it was. Some of the information may not be available due to damages in parts of building. In such cases, the knowledge about the history of buildings, the main features and construction techniques used in that particular era, will be very useful.

2. HISTORY OF BUILDINGS IN SRI LANKA

A typical Portuguese houses which were built in 17^{th} century, were two storied with a very steeply pitched roof. Thick masonry walls, small windows and timber flooring with low ceiling height were some common features.

The Dutch captured the most important Sri Lankan possessions of the Portuguese in the middle decade of 17^{th} century. They converted a large number of Portuguese buildings to Dutch use.

In the final period of Dutch Architecture in Sri Lanka from 1770 to 1796, the building became higher and the door and windows grew more elegant.

Fig 2.1 – An old building in Galle Fort planned for renovation

Fig 2.2 – The timber floor of an Old building in Galle Fort

The British occupied Sri Lanka in 1796 and their early building characters were influenced by French Engineers serving with the Dutch forces of whose skills the British apparently used. But by the first decade of 19th century, true British styled buildings were erected.

From the middle of 19th century new materials and techniques were introduced from the industrial countries of Europe and United States of America. Wood could now be mechanically sawn and shaped using imported circular saws. Machine made nails and bolts were available. Columns and hand rails were mass produced from imported wrought iron and cast iron. Roofs and walls have been built using the revolutionary, new light weight corrugated cast iron.

The industrial structural materials, rolled steel and reinforced concrete, began to be used at the end of 19th century and the beginning of the 20th century.

3. STRUCTURAL FORMATIONS OF OLD BUILDINGS

Most of the buildings which were built during Portuguese and Dutch periods, were either single story or two storied with thick masonry walls, timber floors and high pitch tile roofs. The masonry walls have been built with rubble or clay bricks with lime mortar. These walls have been built on rubble or brick strip foundations.

The reinforced concrete was introduced in 19th century and the concrete suspended floors were begun to use as an alternative to timber floors.

Fig 3.1 The timber floor of Dutch Burger Union, Colombo

Fig 3.2 Timber used for external corridor of Dutch Burger Union, Colombo

The concrete frame structures were not common during early days and the load bearing masonry walls have been used to support upper floor slabs. Most of the floor slabs which were constructed in Colombo during early part of 20th century , have been designed with 75 mm to 100 mm thick slabs reinforced with fabric mesh or mild steel round bars. The floor slabs have been supported by evenly placed steel beams at 2.0 m intervals, in the absence of walls and these steel beams are usually covered with plaster and decorated with mouldings.

The architecture and the technology used to build bungalows in coconut states in the costal line were different. The large single story houses with verandah, internal court yards, entrance car porch …etc were common features for these houses. The rubble with lime mortar has been used for foundations and even for thick walls.

The wall thickness at the ground level is about 375 mm to 400 mm in most of the buildings and it reduces as it goes up forming a tapering on one side or both the sides of the wall.

The bungalows which were built in up country hills, are again single storied buildings with large windows in the living area to capture more light. A fire place has become a unique feature for such buildings. The timber flooring has been used in cold climates.

The common feature in all these buildings is the use of natural light and ventilation to comfort the living. In construction of these real green buildings, more attention has been paid for the durability of buildings with proper eaves. The old clay tile roof has been provided gaps with its tapered shape to ventilate and to escape hot air. The Calicut roof tiles do not have this facility. Roof terraces and exposed walls have not been used for dwellings.

Fig 3.3 Rubble and clay mortar used to build walls in a bungalow at Hanthana, Kandy

4. APPROACHES FOR RENOVATIONS

The two major challenges that Structural Engineer faces are

1 The assessment of the durability of structural elements such as foundations, walls, roof timber, roof tiles …etc and preparing a proposal for improvement method without rejecting them as much as possible in order to make the project feasible and cost effective.

2 Making structural changes such as introducing new walls, changing door window positions, making openings on load bearing walls, adding floors, re arranging roof layouts etc with minimum damages to the structural formation of the building.

4.1 Assessment of buildings

The strength of walls at the time of renovation varies on the material used as mortar to bond brick or rubble used for the wall. The compressive strength of brick samples collected from internal and external walls can be tested from a testing laboratory. But in situ testing is required if the overall strength of wall is required. The roof timber samples can be collected from the site for testing. Having some idea in the depreciation of strength in different type of materials will also be very useful for the assessment. The load testing could also be used to ascertain the current load carrying capacity.

The load acting on these members can be transferred partially to a new concrete / timber or steel frame without effecting aesthetic appearance, if they are not strong enough to take the load. The members such as walls, roof timber …etc can be strengthened by introducing strong members to act finally as a composite element.

In most of the old buildings, only the plaster has deteriorated due to exposure to different climatic conditions. Therefore, it is necessary to replace the plaster in order to extend the life time of the buildings. The properly burned clay bricks found in ancient kingdoms are much stronger than the strength of present bricks.

Sometimes, the treatment for termites and wood bores will be useful to improve the lifetime.

4.2 Making structural changes

Making an openings on a load bearing wall or removing a wall, should be done while the weight above the level concerned transfers to the ground without making them unsupported.

The arch action in a masonry wall can be used to support openings to some extent. If the arch action cannot be maintained due to the span or the shape of opening, a concrete or steel frame can be introduced. A ring which takes the load above opening, transfer it to the existing foundation through the ground beam using two columns at either sides of the ground beam.

The technology is available to improve existing foundations by improving bearing capacity or by increasing the foundation width. Introduction of concrete columns at some intervals and tying with the existing rubble foundations could also improve the carrying capacity.

5. PROJECT PHOTOGRAPHS

Fig : 5.1 - 100 years old bungalow at Hanthana, Kandy

Fig 5.2 – 100 years old bungalow at Hanthana, Kandy

Fig 5.3 – Bungalow at Kakenadura, Matara before renovation

Fig 5.4 – Bungalow at Kakenadure, Matara after renovation

Fig 5.5 – Bungalow at Kakenadure, Matara after renovation

Fig 5.6 – Building at No. 65, Rosmead Place after renovations

6. CONCLUSION

Renovation or restoration of buildings is an environment friendly process which saves our limited resources. The building debris which cannot be re used will also be a problem in future. So restoration helps to protect the environment by not dumping imperishable materials. But the most important lesson what we should learn from the ancient buildings is the environment friendly designs and details used by our ancestors.

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DEVELOPMENT OF LOCALLY REPRESENTATIVE SEISMIC HAZARD MODELS

N. T. K. Lam[1], H. H. Tsang[2] and J.L. Wilson[3]

[1]Reader, Department of Civil & Environmental Engineering, University of Melbourne, Victoria, Australia.
[1]E-mail: ntkl@unimelb.edu.au
[1]Telephone: +61-3-83447554

[2]Research Fellow, Department of Civil Engineering, University of Hong Kong, China Hong Kong.
[2]E-mail: tsanghh@hku.hk

[3]Deputy Dean and Professor of Civil Engineering, Faculty of Engineering and Industrial Sciences, Swinburne University of Technology, Victoria, Australia.
[3]E-mail: JWilson@groupwise.swin.edu.au

Abstract:. Seismic hazard studies in regions of low and moderate seismic activities often resort to the use of attenuation relationships developed elsewhere but there are always doubts as to whether these imported relationships are representative of local conditions. Numerous well established stochastic attenuation models have also been developed for applications in Central and Eastern North America. However, such intraplate models could not be generalised across the globe to other regions of low and moderate seismic activities. Modelling the spatial and temporal distribution of seismic activities can also be thwarted with difficulties because of the paucity of data. This paper presents the experiences of the authors in overcoming these challenges when undertaking seismic hazard studies in different countries. Topics covered in the paper include attenuation modelling and the evaluation of seismic hazards for the determination of the earthquake loading model for engineering design.

Keywords: Moderate seismicity, seismic hazard, attenuation

1. Introduction

Seismic-hazard assessment (SHA) provides an estimate of ground motion at the site of interest, taking into account instrumental and historical earthquake records, information on tectonics, geology and attenuation characteristics of seismic waves. Seismic-hazard assessment is also used for seismic microzonation study, which is important for decision making on land use, evaluation of the level of earthquake preparedness, economical consideration of earthquake-resistant design, retrofit strategy, economic loss estimation in an event of future earthquake, and also for the design of ordinary structures where site-specific studies are not warranted.

Techniques for modelling the spatial and temporal distribution of seismicity for areas of low and moderate activities are described in section 2. The development of ground motion prediction equations (GMPEs), also known as attenuation relationships, is reviewed in section 3. This is a key component in the probabilistic seismic hazard assessment (PSHA) procedure. In regions of high seismicity such as California where strong motion records are abundant, GMPEs are usually developed by regression of recorded ground motion parameters (e.g. Sadigh et al. 1997). In regions lacking recorded strong motion data, GMPEs may alternatively be developed from stochastic simulations of the seismological model, which characterises ground motion properties by their frequency content. The integration of the seismic activity model with the relevant selected GMPEs provides predictions of the recurrence relationship of ground motion parameters and the associated seismic hazard maps. An innovative integration procedure which is known as the *Direct Amplitude - Based* (DAB) Method (Tsang and Chandler 2006; Tsang and Lam 2010) which can be used to circumvent difficulties encountered by the conventional Cornell's iteration method (Cornell 1968) is illustrated in section 4 by case studies.

2. Seismic Activity Modelling

2.1 Historical developments

The level of earthquake activity at an identified seismic source location is normally modelled by the Gutenberg-Richter (G-R) magnitude recurrence relationship. The earthquake activity model of a region comprises the spatial distribution of the potential seismic sources along with the recurrence relationship of each individual seismic source expressed in the G-R form. It is important that the records are complete. Historical records can be very useful supplement to instrumental records, particularly in areas lacking instrumental records and/or areas with a long history of civilised settlement. Meanwhile, geological data on fault activity often provides useful support to the seismological data in identifying and delineating potential seismic sources (Yeats et al. 1997).

2.2 Modelling in areas of low and moderate seismic activity

In areas of low and moderate seismicity, the low level of earthquake activity rate often results in a lack of seismic activity data. Consequently, it is difficult to identify and delineate potential seismic sources and to define the activity level of each individual seismic source. For this reason, regional fault sources have often been grouped into "areal source zones" in which seismic activity is considered to be diffused over the entire zone and not concentrated at a few faults. An example is the work of Pun and Ambraseys (1992) is relation to the South China region. This lumping of fault sources into a diffused areal source has the advantage of aggregating a larger number of seismic records to define the recurrence behaviour of the source. Further, the diffused model overcomes the problems of not accounting for unidentified fault sources such as a buried blind thrust fault.

Seismicity in such regions has therefore been modelled using very large areal source zones defined, rather arbitrarily, in accordance with broad geographical features (such as coastlines and mountain ranges) together with the mapping of significant historical earthquake events (Scott et al. 1994 for Hong Kong; Jacob 1997 for New York City). These are also known as "Broad Source Zones" (BSZ) (Chandler and Lam 2002). In modelling a BSZ, the actual spatial distribution of seismicity within the zone is ignored and assumed to be uniform. The advantage is that only a single G-R relationship needs to be defined using the entire seismic record. This type of areal source is also known as "seismo-tectonic province". The theoretical basis of the BSZ or seismo-tectonic province is controversial, and hence it is important to be aware of the potential implications when adopting a suitable strategy for a given region.

2.3 Large magnitude distant earthquakes

Large magnitude earthquakes can transmit destructive long period ground shaking over very long distances, particularly in a crustal environment of good wave transmission quality such as the mid-continental regions of CENA (Algermissen 1997) or Central and Western Australia. Thus, the overall chance of a facility, or a centre of population, being affected by a large magnitude earthquake is contributed to by a large number of potential seismic sources (including unknown sources) covering a very large area (within a radius of 400-500 km from the site). Consequently, large magnitude earthquakes warrant serious engineering consideration, despite the fact that they are generated very "infrequently" by any particular seismic source. However, making assessment of earthquake performance due to such potential hazards is generally very difficult in intraplate regions, since earthquakes of sufficiently large magnitude occur so infrequently that their activities are often difficult to study simply by mapping historical events or by instrumental monitoring. Studies based on geomorphology, imaging of sub-surface geological structures and carbon dating of soil samples taken from fault scarps (Yeats 1997) have been carried out to study the recurrence interval of pre-historical earthquake activities. Overall, large magnitude long distance earthquakes form a significant hazard and hence are an important issue in the seismic hazard modelling for low and moderate seismicity regions within continents (Lam et al. 2002).

3. Ground Motion Prediction Modelling

The prediction of earthquake ground motions in accordance with recorded observations from past events is the core business of engineering seismology. A GMPE presents values of parameters characterising the intensities and properties of ground motions estimated of projected earthquake scenarios (which are expressed in terms of magnitude and distance). Empirical attenuation models are developed from regression analysis of recorded strong motion accelerograms. In situations where strong motion data are scarce the database of records has to cover a very large area which may be an entire continent or a large part of a continent in order that the size of the database has statistical significance (Toro et al. 1997 for Central and Eastern North America; Ambrasey 1995 for Europe). Thus, attenuation modelling based on regression analysis of instrumental data is problematic when applied to regions of low and moderate seismicity. This is because of insufficient representative data that has been collected and made available for model development purposes.

An alternative approach to attenuation modelling is the use of theoretical models. Unlike an empirical model, a theoretical model only makes use of recorded data to help ascertain values of parameters in the model rather than to determine trends from scratch by regression of data. Thus, much less ground motion data is required for the modelling. Data that is available could be used to verify the accuracies of estimates made by the theoretical model. The heuristic source model of Brune (1970) which defines the frequency content of seismic waves radiated from a point source is developed for such purpose. The model has only three parameters: seismic moment, distance and the stress parameter. Combining this point source model with a number of filter functions which represent modification effects of the wave travel path and the site provides estimates for the *Fourier* amplitude spectrum of the motion generated by the earthquake on the ground surface. The source model (of Brune) in combination with the various filter functions are collectively known as the seismological model (Boore 1983). Subsequent research by Atkinson and others provides support for the proposition that simulations from a well calibrated point source model are reasonably consistent with those from the more realistic finite fault models.

The *Fourier* spectrum as defined by the seismological model only provides description of the frequency properties of the ground motions and not the phase angles of the individual frequency components of the waveforms. Thus, details of the wave arrival times which are required for providing a complete description of the ground shaking remain uncertain as they have not been defined by the seismological model. With stochastic modelling, the pre-defined frequency content is combined with random phase angles that are generated by the *Monte Carlo* process. Thus, acceleration time-histories based on randomised wave arrival details are simulated. The simulations can be repeated many times (for the same earthquake scenario and source-path-site conditions) in order that response spectra calculated from every simulated time-histories can be averaged to obtain a smooth, ensemble averaged, response spectrum.

The seismological model has undergone continuous development since its inception. For example, the original Brune source model has been replaced by the empirical source model of Atkinson (1993) which was developed from seismogram data recorded in *Central and Eastern North America* to represent conditions of intraplate earthquakes. A similar model was subsequently developed by Atkinson and Silva (2000) which was developed from data recorded in *Western North America* to represent conditions of interplate earthquakes. A model to account for the complex spread of energy in space taking into account the wave-guide phenomenon and the dissipation of energy along the wave travel path has also been developed (Atkinson and Boore 1995). The amplification and attenuation of upward propagating waves taking into account the effects of the shear wave velocity gradient of the earth crust have also been modelled by Boore and Joyner (1997). A comprehensive review can be found in Lam et al. (2000).

4. Seismic Hazard Modelling and Case Studies

The effects of all potential earthquake events expected to occur within a specific exposed period are integrated in PSHA with due considerations given to uncertainties and randomness. PSHA is thus able to provide an estimate of ground motion parameters with an annual probability of exceedance (or any other time period), which is a key input for risk analysis and performance-based design.

As introduced earlier, PSHA has been de-coupled into seismic source modelling, ground motion modelling and the integration as the final step in the procedure. One of the well recognized shortcomings of this well established procedure is with difficulties over constraining the source model based on historical data. Consequently, predictions are highly model dependent given that the manner in which certain modelling parameters are decided upon can be fairly subjective. Thus, the procedure is still filled with ambiguities although being widely used. The DAB Method has been developed as a simple and efficient method as an alternative for PSHA.

In this section, the implementation of the DAB approach for three cities in China, Iran, and India, respectively, has been briefly described. Meanwhile, several insights regarding the procedure of conducting PSHA have also been obtained, which could be useful for future seismic hazard studies.

4.1 Direct amplitude-based (DAB) approach

The DAB approach was derived analytically from Cornell's source-based method, yet does not require detailed characterization of seismic sources. Whilst the method possesses the simplicity of the historic method, it could be extended to account for characteristic earthquakes and potential large events that have not been observed historically, in order to improve the reliability of hazard calculation at low probability. Hence, it can also be regarded as a "parametric-historic" method. On the other hand, any site-specific and event-specific characteristics that influence ground motions, such as non-linear site effects, rupture mechanism and directivity, can be incorporated in the early stage of the numerical procedure, which is considered beneficial for microzonation study. The DAB approach can be analytically represented by Equation (1) and details of the derivation process can be found in Tsang and Chandler (2006) and Tsang and Lam (2010).

$$P[Z > z] = N(\Delta_{\min})\int_{\Delta_{\min}}^{\Delta_{\max}} P[Z > z \mid \Delta] f(\Delta) d\Delta$$

(1)

where □ is the median ground motion or spectral response amplitude, obtained from GMPEs for each earthquake scenario (discarding the standard deviation); *f*(□) is the probability density function (*PDF*) of the median amplitude (□), which can be obtained by differentiating the cumulative distribution function (*CDF*), derived from the amplitude-recurrence relationship. □$_{min}$ and □$_{max}$ are minimum and maximum median ground motion or spectral response amplitudes, respectively, and *N*(□$_{min}$) is the mean annual rate of the median amplitude (□) exceeding the minimum value (□$_{min}$).

4.2 Case studies

Implementation of the DAB method is illustrated herein using the case studies of Hong Kong, China; Tehran, Iran; and Bangalore, India. The seismic hazard curves computed by the new method are plotted in Figure 1. Comparison with previous results computed by Cornell's source-based approach has been presented in Tsang et al. (2010). Some insights regarding the procedure of conducting PSHA have also been summarised in the followings.

The case study for Tehran revealed that the assumption of uniform seismicity when characterizing seismic sources in the source-based approach might result in an overestimation of the seismic hazard. Whilst the concept of uniform seismicity as introduced in section 2 seems to be rational, there are difficulties with accurately ascertaining the level of seismic activity for an area in which the disposition of activity is assumed to be uniform. In the case of Tehran, large magnitude near field earthquake scenarios that have been predicted for certain areas are clearly unrealistic as the predictions are contrary to geological evidences. For instance, M7.5 – M8 earthquakes at very short epicentral distances (R < 5 km) have been predicted in locations where no major faults have been identified. Such anomalies were purely resulted from the very assumption of uniform seismicity in a somewhat arbitrarily defined source zone. A joint *PDF* incorporating both magnitudes and distances, $f(M, R)$, should be used in the calculation of seismic hazard.

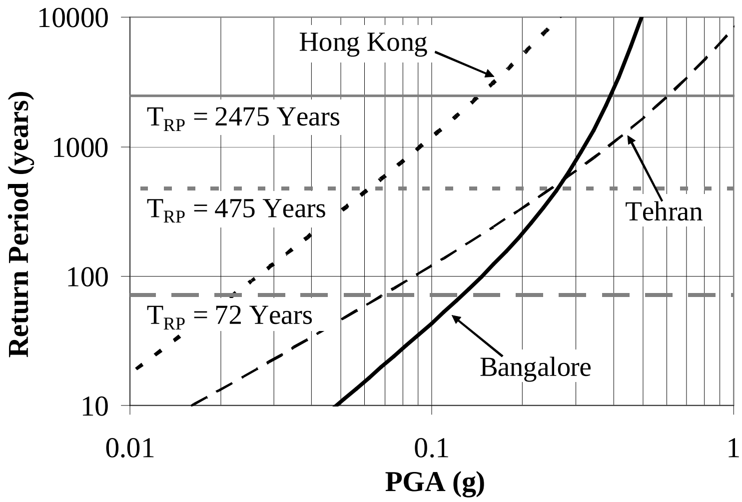

Figure 1: Seismic-hazard curves showing the return period against PGA for Hong Kong, Tehran, and Bangalore

In the Bangalore case study, there is an apparent discrepancy between the hazard values calculated using the source-based method and the DAB method. A careful review of the earthquake catalogue revealed that there were abnormally few data in the period 1901–1966 (refer Table 1, Figures 3, 4 and 6 in Anbazhagan et al. 2009), which is unusual for such a large region in Southern India. Hence, it is likely that the catalogue is incomplete in this period of time. Also, in the period 1997–2006, instrumental records for small magnitude earthquakes are lacking. Such incompleteness of catalogue would result in an underestimation of the rate of seismic activity, if appropriate treatment has not been applied in conducting PSHA. It is noted that in this case study when the DAB method was employed, all events in the above-mentioned (two) periods have been removed and have not been included in the calculation of seismic hazard. The completeness criteria adopted in this study are as follows: $M > 5$ for periods 1800–1900 plus 1967–2006 (a total of 140 years) and $5 > M > 3$ for period 1967–1996 (30 years) (supported by Anbazhagan et al. 2009).

PGA-recurrence relationships as obtained from the DAB method for Bangalore and Tehran revealed some interesting findings. A much higher recurrence rate has been identified with Bangalore at low levels of ground shaking. In other words, the number of earthquake events surrounding Tehran, of PGA in the range 0.01g and 0.1g, is much smaller than that surrounding Bangalore. This is in spite of Tehran apparently possessing a much higher level of seismic hazard as reflected in its level of design PGA. The revealed anomalies might be related to the very different area of coverage by historical events that have been incorporated into the earthquake catalogue of the respective city. The largest source-site distance of earthquake events in the Tehran catalogue was only 200 km, whilst that of Bangalore and Hong Kong were respectively 350 km and 500 km. The area of coverage for Tehran and Bangalore differed by a factor of 3. Such disparity offers a plausible explanation for the much lower recurrence rate of low-to-moderate levels of ground shaking generated by distant earthquakes (with source-site distance greater than 200 km).

Another note to make is that reliable modelling of the ground motion (or spectral response) parameters by the ground motion prediction equations (GMPEs) is essential for a credible outcome of the PSHA. The standard deviation $\sigma_{\log(PGA)}$ of the GMPE would also significantly influence estimates of the seismic hazard level, especially at long return periods. With Bangalore, the very low rate of increase in the seismic hazard level with increasing return period (as shown in Figure 1), compared to those of Hong Kong and Tehran, could be explained by the much lower standard deviation with the GMPEs developed for the city.

5. Conclusions

This paper reveals various problems associated with constraining factors that would influence the prediction of seismic hazard for regions lacking representative historical seismic data. Innovative improvements to the current modelling methodologies have been illustrated using case studies undertaken in different countries by the authors.

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About the Authors

N. T. K. LAM, B.Sc. Hons. Leeds, M.Sc. D.I.C. London, Ph.D. Melbourne., C.Eng. M.I.StructE M.I.C.E. M.I.E.Aust. is Reader at the Department of Civil & Environmental Engineering, University of Melbourne. His research interests are in the areas of earthquake engineering, structural dynamics, impact dynamics and sustainable developments.

H. H. Tsang, B.Sc. Hons. Hong Kong., Ph.D Hong Kong is currently research fellow at the Department of Civil Engineering, University of Hong Kong. His research interests are in the areas of earthquake engineering, structural dynamics and use of recycled products in sustainable developments.

J. L. Wilson, B.E. Hons. Monash, M.Sc. Berkeley, Ph.D. Melbourne., F.I.E.Aust. is Deputy Dean and Professor of Civil Engineering at the Faculty of Engineering and Industrial Sciences, Swinburne University of Technology. His research interests are in the areas of earthquake engineering, structural engineering, reinforced concrete and sustainable developments.

EVALUATION OF EFFECTS OF RESPONSE SPECTRUM ANALYSIS ON HEIGHT OF BUILDING

[1]Prof. Dr. Qaiser uz Zaman Khan

[1]Department of Civil Engineering, University of Engineering and Technology, Taxila, Pakistan
e-mail: dr.qaiser@uettaxila.edu.pk

Abstract
The main aim of the proposed research is to provide guidelines for critical structure height above which Response Spectrum Analysis (RSA) significantly affects the structure design. Response spectrum analysis of 20 story building has been discussed in detail and comparison of static and dynamic analysis and design results of buildings up to 400 ft height (40story) in terms of percentage decrease in bending moments and shear force of beams, bending moments of columns, top story deflection and support reaction are presented in this paper. Percentage decrease in reinforcement area requirement for different members has also been discussed.

1. Introduction

After the devastating October 2005 earthquake in Pakistan, the building regulatory authorities reconsidered the existing structure design requirements for the area surrounding the Islamabad, Peshawar and Azad Jammu & Kashmir. Earthquake zoning of the area was changed and Islamabad, Peshawar and AJK were put under seismic zone III. Also dynamic analysis was made an essential part of structure design for the buildings to be constructed in seismic zone III. Two types of dynamic analysis can be used to make the structures sound against seismic activity. One is Time History dynamic analysis and the other is Response spectrum analysis. For Time History analysis, we do need a bunch of site specific data (period, amplitude) to define the required time history functions. On the other hand, to perform Response spectrum analysis, idealized response spectrum curves provided by UBC can be selected to use by providing seismic coefficients Ca and Cv of the respective seismic zone.

The requirement therefore emerges for a study that can provide guidelines for critical structure height above which response spectrum analysis significantly affects the structure design results and financial aspects of building construction so as to make the structure design process more efficient and to avoid extra effort on structures below that critical height.

First, a regular square building with five stories (50 feet) height was assumed which was supposed to be constructed in Islamabad. A typical Moment Resisting Framing system was used in the research with due consideration to local requirements. The above said structure was analyzed by using Static Lateral load method as well as RSA and structural members were designed against the most critical load combinations for both methods of analysis. Further, building height was increased by 5 stories (50ft) in each steps up to 40 stories (400ft) and were analyzed using both static and dynamic analysis methods.

2. Requirement of Dynamic analysis

Behavior of structure during an earthquake is basically a vibration problem. The seismic movement of the ground causes the structure to vibrate and causes structural deformity in the building. Different parameters regarding this deformity like frequency of vibration, time period and amplitude are of significant importance and defines the overall response of the structure. This overall response also depends on the distribution of seismic forces within the structure which again depends on the method which is used to calculate this distribution.

The lateral force requirements of UBC-97 suggest several methods that can be used to determine of the distribution of seismic forces within a structure.

Different methods of 3-Dimensional dynamic analysis of structures have become more efficient in use along with the development of technology. Response spectrum analysis method for seismic analysis is one of them which also can give more accurate results than an equivalent static approach. [1]

The major advantage of using the forces obtained from a dynamic analysis as the basis for a structural design is that the vertical distribution of forces may be significantly different from the forces obtained from an equivalent static load analysis. Consequently, the use of dynamic analysis will produce structural designs that are more earthquake resistant than structures designed using static loads.

3. Response spectrum analysis (RSA)

3.1. Response Spectrum

A response spectrum is simply a plot of the peak or steady-state response (displacement, velocity or acceleration) of a series of oscillators of varying natural frequency, that are forced into motion by the same base vibration or shock. The resulting plot can then be used to pick off the response of any linear system, given its natural frequency of oscillation. [2]

Response spectra are very useful tools of earthquake engineering for analyzing the performance of structures and equipment in earthquakes, since many behave principally as simple oscillators (also known as single degree of freedom systems). Thus, if you can find out the natural frequency of the structure, then the peak response of the building can be estimated by reading the value from the ground response spectrum for the appropriate frequency. In most building codes in seismic regions, this value forms the basis for calculating the forces that a structure must be designed to resist (seismic analysis). [2]

3.2. Description of (RSA) Procedure

RSA is an elastic dynamic analysis of a structure utilizing the peak dynamic response of all modes having a significant contribution to total structural response. Peak modal responses are calculated using the ordinates of the appropriate response spectrum curve which correspond to the modal periods. Maximum modal contributions are combined in a statistical manner to obtain an approximate total structural response. [3]

Important parameters required for performing RSA are as under

1. Ground Motion and representation of Response Spectrum
2. Modal Analysis
3. Method for combining Modal Maximum Responses
4. Scaling of Elastic Response Parameters
5. Directional Effects

3.3. Ground Motion and Representation of Response Spectrum

An elastic design response spectrum constructed in accordance with Figure 1.3, using the values of Ca and Cv consistent with the specific site. The design acceleration ordinates shall be multiplied by the acceleration of gravity, 386.4 in./sec^2 (9.815 m/sec^2). [3]

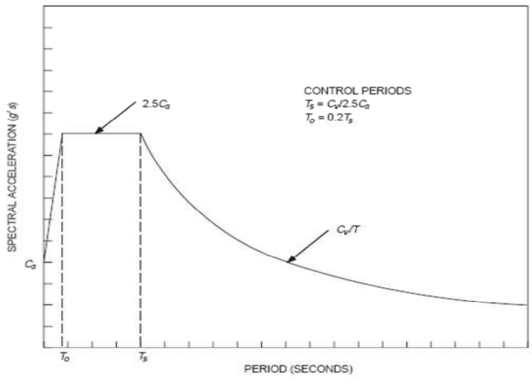

Fig.1: *Typical Response Spectrum Curve*

Most of the structural analysis software packages provide facility to get the response spectrum curve by entering values of Ca and Cv.

3.4. Modal Analysis

Modal analysis is the study of the dynamic properties of structures under vibration excitation. The goal of modal analysis in structural mechanics is to determine the natural mode shapes and frequencies of an object or structure during free vibration. Response Spectrum analysis requires to include the response of all significant modes to calculate the structure response. To satisfy this requirement number of modes considered should be such that, at least 90 percent of the participating mass of the structure is included in the calculation of response for each principal horizontal direction. [3] When vertical dynamic response of structural elements are required to be calculated e.g. vertical vibration of beams and floor systems and exact frequencies and mode shapes are used in the analysis , Ritz vector approach is recommended because for eigenvector analysis hundreds of modes will be required to capture 90% mass participation as is required by UBC-97[4].

3.5. The CQC method of modal combination

The most conservative method that is used to estimate a peak value of displacement or force within a structure is to use the sum of the absolute of the modal response values. Another very common approach is to use the Square Root of the Sum of the Squares, SRSS, on the maximum modal values in order to estimate the values of displacement or forces. [5]

The relatively new method of modal combination is the Complete Quadratic Combination, CQC, method [6] that was first published in 1981. It is based on random vibration theories and has found wide acceptance by most engineers and has been incorporated as an option in most modern computer programs for seismic analysis. Most of the structure analysis software provides all CQC, SRSS, ABS and GMC modal combinations methods as options. The use of CQC method is highly recommended to use for modal combination.

3.6. Scaling of Elastic Response Parameters

UBC-97 provides guidelines for scaling of response spectrum parameters in its clause 1631.5.4 which is as follows.

Elastic Response Parameters may be reduced for purposes of design in accordance with the following items, with the limitation that in no case shall the Elastic Response Parameters be reduced such that the corresponding design base shear is less than the Elastic Response Base Shear divided by the value of *R.*

1. For all regular structures where the ground motion representation complies with Section 1631.2 (Ground Motion Representation), Item 1, Elastic Response Parameters may be reduced such that the corresponding design base shear is not less than 90 percent of the base shear determined in accordance with Section 1630.2 (Static Force Procedure).

2. For all regular structures where the ground motion representation complies with Section 1631.2, Item 2, Elastic Response Parameters may be reduced such that the corresponding design base shear is not less than 80 percent of the base shear determined in accordance with Section 1630.2.

3. For all irregular structures, regardless of the ground motion representation, Elastic Response Parameters may be reduced such that the corresponding design base shear is not less than 100 percent of the base shear determined in accordance with Section 1630.2. [3]

The structural model that we have used for this research work falls in category 3. Detailed procedure for scaling of response spectrum parameters have been described in the next chapter.

3.7. Directional Effects

A weakness in the current code is the lack of definition of the “principal horizontal directions” for a general three dimensional structure. If an engineer is allowed to select an arbitrary reference system, the “dynamic base shear” will not be unique and each reference system could result in a different design. One solution to this problem that will result in a unique design base shear is to use the direction of the base shear associated with the fundamental mode of vibration as the definition of the “major principal direction” for the structure. The “minor principal direction” will be, by definition, ninety degrees from the major axis. This approach has some rational basis since it is valid for regular structures.[7] The required design seismic forces may come from any horizontal direction and, for the purpose of design, they may be assumed to act non-concurrently in the direction of each principal axis of the structure.

For the purpose of member design, the effects of seismic loading in two orthogonal directions may be combined on a square-root-of-the-sum-of-the-squares (SRSS) basis.[7]

4. Dynamic analysis of a 20 Story Building

As many structures with different stories heights are analysed. Therefore, in order to avoid the description of similarity of the work, in the following section, a comparative study of analysis and design results of dynamic and static analysis of only a 20 story building is presented.

4.1. Description of Building

The buildings considered for this research work are Commercial cum Residential buildings.
The first four floors will be considered as commercial with showrooms and display centers on two lower floors and studio apartments (Official Use) on upper two floors and live load will be assigned accordingly. All floors above first four, which will be added at different stages of this research will comprise of two bedroom residential apartments.

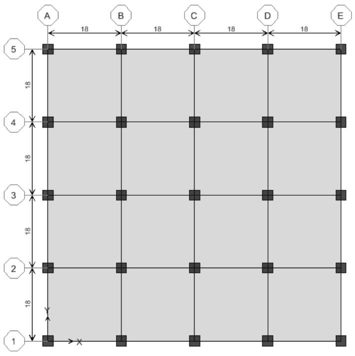

Fig.2: *Floor Plan of 20 Story Building*

4.2. Earthquake load

For static analysis seismic forces on the building will be determined in accordance with chapter 16 Div IV of UBC-97, Design provisions for earthquake resistance of structures.

Seismic zone:	Zone 3
Seismic zone factor:	Z = 0.3
Soil Profile Type	SD (Stiff Soil Profile)
Seismic Importance factor	Essential facility (I = 1.00)
Response Modification Factor	8.5 (For concrete MRF system)
Seismic Coefficient	Ca = 0.36, Cv = 0.54

4.3. Response Spectrum Function

In ETABS Spectrum function is selected as per UBC-97 against Ca=0.36 and Cv=0.54. The spectral values are as under.

Table 1: *Live Loads for the Structure*

Period	**Acceleration**	**Period**	**Acceleration**	**Period**	**Acceleration**
0.0	0.36	2.0	0.27	6.5	0.0831
0.12	0.9	2.5	0.216	7.0	0.0771
0.6	0.9	3.0	0.18	7.5	0.072
0.8	0.675	3.5	0.1543	8.0	0.0675
1.0	0.54	4.0	0.135	8.5	0.0635
1.2	0.54	4.5	0.12	9.0	0.06
1.4	0.3857	5.0	0.108	9.5	0.0568
1.6	0.3375	5.5	0.0982	10.0	0.054
1.8	0.3	6.0	0.09	-----	-----

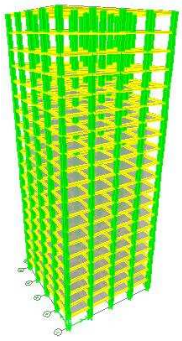

Fig.3: 3-*Dimensional extruded view of 20-Story Building*

5. Sample Beam Result Comparison

A typical bending moment diagram from level 10 is presented hereunder. In dynamic analysis a decrease of 31.3 % in negative bending moments and 46% decrease in positive bending moment, in comparison with static analysis, was observed.

In dynamic analysis a decrease of 35.6% in negative longitudinal reinforcement area was observed while the decrease in positive longitudinal reinforcement area was 50%

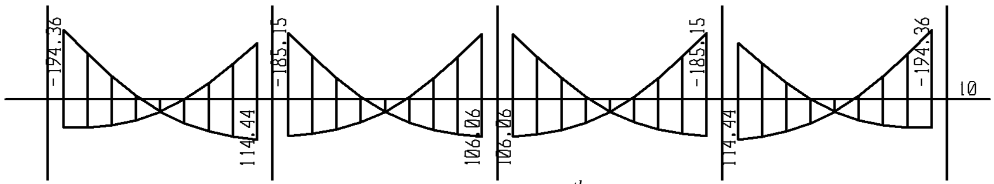

Fig. 4: *Bending Moment diagram of Beams at 10th story-Static analysis*

Fig. 5: *Longitudinal Reinforcement Detail of Beams at 10th story---Static analysis*

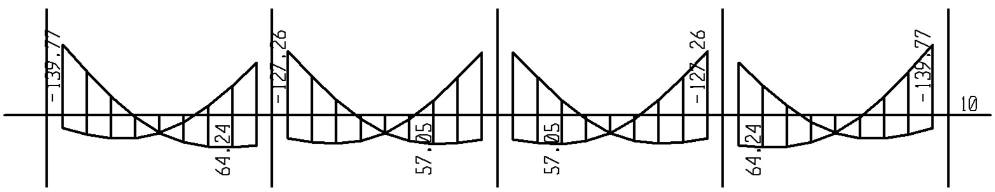

Fig. 6: *Bending Moment diagram of Beams at 10th story---Dynamic analysis*

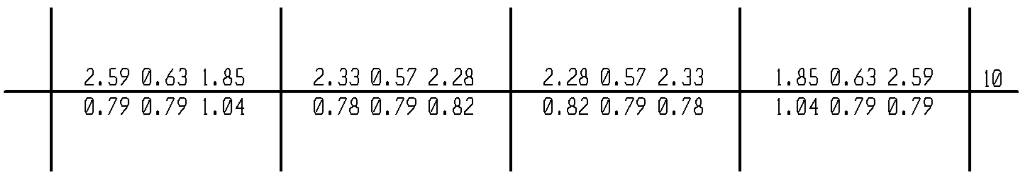

Fig.7: *Longitudinal Reinforcement Detail of Beams at 10th story---Dynamic analysis*

6. Summary of Result Comparison

Height of Building	Avg. Change in Beam Forces		Avg. Change in Column Forces	Avg. Change in Max. Deflection	Avg. Change in Support Reactions
	Bending Moment	Shear Forces			
60	2.2	1.25	1.46	6	1.78
100	8.53	4.24	1.64	17	1.82
150	16.15	10.35	3.66	27	3.66
200	31.3	21.7	18.9	44	18.90
250	33.2	24.8	16.04	45	16.00
300	37.8	29.456	20.8	48	20.94
350	35.6	29.2	15.8	45	15.84
400	42.1	36	26.5	50	26.60

Table 2: *Summary of comparison of member forces resulting from static and dynamic analysis*

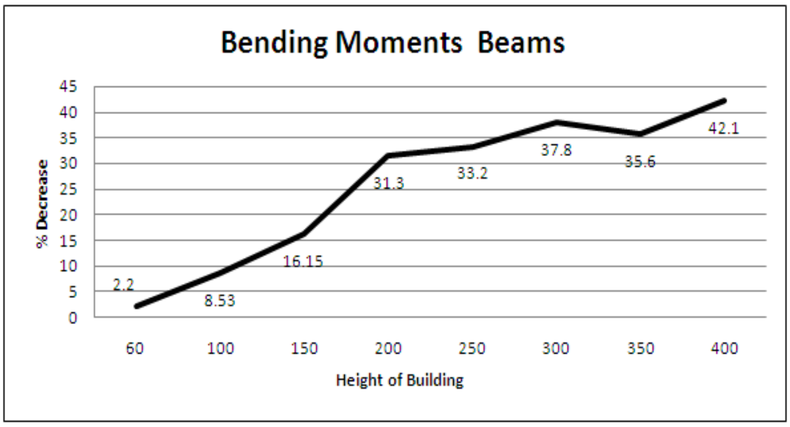

Fig. 8: *Graph showing percentage decrease in bending bending moment of beam Vs height of building*

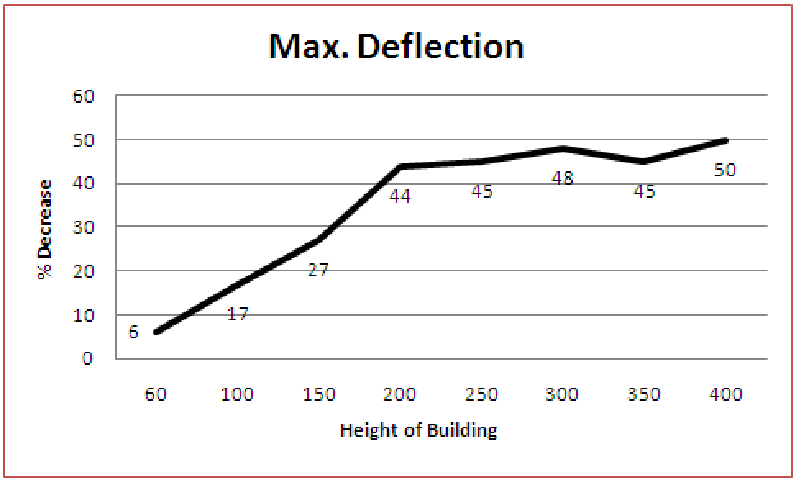

Fig. 9: *Graph showing % decrease in top story deflection Vs height of building*

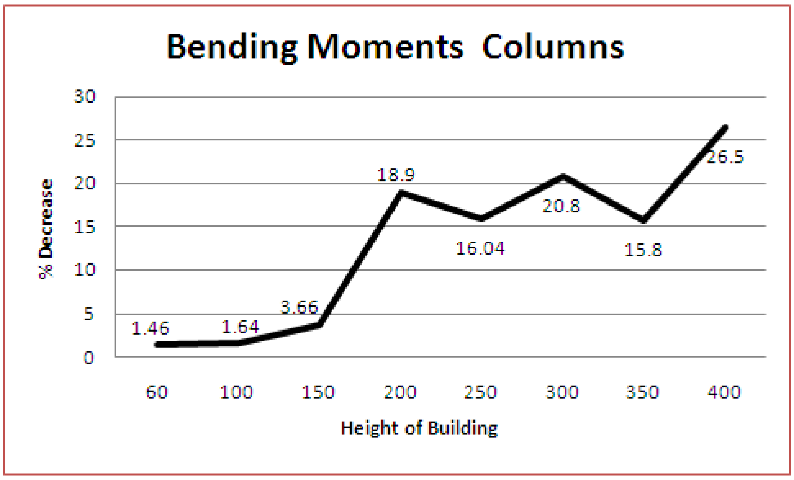

Fig. 10: Graph showing percentage decrease in bending moment of column Vs height of building

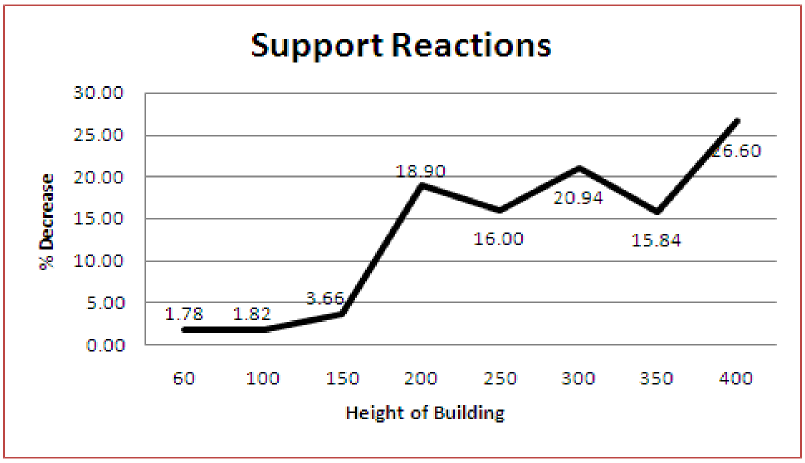

Fig. 11: *Graph showing percentage decrease in support reaction Vs height of building*

Height of Building	Avg. Decrease in Beam Reinforcement (%)		Avg. Change in Column Reinforcement (%)
	Negative Rein.	Positive Rein.	
60	2.6	Not significant	2.9
100	9.2	5.5	3.5
150	16.4	33	4.6
200	35.6	50	38.8
250	37.6	47.5	34.5
300	40.1	50.98	23.4
350	35.5	43.3	19.2
400	39.8	52.15	40.6

Table 3*: Summary of comparison of reinforcement area requirements in static and dynamic analysis*

7. Discussion of results

It is evident from the work presented that the vibration parameters of structure responding to an earthquake, depends upon the method of calculating the seismic force distribution within the structure. Equivalent static lateral force method gives member forces and displacements of the structure larger than the more precise Response Spectrum analysis.

This difference in the analysis results, which obviously affects the structural design of the buildings, increases with the increase in height of the building.

Requirement of reinforcement can be reduced on the basis of Response Spectrum Analysis which consequently reduces the budget required for the building.
Support reaction is reduced up to 18% resulting in economical foundation design.
The column and beam sizes where the required reinforcement is minimum in both static and dynamic analysis results, sizes of the members can be reduced on the basis of Response Spectrum Analysis results which show lesser stress in the members as compared to the static analysis.

8. Conclusions

- There is a significant reduction of about 31.3% in beam moments if Response Spectrum Analysis is performed instead of static analysis for 20 story building (200 ft height) or above in seismic Zone-3. This results in 35.5% decrease in negative reinforcement and 50% decrease in positive reinforcement area.
- Top story deflection is reduced up to 44% and more if Response Spectrum Analysis is performed instead of static analysis for 20 story building (200 ft height) or above in seismic Zone-3.
- There is a reduction of about 18.9% in columns sizes if Response Spectrum Analysis is performed instead of static analysis for 20 story building (200 ft height) or above in seismic Zone-3. This results in 38% decrease in column reinforcement.
- On the basis of Response spectrum analysis member sizes can be reduced (as compared in static analysis) which will further result in lesser dead weight of the structure and will result in an economical foundation design.
- Structure with 200ft height and higher should be designed on the basis of analysis results from response spectrum analysis and not by the static analysis to get an economical and safe structure.

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REVIEW OF CONFIDENCE FACTOR IN EC8-PART3: A EUROPEAN CODE FOR SEISMIC ASSESSMENT OF EXISTING BUILDINGS

P. Rajeev[1], P. Franchin[2], P. E. Pinto[3]

[1]Research Fellow, Department of Civil Engineering, Monash University, VIC 3800, Australia.
[1]Email: Nathan.Rajeev@monash.edu
[1]Telephone: + 61 39905 1145; Fax: +61 39905 4944

[2]Assistant Professor, Department of Structural & Geotechnical Engineering,
Sapienza University of Rome, Rome, Italy.

[3]Professor, Department of Structural & Geotechnical Engineering,
Sapienza University of Rome, Rome, Italy.

Abstract: The built environment, both historic and of recent construction, is exposed to high level of seismic risk due to increasing level of seismicity around the world. Thus, it is necessary to assess the existing building performance to current level of seismicity in order to perform the cost-effective interventions. EC8-Part 3 is devoted to seismic assessment/retrofitting of existing buildings. This document introduces an adjustment factor to account for epistemic uncertainty, called "confidence factor (CF)". CF is based on the level of knowledge of the structural properties such as geometry, reinforcement layout and detailing, and materials. This solution, plausible from a logical point of view, cannot yet profit from the experience of use in practice, hence its soundness needs to be investigated in real applications. This paper proposes a probabilistic based method to calibrate CF, which can simulate the entire assessment procedure conditional on the acquired knowledge. The method is then applied to six-storey three-bay reinforced concrete frame to assess the role of CF. The obtained CF's values are then critically examined and compared with code-specified ones. The critical problems of using a single factor in seismic assessment of existing buildings are pinpointed.

Keywords: Reinforced concrete, Testing and Inspection, Reinforcement details, Material properties, Modelling assumptions, Analysis method, Knowledge level

1. Introduction

A detailed seismic assessment (or evaluation) of an individual building is required to determine the need for seismic retrofitting, and also to identify the particular weaknesses and deficiencies to be corrected. For this reason, during the past two decades considerable work has been done in the direction of developing seismic assessment methodologies, usually under the auspices of national or international organizations. The first document aligned with the modern anti-seismic philosophy can be considered to be the NEHRP guidelines, prepared in 1997 under the sponsorship of the FEMA (FEMA, 1997), followed in 2000 by the FEMA 356 (FEMA, 2000). In the same years work started on Eurocode 8 Part 3 which was finally approved in 2005 (CEN, 2005). Ofcourse, it could not be asked of these documents to provide a knowledge that did not exist and, given the relatively short period during which they were developed, it could also not be expected that they were validated through a sufficiently long experience of application. As a result, they should still be looked at as experimental and subject to further progress.

This paper focuses on one particular aspect of the assessment procedure put forward in EC8-3: the so-called confidence factor (CF), analogous to the knowledge factor in FEMA 356. Actually, the role of this factor is central in the context of the overall procedure. The paper discusses the inadequacy of the present format due the associated uncertainties in structural properties as well as the degree of freedom left to the analyst. Next, the proposed rational calibration procedure is explained and applied to the six-storey three-bay RC frame. The results seem to indicate that the CF format currently specified in the code requires modification.

2. Assessment procedure in EC8-3 and role of CF

The assessment procedure starts with the information base initially available about the existing structure. From this point on, at each step of the procedure given in EC8-3, analysts are faced with a number of options that, as it will be discussed below (with reference to Fig. 1), cannot but lead to different outcomes of the state of the structure.
First of all, the analysts may choose to attain three different knowledge levels, for which different minimum amounts of tests are required by the code (Fig. 1a). For the same target KL, the same percentage of tests per floor may be obtained with different test types and locations (Fig. 1b). Each test type involves a different measurement error and, for indirect tests, a different dispersion in the associated correlation equation. Further, once the results have been collected, these have to be integrated with the initial data set (Fig. 1c): what to do then if the additional information contradicts the design documents? One analyst might accept the discrepancy, within certain limits, while another may choose to rely entirely on in-situ information adopting a full survey, together with extended test/inspection plans, i.e. moving up in the knowledge scale to KL2. Another issue is related to the two higher levels, KL2 and KL3, for which the two options: "initial information plus verification" and "complete reliance on in-situ information" are given as equivalent alternatives. It is quite likely that they are not exactly equivalent and this represents one further source of difference in the final assessment results.

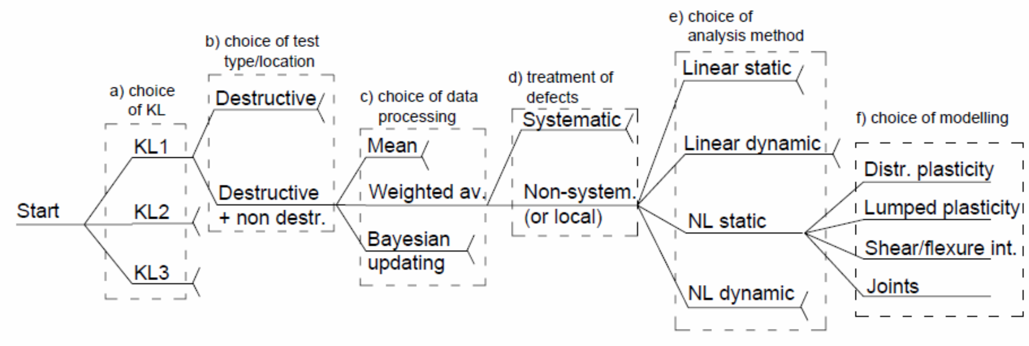

Figure 5. *Degrees of freedom left to the analyst*

The next branching point has to do with the so-called defective details, such as, for example, insufficient anchorage length of rebars, 90° hooks and inadequate diameter/spacing in stirrups, absence of joint reinforcement or wrong detailing of the anchorage of longitudinal bars into the joint, etc. This is a multi-faceted problem. Once a type of defect is discovered, the question arises whether its presence should be considered systematic over the structure or a portion of it only, or as an isolated local feature (Fig. 1d). An informed answer to this question would require extensive and intrusive investigations that are seldom compatible with the continued use of a building.

The next choice of the analysts is the method of analysis to be employed (Fig. 1e), which is intimately related to that of the modelling (Fig. 1f). Obviously, if the selected analysis is linear, the cyclic degradation due to defects cannot be included at this stage. But even if it is nonlinear static, such behaviour cannot be easily included. Exclusion of these defects from modelling may lead to a response quite different from the real one. On the other hand, nonlinear dynamic analysis including behavioural models for defective members would trade the model uncertainty on capacity with that on hysteretic degrading response.

The different sources of uncertainty and multiple choices facing the analysts during the assessment, all contribute to a relatively large dispersion in the estimated state of the structure. The interpretation that is proposed herein for the CF is that of a factor which aims at ensuring that, out of a large number of assessments carried out in accordance with EC8-3, only a predefined, acceptably small fraction of them leads to an unsafe result, i.e. to overestimating the actual safety. Admittedly, the idea that a single factor, with values depending only on the knowledge level, and not on all the aspects recalled above, may achieve the stated objective may appear as unrealistic. The paper represents an attempt to investigate to what extent this idea maintains some value. Further it provides a limited exploration on the magnitude of the CF values needed to reach the stated goal.

3. Proposed procedure for the evaluation of CF values

The proposed procedure consists of a simulation of the entire EC8-3 assessment process with the purpose of quantifying the dispersion in the assessment results due to the many choices/uncertainties described in the previous section.

The starting point of the procedure is to imagine an existing building, with all its properties, including the defects and spatial fluctuation of materials, geometry, etc. completely known. This ideal state of perfect knowledge can never be obtained in practice and it represents a state of knowledge higher (the highest possible) than the state of so-called complete knowledge described in the code (KL3).

In each simulation run choices (knowledge level, type and position of tests, how to process the results, analysis method, modelling options, etc) are made randomly to reflect the arbitrary choices made by different analysts. This obviously requires the spelling-out of all the steps described in the previous section, discretizing the possible choices in a finite number of options and filling the gaps of the code with practices coming from common-sense and experience in real-case assessments. It is imagined that the generic analyst will follow his trail down the procedure arriving at a different evaluation of the safety of the structure. This simulation is carried out without employing the confidence factor (i.e. CF=1). By repeating the process for a sufficiently large number, say n, of analysts a statistical sample (of size n) of the structural safety is obtained and can be used to estimate its distribution.

At this stage the statistical sample of structural states, quantified by the global state variable Y (a *critical* demand to capacity ratio, see Jalayer *et al* 2007) is compared with the *true* state of the structure. It is expected that a portion of the assessments will result in a *conservative* estimate (i.e. in a state worst than the real one) while the remaining will be on the *unconservative* side.

The goal of the last part of the procedure is that of reducing the fraction of un-conservative estimates to an acceptably small value. This is done by re-evaluating the structural state, using the same sets of choices of the previous evaluation, with a value of CF larger than one (i.e. decreasing capacities). If the procedure works as intended the new sample of structural states will have the predefined target fraction of unconservative estimates. The procedure can be split into the following steps:

- Step 1: Generation of the existing and perfectly known structure
  Once all the material properties and possible defects have been assigned a probability distribution, a structure can be generated by sampling a set of parameter values from the above distributions. This structure is by definition completely known and is termed the *reference* structure.
- Step 2: Generation of a sample of imperfectly-known structures from the reference structure
  A number N_{VA} of virtual analysts is given the task of assessing the structure. This step consists of simulating the process of inspection/information-collection, and produces N_{VA} different states of (imperfect) knowledge from the reference structure. These states are the starting point for the assessment by the virtual analysts. In order to reflect the different test plans designed by different analysts, this step requires the randomization of the test locations and test types.
- Step 3: Assessment of the reference structure
  The reference structure is assessed according to the code and the seismic intensity that induces the attainment of the limit-state (LS) under consideration is recorded. The attainment of the limit state is marked by a unit value of the global variable Y=1. This result is considered the true state of the structure.
- Step 4: Assessment of the imperfectly known structures
  The virtual analysts apply the code-based assessment procedure with a unit value of the CF and the same intensity as determined in Step 3. This produces a sample of N_{VA} values of the global state variable Y. This step requires a further randomization, reflecting the freedom left to the code-user in choices such as inclusion/exclusion of defects from modelling and the selection of the analysis method (linear vs. nonlinear, static vs. dynamic).

- Step 5: Statistical processing of the sample states and determination of CF
  Statistical processing of the sample of values of *Y* produces a distribution that exhibits a certain amount of variability around the value Y=1. This is shown in Fig.2 (left). The value of the CF can now be determined by enforcing the condition that a chosen lower fractile of *Y* (say, 10%) is equal to 1, i.e. the *true* state of the structure (as shown in Fig.2 (right)).

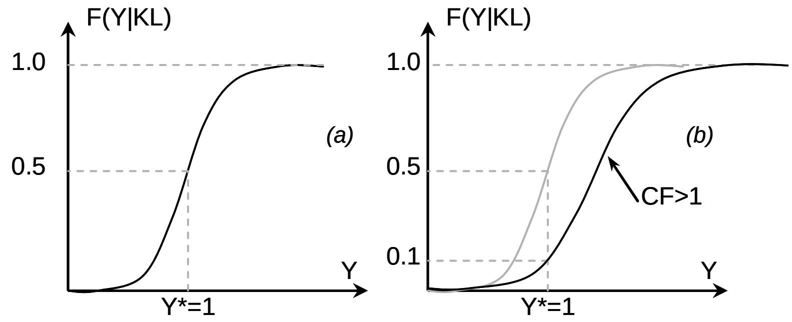

Figure 6. *Distribution of Y conditional on the KL (left), Enforced distribution of Y conditional on the lower fractile of Y (right)*

4. Application

The calibration procedure has been applied to a six-story three-bay RC frame structure (Fig.3). For the purpose of data collection (material tests, reinforcement details etc.) and post processing the structure is considered homogenous, in the sense that the spatial distribution of the properties/defects belongs to a single population.

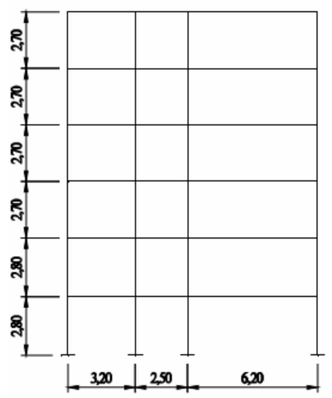

Figure 7. *Six-story, three-bay asymmetric frame*

The assessment has been carried out with the nonlinear static and dynamic methods. CF values have been evaluated both separately with each of the two methods, and jointly, to investigate dependence on the analysis method. For the purpose of dynamic analysis the seismic action is represented by seven recorded ground motions selected to fit on average, with minimum scaling, the EC8-specified spectral shape scaled to a PGA of 0.35g for soil class A (Iervolino et al, 2008). In the case of static analysis, the average spectrum of the recorded ground motions is considered as the demand spectrum.

In terms of modelling the nonlinear degrading response of the structure, account has been taken of flexure-shear interaction and joint hysteretic response. The model is set up in OpenSEES, employing flexibility-based elements for the members with section aggregator to couple a fibre section (flexural response) with a degrading hysteretic shear force-deformation law. Joints have been modelled with a "scissor-model" with a degrading hysteretic shear force-deformation law. Tangent-stiffness proportional damping has been used, calibrated to yield a 5% equivalent viscous damping ratio on the first elastic mode. Since the effect of brittle failure modes, such as shear in members and joints, has been included in the modelling (for both static and dynamic analysis), the structural performance is

checked in terms of deformation quantities only. Hence, $Y = \theta_{max}/\theta_C$, where θ_{max} is the demand peak inter-storey drift ratio, and θ_C the corresponding capacity. Detailed information on the characteristics (geometry, reinforcement, etc) of the structure and the adopted response models can be found in (Rajeev, 2008).
The purpose of generating the *reference structure*, material properties (concrete strength, steel yield stress, and hardening ratio) and structural defects (transverse reinforcement spacing in columns and beams, and column longitudinal reinforcement ratio) are sampled from predefined probability distribution functions (see Table.1).

Table 1. *Distribution type and parameters for random variables*

Random variable	Distribution	Mean or Min	CoV or Max
Column stirrup spacing	Uniform	200 mm	330 mm
Beam stirrup spacing	Uniform	150 mm	250 mm
Reinforcement ratio	Uniform	0.008	0.014
Concrete strength	LN	20 MPa	0.10
Steel Yield stress	LN	275 MPa	0.05
Hardening ratio	LN	0.04	0.25

A value of concrete strength has been sampled at each integration point along a member, while a single pair of values of steel properties has been sampled for all members of each floor. Correlation has been introduced amongst the concrete strength values according to an exponential decay model. The Nataf joint distribution has been adopted for simulation of the concrete strength field values (Liu and Der Kiureghian, 1986).

For the purpose of generating the *imperfectly known structures* (Step 2 of the procedure), the data collection procedure, consisting of tests on material samples from the structure and verification of reinforcement details, is randomized. The *number* of test/inspection locations is determined based on the minimum requirements in the code. These latter are specified as a function of the target KL. Test/inspection levels for KL1, KL2 and KL3 are denominated as *limited*, *extended* or *comprehensive*, respectively, when initial information is poor (relative to each KL requirements).

The actual test location chosen by each analyst is determined by randomly sampling (uniform integer distribution) first the member and then the location within the member (for this purpose each integration point is regarded as a possible test location). At each location, the testing/inspection consists of reading the value of the sought property from the reference structure (value generated during Step 1). Measurement errors are not considered. Since the reference structure is homogeneous by assumption, all the data gathered are *averaged* to obtain the values to be employed in the assessment.

The assumed scarceness of initial information, and in particular the lack of a complete set of construction drawings, influences the knowledge of the geometry of the structure. In particular, this may refer to the presence/absence of elements (a typical case being represented by beams in flat-slab structures) or the actual cross-section dimensions (significant variations in plaster thickness or the presence of cavities for ducts are common and cannot practically be ascertained for all members), or, finally, the precise unit-area weight of the floor system.

To model this kind of "geometrical" uncertainties (denoted as "residual" in the following) two types of additional random variables are introduced: the unit-area weight of floors (one variable per floor typology, e.g. typical floor and roof) and the cross section height of elements (one variable per element type: beams and columns). These random variables are sampled for each imperfectly known structure during Step 2. Detailed information can be found in Rajeev 2008.

5. Result

The assessment of the reference structure has been done with IDA (Vamvatsikos and Cornell, 2002) subjecting the structure to the seven natural ground motions selected to match EC8 spectrum. Consistently with the code indication of using 7 records and taking the average of the maxima, the intensity (PGA of 0.216g) where the mean IDA curve crosses Y=1 is recorded and used in Step 4. The capacity has been set for this structure to the deterministic value of θ_C = 2.5%.

Step 4 of the procedure consists of the assessment by each virtual analyst of its imperfectly known structure (the result of Step 2). As already mentioned, the number of analysts has been set to N_{VA} = 200, and each of them can choose between nonlinear static and dynamic analysis for the assessment. Actually, in this application each analyst has performed both analyses (dynamic and static). The results are first presented separately by method (200 samples each) and then *mixed* (400 samples). This, as anticipated, serves the purpose of investigating the dependence of CF on the analysis method.

The empirical distributions (conditional on KL) of the 200 Y-values obtained are shown in Fig. 4 (top left). In the figure the value Y=1 is marked by a dashed vertical line. For the employed seismic intensity of PGA = 0.216g this is the state of the reference structure. A second vertical (solid) line marks the value Y= 0.79. This is the state of the *mean* structure, i.e. a structure identical in geometry to the reference one, but with spatially homogenous properties equal to the average values of the samples generated in Step 1. As it can be seen, with increasing KL the distributions get steeper (lower dispersion) and closer to the *mean* rather then the *reference* structure. In all cases a large proportion of the analysts overestimate the safety of the structure (i.e. they find Y<1): roughly 40% with KL1, 70% with KL2 and 100% with KL3.

Next, the analysis is repeated with CF-values larger than one in order to reduce the above percentages to the same acceptably low value. For the purpose of this application this value has been set to 10%. Sensitivity of the results to this choice can be found in (Rajeev, 2008). Fig. 4 (top left) shows the corresponding distribution for KL1 only, for clarity (CF=1.34).

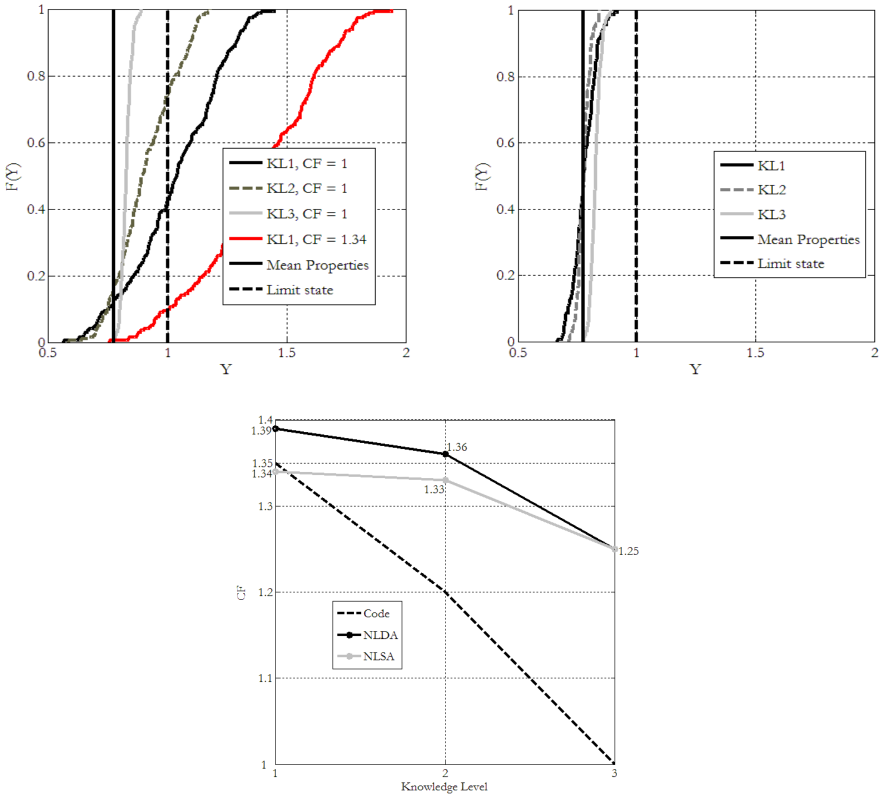

Figure 8. *Distribution of N_{VA} Y-values obtained by static analysis (top left), distribution by static analysis neglecting residual geometric uncertainty (top right), CF-values and analysis types (bottom)*

The relevance of the residual geometric uncertainty can be appreciated by comparing the curves in Fig. 4 (top left) with those in Fig.4 (top right), obtained disregarding this contribution (the difference between the structures analysed by the virtual analysts is only due to material properties and construction defects). The CF values obtained for the considered structure are summarized in Fig.4 (bottom). The figure reports separately the values obtained by static (grey) and dynamic (black) analysis, together with the code-specified values. It can be observed how the dependence of CF on KL is in all cases milder than that specified in the code, and that CF depends on the analysis method.

6. Findings and Conclusions

Based on the results, the most relevant findings are:

- In the code CF values are specified as a function of KL only, implying that KL is the single most important factor influencing CF. Results appear not to clearly support this expectation of the code. The dependence is found to be generally mild.
- The code does not differentiate CF values with respect to the analysis method, implying that epistemic uncertainty has the same effect with all analysis methods. Results appear again not to clearly support this assumption. When considered separately (i.e. assuming that all analysts will chose the same analysis method) nonlinear static results show a much reduced dispersion than those obtained by nonlinear dynamic analysis. This, according to the proposed procedure, leads in general to smaller values of CF to be employed with static than with dynamic analysis.
- The code specifies that the geometry of the structure must be completely known before setting up a model for the analysis. Experience with real-case assessments shows that it is usually not possible to obtain accurate measurements over the entire structure and that even when member centrelines are known, a residual uncertainty on the cross-section dimensions is unavoidable. This source of uncertainty has been modelled in the applications. Results show it to be, for the examined case, at least of the same order of importance of that associated with material properties and defects.

In conclusion, within the limits of the analyses carried out, it appears that current CF-based format of Eurocode 8 Part 3 doesn't show to be entirely adequate for its purpose, and should be improved since:

- CF values are not differentiated with respect to analysis method/modelling options;
- CF values are not differentiated with respect to structural type (size, regularity, construction material, load-resisting system, etc);
- the so-called complete knowledge (KL3) does not actually correspond to a state of perfect knowledge, hence, it should be penalised with a CF value larger than one.

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PRE-COMPRESSED CONCRETE-FILLED STEEL TUBE FOR HIGH EARTHQUAKE RESISTING PERFORMANCE OF STEEL COLUMNS

Susantha K. A. S.[a*] and Aoki T.[b]

[a] Senior Lecturer, Department of Engineering Mathematics, Faculty of Engineering, University of Peradeniya, Peradeniya, Sri Lanka. Email: samans@pdn.ac.lk, Tel: +94 81 2393351.

[b] Professor, Department of Civil Engineering, Aichi Institute of Technology, Toyota, Japan. Email: aoki@aitech.ac.lk, Tel: +81 565 48 8121.

Abstract

Strength of the circular-shaped concrete-filled steel tubes (CFT) is enhanced significantly due to the confinement provided by the surrounding steel plates. The effectiveness of the confinement depends on several factors such as column slenderness, diameter to thickness ratio, strengths of steel and concrete, the loading method and boundary conditions, and the interface condition between steel and internal concrete. A new technique is introduced in this study to increase the effectiveness of the confinement in order to improve the strength and the ductility of CFT columns. The CFT column used in this study is different from the conventional CFT column in that the concrete is compressed prior to hardening using two circular steel plates placed at both ends of the column.

Keywords: *concrete-filled steel tubes, confinement, strength, ductility*

1.0 Introduction

A great effort is made in seismic design of highway bridge piers made out of steel plates to control the formation of local buckling deformation in plates in order to achieve high ultimate strength and ductility capacity. Concrete infilling, use of stiffened sections, double-skin tubes, tapered plates, low yield strength steel plates, introducing shear walls are few techniques that have been proposed so far for this purpose [1-6]. Among these, concrete-filled steel columns have a widespread application in building and civil engineering structures in areas where severe earthquakes are expected to occur because of the enhanced axial strength and ductility of such members due to the confinement provided by steel plates. The effectiveness of confinement varies with the column slenderness, diameter to thickness ratio, strength of steel and filled-in concrete, the loading method, boundary conditions, and the interface condition between steel and internal concrete [7-10]. An attempt is made in this study to explore the possibility of increasing confinement by pre-compressing filled-in concrete before it get hardened. In addition, the new CFT member is designed so that axial load is applied only to concrete segment. The pre-compressing is to be done using two circular steel plates placed at each end of the tube. The new pre-compressed concrete-filled circular steel tube (PC-CFT) is intended to be placed inside a hollow box or circular steel piers with a special loading device from which axial load from superstructure transfers to the PC-CFT member while lateral loads exerting on piers due to earthquakes will be taken by the main pier. This will greatly enhance the ultimate strength and the ductility of bridge piers when subject to earthquake loads.

2.0 Test of Concrete-Filled steel Tubes (CFT)

Concrete infilling has become increasingly popular in building and bridge pier construction over the past few decades. A large number of experimental and analytical studies has been carried out to investigate the behaviour of concrete-filled steel tubes. The attention has been paid in particular on examining the strength enhancement, ductility improvement, and energy absorption capacity because these are the key factors considered in designing of earthquake resistant structures [11, 12]. It has been well known that the CFTs have excellent earthquake resisting characteristics. These improvements are mainly due to the confinement of concrete from surrounding steel plates. The mechanical behaviour of CFT columns when load applied to: (a) the concrete section; (b) the steel section; and (c) the entire section has been investigated extensively in a past study, and it has been

* *Corresponding author*

revealed that the axial deformation of columns with load applied only to the concrete section was higher than the other two cases [8].

In this study, series of axial load tests of short CFT columns were carried out prior to the testing of PC-CFT columns in order to check the effect of interface condition and the time of concrete compressing (t_i). The axial load was applied only to the concrete through a loading cap. The concrete is slightly compressed at the beginning so that the confinement could be much effective and a significant strength gain could be expected. For this, nine specimens in three sets (i.e., Set-1, Set-2 and Set-3) were prepared. The details of the specimens are presented in Table 1 where t_0 is the thickness of tube, D is the outer diameter, and h is the specimen height. The steel was of grade SM490 having nominal yield strength of about 325 MPa. Since the actual yield strength usually differs from the nominal value, tensile coupon tests were carried out to check the actual yield strength. These tests showed that the yield strength is around 414 MPa, Young's modulus is 208 GPa, and Poisson's ratio is around 0.29. The loading arrangement is shown in Fig. 1.

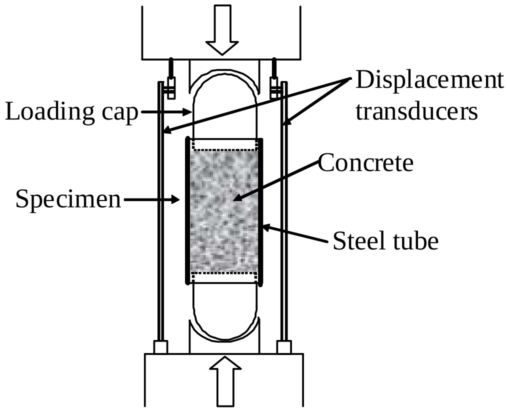

Fig. 1 *Loading arrangement*

Table 1 *Details of specimens (concrete-filled steel tube and concrete cylinders)*

Parameter	Set-1			Set-2			Set-3		
	A1	A2	A3	B1	B2	B3	C1	C2	C3
t_0, mm	6.8	6.9	7.2	7.1	7.1	7.2	7.0[a]	6.9[a]	6.9[a]
D, mm	165.9	165.9	165.9	165.9	166.0	165.6	165.8	166.0	166.0
h, mm	450.0	450.0	450.0	450.0	450.0	450.0	450.0	450.0	450.0
Interface	Paraffin	Paint	Grease	No	No	No	-	-	-
t_i, min	90	90	90	90	180	270	90	180	270

[a] Tube removed after concrete compressing

Axial shortening was measured from displacement transducers. The inner surface of steel tubes in Set-1 was applied with paraffin, grease, and paint and concrete was compressed 90 minutes after pouring. In Set-2, no interface material was used and concrete was compressed after 90, 180, and 270 minutes respectively after pouring. In Set-3, specimens had the same dimensions as those of Set-1 and Set-2 but outer steel tube was removed before concrete get fully hardened. The concrete was compressed after 90, 180, and 270 minutes respectively after pouring similar to those of Set-2.

The results of the specimens in these three sets are shown in Fig. 2. As seen in Fig. 2(a), the paint and grease have the same effect on the strength, but paraffin causes comparatively low stiffness and slightly high strength of concrete. There is no any apparent effect from the time of concrete compressing when tested with the outer tube as seen in Fig. 2(b). The strength of concrete in specimens Set-2 and Set-3 was nearly twice that of Set-3.

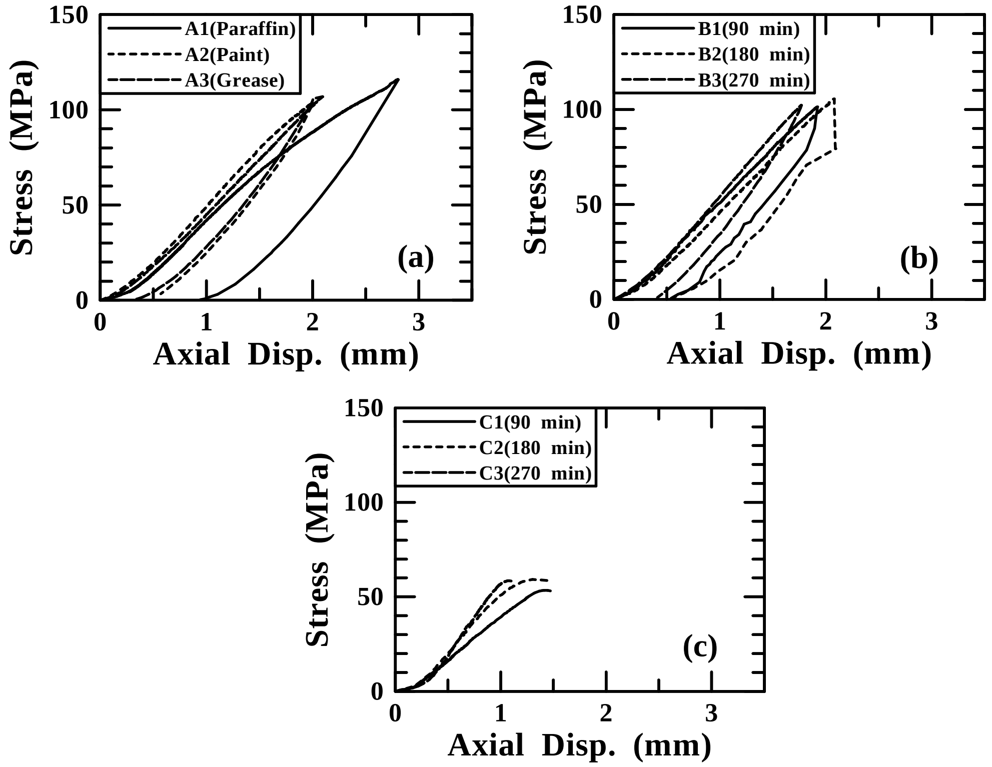

Fig.2 Axial stress versus axial displacement of specimens

3.0 Test of Pre-Compressed Concrete-Filled steel Tubes (PC-CFT)

The proposed PC-CFT column is shown in Fig. 3. The concrete inside the PC-CFT is compressed before getting it hardened, and the standard cylinder tests were carried out simulating the same condition of filled-in concrete. This means that the concrete cylinders were prepared by compressing the concrete after 90 minutes as same as the concrete in PC-CFT. Six cylinders were prepared and tested after 14 days. The average compressive strength was found to be 49.1 MPa. The axial load from the super-structure is directly applied to the concrete core through a loading cap. The inside concrete is compressed from 60 to 90 minutes after pouring, using two steel plates placed at both ends of the tube and a steel rod connecting these two plates. After compressing, the plates are bolted to the steel rod. Thus, the steel rod is in tension and concrete is in compression. The interface between steel and concrete is made as much frictionless as possible by applying paraffin, grease or paint. The air voids inside the concrete wiped out during compressing the concrete. The concrete should be compressed thoroughly so that the fully hardened concrete should have effective confinement from the surrounding steel plates. Some stress will be released when concrete get hardened due to shrinkage.

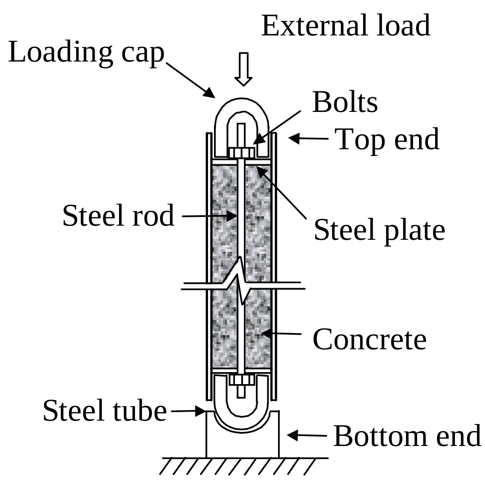

Fig. 3 PC-CFT specimen

Nine specimens as described above were prepared in this study to estimate the axial strength of the PC-CFT columns. They were divided into three sets each having three specimens. Each set was tested 14, 21, and 28 days respectively after casting. The average strength of each set was found to be 78.4, 82.6, and 88.5 MPa, respectively. The results implied that the PC-CFT columns have much higher strength than the normal CFT columns.

4.0 Use of proposed PC-CFT

The proposed PC-CFT is intended to be used in a new structural system where PC-CFT is placed inside a hollow box- or circular-shaped steel column of highway bridge pier. The structural system is designed so that only axial load from superstructure transfers to the PC-CFT while main pier is reserved to take lateral loads exerted due to ground acceleration. This could be done using a special loading device.

5.0 Conclusions

Conventional concrete-filled steel tubes are well known for their high axial strength and ductility performance. Concrete confinement has been identified as a key factor that affects the performance of CFT members. Pre-compressing of in-filled concrete was found to be very effective in further improving the confinement. Based on the test results, it has been found that the condition of steel-concrete interface has moderate effect on the axial stiffness of the CFT members. And, the axial strength of pre-compressed CFT was found to have very high strength level such as 88.8 MPa.

Acknowledgment

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REUSE – ISSUES AND CHALLENGES IN STRUCTURAL ENGINEERING APPLICATIONS

N. Haritos[1] and N. T. K. Lam[2]

[1,2]Associate Professor and Reader, Department of Civil and Environmental Engineering,
The University of Melbourne, Grattan Street, Parkville, Victoria, 3010, Australia
[1]E-mail: nharitos@unimelb.edu.au
[1]Telephone: +61-3-83446829; Fax: + 61-3-83444616

[2]E-mail: ntkl@unimelb.edu.au
[1]Telephone: +61-3-83447554; Fax: + 61-3-83444616

Abstract: Reuse of structural elements, frames or modules in buildings and structures, as a concept towards improving sustainability in the built environment poses a number of issues and challenges. These issues and challenges arise as early as the conceptual stage. What are the structural elements to be considered for reuse ? How are they to be designed to allow for this possibility and to facilitate the process of reusing them as elements, elsewhere ? Are new concepts for such element types (ie non-traditional structural forms) needed or can traditional forms, but with reuse concepts in mind, still be viable propositions ? For example, there are plenty of opportunities for innovation in which the structural integrity of concrete is provided mainly by external confinement in order that the amount of cement binders in the concrete can be reduced to facilitate reuse of the aggregates. What are the limitations (Architectural as well as Engineering) that would be associated with designing such elements for their possible reuse ?
These and a number of other questions are discussed and some ideas offered towards addressing them are then compiled in this paper.

Keywords: reuse, recycle, reduce, sustainability, structural system innovation

1. Introduction

Sustainability, in its many forms and how it may be implemented in buildings and building construction, is receiving a great deal of attention from designers, owners, occupiers and other interested stakeholders, nowadays. Much of this attention has been directed towards improving efficiencies in energy consumption and in the air conditioning of these buildings, when in service – the Reduce part of the so-called 3R's of sustainability (Reuse, Recycle and Reduce). Whilst some attention has also been directed to possible Recycling options in buildings (eg in waste disposal, water supply, in the salvaging of copper in electrical wiring and even in the construction materials themselves (eg concrete from building demolition being recycled after processing for use as road base material, [1], in general concrete applications, [2], or for high-strength concrete, [3]), not so much attention has however been directed towards Reuse possibilities in building construction.

It is quite easy to see the reasons why this is so but not so easy to see how these obstacles may be ameliorated, or even overcome, if we are to see reuse options being seriously exercised in buildings and building construction, [4]. This paper therefore explores the inhibitors of reuse concepts in buildings and building construction and provides some suggestions and ideas that are needed to be able to "move forward" with reuse concepts in the construction industry.

2. Inhibitors of Reuse Concepts in Buildings

When one starts to consider the reasons behind why possible reuse concepts may be difficult to implement, often to the point that they are therefore not exercised (allowing for some few exceptions), these are found to be easy to identify. Some scenarios of possible reuse strategies or ideas are presented here to illustrate these inhibitors depending upon the situation.

2.1 Reuse of entire buildings

When an existing building of some years comes onto the market, its attractiveness to a prospective purchaser for reuse, either for its original intended purpose or for some viable alternative, and this only when any modifications necessary are minor, is dependent primarily on how "dated", or conversely, how "fashionable" it may be perceived as a marketable proposition. Here we may be considering medium to high-rise office buildings, say of 30 or 40 years of age. Architects may advise to level the building concerned and to create a "fresh new one", with more modern layouts, fixtures and features that would attract prospective tenants a lot more than would the original building after minor or even significant refurbishment.

Some exceptions here, drawn from the experience in Australia, would be:

- conversion of an office building to apartment style housing in the centres of such major cities as Melbourne say, where inner city apartment style living is "catching on" from when it was once (not so long ago) virtually non-existent,
- heritage buildings which are protected from demolition by local legislation and which have significant restrictions placed on the style of refurbishment and reuse that may be considered for the building structure, where this is seen as a marketing edge by the owners (as opposed to an inhibitor). Examples here, may range from heritage ex-church, council and bank buildings, (often converted to dwellings/apartments, restaurants or even fast food outlets, see Fig. 1), and older style cinemas (often converted to ballrooms/reception centres), which although do not appeal to everyone do attract a significant clientele to become viable or even attractive,
- warehouse buildings in inner suburbs, which can be appealing because of their generally high ceilings, after significant refurbishment can be converted to restaurants and dwellings/apartments.

So two inhibitors of reuse of buildings (when perceptions are adverse) are identified here – "marketability" and "fashion", though one may consider these to not necessarily be entirely mutually independent.

2.2 Reuse of building modules or components

When buildings are to be demolished, attractive elements for reuse (and recycling) are identified and salvaged prior to the more rigorous and damaging process of serious demolition taking place. These elements would include:

- Copper piping and wiring (attractive because of its value for recycling).

Figure 1: Examples of Heritage and Church Building Reuse

- Bathroom and other fixtures that can be reused, especially if of heritage or artistic value. A striking example “close to home” is the entrance to the underground car-park at The University of Melbourne - itself a heritage listed structure because of its unique use of a regular grid of hyper-parabolic shell roof elements supported by hollow circular columns, [5]. The doorway to the now demolished Colonial Bank originally in Elizabeth Street has been reused at one entrance and a 1745 wooden door from a house in St Stevens Green Dublin, donated by the government of Eire to the University, has been reused at another entrance, (see Fig. 2).
- Items that could be of limited intrinsic value other than in terms of memorabilia, eg portions of carpet, [6], wall paneling, brass coat-hooks and other fixtures when the old grandstand portion of the MCG was demolished to make way for the construction of the Great Southern Stand.

Seldom would building elements or even building materials be salvaged for reuse from a building demolition site for a number of reasons (or “inhibitors”), which would include:

- Cost – construction elements and materials are generally not designed to facilitate their removal intact, so this tends to make it difficult and costly to salvage them
- Safety issues – as, again, because ease of removal has not been considered in their design, to access primary elements, in particular, by attempting to remove tertiary and secondary elements and to then disconnect them would often pose high risk.
- Integrity issues – the fitness for purpose (or reuse) of construction elements and materials may be questioned as their strength and integrity may have been compromised from adverse loading effects, during their history of operation, or as a result of the removal process itself.

There are, however, some notable exceptions to the reuse of entire structures and building components that can be found in the offshore oil and gas industry. For example, jack-up rigs can be reused in their entirety by the offshore wind industry [7], (see Fig. 3), and modular topsides elements can be refitted to other platforms, once no longer required at their original site.

A particular driver for reuse of building materials is in situations of extreme poverty which virtually dictate this to be the only option as the cost of producing a structure anew is prohibitive.

This situation is notably exemplified in the case of Toni ‘el Suizo’ Rüttimann – bridge-builder, [8]. Toni is indeed a unique individual who, through his bridge-building, based upon a suspension bridge design that he has more-or-less perfected over 23 years, a design that is based virtually entirely of reused components/materials, has transformed the lives of many thousands of residents in remote locations in South America and South-East Asia. Over 500 bridges have been constructed by the local inhabitants/villagers in these remote locations with Toni’s help. Figure 4 illustrates the design concept for Toni’s suspension bridge via an example of a bridge under construction and another of the completed product.

Figure 2: The two entrances (both examples of reuse options) and the Underground Car-park at The University of Melbourne

Figure 3: Example of a Jack-up Oil

Figure 4: Rüttimann’s Suspension Bridges– under construction (Ecuador), completed (Vietnam)

The rectangular frames, in this design, are welded tubular members salvaged from the offshore oil industry and the cables are ones that have been removed from service (according to statutory requirements) that once supported cable-cars in Switzerland. The vertical stringers have also been salvaged. The only “new” material tends to be that used for the wooden/steel plate decking.

3. Reuse Innovation in Buildings and Building Construction

In the situation of a building which is free of heritage protections, the owner may decide to demolish and rebuild in view of marketability and fashion considerations. Whilst the original building cannot be preserved in its entirety, there is plenty of scope for innovation to be introduced to salvage materials from it for reuse in the building that is intended to replace it and hence reduce the consumption of energy and the need for new materials in the rebuilding process. This can be achieved by either (i) the reuse of members or (ii) the reuse of materials.

3.1 Reuse of Members

Significant savings in both energy and materials can be achieved by reusing structural members that can readily be detached from the existing structure. The reuse of structural steel girders and columns is already common practice given that bolted connections require a relatively small amount of effort to undo. Reuse of dismantled components for rebuilding on the same site is the ideal arrangement from the perspective of maximising savings. However, this *direct reuse* approach is not always viable with contemporary design practices. The concept of direct reuse should therefore be incorporated into the architectural and structural design of buildings to facilitate this practice in the future. For example, an existing building and its replacement could adopt a similar modular design in order that beam and column lengths are kept the same. Consequently, members can be directly reused in the replacement structure on the same site.

Alternatively, dismantled members can be distributed to different sites for reuse which has the obvious advantage of increased flexibility in design. However, the challenge with this *re-distributed reuse* approach is the development of an efficient and effective co-ordination scheme for stockpiling, sorting and redistribution of such members and components.

Whilst salvaging bolt connected steel members is immediately practical, extending this reuse approach to floor slabs that are typically built of concrete would represent a major challenge. Building floors are commonly built using *in-situ* concrete, concrete cast over corrugated steel, precast hollow core units or waffle slabs. These concrete units are difficult to separate given that their connectivity is typically achieved through the use of grout or *in-situ* concrete. The same can be said of concrete walls and facades. The sheer weight and size of precast concrete also means it would generally be costly to handle and stockpile precast units following their detachment from the structure concerned. Thus, the reuse of concrete is not as straightforward as steel. However, there is plenty of scope for innovation in the design of concrete floors in terms of facilitating the removal of precast concrete planks (or similar floor elements) from their supporting girders.

3.2 Reuse of Materials

The authors have identified considerable scope for future innovation with the reuse of materials as opposed to the reuse of prefabricated units. The recycling of concrete aggregates is a well known example of the reuse of building materials. A drawback with recycling is that a considerable amount of energy needs be expended for breaking up concrete into smaller particles, separating the aggregates, and re-introducing cement to bind the aggregates to form a new structure. Reuse should be distinguished from recycling in that reuse effectively short circuits the process of renewing in order to save energy as illustrated in the schematic diagram of Figure 5.

The primary challenge with the reuse of concrete materials is the irreversible binding actions of the cementitious materials. A potential breakthrough with the innovative reuse of concrete is circumventing the use of the cement binder. As is widely known, the intrinsic compressive and shear strength of concrete is primarily attributed to the cement binders. However, similar strength could be achieved by exploiting the advantages offered by confinement and arching action through suitable choice of geometric configuration. Importantly, the cement binder is not an essential ingredient in achieving the desired strength provided that the required (compressive) load paths have been facilitated in the structural form. This “innovation” is nothing new given that the design of stone arches in historical structures has been based on this concept. The major challenge is in adapting this old concept into contemporary construction, (here exemplified in terms of a floor system in a building), without significantly altering its form.

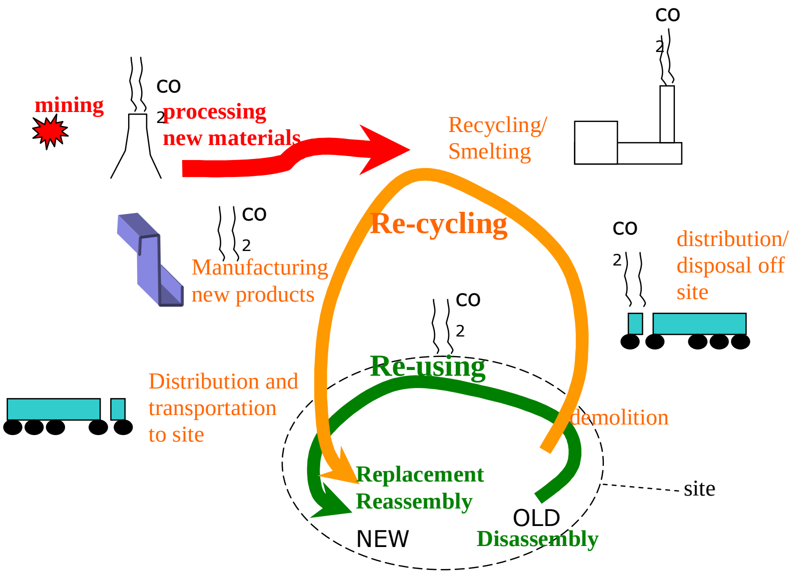

Figure 5: Reusing versus

The authors have designed and built a 1:100 scale model of a building out of pebbles and 1.2 mm thick cardboard to demonstrate the feasibility of utilising arching action in a slab-beam-column form of contemporary construction as depicted in Figure 6. Arching action in support of the floors was enabled via the curved shape of the floor cross-section formed by stiff cardboard and the use of cardboard tie-plates. The columns were essentially supported by the confinement of pebbles in tubes also made of cardboard. Some 20 kg of pebbles essentially were able to be supported "off the ground" by 1.2 mm thick cardboard sheets. Obviously, the reuse of materials with this type of construction would not be hampered by cement binders because such binders are not used, nor required, in this design concept.

Aggregates that have been salvaged through a reuse strategy based upon this design concept can be pumped into silos and extracted there-from as may be required for use elsewhere. The need to handle and stockpile detached units is hence eliminated, as you are dealing more-or-less with the raw materials themselves.

It should be clarified here that the *cardboard-pebble* model is intended only to shed light onto the "character" of the potential innovations that would be needed to allow for effective reuse strategies in building construction. Challenges still remain for delivering robustness, safety, reliability and durability with this form of construction. There would also be additional logistical challenges with construction and re-construction posed by reusing materials from the same site.

This plethora of potential challenges clearly indicates that reuse in construction is a very fertile area for research and development work in structural engineering.

4. Concluding Remarks

This paper has explored a number of issues and inhibitors for implementing reuse strategies in the construction industry. These issues and inhibitors pose a plethora of challenges to structural engineers who seek to introduce and facilitate reuse concepts for building elements and materials in their designs of buildings, in particular.

A concept for capatalising on the property of arching action to ensure compression only conditions in a shallow arched floor system has been tested by using simple physical models to demonstrate feasibility of such a system. The idea here is that "loose" lightweight aggregate can form the "fill" for the material in the arch, without the need to "cement" the fill using cementitious binders. The "fill" can therefore be introduced and removed from the floor system housing structure, quite readily thereby facilitating a reuse strategy for floor systems that adopt such a design concept.

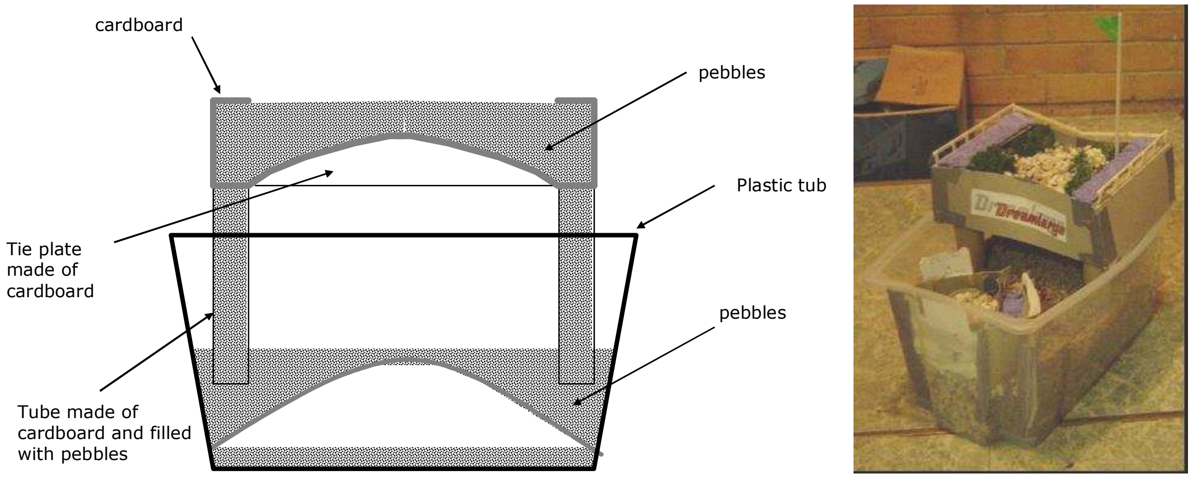

Figure 6: Cardboard-pebble model of a building supported by arching actions

The authors recognise that whatever novel concepts for implementing reuse strategies may be devised by structural engineers, challenges remain for ensuring these concepts meet robustness, safety, serviceability, reliability and durability requirements in the structures they design that incorporate such strategies.

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About the Authors

N. HARITOS, B.Eng, M Eng Sc.., Ph.D. The University of Melbourne., FIE(Aust), CPEng. is an Associate Professor and Reader in the Department of Civil and Environmental Engineering, The University of Melbourne. He has over 30 years of post-graduate experience in research, academia and specialist consulting. His research interests include: structural dynamics of both land-based and offshore structures; hydrodynamics; condition/capacity assessment of bridge structures; experimental modal analysis testing and computers in modeling, simulation and control.

N. T. K. LAM, B.Sc. Hons. Leeds, M.Sc., D.I.C. London, Ph.D. Melbourne, C.Eng. M.I.StructE, M.I.C.E., M.I.E.Aust. is an Associate Professor and Reader in the Department of Civil & Environmental Engineering, University of Melbourne. His research interests are in the areas of earthquake engineering, structural dynamics, impact dynamics and sustainable developments

FACTORS AFFECTING THE PERFORMANCE OF SEPTIC TANK SYSTEMS TREATING DOMESTIC WASTEWATER IN DEVELOPING COUNTRIES

R. Jayarathne[1], M.A. Connor[2], S.T.S. Yuen[3*] and P. Pivonka[4]

[1, 3]Department of Civil and Environmental Engineering, University of Melbourne, Vic. 3010, Australia
[2]Department of Chemical and Biomolecular Engineering, University of Melbourne, Vic. 3010, Australia
[4]Engineering Computational Biology Group, School of Computer Science and Software Engineering, University of Western Australia, WA 6009, Australia

*Corresponding author: Dr Samuel T. S. Yuen
[3]Department of Civil and Environmental Engineering, University of Melbourne, Vic. 3010, Australia.
[3]E-mail: stsy@unimelb.edu.au; Phone: +61 3 8344 6716; Fax: +61 3 8344 4616

ABSTRACT

Septic tank systems are in widespread use for treating domestic wastewaters. They are particularly attractive to developing countries because they are simple in concept and, if correctly designed, installed and maintained, require little attention from householders. However, present design guidelines disregard the influence of climatic conditions and surface vegetation on septic tank system performance. A combined field monitoring and numerical modelling study was undertaken to evaluate how these factors affect the hydrological performance of septic tank absorption trenches. The study revealed that a substantial proportion of water inflows to trenches is lost via evaporation and transpiration.

Keywords *- absorption trench; evaporation; numerical model; on-site soil absorption systems (OSAS); transpiration.*

1. INTRODUCTION

Septic tanks, the commonest type of on-site absorption systems (OSAS), are widely used to dispose of domestic wastewaters in rural and peri-urban areas that lack centralised wastewater collection systems. This is the case in both less developed and industrialised countries. In Australia, for example, around 800 000 households rely on septic tanks for wastewater disposal (Beal *et al.*, 2005). Provided they are well designed, well managed, and properly installed on an appropriate site, septic tank systems can be an effective, safe and relatively cheap means of disposing of household wastewaters.

For developing countries in particular, the design simplicity of septic tank systems is an important advantage. As shown in Figure 1, septic tank systems have only two main components, a settling or septic tank, and an absorption (dispersion) trench. Wastewaters flow by gravity to the septic tank where most solids settle out. The clarified wastewater overflows into a distribution box from where it passes to the absorption trench and thence into the surrounding soil. Some contaminant removal occurs in the trench itself but much takes place as the wastewaters disperse outwards and downwards through the soil.

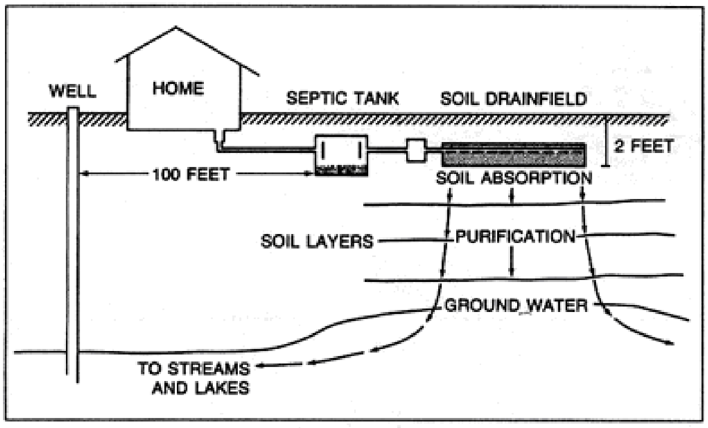


Figure 1. *A typical septic tank system*

In the tank the settled solids undergo anaerobic decomposition, with up to 80% of the solids breaking down over time. A slow build up of solids occurs, however, making it necessary for the tank contents to be pumped out every 3 to 5 years (Bitton, 2005). For the rest of the time the system operates passively, and once installed requires little attention and no external energy inputs. This feature, together with the very low operating costs, further enhances the potential suitability of septic tanks for use in developing countries.

Despite their merits, septic tanks also have a number of shortcomings. If they are poorly designed, overloaded, or installed in inappropriate locations, pollution of surface and ground waters can result, with potential risks to health. Concerns about septic tank failures are widespread in both developed and developing countries and a growing reluctance to install new septic tank systems is becoming evident. This situation has developed in spite of the fact that in most countries quite extensive guidelines on the design, installation and operation of septic tank systems are available. The main problem seems to be that these guidelines are based largely on experience and empirical rules of thumb and are not necessarily underpinned by good science. Where authorities are aware of the shortcomings of existing guidelines, large safety factors tend to be incorporated into local regulations in order to obviate failures, and so septic tank systems can end up being heavily overdesigned. Whilst such overdesign may not be that important in more affluent areas, poorer communities are much less able to afford the extra costs associated with overdesigned systems.

It was concerns about the appropriateness of regulations and guidelines applicable to septic tank installations that led to the study described in this paper. The aim of the study was to develop a better understanding of how septic tank systems work and how designs can be improved to reduce the likelihood of failures. Whilst the project focused on conditions on the outer fringes of Melbourne in Australia, its findings should be applicable in many parts of the world where septic tanks are used.

2. WHY DO SEPTIC TANKS FAIL?

One of the initial tasks in the study was to look at what causes septic tank systems to fail. Interviews with homeowners and local authorities indicated that both technical and non-technical problems contribute to poor septic tank performance.

2.1 Non-technical problems

On the non-technical side, it became clear that there was a good deal of complacency and ignorance about septic tanks. Prescribed installation procedures were often poorly observed and inspections of new installations by local authorities were sometimes quite perfunctory. In addition, householders often knew very little about septic tank processes and their management. This brought home the importance of providing sufficient funds for communication, education and training, of both local authority personnel and also householders, when on-site wastewater treatment systems like septic tanks are being installed. Whilst these findings relate specifically to Melbourne, we believe them to be of equal or even greater relevance to Third World communities.

2.2 Technical problems

The major technical problems encountered with septic tank systems relate to their design. Development of designs that are appropriate to a specific location requires a depth of understanding of the fundamentals underlying septic tank operation and the performance of dispersion trenches that we presently do not have. Guidelines in the form of codes and/or recommended loading rates exist for most countries but these tend to be rather general, with only limited cognisance taken of local conditions. Evidently, better ways are needed of predicting how septic tanks will perform in specific locations, and what the risks of system failure are.

According to the Australian/New Zealand Standard AS/NZS 1547:2000, failure (of a septic tank system) can be defined as an unsatisfactory performance of the system or an undesirable and unfavourable impact on the environment (Standards Australia and Standards New Zealand, 2000). (This is not particularly helpful since deciding whether or not failure has occurred tends to be based more on people’s perceptions than on quantitative criteria.) Two types of failures need to be considered. The most readily detected is *hydraulic failure*, which occurs when water flows into the septic tank dispersion trench faster than it can be absorbed by the surrounding soil, and the excess water rises to the soil surface. This water will often contain sufficient viable pathogens to create a significant health risk. The second type is *treatment failure*; this occurs when waters passing through the soil merge with ground waters or nearby surface waters before adequate removal of contaminants and pathogens has occurred.

Depending on the local situation, both types of failure can be of concern. In areas such as Melbourne, where only limited use is made of groundwater, avoiding hydraulic failures is probably the major concern of local authorities. Elsewhere in the world, especially in areas where water for domestic use comes mainly from sub-surface sources such as wells, treatment failures would be expected to be of much more importance; if wastewater disposal systems and water sources are too close together, contamination of water sources with pollutants and pathogens is likely, with potentially serious consequences for the health of local communities.

3. FIELD STUDY

Field investigations were undertaken at eight different septic tank sites around Melbourne over a one year period from November 2006 to November 2007. Soils on these sites were all of the silt or silt loam type. When choosing the study sites care was taken to ensure that: systems were properly installed and well maintained; hydraulic loading rates could be estimated with reasonable accuracy; and sites were flat and readily accessible. The fieldwork included: the collection of soil samples for classification and laboratory testing; in-situ testing to determine soil physical and hydraulic properties; a twelve month monitoring program to establish how moisture levels in the vicinity of the absorption trenches altered over a full annual cycle; and regular monitoring of ponding levels inside the trenches.

Nine 50mm diameter soil sampling holes were drilled at the upstream end of each trench (Figure 2). Two or three similar holes were also drilled close to each trench but in an area not expected to be affected by the dispersing wastewater; these holes were used as controls. Three PVC standpipes were installed along the centre line of each trench, as shown in Figure 2, to enable ponding levels inside each trench to be monitored. Samples of surface vegetation were collected and the major plant species present identified. Leaf Area Index (LAI) and root depth characteristics for each site were established. Full details of the field and laboratory programs, and all test results can be found in Jayarathne (2008).

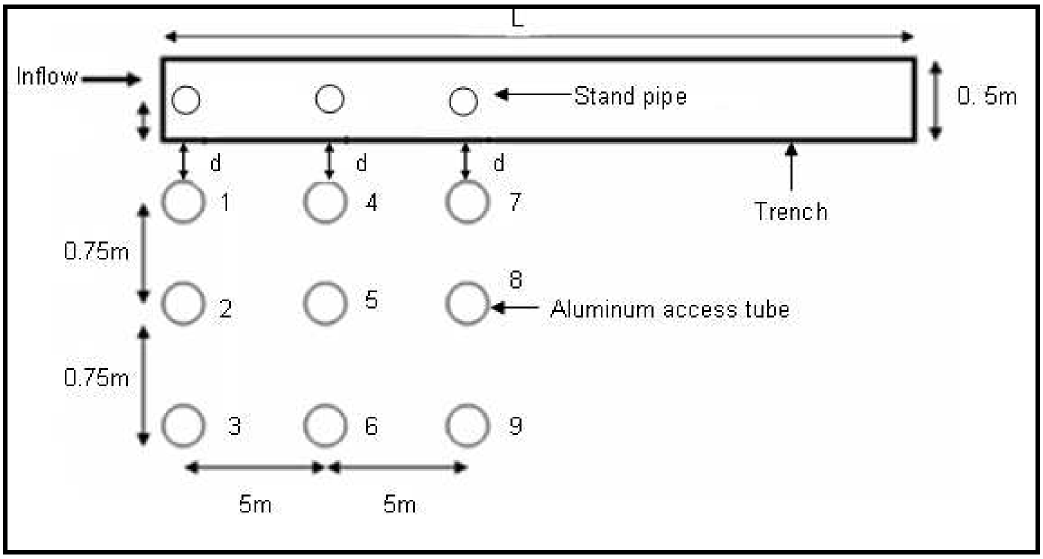


Figure 2: *Plan view showing soil moisture and ponding level monitoring points relative to each study trench (not to scale).*

4. THE HYDROLOGICAL MODEL

To complement the fieldwork component of the study, a dynamic hydrological model was developed to describe the behaviour of wastewaters dispersing into soils around septic tank trenches. Such a model would have the capacity to predict when hydraulic failure was to be expected and would also provide information helpful in assessing the risk of treatment failure occurring.

The model was constructed using the VADOSE/W module of the "GeoStudio 2004" software package developed by GEO-SLOPE International Ltd, Canada. This module uses a finite element approach to determine moisture flow patterns in soil as a function of both spatial position and time. It is able to take into account variations in soil type, soil horizons, surface vegetation parameters and climatic conditions. It was chosen specifically because of its ability to allow for variations in the latter two parameters as these have hitherto been largely neglected in septic tank studies. Figure 3 shows the various hydrological processes and features incorporated into the model. Details of how the model was developed as well as technical details about the model domain, finite element grid sizes, boundary conditions, and correlations used can be found in Jayarathne *et al*. (2007) and Jayarathne (2008).

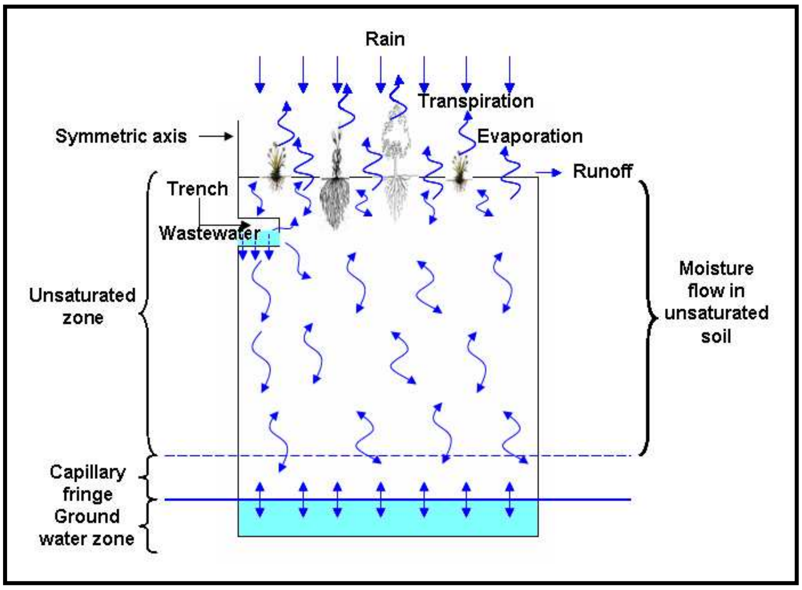


Figure 3.
The hydrological processes affecting moisture flows in soils around septic tank absorption trenches.

Preliminary evaluation of the model's capabilities was carried out using the published results from an earlier study at Mount Macedon, northwest of Melbourne (Brouwer and Bugeja, 1983). Inputs to the model included published soil property data from the above study together with data on local climate conditions during the study period. A vegetation-related boundary condition was applied to the top surface of the model domain. Soil moisture content profiles predicted by the model compared encouragingly well with those measured by Brouwer and Bugeja.

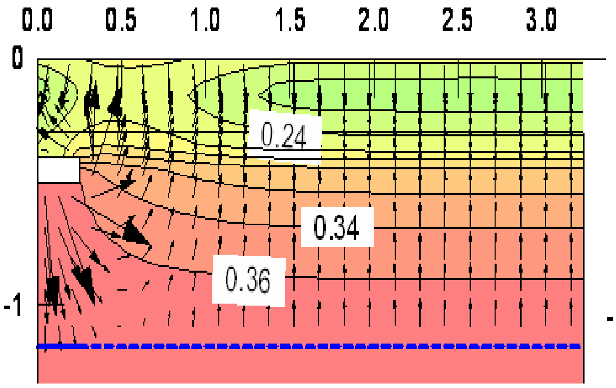
More extensive calibration of the model was then undertaken using soil property and hydrological data obtained from the eight septic tank installations in the greater Melbourne area referred to earlier. Hydrological information was collected monthly at each of these sites over a full year. A wide range of on-site and laboratory tests were undertaken on soil samples from each site to characterise soil properties.

Water balances calculated for each of the sites using the model showed that 38-65% of the total water entering the trenches (this includes rainfall as well as wastewater inflows) was lost by transpiration. This shows just how significant the uptake of water by surface vegetation from the absorption trench and surrounding areas can be. An additional 10-13% of the incoming water was lost by evaporation. This means that, according to the model, more than half the water received by the trench system is lost through evapotranspiration. The model was then used to compute water balances around trenches under climatic conditions typical of those at a number of locations across Australia. Again evapotranspiration was predicted to be a major, and often the predominant, water loss pathway.

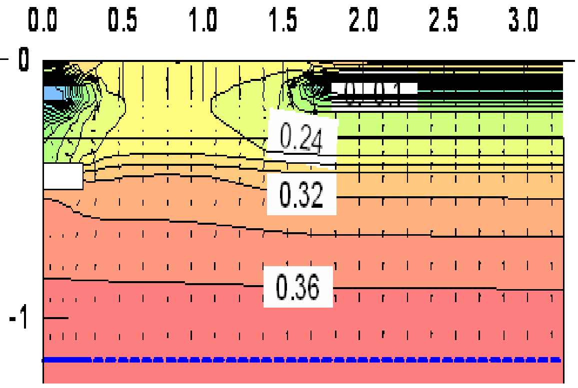
The above results were obtained for systems that were not excessively loaded and for which a significant layer of surface vegetation was present. In heavily loaded systems with few plants growing above the absorption trench losses by transpiration would be much lower, though this would in part be offset by higher evaporative losses. However, what makes the above findings so important is their demonstration that a substantial fraction of the flows entering an absorption trench can be lost to the atmosphere via evaporation and transpiration, something that is not properly accounted for in current design guidelines. The findings also imply that maintaining a clear well-vegetated surface above absorption trenches when trenches are heavily loaded reduces significantly the risk of hydraulic and treatment failure.

The model was then used to predict moisture dispersion patterns around septic tank trenches under a variety of soil, climate, vegetation, loading rate and other conditions. Of particular interest was the impact that groundwater table depths might have on soil moisture content and moisture dispersion patterns. Model runs were undertaken to establish how predicted moisture distribution patterns around a specific trench would be affected by changing the depth of the groundwater table from 4 m to 1.2 m (the minimum distance to the water table permitted in the Standard AS/NZS 1547:2000).

Figures 4 and 5 show model predictions for a silt loam soil with a saturated hydraulic conductivity (K_{sat}) of 0.003 m/day and a volumetric water content (VMC) at saturation of 0.36, conditions similar to those at one of the study sites. Climatic conditions typical of those used in Melbourne were applied and a typical surface vegetation cover was assumed. The left hand diagram in each Figure shows predicted patterns for the case when the trench was subjected to a hydraulic loading of 5 mm/day, the maximum permitted in the Australian/ New Zealand Standard AS/NZS 1547:2000 for the given soil type.

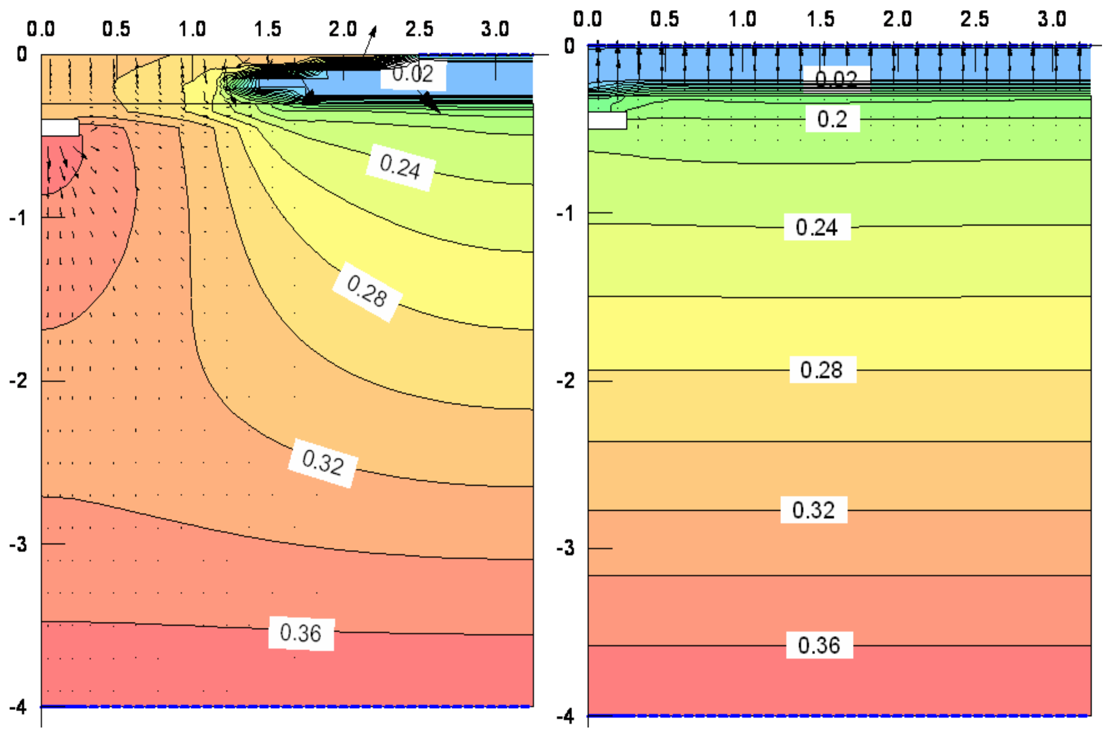


(a)



(b)

Figure 4. *Predicted soil moisture content distributions around an absorption trench for a groundwater table depth of 1.2 m and wastewater loading rates of: (a) 5mm/day; (b) 0 mm/day*



(a) (b)

Figure 5. *Predicted soil moisture content distributions around an absorption trench for a groundwater table depth of 4 m and wastewater loading rates of: (a) 5mm/day; (b) 0 mm/day*

The right hand diagram in each Figure shows predicted patterns for the case where no wastewater is supplied to the trench and the only water inflows are those due to rainfall. In all cases the top boundary of the diagram lies along the soil surface and the boundary at the left lies along a vertical centreline through the trench; the white rectangle adjacent to the left hand axis shows the position of the right half of the trench, which has its base 0.5 m below the ground surface. The right hand boundary of each diagram lies along a vertical line 3.5 metres from the trench centreline. The blue horizontal line shows the position of the groundwater table. Soil moisture content distributions are shown by a combination of labelled contour lines and shading – zones of saturated soil are present whenever the VMC > 0.36. In all cases the patterns shown are those predicted for the wettest day of the year (when the probability of failures is highest).

5. DISCUSSION

Figure 4 shows predicted moisture distribution patterns for the case where the groundwater table is 1.2m below the ground surface (effectively a worst case scenario). Comparison of Figures 4(a) and 4(b) shows how the introduction of wastewater into the trench causes moisture distribution patterns to change from those determined by climate and surface vegetation conditions alone. It is apparent that even with the water table as close as 1.2m to the ground surface, soil moisture contents above the level of the trench base stay well below saturation levels; hence hydraulic failure would not be expected. However, Figure 4(a) also shows that a saturated soil zone extends upwards from the groundwater table right to the base of the trench. This suggests that much of the wastewater dispersing from the trench will not pass through an unsaturated zone. Since passage of wastewater through unsaturated soil zones is known to help greatly in the inactivation of pathogens, the absence of such an unsaturated soil zone between the trench and the groundwater surface indicates an increased risk of treatment failure.

How moisture distribution patterns are affected when the distance to the groundwater table is increased to 4 m is shown in Figure 5. From Figure 5(a) it is evident that in this case there is no longer an uninterrupted zone of saturated soil connecting the trench to the groundwater below. Instead there is a 2.5m wide unsaturated zone, which ensures that all wastewaters will spend a substantial period in an unsaturated zone before entering the groundwater. This should markedly reduce the chances of treatment failures occurring.

6. CONCLUSIONS

The model developed during our study provides useful insights into what happens around and beneath septic tank absorption trenches. It showed that evapotranspiration can be the major water loss pathway from soils around absorption trenches. This highlights the value of keeping the ground above absorption trenches clear and of maintaining a good surface vegetation cover over the trench area. Use of the model also demonstrated that while groundwater depth limits in current guidelines and Standards may be adequate to prevent hydraulic failures they may not be appropriate where treatment failures can have serious consequences. Revision of guidelines to address the above findings is highly desirable, especially where there is heavy reliance on groundwater for potable use, as is the case in many developing countries.

Whilst the findings of our study relate primarily to conditions around Melbourne, we believe they have considerable relevance throughout the world wherever installation of septic tank systems is under consideration.

7. ACKNOWLEDGEMENTS

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ON-SITE CONSTRUCTION WASTE MANAGEMENT: ACTIVITY-BASED WASTE GENERATION

B. A. D. S. Wimalasena[1], J. Y. Ruwanpura [2], J. P. A. Hettiaratchi[3]

[1]PhD Candidate, Project Management Specialization, Department of Civil Engineering, University of Calgary, 2500 University Drive NW, Calgary AB, T2N 1N4, Canada.
[1]E-mail: swimalas@ucalgary.ca; sujeewaw@gmail.com
[1]Telephone: 1 403 220 4812; Fax: 1 403 282 7026

[2]Director and Associate Professor, Project Management Specialization, Department of Civil Engineering, Schulich School of Engineering, University of Calgary, 2500 University Drive NW, Calgary AB, T2N 1N4, Canada.
[2]E-mail: janaka@ucalgary.ca
[2]Telephone: 1 403 220 6892; Fax: 1 403 282 7026

[3]Professor, Centre for Environmental Engineering Research and Education (CEERE), Schulich School of Engineering, University of Calgary, 2500 University Drive NW, Calgary AB, T2N 1N4, Canada.
[3]E-mail: jhettiar@ucalgary.ca
[3]Telephone: 1 403 220 5503; Fax: 1 403 282 7026

Abstract: Sustainability of a construction project is often considered as a very important parameter in evaluating the success of the project. Though the management of material waste that generated as a result of on-site construction activities were initially regarded as less important to the overall sustainability of the project, current trends proves it otherwise. Leadership in Energy and Environmental Design (LEED) is one of those driving-forces that make the construction waste management an important sustainability indicator of a construction project. As a result of LEED as well as other assessment tools, construction waste management is very common in practice. However the effectiveness of the current management practices is questionable because prediction of waste material quantities is often neglected during the process. Therefore, to accomplish effective construction waste management (minimization, recycling, reuse, etc.) it is essential to predict quantities of construction waste which essentially depends on identifying the sources of waste generation and their relationships to quantity of waste.

This paper presents the findings of a current research work on prediction of construction waste based on activity based construction waste generation method. The proposed activity-based construction waste generation modeling facilitates material waste predictions using several parameters including activity specific factors, environmental factors, worker related factors, etc. Statistical model that Predicts the drywall waste generation was presented in this paper. The study was based on the work carried out at several building construction sites in Calgary, Alberta. The findings can be incorporated into a planning tool which can essentially be used for the construction waste management process at sites.

Keywords: Construction waste management, Planning, Prediction

1. Introduction

Sustainable development practices, which ensure societal and environmental advancements in addition to economic benefits, are well recognized and enforced by almost all municipalities/local governments in Canada if not across the globe. Being one of the largest business sectors, construction industry plays a significant role in providing social and economical development to the society. For instance, the Canadian construction sector contributes 5.95% of the GDP through employing over one million individuals [1]. Beside that, construction industry consumes large amounts of natural resources and generates large amounts of material wastes (the amount of material waste produced over the year 2000 being 11 million tonnes from the construction sector [2].

It is the reality that construction industry's profit margin is tight and that construction companies have to streamline their processes and activities in such a way to survive in the industry [3]. Because economic benefits are not usually revealed through implementation of waste management programs on-site, it seemed to be common that contractors give little consideration to waste management aspects of construction compared to meeting other targets and schedules [4]. Therefore, the most common solution for construction waste materials generated during construction was to deposit at landfills. At times construction waste materials were considered harmless for the environment, and therefore social and environmental acceptance for such practices were also evident [5]. However with the evolution of research in the area of solid waste management, and with global acceptance on sustainable construction principles, landfill disposal of construction waste materials is now considered the last available option in the waste management hierarchy (Figure 1).

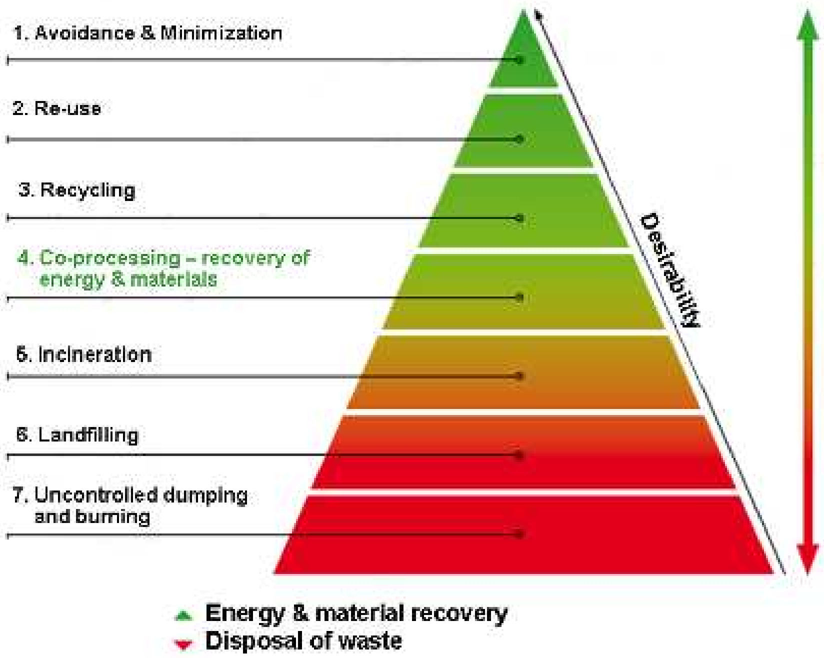

Figure 1: Waste Management Hierarchy

Although landfill disposal of construction waste materials is still the most preferred option for many construction companies, more than 75% of construction waste materials have the potential for reuse or recycling [6]. In fact sustainable rating systems such as Leadership in Energy and Environmental Design (LEED) encourage such practices in new building construction projects.

LEED rating system that launched in 2004 by LEED Canada provides relatively more comprehensive tool to evaluate the sustainability of a building, especially in the Canadian context. It recognises leading edge buildings that incorporate design, construction and operational practices that also ensure healthy, high-quality and high-performance in the process with reduced environmental impacts. Presently, LEED is one of the widely accepted sustainable building rating systems [7] that has been adopted by almost all construction companies in Canada. LEED measures sustainability of a building using five key areas under which credits are awarded for each sustainable practice recognized within the area. Major categories in the rating system are: Sustainable sites, Water efficiency, Energy and atmosphere, Materials and resources, and Indoor environmental quality. LEED has four performance ratings available as illustrated in Table 1.

Table 1: LEED Canada Performance Rating

Level	Points
Certified	26 – 32
Silver	33 – 38
Gold	39 – 51
Platinum	52 - 70

LEED recognizes the importance of on-site construction waste management within its rating system allocating maximum of 6 points dedicated for waste management from the materials and resources category. Further, it is noteworthy that construction waste management credits are the most common to obtain by the Canadian construction companies to attain their desired LEED certification [8].

Because Leadership in Energy and Environmental Design (LEED), one of the mostly accepted and widely recognized sustainable building rating systems [7], gives an impeccable place to on-site construction waste management within the rating system; there now exists a growing trend towards the implementation of sustainable waste management techniques. However, the economic viability of waste management programs has rarely been studied while a previous study confirms all such programs may not be delivering the sustainability goals [9].

2. Background: Construction waste, generation and management

This study defines construction waste as "waste materials produced in the process of construction of structures; the structures include both residential and non-residential buildings as well as roads and bridges". Building construction waste is the main focus of the study. Typically building construction waste stream consists of materials such as concrete, brick, wood, rubble, metals, drywall, cardboard, floor tiles and roofing materials.

Construction waste generation

The severity of the construction waste problem can be identified from the studies performed in different parts of the world on building waste material quantities [10, 11, and 12]. Skoyles (1976) identified thirty seven building materials of having material wastages from 2 to 15% of the designed amount of material [10]. 1-10% wastes from the purchased material quantities based on a study in Netherlands [11]. Another study based on the construction projects in Australia, indicates the material wastage to be 2.5-22% of the total material purchased [12]. Though the percentages of waste from construction materials are different from region to region, the important finding is that the quantity of construction waste generation is significant irrespective of the location. Evidently the type of construction, construction technology, and the rules and regulations imposed by the local authorities can have an impact on the material wastages indifferently. It is evident that the generation of construction waste is increasing over the years creating a series of problems in various regions in Canada. For instance in Alberta, one of the rapidly growing provinces in Canada has reported a 68.6% increase of construction and demolition waste generation over the period 2000-2006 [13] where, approximately one third of C&D waste is coming from new constructions in Alberta.

Significance of annual construction waste generation and its impact on the environment and the society as a whole has created a situation that encourages every builder to consider construction waste management seriously. It is the current trend to seek socially accountable building/construction practices from the industry. However for effective management should be preceded by planning and scheduling and to facilitate front end planning of the waste management process for a given construction project, it is essential and necessary to predict the waste quantities. To the astonishment, the studies that focus on construction waste management and cost effectiveness of the waste management programs do not include predictions.

3. Planning construction waste management

Prediction gives us an opportunity to see the future and plan events beforehand. Predictions can be based on experience or knowledge, but not always. Scientifically, prediction can be identified as a rigorous, often quantitative statement forecasting the future events under specific conditions. Prediction has become a challenging task because of the unavailability of construction waste quantity and quality related data in the industry [14]. Unavailability of data may be considered as a result of many reasons identified by previous researchers [14, 15, and 16] and could be listed as follows:

- not keeping construction waste records due to reasons such as not having or not adapting regulatory requirements
- not motivated to keep records or manage in any form because it has been considered as a non-value added task
- Considered as a potential trouble for other activities' progress

This paper focuses on prediction of construction waste using activity-based waste generation principle. Principle of activity-based waste generation assumes that total quantity of construction waste generated at a particular time in a construction site is the accumulation of waste quantities from each construction activity that is being executed at that moment. Therefore, prediction of total quantity of construction waste is possible only if each and every activity's waste generation can be predicted.

Factors of Waste Generation:

Prediction of construction waste quantities starts with identifying relationships with other measurable factors in the environment where waste is generated. More importantly, identification of causal relationships is the key to prediction. Construction being a highly labour intensive industry research on construction waste management should also consider on people's attitudes and behaviour as well. More importantly the labourers, foremen, leadhands and tradesmen who directly involve with the construction activities need to part of the study. Construction waste generation cannot narrow to the construction phase because recent findings confirm that causes of construction waste generation spans over almost all the stages of the project [3, 11, 17, and 18].

After extensive literature reviews and the pilot study which was carried out in a Calgary building construction site, the authors identified the factors identified in Table 2 as of important to waste generation and considered for further study aiming for the purpose of prediction of waste quantities. Further, it must be noted that these causes were of great interest specifically for the main focus which is drywall construction waste generation predictions. Some of the human and non-human factors that considered for the study were grouped to facilitate statistical inferences and the detailed procedure of factor grouping is available in Wimalasena et al. (2010) [19].

Table 2: Factors of waste generation

Non-Human Factors	Comfort Index (CI)	Working Temperature (C)
		Relative Humidity (%)
		Wind Speed(km/h)
		Precipitation(mm)
		Light Level(lux)
		Work Space(m^2)
		Distance to Material Store (m)
		Labour Hours (h)
		Work Quantity (m^2)
		Material Size Required/Material Size Ordered
Human Factors	Competency	Labour skill

		Adaptability to the Organization
		Adaptability to the Job site
		Knowledge about waste generation methods
	Satisfaction	Satisfaction over the method of communication (to receive instructions, etc.)
		Satisfaction over the Working hours

4. The proposed model

Developing waste generation prediction model was conducted using multiple regression analysis and the computation procedure includes the following main steps:

1. Calculate the correlations between different factors (Bivariate correlation analysis). The Statistical Package for Social Sciences (SPSS) was used to perform this analysis.
2. Select appropriate independent variables employing the backward elimination regression procedure and then the possible variable interactions were also considered.

The resultant model, waste generation function for drywall construction activity, which explains 71.5% of the variability (R square =0.715) of the dependant variable is given below:

$$QW = \beta_0 + \beta_1 WQ + \beta_2 LH + \beta_3 LL + \beta_4 CI + \beta_5 CI^2 + \beta_6 DM + \beta_7 SL + \beta_8 SLLH + \varepsilon_1$$

where, coefficients of the reduced model is shown in Table 3.

Table 3: Model Coefficients

Variable	**Variable Description**	**Coefficient Label**	**Coefficient Value**	**Significance (p value)**
Constant	-	β_0	330.035	0.086
WQ	Work Quantity	β_1	1.193	0.000
LH	Labour Hours	β_2	-50.508	0.007
LL	Light Level	β_3	0.033	0.000
CI	Comfort Index	β_4	-22.937	0.026
CI^2	Comfort $Index^2$	β_5	0.974	0.014
DM	Distance to Material store	β_6	-2.431	0.075
SL	Skill Level	β_7	-271.119	0.177
SLLH	Skill Level*Labour Hours	β_8	70.539	0.006

The fitted model implies that there is a positive impact of light level (p value < 0.0001), work quantity (p value < 0.0001) on the drywall waste quantity, and a negative impact of Distance to Material Store (p value = 0.075) on waste quantity after controlling for Labour skill Level and labour hours. However, there is a quadratic effect of CI (p value = 0.014) on waste quantity and interaction effect (p value = 0.006) of Labour Hours and Labour skill Level on the waste quantity.

ANOVA table (Table 4) tests the acceptability of the model from statistical perspective. It confirms that more than 71% of the variation of dependent variable is explained by the model. Because the significance of the F test is less than 0.05, the variation explained by the model is not due to chance. Therefore the ANOVA test confirms the model’s strength in explaining the variation of the dependent variable.

Table 4. ANOVA

Model	Sum of Squares	df	Mean Square	F	Significance
Regression	202166.507	8	25270.813	19.475	.000
Residual	80452.703	62	1297.624		
Total	282619.210	70			

5. Practical Applications of the model

There are two main practical implementations of the waste generation prediction model in building construction projects:

1. The model helps identifying the significantly correlated factors to quantity of waste generation from a construction activity. This is useful for developing material waste reduction strategies, as it enables focusing on important environmental factors.
2. Prediction model is an essential part of the on-site waste management planning process. A planning tool which can easily be integrated to a simulation model is useful for on-site waste management operations planning and even at the pre-planning stage

The following are applications of the prediction model for a planning tool:

1. Simulate the quantity of waste generation from construction activities accounting for the randomness of activities and dynamic nature in the representation.
2. Simulate the cost and benefits of the entire waste management process to identify the costs or benefits of practicing alternative waste management options, reuse, recycle and landfill disposal for all waste types. This will be helpful to determine the cost-effective waste management alternative for each material type.
3. Simulate waste material storing process to determine site space requirement for the waste management process. This will be an important finding to make specially when the construction site is located in a highly populated, congested area.
4. Simulate cost benefits of the process to determine the cost-effective hauling schedule

In order for a model to be successful in the construction industry, model requires to be easily learnt and used by a person without much simulation knowledge. Also the model must be able to change with the change of project as industry is dynamic by nature. It is necessary to accommodate already existing project information without further processing into such a planning tool to ensure it saves time and energy avoiding duplicate work.

6. Conclusions

On-site construction waste management is an important component of a construction project; thus plays a significant role in project's sustainability. However, success of the waste management program mostly relies on planning and scheduling as other construction activities do. The paper introduces a drywall waste prediction model based on a novel concept "activity-based construction waste generation principle" and based on the data collected from several building construction projects in Calgary, Alberta. The model which can easily be integrated into a planning tool will be useful for decision making at different stages of the project, construction operations as well as pre-planning stages. Other main applications of the model in building construction industry are also discussed in the paper. The proposed planning tool incorporating the prediction model, other project information and a simulation model which would be a ready-to-use tool for the construction industry is also included. This would be a useful tool to evaluate economic viability of the on-site waste management programs such as recycling, and reuse.

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About the Authors

B. A. D. S. WIMALASENA, B.Sc. Hons. Moratuwa., is a PhD Candidate at the Department of Civil Engineering, University of Calgary, Canada. His research interests are in the areas of sustainable construction, environmental consideration in built environments, and project planning and controls.

J. Y. RUWANPURA, PhD. Alberta., PQS (Canada), is an Associate Professor and Director of Project Management Specialization at the Department of Civil Engineering, University of Calgary, Canada. He also holds the Canada Research Chair in Project Management Systems. His research interests include: Integrated Risk Management and Simulation, Modeling tools, sustainability, technological advancements and productivity improvement.

J. P. A. HETTIARATCHI, M.Eng., PhD. Alberta., is a Professor at the Department of Civil Engineering, University of Calgary, Canada. His research interests are sustainable landfill technology, complex hydrocarbon contaminated sites remediation, green house gas emission technology development, optimization of landfill construction and operation, and recycling of construction waste.

ENHANCING SUSTAINABILITY OF CONSTRUCTION PROJECTS THROUGH WASTE MINIMIZATION

Parijat Naha[1], Yogendra Kumar[1], Koshy Varghese[2]

[1] M.Tech Student, Department of Civil Engineering, Indian Institute of Technology Madras, Chennai – 600036, India. E-mail: parijat.naha@gmail.com

[2] Professor, Department of Civil Engineering, Indian Institute of Technology Madras, Chennai – 600036, India. E-mail: koshy@iitm.ac.in

Abstract: Construction activity is a critical indicator of development. As developmental activities in emerging countries are increasing, the construction industry is being viewed with increased interest as an area which needs sustainable practices. The construction industry uses 55% of the wood cut for non-fuel uses and 40 % of the world's energy and a large proportion of the material manufactured can be traced to buildings and their construction. To make construction sustainable, all of its stages from conception to deconstruction need to be considered with the viewpoint of waste minimization. This paper looks at the execution stage of a construction project and seeks to identify the sources of waste generation at the planning and execution stage. A detailed field study and subsequent root cause analysis of the execution planning process has been done to identify the factors that lead to erratic and variable execution performance and thus create waste in the form of inventory and rework. An attempt has been made to study, categorize and quantify waste related to MEP works on building sites which typically account for about 40% of the project construction cost. The studies have incorporated expert views, interviews with on-site personnel, study of documents and actual field sampling. Based on the observations, multiple solution concepts has been proposed. The proposed solutions aim at reducing the generation of waste through better execution planning and control.

Keywords: Waste minimization, planning & execution, MEP

1. Introduction

The Conseil International du Batiment (CIB) defines sustainable construction as "… creating and operating a healthy built environment based on resource efficiency and ecological design." Huovila & Koskela (1998) identified energy efficiency, non-toxics, recyclability, preserving property value, flexibility, long service life, use of local resources, information dissemination, use of by-products and efficient mobility as the important sustainability criteria for the built environment.

The construction industry warrants a special focus from the standpoint of sustainability considering the nature of its material and energy inputs.. The construction industry uses 55% of the wood cut for non-fuel uses and 40 % of the world's energy and material consumption can be related to buildings and their construction. 54% of energy consumption in the U.S. is directly or indirectly related to buildings and their construction. Nearly one-quarter of all ozone-depleting chlorofluorocarbons (CFCs) are emitted by building air conditioners and the processes used to manufacture building materials. Also, construction and demolition waste has typically accounted for almost 65% of Hong Kong's and about 50% of UK's landfills at the peak of construction activity.

Construction waste is thus a major area of concern. The Indian construction industry is unique due to its labour intensive nature and consequent aversion towards mechanization and automation. Despite the booming size of the Indian construction industry, the issue of sustainability is still not being given due consideration.

The objective of the research study was to analyze at the project and execution planning process and its influence on the level of waste generation. This is based on the premise that poor planning and coordination results in poor procurement schedules, rework and improper waste management, which leads to the generation of waste. The study specifically focuses on Indian building sites to find the impact and root causes of planning inefficiencies. As MEP works typically account for about 40% of the project cost on modern building sites, the study also attempts to analyze, categorize and quantify

MEP waste on the projects. Based on the findings from the study, solution concepts that would help in making the construction process more sustainable are proposed.

2. Literature survey

Abundant literature exists in the field of construction and demolition (C&D) waste. Numerous studies have looked at quantifying and analyzing the nature of construction waste generated in construction in various parts of the world. Studies by Apotheker (1990), Craven et al (1994), Ferguson et al (1995) and Stokoe et al (1999) have quantified C&D waste in numerous countries and found it to be a significant impediment to sustainability. Concrete and mortar have been identified as the major components of C&D waste. Also, the presence of materials such as asbestos and Volatile Organic Compounds (VOCs) in C&D waste makes it extremely hazardous. Studies also document and suggest the various strategies that have been devised to manage the generation, handling and recycling of C&D waste. But relatively little work has been done to study the causes of waste generation and to quantify the details.

Bossink & Brouwers (1996) found that the amount of construction waste depends on the construction techniques employed, work procedures and common practices. They also identified the sources and causes of C&D waste which, among other factors, included (a) error in contract documents (b) lack of knowledge of construction methods (c) improper material handling (d) improper planning of material procurement and (e) damage caused by subsequent trades. Esin & Cosgun (2006) found that change in specifications due to change in end user requirements generated a significant amount of waste. This leads to the conclusion that poor project and execution planning contributes significantly towards C&D waste.

Over the years, research has attempted to analyze the problems and challenges associated with the project planning process. Collingridge et al (1994) predicted that technologies with high capital intensity, large unit size, long lead time, and high infrastructural requirements are susceptible to large schedule delays. Goldratt (1997) identified three basic reasons why even well-planned projects run into problems which are:

1. Activity level contingencies: Unduly large amount of safety built into individual activities
2. Student syndrome: Tendency to delay the application of peak effort to the last possible moment
3. Multitasking: Tendency to distribute available resources over multiple concomitant activities

Xiao & Proverbs (2002) evaluated and compared the construction time performance of construction contractors in Japan, the United States of America and the United Kingdom. They concluded that while Japanese construction contractors achieved superior levels of time performance through detailed planning & by working more closely with subcontractors, contractors in the US and UK suffered because of less intensive schedule planning & monitoring, adversarial relationships & lack of pragmatic thinking. Kar (2009) reports lack of advance planning, a holistic approach, inconsistency in monitoring and follow-up, coordination and communication lapses and absence of a methodical approach as major causes of project failures in developing countries.

Koskela (1992) proposed new concept of waste as incident of material losses and the execution of unnecessary works, which generate additional costs but do not add value to the product. Only processing activities were assumed as value adding activities. Hence target for continuous improvement can be achieved by eliminating / reducing the share of non value adding activities and increasing the efficiency of value adding activities.

The most classical waste categorization according to lean production philosophy was given by Ohno (1988) which has been quoted by Formoso et al. (1999) and Koskela (2000). Following is the 7 wastes proposed by Ohno, of which the first five refer to flow of material and the last two is due to work of men: (a) Waste of overproduction (b) Waste of correction (c) Waste of material movement (d) Waste of processing (e) Waste of inventory (f) Waste of waiting (g) Waste of motion.

Instead of classifying waste of productive time, Serpell et al. (1995) have classified these wastes in relation to work categories. There are three types of work categories as proposed:

1. Productive work (value-adding activities)
2. Contributory work (non value-adding activities but essential for conversion process)
3. Non-contributory work (non value-adding activities)

Result of research studies carried out on Brazilian building construction projects by Formoso et al. (2002) indicates that the amount of material waste is very high and there is a large variability in waste incident across different projects. Research conducted in Netherland by Bossink and Brouwers (1998) indicated material waste in range of 1 - 10% in weight of the purchased amount of materials. The main causes of waste were identified as upstream process, such as design and material supply, as well as poor handling of materials in transportation and storage. Garas et al. (2001) conducted research in the Egyptian construction industry to find out the causes of the waste present. The study indicated that late information and changes to design were the most fundamental causes of material waste.

A general frame work for construction improvement and waste reduction was developed by Serpell and Alarcon (1998) which has been successfully applied to several construction sites in Chile. An analysis of the relationship between buffers (inventory) and construction labour performance done by Horman and Thomos (2005) in Brazil shows that some buffers helps in achieving the best labour performance in construction operations.

Although past work has addressed the issue of waste due to inadequate planning, no formal study has identified the factors and investigated root causes relevant to the Indian industry and quantified the levels of waste.

3. Methodology

The data for the study to identify the factors related to inadequate planning in the Indian construction industry was collected through a variety of methods. Site visits were made to multiple Indian building projects in various stages of completion. Interviews and meetings were held with the project managers, planning staff, section in-charges and execution engineers of the sites. Foremen, supervisors and workers were interviewed to get a clear perspective of project execution at the operational level. Project BoQs and contract documents were studied. The project planning and execution planning documents like construction schedules, monthly reports, Daily Progress Reports, Minutes of Meetings, catch-up schedules, method statements, clearance certificates and other formats were studied to understand the existent project planning, control and monitoring mechanisms. The observations were then collated and a root cause analysis was done.
A model to quantify the levels of waste was developed with the help of literature survey. Empirical formulae were used to measure the cost of waste due to labour inefficiency, material scrap and excess inventory. Tour based work sampling (Liou and Borcherding 1985) was conducted to get the problem areas related to workers inefficiencies. Crew work sampling was then done to get to know the actual problems associated with those areas. All activities were listed and categorized in three categories as value added, non-value added and non-value added but required. Finally modified work sampling was done for all processes to measure time spent by labourers in these work categories. Store records like material request forms, material receipt & issue details, inventory ledgers were and cost statements were studied to analyze the material handling and management at the sites.

Based on the studies conducted, a techno managerial solution concept was developed. The Theory of Constraints was used to develop a new execution planning logic. A simulation model was developed in STROBOSCOPE using Microsoft Visio, to replicate FPS process as practiced at various sites. This model was tested using work sampling data collected at site and through questionnaires. Four ways to minimize waste were suggested and implemented through simulation and results were discussed.

4. Study of the planning process

4.1 Definitions

It is worthwhile to define the terms project planning and execution planning at this stage. These are explained below.

1. ***Project planning***
   a. Focuses on overall scheduling for a project, to meet broad targets of cost, time, quality and safety
   b. Considers only precedence relationships, as resource and information availability data may be both unavailable and irrelevant at this stage
   c. Is relevant at senior management level
   d. Has limitations with respect to execution planning
2. ***Execution planning***
   a. Focuses on scheduling at activity level, to provide a realistic execution plan
   b. Should consider both precedence and resource/information constraints
   c. Needs to study and factor actual field conditions
   d. Is more relevant to the operational level task force

Ideally, a robust execution plan depends on, and is to be derived from a robust project plan. While the project plan provides the broad framework for project execution, the execution plan uses this framework to work out the details of execution and presents an executable plan to the field level operational force, taking into account all realtime project constraints and conditions. So, there should be a tight correspondence between a project plan and an execution plan.

4.2 Observations

4.2.1 The execution planning process

Based on the observations made during the course of the present study, the problems in the execution planning process were identified (Table 1).T As a result of these problems, the planning process tends to be more reactive than proactive, this frequently leads to cost and time overruns. Thus, there is a large amount of variability in the construction process which is not factored into planning. This variability can only be reduced if a structured planning process with a sound tracking and feedback structure is implemented.

Table 1: Problems existent in the Execution Planning Process

Problem	Elucidation
Inadequately detailed project master schedule	• Too broad • No logical precedence linking
Insufficient schedule interaction	Inconsistent interaction between Master Construction Schedule (MCS) and package schedules
Discontinuous schedule updation	• Deviations with project progress, the large number of activities and the complex interrelationships. • Lack of manpower trained to work on packages like MSP and Primavera
Inadequate detailing of execution plan	• Only dates, no quantities • Co-ordination aspects not considered • Inaccurate duration estimation
Insufficiently developed coordination planning mechanism	• Critical for finishing and services • Co-ordination drawings prepared to resolve spatial conflicts • The sequence in which various trades have to enter and work in an area is done through verbal meetings
Inadequate project monitoring systems wrt activities	Cost implications of activity crashing and delays due to coordination problems, material unavailability etc. not captured easily.

Apart from the flawed planning process, problems were also found during execution. Interviews to execution engineer, foremen and workers gave insights to some problems associated with execution which were responsible for delayed project completion. These problems were playing crucial role in variable project duration. Some of the problems associated with execution, observed during work sampling are listed in Table 2.

Table 2: Observed Problems in the Execution Process

Problem	Manifestation
Design Problems	• Lack of co-ordination b/w structural and services drawings • Frequent Changes in design during construction • Delay due to drawings / finalization of interiors • Level difference due to mistake in drawing / execution
Execution Problems	• Co-ordination with other services • Co-operation of specialized agency to the engineer. • Damages to the fixtures holes due to civil works (plastering) • Clearance from other departments (present at all sites) • Use of superseded drawings • Space constraints • Structural accuracy of constructed structure
Material Problems	Material procurement, Storage and Shifting
Manpower Problems	Lack of skilled Manpower & Contractor

Based on the findings of the field study, a root cause analysis of the execution planning mechanism in Indian construction projects was done the results are presented in Figure 1 in the form of a fishbone diagram.

A more detailed study on quantifying the waste due to MEP processes was then carried out. It was found that many of the issues faced in executing MEP was similar to general construction.

5. Assessment and quantification of MEP waste

Execution problems associated with manpower was assessed by conducting work sampling. Activities performed by workers were categorized under Value Added, Non-value Added and Non-value Added but Required. Modified Crew Work Sampling (Liou & Borcherding 1985) was done to assess worker efficiency for all major processes of MEP.

Waste was measured for labour and material in terms of cost using empirical formulae. Cost of labour inefficiency represents the costs spend on labourers for their NVA and NVAR works. Material waste is a measure of cost of material scrap and cost of excess inventories. Cost of excess inventories is the loss of opportunities of interests on the investments which are kept excess in store.

Figure 2 represents the waste in percentage of total project cost

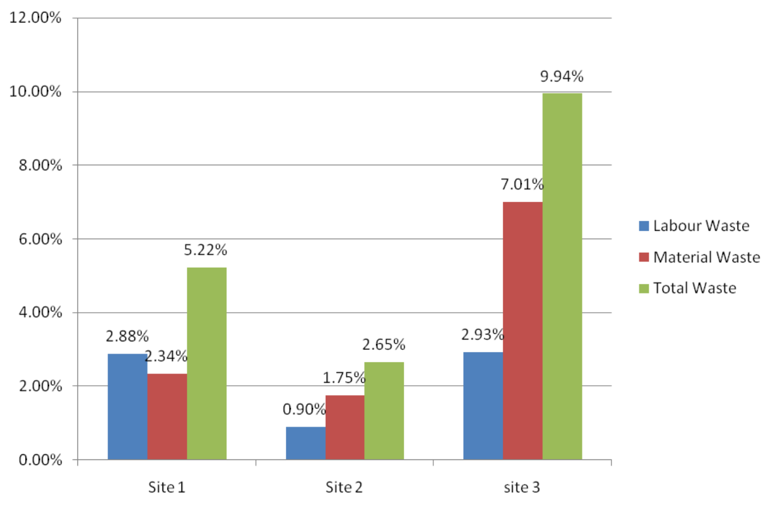

Figure 2: MEP Waste as percentage of Project Cost

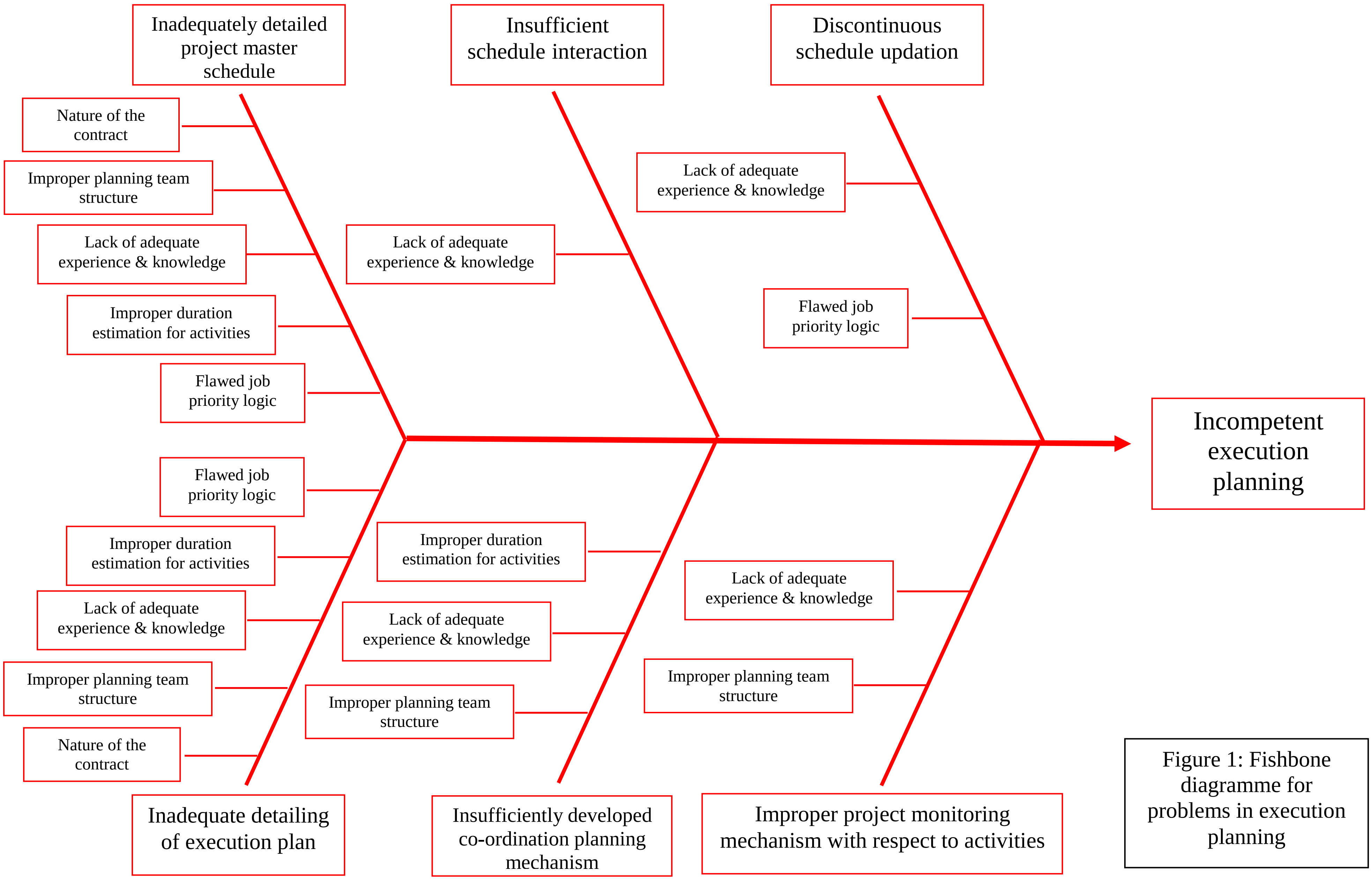

Figure 1: Fishbone diagramme for problems in execution planning

Though MEP execution is not labour intensive, cost of labour inefficiency was almost 3% of the project cost which is alarming. Direct cost may be small but improving productivity could decrease project duration marginally and would have more benefits. Large portion of material waste was found due to excess inventory at site. Excess inventory was attributed to the fact that material was procured in bulk if it was to be imported, or to achieve price advantage from the vendor.

6. Root Cause Analysis

As the study analyzed and concluded, the major reasons directly or indirectly causing these improprieties can be categorized as shown in Table 3.

Table 3: Roots Causes behind Problems in the Execution Planning Process

Root cause	Elucidation
The nature of the contract	• LSTK or BOT contracts – milestone based • This is reflected in the detailing of the MCS
Improper planning team structure	• Top-down scheduling • Inputs from site staff not fully utilized
Lack of adequate knowledge experience	• Significant problem for finishing and services • Use of thumb rules – risky approach
Improper duration estimation for activities	• Lack of experience, absence of benchmarks • Imposition of forced completion dates
Flawed job priority logic	• Traditional emphasis on progress reporting • Commercial considerations override technical considerations • Client imposed handover dates

While doing work sampling many issues were observed which were causing inefficiency to workmen. These problems were analyzed in detail to get the root causes. Table 3 represents the root cause analysis of execution problems observed.

Table 4: Root Causes behind Observed Execution Problems

Problem	Root Cause
Idle and Waiting Time	• Inappropriate crew size • Poor assignment of work • Unavailability of interdependent team mates
No contact	• Lengthy process of material issue • Labour used for arranging snacks
Poor housekeeping	• Lack of control of site engineers over workers • Lack of knowledge towards ill effects of poor housekeeping
Late Start of work	• Lack of management commitment
Same crew working at different levels	• Competition among crew to get more work • Management wish to allot similar works
Lack of co-ordination	• Poor scheduling • Unavailability of all details from beginning

7. Results and Discussion

The field studies conducted in some building construction sites in India helped understand the process of execution planning and its influence on sustainable practices. The study to assess and quantify of MEP waste found that the total MEP waste was 9.94% of the total Project Cost, which is a significant and alarming fraction. The root causes behind the existent problems with project execution were also identified.

This study helps us to identify the role of proper planning at the project and execution level in determining the sustainability of construction projects. Thus, this issue needs to be approached in significant detail. Several solution approaches are being considered by the authors. The concept of Critical Chain Project Management has been used to design a new execution planning logic that utilizes realtime field data to come up with realistic execution plans. The potential of Building Information Modelling is being utilized in constructing 4-D simulations of structures which help in better visualization at the planning stage and thus more effective and less variable execution. Simulation packages like Stroboscope have been used to simulate scenarios with improved construction practices with respect to labour and material utilization. Assuming that all the levels of the organization are ready to internalize and utilize the concepts, these can be very effective tools in reducing the levels of waste and making the construction process more sustainable.

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About the Authors

PARIJAT NAHA, is an M.Tech in Construction Management from the Indian Institute of Technology Madras.

YOGENDRA KUMAR, is an M.Tech in Construction Management from the Indian Institute of Technology Madras.

KOSHY VARGHESE, Ph.D., University of Texas - Austin, is a Professor at the Department of Civil Engineering, Indian Institute of Technology Madras. His research interests are construction project management and computer integrated construction.

TOTAL WASTE MANAGEMENT USING CLEANER PRODUCTION AND INDUSTRIAL ECOLOGY PRINCIPLES IN SRI LANKAN HOTEL SECTOR

*A.M.G.E Weerawansa[1], W.M.S.M. Samaraweera[1], A.K. Kulatunga[1], M.T.R Dissanayake[2]

[1]Dept. of Production Engineering, University of Peradeniya
[1]E-mail:*gayan.weerawansha@gmail.com[1], sumith.samaraweera@gmail.com[1], aselakk@pdn.ac.lk[1]
+94716619146, +94772399079, +94718222535

[2]Institute of Post Harvest Technology, Anuradhapura
[2]mrdisanayake@yahoo.com

Abstract: Hotel sector is one of the main revenue earners in current Sri Lankan economy. This sector is rapidly growing post war period. Since tourists arrive Sri Lanka comes for diverse reasons hotels are located in all around the country to attracting these tourist groups and mostly this industry is located adjacent to environmentally sensitive places such as virgin forests, beaches and archeologically significant locations. Due to this number of protests and demonstrations also happened in certain places for opposing this industry expecting that there would be environmental damages sometime ago. Therefore, proper environmental management practices are of paramount importance. Waste disposal is one major area which needs to be addressed first and foremost.

Waste generated in this industry can be divided into solid waste and waste water. Furthermore, solid waste can be divided into biodegradable and non degradable waste. Waste water is generated from the laundry, kitchen, toilet /bathroom cleaning etc. Some of the waste categories depend on the occupancy of the hotels and some depend on the reception functions which are held in hotels (ex. Kitchen waste generated after wedding functions etc). However, it is impossible to minimize this type of waste generation or to educate the guests on waste generated issues due to the nature of the industry and high competition in this sector. Therefore, best possible options remains are to have proper waste management system to run this industry in sustainable manner. If there is a way to reuse waste generated then there will not be much impact to the industry. Conversely Cleaner Production (CP) and Industrial Ecology (IE) principles are being used successfully in manufacturing sector for many years to address the waste generated in those industries.

Hence in this research these two concepts are used in local hotel sector to manage waste generated. There are number of ways waste is reused in productive manner. Out of them energy harnessing from biogas generators are significant. The biogas generator which converts all biodegradable waste generated to useful flammable biogas Methane (CH_4). This gas is used completely as a fuel to pre-heat water which is used in steam boiler for laundry purposes thereby reduces diesel consumption for boiler firing considerably. Only percentage of waste water is used for biogas generation, rest of the wastewater and wastewater mixed with chemicals are treated in treatment plant and treated water is used for gardening purposes and organic farming and some studies are carried out to use them for water fountains etc. Furthermore, non-degradable wastes are segregated at the point of generation and sell them for recycling purposes. A pilot project is carrying out in one of the leading hotels in Kandy and preliminary studies were done with already established hotel which has bio gas generator in Uva province. Results reveals that shows that diesel and LP gas consumption can be reduced considerably.

Keywords: *Wastewater, solid waste, Cleaner Production, Industrial Ecology, bio gas, organic farming*

1. Introduction

Sri Lanka has all of a sudden become one of the safest tourists destinations post war era. To support this fact, many western countries which previously imposed travel restrictions have lifted. Even US times named Sri Lanka as the best place to travel in the recent past. Conversely, many popular destinations of other countries such as Indonesia (bali), Thailand (Puket), India etc. suffering from unrests and terrorists activities. This creates overwhelming advantage over the other regional tourist destinations. These geo-political issues leads to arrive large number of tourists in the first half of the year and it is expected that this year the foreign tourist arrival will hit record 700,000 figure for the first time. The Sri Lankan government expects to increase room capacity to 16,000 by year 2012 where present capacity is around 6,000. Due to these expansions, many damages can happen to the environment since most of the hotels put up in the vicinities of beaches, forest reservations or archeological sites. In addition during the operations many environmental impacts can happen (waste generation from the hotels, Green house gases emissions etc.). Therefore, proper environmental management practices are paramount importance. Waste disposal is one major area which needs to be address first and foremost.

Waste generated in hotel sector can be divided into solid waste and wastewater. Furthermore, solid waste can be divided into biodegradable and non degradable waste. Waste water is generated from the laundry, kitchen, toilet /bathroom cleaning etc. Some of the waste categories depend on the occupancy of the hotels and some depend on the reception functions which are held in hotels (ex. Kitchen waste generated after wedding functions etc). However, it is impossible to minimize this type of waste generation or to educate the guests on waste generation due to excessive usage food due to the nature of the industry. Therefore, best possible options remains are to have proper waste management system to drive this industry in sustainable manner. If there is a way to reuse waste generated then there will not be much impact to the industry. Conversely Cleaner Production (CP) and Industrial Ecology (IE) principles are being used successfully in manufacturing sector for many years to address the waste generated in those industries.

Therefore, in this research we proposed hotel ecological cycle based on CP and IE concepts to convert solid waste into useful products. These two concepts already proven that business performance can be enhance by practicing them [1]. This cycle uses already discovered technologies yet the concepts can be used to operate the hotel sector more environment friendly atmosphere. In developed countries already proven that good environmental practices helps to increase the hotels performance [2]. Therefore this concept is useful to operate this industry in sustainable manner in the long run. There are number of ways waste is reused in productive manner. Out of them energy harnessing from biogas generators are significant. The biogas generator which converts all biodegradable waste generated to useful flammable biogas Methane (CH_4). There are many types of biogas generators. Depends on the feeding frequency and operating atmospheric temperature, type varies [3]. The gas generated can be used completely as a fuel to pre-heat water which is used in steam boiler or even for cooking purposes. Only percentage of waste water is used for biogas generation, rest of the wastewater and wastewater mixed with chemicals are treated in treatment plant and treated water is used for gardening purposes and organic farming. Furthermore, non-degradable wastes are segregated at the point of generation and sell them for recycling purposes. Furthermore, non-degradable wastes are segregated at the point of generation and sell them for recycling purposes.

Rest of the paper is arranged as follows; in section 2, overview of biogas plants is given. In section, three the proposed hospitality industry cycle is presented. This is followed by a case study and finally the conclusion and discussion.

2. Anaerobic Digestion

Anaerobic digestion is a series of processes in which microorganisms break down biodegradable material in the absence of oxygen, used for industrial or domestic purposes to manage waste and/or to release energy. It is widely used as part of the process to treat wastewater. As part of an integrated waste management system, anaerobic digestion reduces the emission of landfill gas into the atmosphere. Anaerobic digestion is widely used as a renewable energy source because the process produces a methane and carbon dioxide rich biogas suitable for energy production, helping to replace fossil fuels. The nutrient-rich digestive which is also produced can be used as fertilizer. The digestion process begins with bacterial hydrolysis of the input materials in order to break down insoluble organic polymers such as carbohydrates and make them available for other bacteria. Acidogenic bacteria then convert the sugars and amino acids into carbon dioxide, hydrogen, ammonia, and organic acids. Acetogenic bacteria then convert these resulting organic acids into acetic acid, along with additional ammonia, hydrogen, and carbon dioxide. Finally, methanogens convert these products to methane and carbon dioxide. Anaerobic digestion is particularly suited to organic material and is commonly used for effluent and sewage treatment. Anaerobic digestion is a simple process that can greatly reduce the amount of organic matter which might otherwise be destined to be dumped at sea, land filled or burnt in an incinerator. Almost any organic material can be processed with anaerobic digestion. This includes biodegradable waste materials such as waste paper, grass clippings, leftover food, sewage and animal waste. In developing countries simple home and farm-based anaerobic digestion systems offer the potential for cheap, low-cost energy for cooking, lighting, heat generation and electricity generation. The conversion process is given in Figure 1.

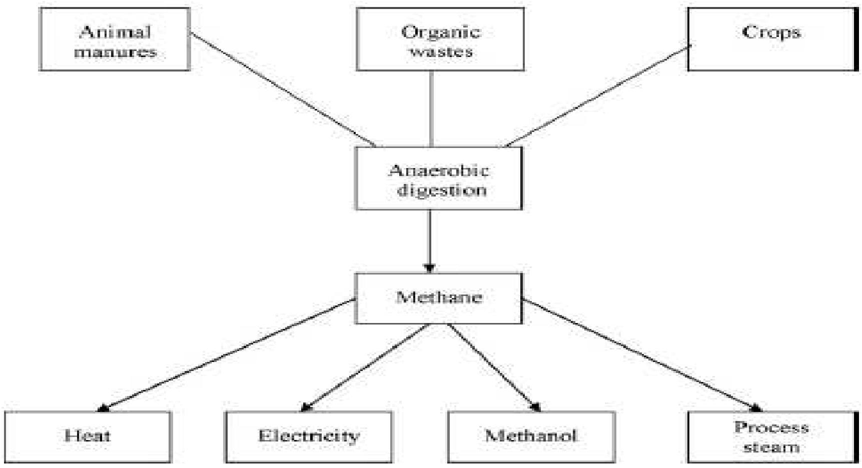

Figure 1: *Process of methane generation [4]*

3. Methodology – Hotel eco cycle

There are number of methods already adapted in many industries to manage waste generated in their own industries. Early days everybody tried to dump waste wherever possible. However due environmental pollution related issues, due to rules are regulations industrialists had to stop discharging wastewater and even dumping solid waste to environment and began to start waste management. During this era end of pipe wastewater treatment plants became famous to handle wastewater and compost fertilizer processing was the solution or to dump solid waste and covered them with soil were the methods practiced. However, when resources used in the industries became scar especially the water, fuel and energy, many industries realized that they are using the resources inefficiently. In order to address this Cleaner production (CP) tool was developed in early 1990s. Instead of treating the waste at the end of the pipe, CP tries to reach the point of waste generation and to address problem with the view of minimizing waste generation. In addition, due to same resource scarcity situation, Industrial Ecology concept became famous among the cluster of industries to reuse the waste generated at one industry as raw material in anther industry. In the proposed hotel-ecology cycle, this research focused to used both CP and IE concepts together for the single industry: the hotel sector since many hotels at the moment experiencing high operational costs due to high fuel/electricity and food items prices from one side and environmental management problems due to higher waste quantities generated daily in solid and liquid from the other side.

In the first phase of the proposed cycle, main waste streams are identified by walk through assessments and material balancing. Here in hotel sector, and main waste streams are coming from restaurants and kitchen. Higher amount of wastewater is generating from cleaning and housekeeping operations. In the second phase, waste types are segregated into different categories and later addressed each category separately. In the third phase bio-degradable waste is used for useful energy generation purposes through anaerobic digesting process and biogas is harnessed and thereby energy requirements of the hotel can be reduced certain percentage. In addition, there will be compost fertilizer as by product from bio-gas plants which can be used for gardening and organic farming. Furthermore, another waste generated from hotels kitchen: the fried cooking oil is used to mix with boiler fuel after filtering and used after blending and this also helps to dispose waste and to reduce the energy bill even at small scale. In the fourth phase, untreatable and unrecyclable waste at the location is segregated and stored at controlled conditions thereby they can be easily sell to potential buyers. These waste groups include solid non-degradable wastes such as empty bottles, waste papers etc. Infect, waste papers also can be recycled and used for rapping and other requirements which will attract foreign tourists and this can be used as marketing purposes as a sustainable eco friendly organization. The wastewater as usual treated by treatment plant, however, the wastewater load to be treated will be less since waste streams are minimized at the source of generation and some of the solid waste mixing with water. Ultimately this will minimize energy requirements to operate wastewater treatment plant. Furthermore, treated water is used for farming and gardening purposes thereby to minimize fresh water usage. The proposed Hotel-ecology cycle is presented in Figure 2.

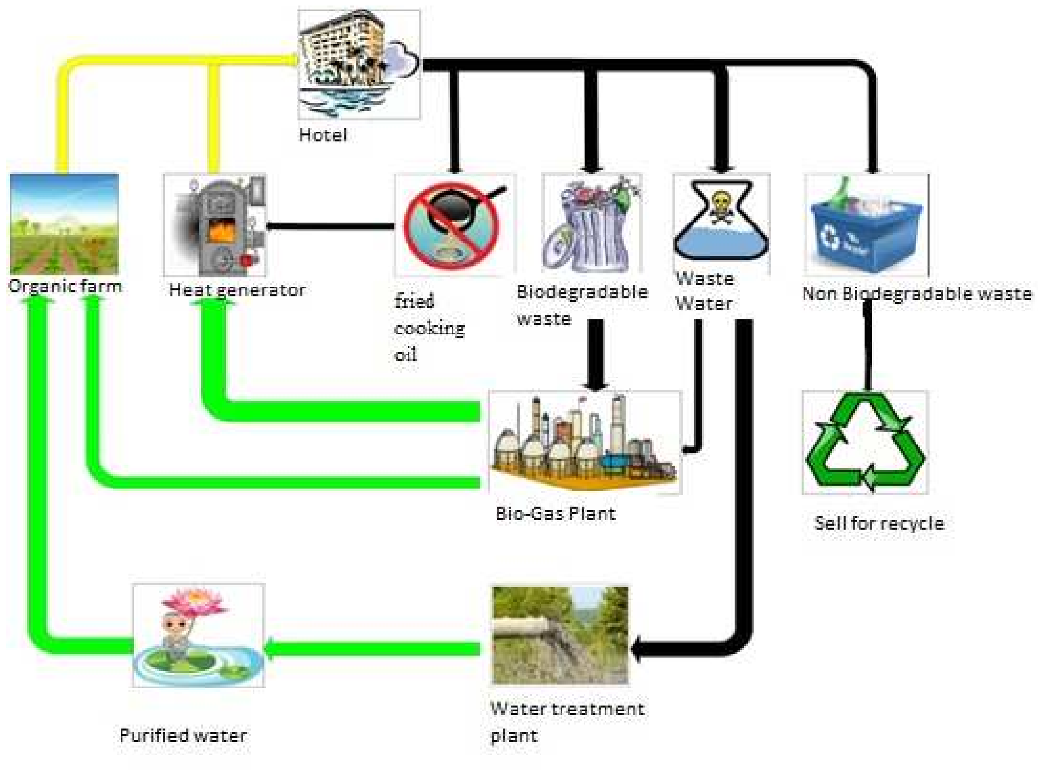

Figure 2: – *Hotel Eco Cycle*

4. Case Study

A pilot project was carried out at a hotel maintained by Uva Provincial council situated at the beneath of Namunukula mountains near to palgahatenna town. This hotel consists of 200 numbers of rooms and a garden with small area for organic farming. As our hotel-ecology cycle proposes, initially waste steams were identified and tried to minimize at the point of generation and latter kitchen and restaurants bio-degradable waste based bio gas plant was put up with the technical assistance from Uva management Development and training Institute of Uva provincial council. In order to operational difficulties fixed dome type biogas plant was designed to match with average daily bio degradable solid waste of 50kgs which accounts from approximately 100 plates for breakfast, 200 plates for lunch and 150 plates for dinner. Main advantage is of this type of bio gas plants is that waste can be fed daily basis and digested sludge also can be collected regularly. Basically it has one dome and it is separated to the two partitions by a wall. And one side is open to the inlet and other side is open to the outlet. The generated bio-gas also contain in the same dome. It is very easy to maintain that kind of bio-gas plant with very low operational cost. Since there is no separate tank to collect biogas, pressure of the dome has to be maintained at safety limits. In order to facilitate this, separate pressure relief valve is connected with auto igniting burner since the main constituent of biogas CH4, is a green house gas.

In order to minimize waste load on wastewater treatment plant and to get more inputs to convert useful biogas, sewage system of the hotel also connected to the biogas plant which facilitated solid waste – water ratio to maintain anaerobic reaction efficiently. The figure 3 shows the schematic of the fix dome type bio gas plant.

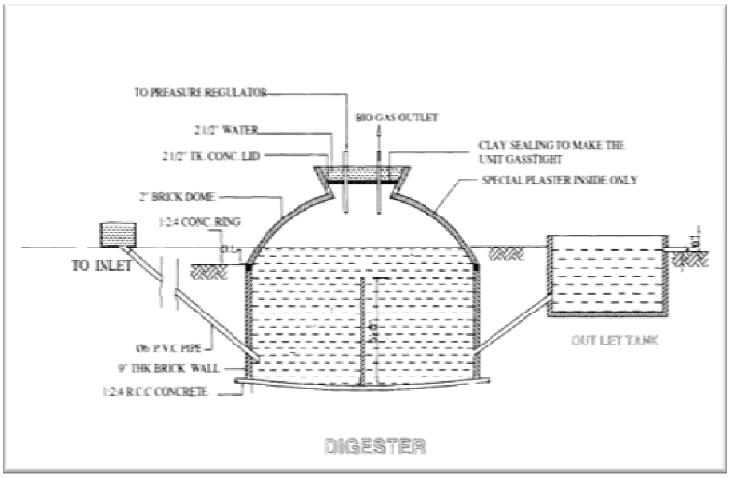

Figure 3: *Schematic of the Chinese type fix dome bio gas plant* [5]

The Figure 4 and Figure 5 show the solid waste handling before and after the pilot project. Before the project all the waste dumped together, however after the projects solid waste is segregated as much as possible. Therefore unusable wastes such as bottles, plastics etc. can be easily sell to interested parties.

Figure 4: *Display of the bio gas plant in the hotel*

Figure 5: *Solid waste dumps before the project*

Figure 6: *Waste segregation after the project*

The biogas rector has the capacity of 40m^3 and daily input of solid waste is 50kgs and water content is 40l, after 15 to 21 days of initial commissioning biogas generation was started slowly currently per day around 1 m^3 is harnessed and savings from LP gas for cooking is 12.5 kg cylinder within every two days. The design calculations and breakeven analysis of the biogas rector is given in the appendix and the specific details of the biogas plant and cost for the project, daily harnessing and savings are given in Table 3. The other saving aspects of the pilot project are not presented in this study though the different phases of the proposed cycle are adapted. The second pilot project is already started in medium scale hotel in Kandy and preliminary studies shows that this project also will deliver good results.

Table 3: *Specific details of the biogas rector*

Capacity	$40m^3$
Type	Chinese fix dome
Daly waste input	40-50 kg
Daily output Biogas	$1.5m^3$
Saving per day	1/4 to1/2 LP gas cylinder
Daily fertilizer gain	15-20 Kg

Figure 9: *Hotel biogas plant and the Natural beauty after project*

5. Conclusion

This paper presented an environmental friendly self sustainable hotel ecology cycle to minimize waste generated and to use waste generated into useful products such as energy and fertilizer. This cycle was based on the two popularly used environmentally friendly concepts of CP and IE which are currently practiced in manufacturing sector extensively. The main outcome of the project was the design and development of biogas rector which transform bio degradable waste into biogas which can be used for cooking purposes. Results reveal that from daily waste of 50 kgs the hotel can save up to Rs 35000/- per month and will get dry compost fertilizer of 30-35 kgs per month.

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7. Appendix

Feasibility Analysis (change according to Uva Project)

Daily Biodegradable Kitchen Waste generation =50 kg (per day)

Considering the previous experimental results

Amount of bio gas generation = 1500 liters ($1.5m^3$)

Volume of CH_4 = 1.5X60/100 (60%of CH_4 in Biogas)
= $0.9m^3$

Bio gas will be generated just 500 Pa at room temperature

Mass of methane = 6.528X0.9

= 5.9kg

Energy generated = 5.9X55.7 (Methane produces more heat per mass unit =55.7 MJ/kg)

= 329Mj

Equivalent LPG saving =329/46.1 (LP Gas typically releases about 46.1 MJ/kg)

= 7.1kg

Saving from LPG per month =7.1X1700X30/13.5 (13.5kg Gas cylinder is to be Rs 1700.00)

= **Rs 26282.00**

Compost fertilizer generation per Day =15kg

Economic value of fertilizer =15X30X10(Rs 10/- per 1kg)

=Rs**4500.00**

PRE-TREATMENT OPTIONS FOR HYBRID CERAMIC MICROFILTRATION SYSTEMS FOR SURFACE WATER TREATMENT

Amila Abeynayaka[a*], Le Anh Tuan[a], Chettiyappan Visvanathan[a], Hitoshi Yonekawa[b], Yoshimasa Watanabe[c]

[a] Environmental Engineering and Management Program, Asian Institute of Technology, P.O. Box 4, Klong Luang, Pathumthani 12120, Thailand

[b] Membrane Engineering Department, Business Development Division, MetaWater, Nagoya 467-8530, Japan

[c] Division of Built Environment, Graduate School of Engineering, Hokkaido University, Sapporo 060-8628, Japan

*Corresponding author Tel.: +66 2 524 6189; fax: +66 2 524 5625. E-mail address: amila@ait.ac.th

Abstract
Membrane filtration is one of the alternatives suggested for the future drinking water production. Worldwide, the total number of ceramic membrane usage for water treatment is increasing swiftly and this is the first ceramic filtration study which is applied to a surface water treatment in a tropical region. This study was conducted to investigate the removals of natural organic matter and pathogens and, the fouling behaviour of the ceramic microfiltration system (CMF) when combined with the pre-treatments namely coagulation-flocculation and adsorption. Further, this study was carried out for three different scenarios namely: direct CMF (Scenario 1), coagulation-flocculation (PACl) CMF (Scenario 2), and coagulation-flocculation (PACl) and adsorption (PAC) CMF (Scenario 3). The outcome of the study revealed that the suspended solid, total coliform and fecal coliform were removed completely in all three scenarios. However, the removal efficiency on *Giardia* and *Cryptosporidium* was the highest (99.92%) with scenario 3 when compared with that of the other two scenarios. In addition, both the highest TOC and DOC removals were noted as above 80% with scenario 3. The fouling behaviour of the system varied with the scenarios, and long filtration period and high TMP recovery were achieved with the third scenario.

Key words: Surface water; Ceramic membrane; Microfiltration; Coagulation-flocculation; Adsorption.

1. Introduction

The outburst of population and the spread of industrialization whip the integrative adverse impact on the environment in the forms of air pollution, water pollution, ozone depletion, climate change, food contamination, poor sanitation, etc. The wayward activities of human kind have seriously disturbed the stability of the surface water cycle due to excessive consumption and pollution. As a result, in the recent past the need for improved surface water treatments became obligatory to provide good quality drinking water to devoid of microbial contamination. However, in the past, the surface water treatments were carried out based on conventional physico-chemical processes including coagulation, flocculation, sand filtration, disinfection, etc. The conventional technologies are used for long time which endow with good quality and portable water, and their design and operation are well understood by the operators. Lately, membrane alternatives have drawn mounting attention because these technologies have highly developed and membrane systems significantly reduce the space requirement to treat a given flow, reduce chemical requirements, and produce water which is easily disinfected and less prone to produce adverse disinfection by-products.

There are different types of membranes available to treat the surface water and they bring several advantages such as effective removal of turbidity, total organic carbon, and micro particles from the surface water. Besides the improvement of prescriptions, development of new concepts and usage of new technologies like nanotechnology have encouraged the use of ceramic membrane. In recent years, ceramic membranes have found to be an attractive alternative to organic membranes, especially for treatment of surface water. In addition, the ceramic membrane has many unique advantages, when compared with the traditional filters and polymer or organic membranes, such as excellent resistance to acid/alkaline and oxidation chemicals, solvent stability, high permeate production at relatively low pressure, high thermal stability, fine separation with narrow pore size distribution, excellent mechanical and abrasive resistance, extremely long working period, high recoveries, hydrophilic membrane surface, and, easy to clean and sanitize with short backwash interval (with air flush). Nevertheless, the membrane fouling is the major drawback of the direct ceramic membrane system. Consequently, it requires appropriate pre-treatment system like chlorination, adsorption by powdered activated carbon (PAC) or coagulation/flocculation to trim down the membrane fouling. Yuasa, et al. (2003) found that when the ceramic membrane system was combined with pre-treatment processes, the filtration achieved high rate of pollutant removal from surface water, where they achieved 99.7% of turbidity removal.

The ceramic membrane is being applied in the drinking water treatment over two decades and there is a great potential for application of the ceramic membrane technology in developing countries. Nevertheless, the operation of this technology depends on the specific characteristics of surface water and other local factors such as temperature, pH, turbidity, etc. Thus, this study was conducted to evaluate the removal efficiency of the natural organic matter and pathogens (bacteria and protozoa) from the surface water using a hybrid ceramic microfiltration (CMF) system, with the combination of pre-treatments such as chlorination, PAC adsorption, and coagulation in the ambient conditions. In addition, the removal of *Giardia* and *Cryptosporidium* was also investigated in this study as both organisms have caused numerous deaths in recent years in USA, Canada and some other countries (AWWA, 1999).

Pasts studies related to drinking water treatment with ceramic microfiltration systems are mainly conducted in developed countries like Japan (Yonekawa et al., 2004; Oh et al., 2007), Germany (Lerch et al., 2005) and Norway (Meyn et al., 2008). The studies were conducted with different but relatively low feed water (dissolved organic carbon) DOC concentrations. Most of the DOC values of feed water were less than 3 mg/L (Yonekawa et al., 2004; Oh et al., 2007; Lerch et al., 2005). However, Meyn et al. (2008) conducted the studies with feed of 5.5 mg/L DOC. It is important to investigate the treatment system behavior under feed conditions with higher DOC concentrations. This study was conducted with feed water of DOC concentration ranging from 6.9 – 10.5 mg/L, which is a relatively higher value than past studies.

2. Materials and methods

2.1 Feed water

The rainwater runoff storage pond water from Asian Institute of Technology (AIT), Thailand was used as a surface water source for this study. The pond water was sent through a raw mesh screen to remove the floating and big particles, and, then stored in a storage tank. Table 1 lists the characteristics of the AIT pond water used in this study.

Table 1: *Characteristics of AIT pond water*

Parameter	Unit	Value
pH	-	6.5 - 8.2
Temperature	°C	26 – 31
Turbidity	NTU	5.18 – 23.1
Conductivity	µs/cm	259 – 505
Micro-particle, 5-15 µm	Count/mL	1,230 – 11,448
TS	mg/L	198 – 315
TSS	mg/L	8 – 21
TOC	mg/L	10.05 – 12.5
DOC	mg/L	6.86 – 10.51
Total Fe	mg/L	0.02-0.09
Total Mn	mg/L	0.07-0.15

2.2 Experimental set-up

A pilot scale experiment was conducted with dead-end mode CMF. The CMF pilot system unit was made in Japan and the process of the pilot system was controlled automatically. The hybrid CMF system was combined with pre-coagulation unit with 2 minutes hydraulic retention time (HRT) to augment the efficiency of the filtration. In addition, the backwashing unit was accompanied to clean the fouling layer inside the channels of the membrane mechanically. The ceramic membrane filter (NGK insulators, Ltd.) with a pore size of 0.1 µm was installed vertically inside a module casing (a stainless steel tube). Furthermore, the 45 cm long ceramic membrane had 55 channels and each channel had inner diameter of 2.5 mm. The membrane was operated at constant flux of 50 $L/m^2.h$ in all the experimental runs. The operational conditions of the CMF are tabulated in the Table 2.

Table 2: *Operational conditions of the CMF system*

Description	Value
Effective membrane area	0.18 m^2
Membrane flux	1.2 $m^3/m^2/day$
Membrane filtration rate	0.2 m^3/day (150 ml/min)
Filtration time (backwash interval)	2 hours
Backwash pressure	500 kPa

Three different scenarios were set-up to achieve the proposed objectives of this study namely, (a) Scenario 1: Direct CMF, (b) Scenario 2: Coagulation and Flocculation + CMF, and (c) Scenario 3: PAC adsorption + Coagulation and Flocculation + CMF. In the scenario 1, the feed water was directly filtered through the ceramic membrane and the performance was compared with the other two scenarios. During the scenario 2, the raw water was pumped into the coagulation tank from the storage and then passed through the flocculation tube. The coagulant poly aluminum chloride (PACl) was added with an optimum dosage of 2 mgAl/L which was determined by the Jar test. In addition, the flocculation process was used to improve the pollutant removal efficiency of the ceramic membrane. The flocculated water was sent to the inlet channels of the ceramic membrane for filtration and the filtrate was collected in the filtrate tank after passing it through the pressurized tank. On the other hand, in the scenario 3 the powder activated carbon (PAC) was added with the dosage of 20 mg/L (the optimum value found through series of Jar tests conducted with PAC doses of 0-250 mg/L) before coagulation-flocculation process to augment the performance of the hybrid system. Further, in all three scenarios, the backwashing unit was automatically activated after every two hours of filtration to reduce the cake fouling in the ceramic membrane. The backwashing unit was operated at 500 kPa using filtrate for 5 seconds which was ameliorated by NaOCl, and air blow down was followed at 200 kPa for 5 seconds. In this study low backwash pressure was maintained as a creative approach to evaluate the fouling potential by lowering the backwash intensity. Besides, ex-situ chemical cleaning with citric acid solution 1% (for 24 hrs) and NaOCl 0.3% (for 24 hrs) was done to remove the irreversible fouling which can not be removed by regular backwashing, when the transmembrane pressure (TMP) reached 100-120 kPa. The overall hybrid CMF process is illustrated in the Fig.1.

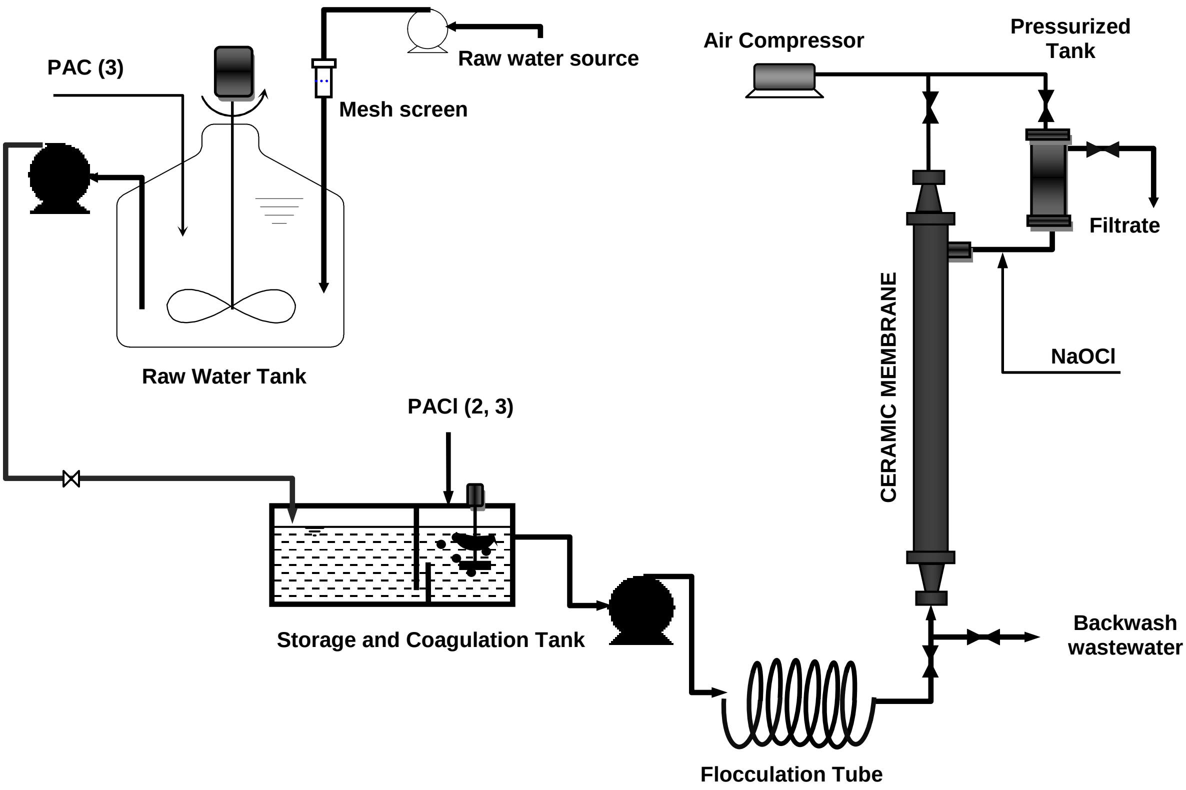

Fig. 1: *Simplified flow diagram of the CMF system*

2.3 Analytical methods

Standard Methods (APHA et al., 1998) were followed to measure the Alkalinity, total solids (TS), total suspended solids (TSS), free chlorine, total coliform and fecal coliform. The total organic carbon (TOC) and the dissolved organic carbon (DOC) were measured using Total Organic Carbon Analyzer (TOC 5000A, Shimadzu, Japan). The DOC is defined as the organic carbon which remains after filtration of the sample through 0.45 µm filter (Jarusuthirak, et al. 2007). In addition, total Fe and total Mn were measured using atomic absorption spectrophotometer (AAS) Z-8230. Conductivity and turbidity were measured with Conductivity Meter (WTW-330i) and Turbidimeter (HACH 2100N) respectively. The measurement of temperature, pH and TMP were noted down daily to monitor the stability of the system. In this study *Giadia* and *Cryptosporidium* were focused as they are the most common protozoa pathogens with the body size of 5 to 15 µm (AWWA, 1999). The particle counter MLC-7P (made in Japan) was used to count the particle numbers of *Giardia* and *Cryptosporidium* (*Protozoa*). The size range of particles that can be measured by the counter was 1 to 25 µm. The particle counter was initially set-up with tap water passing through 0.45 µm. When the particle count was less than 50 Counts/mL the counter was considered as stabilized. Then the pre-treated samples (pretreatment was done by filtering through a screen with pore size of 53 µm) were injected with a flow rate of 50 mL/min. The readings were taken after allowing system to stabilize, where minimum variation of repeated observations was recorded. The concentrations of particle size ranging from 5 – 15 µm were reported as an indicator for degree of presence of *Giardia* and *Cryptosporidium* (*Protozoa).*

3. Results and discussion

3.1. Comparison of filtration time, TMP & TMP recovery

As mentioned earlier, the three different scenarios were compared to optimize the best scenario to treat the surface water. The Fig. 2 illustrates the change of TMP with the filtration time for different scenarios. The direct CMF had showed the shortest filtration duration of 9 days to reach TMP to 100 kPa. However, with pre-treatment by PACl coagulation-flocculation, the system had showed longer filtration time of 17 days to reach TMP to 100 kPa. Here large molecular compounds and colloidal fraction could be easily flocculated and separated on the microfilter membrane surface. Where as the low molecular size compound with the molecular weight around 1000 Dalton is expected to penetrate into the membrane pores and adsorbed on to the pore walls. Same time major fraction of low molecular weight organics could pass though the membrane. DOC removal efficiencies (section 3.2 and Fig. 4) supports that considerable amount of organics are (40% of DOC) passing through the membrane in the scenario 2. However in scenario 3, another portion of organics were eliminated (only 20% of DOC was passing) through the treatment process. Thus, pre-treatment by adsorption of the low molecular weight compounds using PAC facilitated to prolong the period of treatment further (20 days to reach TMP to 100 kPa) which is the longest filtration period achieved when compared with other two scenarios. Hence, it can be concluded that the pre-treatment processes including coagulation-flocculation and adsorption effectively reduce the membrane foulants in the feed water. As a result the irreversible fouling which can not be effectively removed with the periodical (2 hours) backwash has been reduced. Hence the pretreatment effectively reduce the requirement of chemical cleaning to remove irreversible fouling which leads to longer filtration cycle.

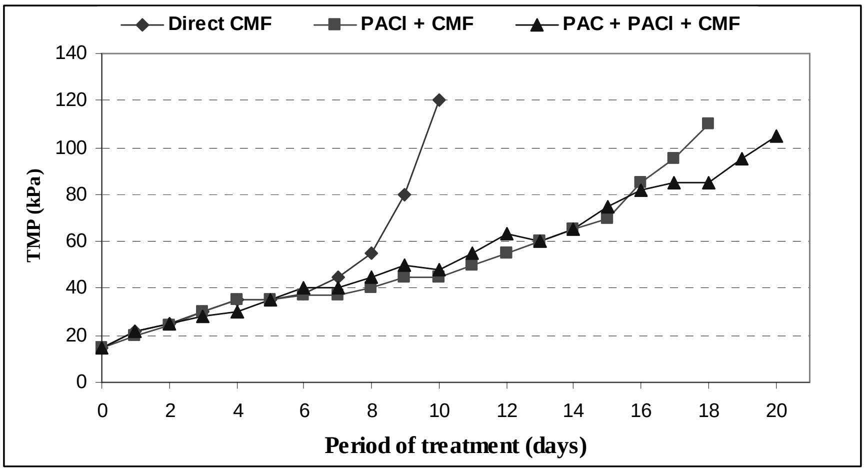

Fig. 2: *Variation of TMP with filtration time*

In addition, the TMP raise in scenario 1 showed rapid growth and low TMP recovery after backwashing. This could be caused by high particle (size less than the membrane pore size) content in the feed water where no pre-treatment was carried out to evade the particle content prior to membrane filtration. In contrast, TMP of the scenario 2 and 3 showed a gentle increment with time (Fig.2). Fig. 3 indicates the TMP recovery after the periodical 2 hour physical backwashing process with 500 kPa pressure (Table 2) during the system operation. The average TMP recovery was higher in the scenario 2 and 3 when compared with scenario 1. The increments of scenario 2 and 3 are 30% and 40% of the scenario 1 respectively. With the pretreatment applications, the part of the fouling components which causes irreversible fouling has been removed and hence the TMP recovery after backwashing process has increased. Study of Yonekawa et al. (2004) with similar type of ceramic membrane obtained 98% of water recovery after backwashing process in a hybrid CMF system as the reversible fouling was removed periodically. In addition, they found that the chemical enhanced backwashing increased the water recovery to 99.2%. Hence, it can be concluded that the reduction of micro particles flow into the ceramic membrane was occurred as a result of pre-treatment process in the hybrid CMF system.

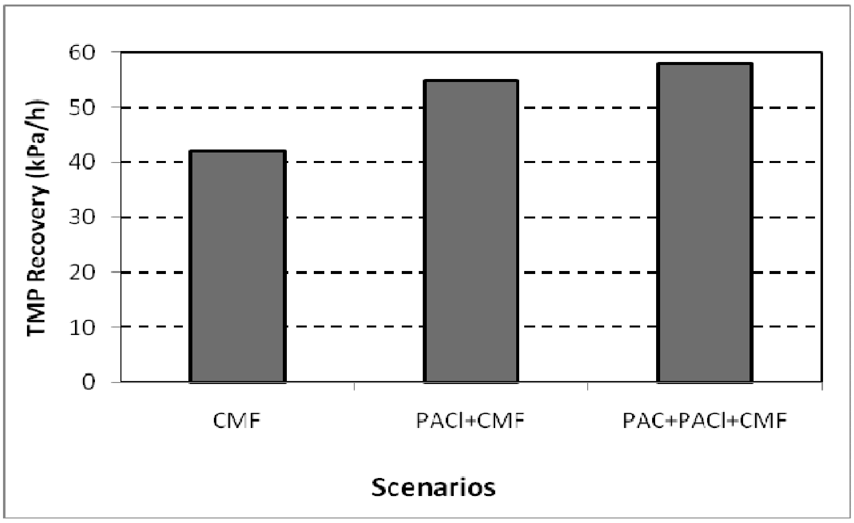

Fig. 3: *TMP recovery after backwashing operation*

Similarly, in this study, on top of backwashing process, the chemical cleaning was done once the TMP reached to 100-120 kPa. As mentioned in section 2.2, the chemical cleaning of a ceramic membrane in this study contains two stages namely, stage 1: cleaning with 1% citric acid and stage 2: cleaning with 0.3% NaOCl. Citric acid was used to dissolve the inorganic matters existing in the form of both inorganic and inorganic-organic complexes like ion-organic compound. During the chemical cleaning stage 1, portion of the TMP values of fouled membranes were recovered. The measured TMP after this step were 60 kPa, 25 kPa and 20 kPa for scenario 1, 2 and 3 respectively. In addition, after stage 2 chemical cleaning the TMP was further reduced to 15 kPa for all three scenarios.

In the scenarios 2 and 3, the feed water was pre-treated by PACl (coagulation-flocculation) alone, or PACl and PAC (adsorption), and as a result significant amount of dissolved organics and colloids were formed into flocs or the dissolved organics were adsorbed by the PAC particles. According to Suzuki & Chihara (1988) during PAC pretreatment process, dissolved organics were removed by adsorption, and suspended solid organics were by heterogeneous coagulation, at the same time. Hence untimely the dissolved organics and colloids were fixed to the larger flocs which can be removed by microfiltration through surface filtration. Accordingly, these flocs were removed effectively from the membrane surface by backwashing process (Zhao et al., 2005). In addition, as cited earlier the ions and colloidal fouling were also brought down due to pre-treatment process in the system. Thus, it can be concluded that the chemical cleaning by citric acid and the pre-treatment process in the hybrid CMF system can increase the TMP recovery. Further, it is noted that the scenario 2 and 3 are the better treatment sequences when compared with the scenario 1 as the pre-treatment processes reduces the chemical cleaning load.

3.2. Pollutant removal of the system

The total coliform, fecal coliform, and TSS were eliminated completely by the system in the case of all three scenarios during the steady state operation (Fig. 4). With the pore size of 0.1 μm, the ceramic membrane was able to remove the bacteria and TSS effectively which is one of the attractive benefits of the CMF. Moreover, the addition of NaOCl into the filtrate tank for enhancing the backwashing process contributed for successful disinfection of remaining bacteria in the filtrate. In addition, the Fig. 4 explains the key roles of the pre-treatment processes based on pollutant removal efficiency. The turbidity removal through scenarios 1, 2 and 3 were more than 99.3%. Other than the above mentioned pollutants, around 25%, 60% and 80% of both TOC and DOC were eliminated through scenarios 1, 2, and 3 respectively. This was due to the removal of both low and high molecular weight organic compounds through physico-chemical treatment of coagulation-flocculation and adsorption processes. Another study conducted with a pilot-scale PAC-MF system had been functioning for one year without withdrawal and replacement of PAC with an average DOC removal rate of around 80% (Kim et al., 2006; Oh et al., 2007). Hence, it confirms that the pre-treatment process can increase the removal efficiencies of the system. The total Fe removal through scenarios 1, 2 and 3 were 92%, 98% and 96% respectively (Fig. 4). In the case of removal of *Giardia* and *Cryptosporidium*, the hybrid CMF system has removed micro particles between 5 – 15 µm up to the undetectable level for all three scenarios. Since maximum reported raw water particle count is 11,448 Count/mL (Table 1), the system has shown over log-3 removal. Where the size of *Giardia* and *Cryptosporidium* (5 – 15 µm) are much grater then the nominal pore size of membrane (0.1 µm), even higher degree removals are expected. Studies of Lerch et al. (2005)

with same type of ceramic membrane (0.1 µm; NGK insulators, Ltd.) has shown over log-4 removal, where the raw water micro particle concentration is much higher than this study.

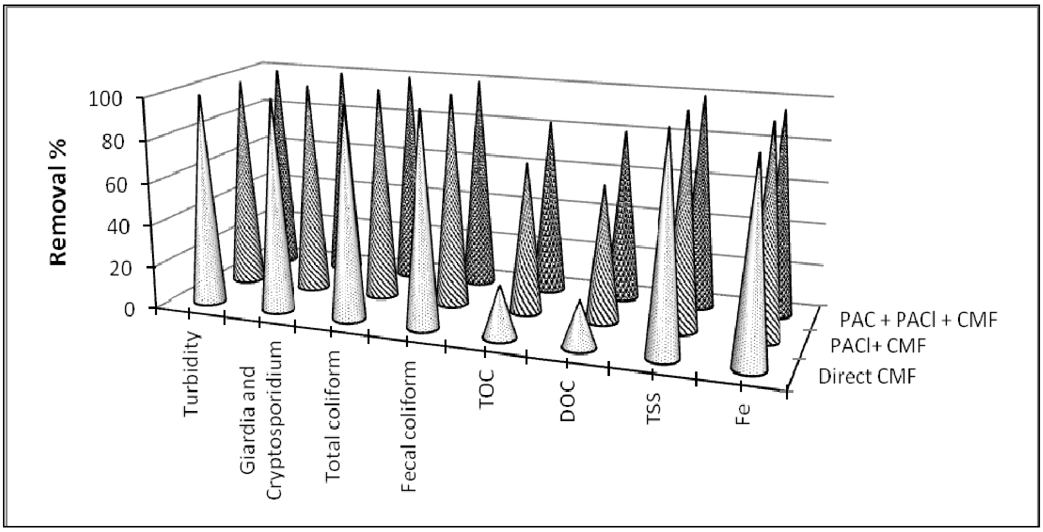

Fig. 4: *Removal rate of major pollutants for different scenarios*

The objective of any treatment technology is to reduce the disease causing organisms and related health hazards to a tolerable or safe level. The level of acceptability varies from country to country and depends on several factors such as environmental, human health and economic conditions. The overall results revealed that the treated water from the system with scenario 3 can be used for drinking purpose as the quality of the water meet the WHO standard requirement. However, the other two scenarios can also be utilized for drinking purpose after further treating for *Giardia* and *Cryptosporidium.*

4. Conclusion

In this study, the key role of pre-treatment processes combined with the pilot scale CMF system on surface water treatment was investigated with different combination of pre-treatment scenarios. It was found out that the pollutants removal efficiencies and performance of the hybrid CMF system differ depending on the pre-treatment process and operational conditions. In conclusion, the effectiveness of ceramic membrane filtration can be enhanced by pretreatment such as coagulation, PAC as well as by chemical backwashing.

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RECEIVING OF THEREPHTHALIC ACID FROM POLYETHYLENE TEREPHTHALATE WASTE BY DEPOLYMERIZATION IN THE MELT.

[1]Lipik V. T.

[1]School of Materials Science & Engineering, Block N4.1, 50 Nanyang Avenue, Nanyang Technological University, 639798, Singapore.
[1]E-mail: vitali@ntu.edu.sg, Tel: +65 83792515; Fax: +65 67909081.

Abstract
The technological method of terephthalic acid receiving by depolymerization of polyethylene terephthalate (PET) in the melt has been developed as the result of research. Mechanism of depolymerization is studied by mean of DSC analysis. It was established, that PET depolymerization in melt can be reached at atmospheric pressure, and temperature range 140-160°C during a short time with the yield of terephthalic acid close to 95-98%. Sodium or potassium hydroxide is more suitable reagents for PET depolymerization regarding technological point of view. The best found amount of alkali to be added to the polymer is 120% from stoichiometrically necessary quantity. Electrolytic method is more prominent for terephthalic acid precipitation because at the time of terephthalic acid receiving, sodium or potassium hydroxide can be regenerated by electrolysis and used in cycle. Possible use of catalyst – zinc acetate which is added into the melt to increase the degree of PET depolymerization is shown. The quantity of zinc acetate of 1% from polymer weight amount was found as optimum.

Keywords
Polyethylene terepthalate, depolymeryzation, alkali, melt, catalyst.

1. Introduction

Therephthalic acid is the base for polyethylene terepthalate (PET) synthesis. Currently therephthalic acid is a strategic commodity, because the volume of PET production continuously increases. PET synthesis based on dimethyl terephthalate makes way for one based on terephthalic acid. Processing of PET waste, presented in the majority bottles and packaging waste, should be developed in several directions. The market of secondary materials cannot contain and does not require a large quantity of secondary PET processed by physico-mechanical methods. Therefore the use of PET chemical recycling with therephthalic acid receiving is necessary as alternative to the physico-mechanical processing method, despite technological complexity.

There are technologies of PET depolymerization with the use of alcohols, water, acids and alkaline reagents and even with ionic liquids use (1-butyl-3-methylimidazolium chloride) [1, 2]. Practically all of them use elevated pressure and high temperature [3]. In some cases, the process of PET depolymerization lasts several hours [4]. The best result, especially yield of terephthalic acid, in PET depolymerization has been achieved at the use of alkaline reagents because PET ester bound is more subjected and vulnerable to rupture under alkali influence [5].

But almost all developed technologies are based on the depolymerization in the solvent. The contact between alkali and polymer is not efficient. The reaction is going only on the polymer surface. Complete depolymerization takes time. Temperature, pressure and catalyst are used to accelerate the process. Zinc acetate, amines chlorides of different metals can be used to accelerate PET depolymerization. [6, 7].

A new method of PET depolymerization in the melt was discovered and the possibility of PET depolymerization in the melt was confirmed and reaction was studied by DCS analysis and realized in laboratory reactor [8]. The advantages of proposed method are the following. Method does not require a solvent; there is no evaporation of solvent in the atmosphere, no solvent recycling technics need to be used etc. Method is fast because of close contact of reagents and reaction of depolymerization is going in the volume and not only on the PET surface. Calcium hydroxide was found as the most promising and perspective reagent for PET depolymerization in laboratory condition because of its wide spreading, small cost and relative simplicity of handling and transportation. It was established, that the quantity of calcium hydroxide for complete PET depolymerization can be within the range 105-140% from stoichiometrically necessary amount. The highest reached degree of PET depolymerization was 95%. The time of depolymerization in the

melt was about 15 minutes. And the second stage of processing of melt product (hydrolysis in boiling water) took 30-60 minutes.
Technically the developed method needed to be improved in the way of reducing the quantity of technological stages. Making step from laboratory experiment to the industry, several technological complicities have been arisen at the application of calcium hydroxide. We established that the use of potassium or sodium hydroxide for PET depolymerization in the melt is more justified in industrial scale.

2. Material and methods

The developed chemical PET recycling process, for which colorless PET bottles crushed to the size of 5×5 mm were used, was performed under the following condition. Cut PET waste are alloyed for a certain time and temperature with an alkaline reagent in laboratory stainless steel reactor, then cooled and hydrolysed in water at steering. Terephthalic acid was regenerated by the adding of an inorganic acid or electrochemically and then filtered, washed and dried. All used alkalis and zinc acetate (chemical degree purity) were bought from Sigma-Aldrich (Singapore) and used as received.

2.1. Differential Scanning Calorimetry (DSC)

PET depolymerization reaction has been studied by differential scanning calorimetry using a TA Instruments Model Q10 DSC machine, which was equipped with a DSC Refrigerated Cooling System to achieve low temperatures. The software used was TA Instruments Control. PET flaxes grinded by mean of spherical mill with liquid nitrogen cooling have been used for DSC analysis. PET and alkalis were mixed in certain stoichiometrical proportion and placed into hermetic aluminum pans. DSC analysis was done by heating of the sample up to 400°C at the rate of 10°C per minute.

3. Discussion

The method of PET depolymerization in melt with calcium hydroxide has positive and negative characteristics. Calcium hydroxide is accessible, cheap, stable and very easy to use. But from technological point of view, calcium hydroxide application has several lacks. The salt formed after melting – calcium terephthalate is insoluble in water. Therefore, simultaneous precipitation of calcium terephthalate, terephthalic acid and impurities takes place during the hydrolysis. It complicates the technology, increases the number of purification stages and leads to low purity of terephthalic acid. Moreover, the melting process should be realized at high temperature (up to 260-280°C) and if ethylene glycol is formed during depolymerization process it would evaporate from the reactor because the temperature of process exceeds its boiling point.

3.1. The use of different alkalis for PET depolymerization in melt.

The use of potassium or sodium hydroxide for PET depolimerization in the melt appeared more preferably due to several technological reasons. Potassium or sodium hydroxide is more expensive and aggressive compared to calcium alkali. But several advantages of the use of potassium or sodium hydroxide cover all negative moment and lacks.
The use of potassium hydroxide allows to reduce the temperature of depolymerization in the melt. It can be seen on DSC curves (figure 1) that depolymerization of PET in the presence of potassium hydroxide occurs at 140-160°C.

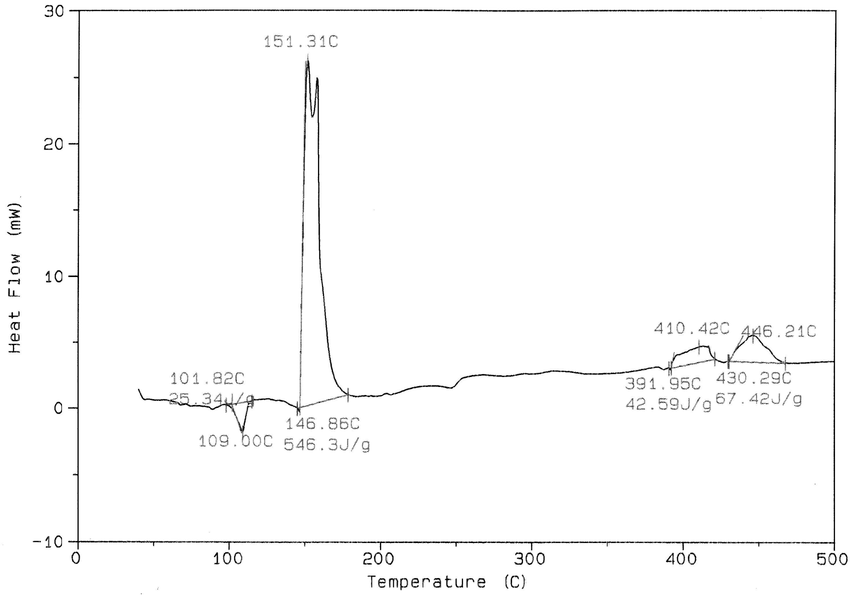

Figure 1. *DSC analysis of PET depolymerization at the presence of potassium hydroxide.*

Considering the splitting of main exothermic peak into two tops, we can suppose that there are two steps of depolymerization. The results shows the possibility to make depolymerization very fast because the process is exothermic and can be realized at lower temperature compared to the use of calcium hydroxide. It saves the time, energy for heating and mixing and decreases the lost of ethylene glycol because its boiling point is 197.3°C [9].

3.2. Use of catalyst to increase the degree of PET depolymeryzation.

Process of PET depolymerization can be improved by catalyst use. Many works have been done in the area of catalyst for PET depolymerization. Catalytic activity in PET depolymerization process has been reveled at the following chemicals: acetates of magnesium, zinc, tin, aluminum, copper etc [10, 11]. We studied the effect of zinc acetate added to the mix of PET and different amount of potassium hydroxide. PET depolymerization has been realized in laboratory reactor with mechanical steering. PET flakes, alkali and zinc acetate were added subsequently to the reactor preheated up to 100°C. One and three weight percents of zinc acetate (from polymer amount) were used to study the influence of catalyst on depolymerization process. Potassium terephtahlate received after depolymerization was dissolved in water and insoluble part (impurities and unreacted PET) was filtered, dried and weighted. Amount of products insoluble in water after PET depolymeryzation with the use of different quantity of zinc acetate is presented in figure 2.

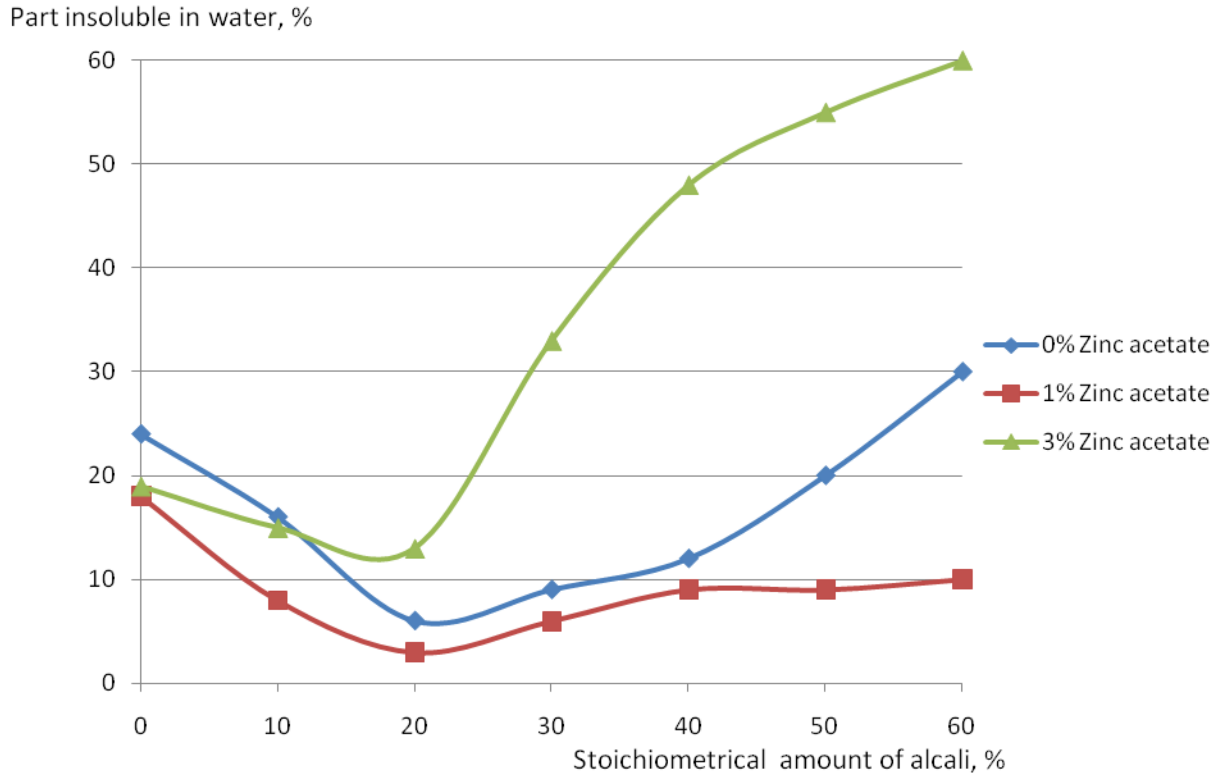

Figure 2. *Influence of catalyst on degree of PET depolymerisation.*

It was established, that the quantity of alkali for complete PET depolymerization can be within the range 105-130% from stoichiometrically necessary amount. The highest degree of PET depolymerization (96-98%) can be reached when the amount of alkali is 115-120%. Larger quantity of alkali invokes additional oxidation and degradation reactions that sharply augment the output of water insoluble products.

Zinc acetate improves the depolymerisation when it was added at the quantity 1% of polymer weight. We suggest that the highest amount of catalyst leads to redundant of transeterification reactions and formation of different condensed products that reduces the yield of potassium terephthalate and increases the amount of insoluble part. But we suppose, the use of catalyst in industrial scale is not well-grounded economically cause of small difference in results obtained for 0% and 1% of catalyst added to the melt (figure 2).

3.3. Terephthalic acid regeneration.

The product of PET processing in the melt represents very fragile material which can be directly put in reactor for hydrolysis. The grinding is no needed because sodium or potassium terephthalate is soluble in water even at room temperature. Formation of sodium or potassium salt of terephthalic acid, compared the calcium one, allows to omit the step of grinding and realize the dissolution at room temperature. Solubility of sodium/potassium terephthalate gives an advantage to filter the solution and separate all insoluble impurities. Filtered solution goes to the next technological stage of terephthalic acid precipitation.

There are some possibilities to regenerate terephthalic acid from solution of sodium or potassium terphthalate. Electrochemical and chemical methods can be applied. The choice of the best method for developed technology depends of the quantity of processed PET waste and amount of impurity and other possible local factors. When the amount of processed PET waste is not large, the use of chemical methods for terephthalic acid regeneration is more justified. For example, formation and sedimentation of terephthalic acid occurs at the addition of nitric or hydrochloric acid to the hydrolysis solution. Then terephthalic acid is filtered, washed out and dried in the vacuum oven. Potassium hydroxide is regenerated from potassium nitrate or chloride solution. Electrochemical method is more complicated from technological point of view, but it can combine several technological steps: precipitation of terephthalic acid and regeneration of sodium/potassium hydroxide. Water can be used several times for hydrolysis, accumulating ethylene glycol which is well soluble in water. When certain concentration of ethylene glycol is reached, solution need to be regenerated by means of distillation or membrane separation. We aimed on terephtalic acid receiving in this method. Ethylene glycol might be considered as a secondary value product, because of two reasons. Theoretical weight yield of terephthalic acid is 2.5 times more than ethylene glycol because, for example, 1 ton of PET after depolymerization gives about 855 kg of terephthalic acid and 320 kg of ethylene glycol. And secondly, market price of terephthalic acid is much higher that one of ethylene glycol.

4. Conclusions

New PET depolymerization method in the melt was developed and maximum approached to the industrial requests as a result of research. Alkaline reagent – sodium or potassium hydroxide allow to realize PET depolymerization during several minutes at the temperature 140-160°C.

Developed process of PET depolymerization differs from existing technologies by simplicity of the main reaction, use of cheap reagents – alkalis in solid state, absence of elevated pressures, supercritical temperature and long time of processing. Presence of impurity such as paper and other polymers does not affect the depolymerization process and therephthalic acid quality that is extremely important at waste processing. PET waste containing 90% of sand formed at the PET synthesis plant was successfully processed, with therephthalic acid received during the trial attempt.

The quantity of technological steps was reduced to the minimum. Method of terephthalic acid receiving consists in the following technological steps: depolymerization of PET in melt in presence of sodium or potassium hydroxide; dissolution of received salt of terepthalic acid in water; filtration of solution to separate insoluble impurities; regeneration and precipitation of terephthalic acid by chemical or electrochemical methods; separation of terephthalic acid by filtration; drying of terephthalic acid in vacuum oven; regeneration of alkali; regeneration of ethylene glycol by distillation of membrane technology application.

Obtained therephthalic acid can be used as a product for chemical synthesis, for PET and heat-resistant polyester fibers production.

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EMBODIED ENERGY IN RESIDENTIAL COST EFFECTIVE UNITS (Single Storied) Up to 50 Sqm Plinth area

Deepak Bansal[1]
Pooja Nandy[2]

[1]Deepak Bansal IE (I), CEAI (FIDIC), ISWE, IASE, BIS, IMS, Assistant. Chief (Projects), Design and Development Wing, Housing and Urban Development Corporation, India Habitat Centre, Lodhi Road, New Delhi, 110003, India
Email: dbansal1969@gmail.com
Telephone: +91-11-24648193/94/94 Extension: 4050

[2]Pooja Nandy, Senior Project Appraisal Officer (Architect), Design and Development Wing, Housing and Urban Development Corporation, India Habitat Centre, Lodhi Road, New Delhi, 110003, India
Email: pooja.nandy@gmail.com
Telephone: +91-11-24648193/94/94 Extension: 4047

Abstract:
It is believed that about 30 % of Energy is used in construction activities alone, and building construction constitutes a major part of all construction activities. Now the emphasis is towards construction of Green buildings with the objective of minimizing the uses of energy in building in terms of Embodied energy and maintenance/operational energy (Electrical, Water, Thermal, Sound, Sanitation, HVAC etc). In Green buildings all the emphasis is on the usage of Fly ash, Construction wastes, materials from the vicinity (To reduce Transportation energy) and materials with less EEV- Embodied Energy Value, besides effective utilization of water, rain water harvesting, Solar, wind , Landscaping and orientation of buildings to minimize the usages of energy in buildings.
There are many agencies like TERI-GRIHA, USGBC, LEEDS & BEE- Energy Conservation Building Conservation code, who are advocating the concept of Green Buildings very aggressively in India, by releasing guidelines on the planning and designing of buildings as well as they are doing certification of the green buildings, however they have not yet quantified the basic Minimum EEV of different types of buildings and their minimum maintenance/operational energy required on area or volume basis, so that, this can be quantified easily, by metering these parameters (Embodied and Operational Electrical Energy) rather than doing highly time/cost consuming model analyses on several complicated but not very conclusive hypothesis.

Keywords: *EEV- Embodied Energy Value, Cost effective houses, HUDCO- Housing and Urban Development Corporation Limited, IHSDP-Integrated Housing and Slum Development Programme, Housing Typology, Interlocking Blocks*

1. Introduction

An attempt has been made in this paper, to quantify the EEV of Single storied house up to 50 sqm of plinth area, as Hudco -Housing and Urban Development Corporation Limited has designed/constructed hundreds of this type of houses in the country. The Typology of the house taken in this paper are : Houses with Strip footing in brick masonry in cement mortar 1:6, DPC 40 mm Thick 1:2:4 PCC, Brick wall 230/115mm thick in cement mortar 1:6, 12mm/15mm cement plaster 1:6 , CC skirting/Dado 12 mm thick 300/1200 mm high, RCC M20 roof 115 mm thick, CC Gola, khurrah, Mud Fuska with Brick tiles, parapet in brick work 900mm high, CC Coping 40mm thick PCC, IPS- Indian Patent Stone flooring, minimum Joinery, etc.
This Exercise is based on the bill of quantities of the single storied tenement type of houses designed/constructed by HUDCO and is cost effective houses. If the Cluster approach is to be adopted, the quantities of Materials will vary.
The bill of quantities of the various types of houses proposed by HUDCO are quantified, analyzed and the basic building materials are calculated as cement, sand, aggregates, steel, bricks on per square meter of plinth area basis, which require the almost all the energy in Construction of the house. The components like plumbing, electrical, finishing items are not calculated as mostly these houses had the basic minimum of them.

2. Objectives and Methodology

The main objective is to study the embodied energy values of materials used for cost effective houses up to 50 sq.m. The most important criteria for judging the energy efficiency of this housing typology has to fixed- whether Embodied energy or Maintenance/ Operational Energy. The following methodology was used:

a) This Exercise is based on the bill of quantities of the single storied tenement type of houses designed/constructed by HUDCO and is cost effective houses.
b) If the Cluster approach is to be adopted, the quantities of Materials will vary.
c) The bill of quantities of the various types of houses proposed by HUDCO are quantified, analyzed and the basic building materials are calculated as cement, sand, aggregates, steel, bricks on per sqm of plinth area basis, which require the almost all the energy in Construction of the house.
d) The components like plumbing, electrical, finishing items are not calculated as mostly these houses had the basic minimum of them.
e) The energy used for running the energy efficient electrical fixtures in the house is evaluated.
f) The comparison in the Embodied energy and Maintenance/ Operational Energy is done over the life cycle of the house say 50 years.

3. Embodied Energy Values for Materials of Construction

Table 1: Basic EEV of Materials: (Reference: IE (I) Journal-AR-Page 47-50, vol 84, October-2003, Dr P S Chani, Dr Najamuddin, and Dr S K Kaushik):
EEV of Different basic construction Materials (1):

Items	EEV (MJ)	Units	Sizes in mm
Soil	0	0	0
Cement	6.70	MJ/Kg	0
Sand	0	0	0
Fly ash	0	0	0
Steel	32.00	MJ/Kg	0
Standard Burnt Bricks	4.50	MJ/Bricks	229*114*76
Clay Fly ash Bricks	2.32	MJ/Bricks	200*100*100
Sand Lime Bricks	2.79	MJ/Bricks	200*100*100
Hollow Cement Concrete Blocks	11.00	MJ/Blocks	400*200*200
Aerated Blocks	11.50	MJ/Blocks	400*200*200
Fal G Blocks	7.90	MJ/Blocks	300*200*150
Solid Concrete Blocks	10.40	MJ/Blocks	300*200*150

The EEV has been taken as the same, without adding for handling/transportation etc inputs.

Table 2: Calculation of EEV of Hydra form block :(Hydra form India (P) Ltd) (2).

Assumptions:

Block Dimension(mm)	**Length (+-)(mm)**	**Width (mm)**	**Height(mm)**
	230	220	115
Production Capacity (Model: M7S2E)	2800	Blocks per shift, 8 working hours	
Weight of Soil based block(kg)	11		
Weight of Fly ash based block (kg)	9.5		
Total weight of Soil based mix (kg)	30800		
Total weight of Fly ash base mix(kg)	26600		
Volume of each Block (in Cu.m)	0.006		
Total volume of blocks produced(Cu.m)	16		
Density of soil base block(per Cum)	1890		
Density of Fly ash base block(per Cum)	1633		

The calculations for the EEV for SEB Interlocking block using Hydra form Technology and Fly Ash interlocking block using Hydra form technology are given below in Table 3 and Table 4 respectively.

Table 3: EEV break up for SEB

Raw Material	**% age**	**Weight (kg)**	**EEV (MJ)**
Soil	62.00%	19096	0
C. Sand/ St. Dust	30.00%	9240	0
Cement	8.00%	2464	16509
Total	100.00%	30800	16509
Power : 18.5 kwh x 8 hr x 3.64 MJ			539
Total EEV per day production			17048
EEV per Hydra form Block (SEB) (size: 230 x 220 x 115)			**6.09**

Table 4: EEV break up for Fly Ash interlocking block using Hydraform technology

Raw Material	**% age**	**Weight (kg)**	**EEV (MJ)**
Fly Ash	65.00%	17290	0.00
C. Sand/ St. Dust	27.00%	7182	0.00
Cement	8.00%	2128	14258
Total	100.00%	26600	14258
Power : 18.5 kwh x 8 hr x 3.64 MJ			539
Total EEV per day production			14796
EEV per Hydra form Block (Fly Ash) (size: 230 x 220 x 115)			**5.28**

4. Model

Some 200 No of typical houses were studied for the purposes of understanding the house Materials economics and this gives the inputs for calculation of EEV. Shown below in plate 2, is the house design for the units proposed by HUDCO for the IHSDP schemes taken up in the city of Meerut, India. The broad specifications are mentioned in plate 1.

Plate 1: Specification for the housing Typology studied:

S.no.	SPECIFICATIONS	
1	Structure	Load Bearing Structure
2	Wall	230mm Thick Brick Masonry In 1:6 Cement : Coarse Sand, Mortar
3	Roof	Flat Rcc Roof (M20),115 Mm Thick With TMT Fe 500d Reinforcement
4	Flooring	40 Mm Thick Cement Concrete (1:2:4) Flooring
5	Skirting/Dado	12 Mm Thick 300/1200mm High, 1:6 Cement :Coarse Sand Mortar
6	Plaster	12/15mm 1:6 Cement : Coarse Sand
7	Mud Fuska	100 Mm Average With Brick Tiles
8	Parapet	900mm High In 115 Mm Thick Brick Masonry In 1:4 Cement Sand Mortar
9	Joinery	Ms Frames With Steel Grills And Glass Panels
10	Cc Gola/Khurrah/Coping	1:2:4 Pcc

Plate 2: TYPICAL LIG HOUSE drawings (3)

Comparison for Masonry used in Construction: The masonry can be chosen from one of the various choices available, the compressive strength and the comparative EEV (MJ/kg) as shown in Table 5.

Table 5: Comparative Chart for Embodied Energy Value (EEV) & Compressive Strength for Different Building Materials

Building Material	**Size (cm)**	**Comp. Strength**	**Weight**	**Density**	**EEV(Block)**	**EEV**
		(kg/ sq.cm)	**(kg)**	**(kg/cu.m)**	**MJ**	**(MJ/kg)**
Brick (conventional)	22.9x11.4x7.6	+- 75	2.75	1386	4.5	1.64
Hollow concrete block	40x20x20	+- 40	26.88	1680	11	0.41
AAC/CLC	40x20x20	+- 40	19.2	1200	11.5	0.60
Solid Concrete Block	30x20x15	+- 75	21.6	2400	10.4	0.48
HF (fly ash block)	23x22x11.5	+- 70	9.5	1633	5.3	0.56
HF (soil-cement block)	23x22x11.5	+- 50	11	1890	6.1	0.55
FalG Block	30*20*15	+- 75	18	2000	7.9	0.44

Basic Construction Materials per Sqm of Plinth area and the EEV of the units per sqm of plinth area are as follows in Table 6 – Table 13:

ENERGY (EEV) OF DIFFERENT BUILDING COMPONENTS (BRICK MASONRY)

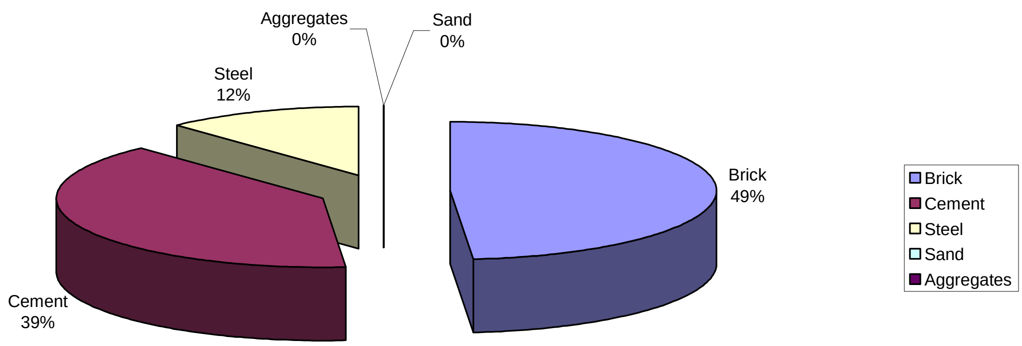


PLATE 2: TYPICAL LIG HOUSE

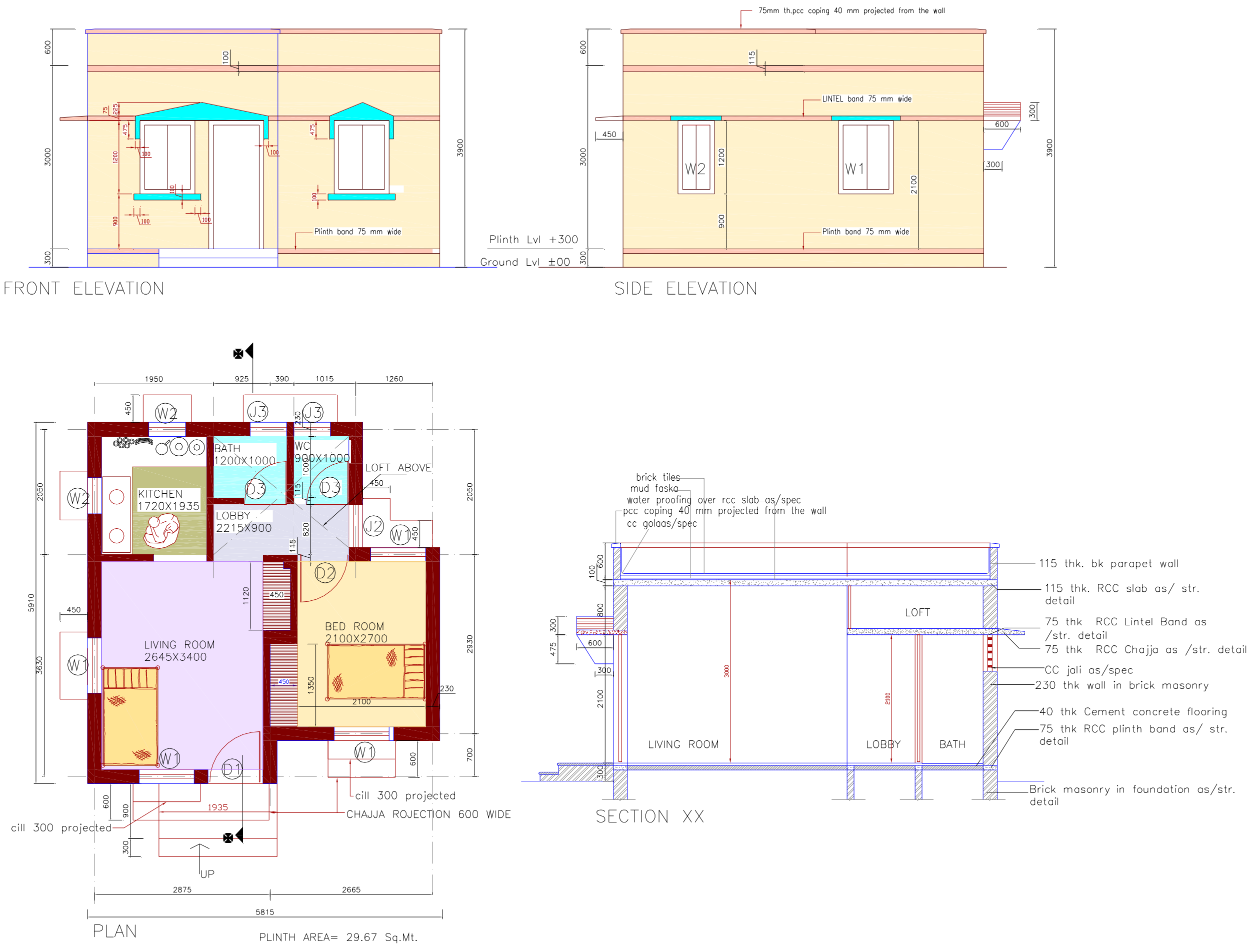


Table: 6 (4) **Computation of EEV in a Single Storied Cost effective house with following masonry option**
Bricks English Bond: EEV (MJ) of Building per Sq.m. of Plinth Area

Item	Material requirement per Sqm of Plinth Area	**EEV**	**EEV per Sq.m. of Plinth Area**
Brick	460 Nos	4.5 MJ/Brick	2070
Cement	5 Bag	6.7 MJ/Kg	1675
Steel	16 Kg	32 MJ/Kg	512
Sand	0.65 Cum	0	0
Aggregates	0.45 cum	0	0
Total			**4257.0**

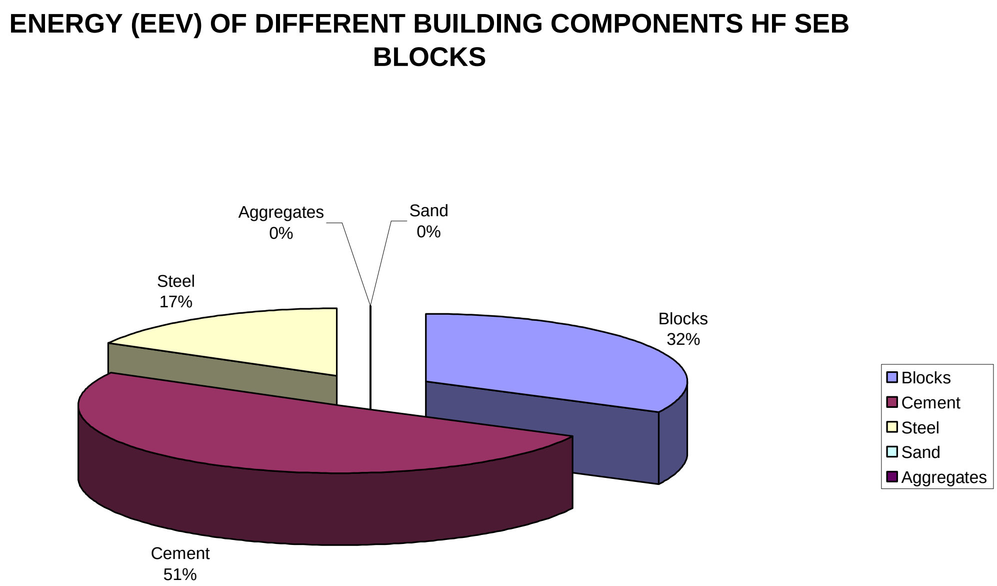


Table 7(4)

HF SEB BLOCKS EEV (MJ) of Building Per Sqm of Plinth Area

Item	Material requirement per Sqm of Plinth Area	EEV	EEV per Sqm of Plinth area
Blocks	160 Nos	6.09 MJ/Brick	974.4
Cement	4.55 Bag	6.7 MJ/Kg	1524.25
Steel	16 Kg	32 MJ/Kg	512
Sand	0.55 Cum	0	0
Aggregates	0.45 cum	0	0
Total			**3010.650**

EEV 29% Less than Brick House

ENERGY (EEV) VALUES FOR BUILDING COMPONENTS (FLY ASH BLOCKS)

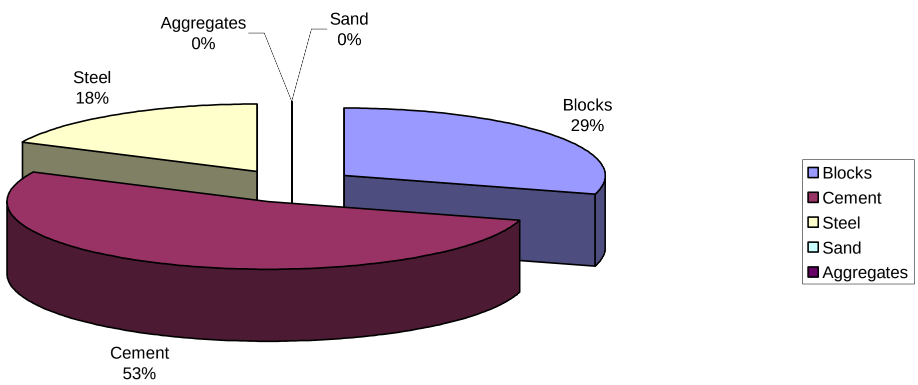


Table 8: (4)

HF Fly ash BLOCKS	EEV (MJ) of Building Per Sqm of Plinth Area		
Item	Material requirement per Sqm of Plinth Area	EEV	EEV per Sqm of Plinth area
Blocks	160 Nos	5.28 MJ/Brick	844.8
Cement	4.55 Bag	6.7 MJ/Kg	1524.25
Steel	16 Kg	32 MJ/Kg	512
Sand	0.55 Cum	0	0
Aggregates	0.45 cum	0	0
Total			**2881.050**

32% Less Than Brick House

ENERGY (EEV) VARIOUS BUILDING COMPONANTS BRICK RAT TRAP BOND

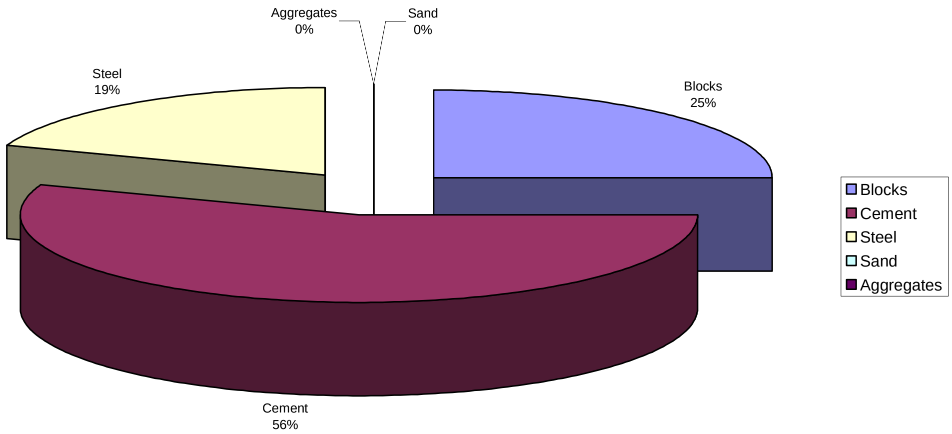


Table 9: (4)

Bricks EEV (MJ) of Building Per Sqm of Plinth Area

Rat-Trap Bond

Item	Material requirement per Sqm of Plinth Area	EEV	EEV per Sqm of Plinth area
Brick	420 Nos	4.5 MJ/Brick	1890
Cement	5 Bag	6.7/MJ/Kg	1675
Steel	16 Kg	32 MJ/Kg	512
Sand	0.65 Cum	0	0
Aggregates	0.45 cum	0	0
Total			**4077.000**

4% Less than Brick House

ENERGY (EEV) VARIOUS BUILDING COMPONENTS HOLLOW CONCRETE BLOCKS

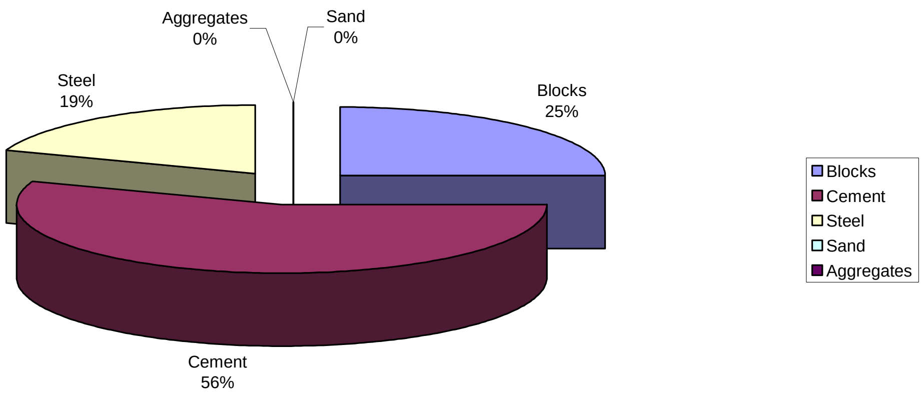


Table 10: (4)

Hollow Concrete Blocks EEV (MJ) of Building Per Sqm of Plinth Area

Item	Material requirement per Sqm of Plinth Area	EEV	EEV per Sqm of Plinth area
Blocks	60 Nos	11 MJ/Brick	660
Cement	4.4 Bag	6.7/MJ/Kg	1474
Steel	16 Kg	32 MJ/Kg	512
Sand	0.55 Cum	0	0
Aggregates	0.45 cum	0	0
Total			**2646.000**

38% Less than Brick House

ENERGY (EEV) VARIOUS BUILDING COMPONENTS FAL G BLOCKS

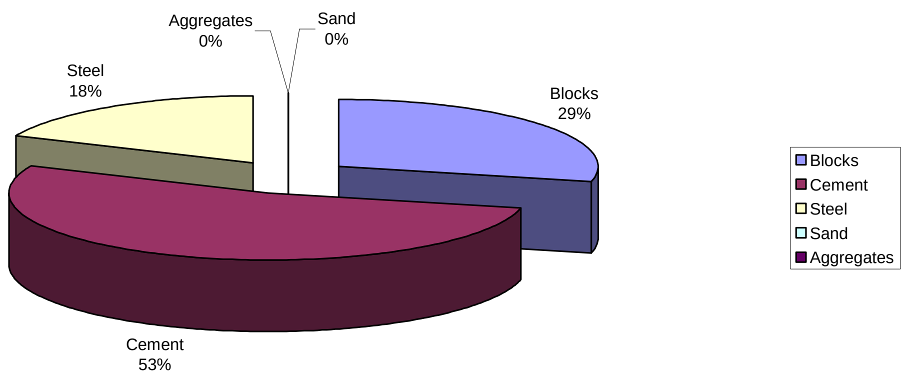


Table 11: (4)

FalG Blocks EEV (MJ) of Building Per Sqm of Plinth Area

Item	Material requirement per Sqm of Plinth Area	EEV	EEV per Sqm of Plinth area
Blocks	102 Nos	7.9 MJ/Brick	805.8
Cement	4.45 Bag	6.7/MJ/Kg	1490.75
Steel	16 Kg	32 MJ/Kg	512
Sand	0.55 Cum	0	0
Aggregates	0.45 cum	0	0
Total			**2808.550**

34% Less than Brick House

ENERGY (EEV) VARIOUS BUILDING MATERIALS SOLID CONCRETE BLOCKS

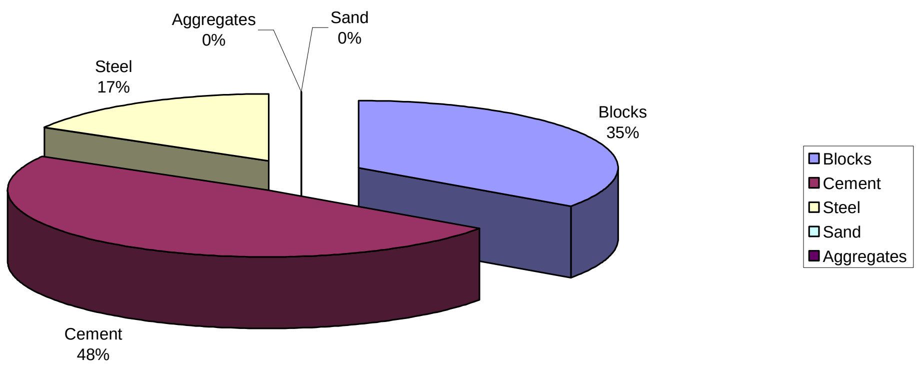


Table 12: (4)

Solid Concrete Blocks EEV (MJ) of Building Per Sqm of Plinth Area

Item	Material requirement per Sqm of Plinth Area	EEV	EEV per Sqm of Plinth area
Blocks	102 Nos	10.4 MJ/Brick	1060.8
Cement	4.45 Bag	6.7/MJ/Kg	1490.75
Steel	16 Kg	32 MJ/Kg	512
Sand	0.55 Cum	0	0
Aggregates	0.45 cum	0	0
Total			**3063.550**

28% Less than brick house

ENERGY (EEV) BUILDING MATERIALS AAC/ CLC BLOCKS

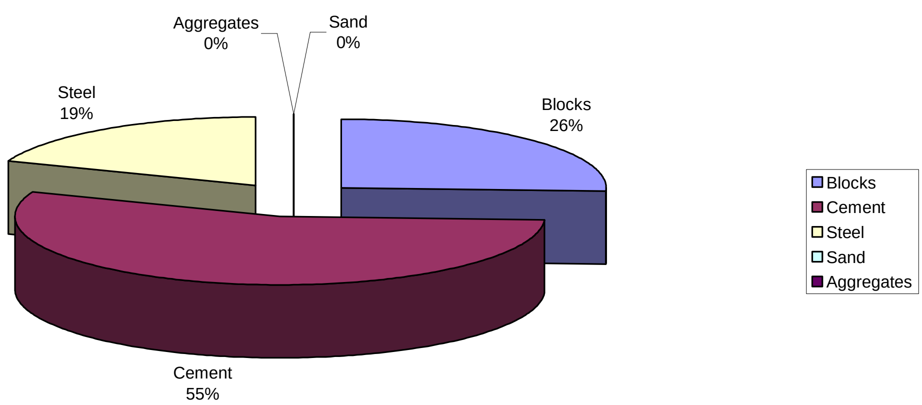


Table 13: (4)

AAC/CLC Blocks EEV (MJ) of Building Per Sqm of Plinth Area

Item	Material requirement per Sqm of Plinth Area	EEV	EEV per Sqm of Plinth area
Blocks	60 Nos	11.5 MJ/Brick	690
Cement	4.4 Bag	6.7/MJ/Kg	1474
Steel	16 Kg	32 MJ/Kg	512
Sand	0.55 Cum	0	0
Aggregates	0.45 cum	0	0
Total			**2676.000**

Table 14:

Comparative chart for EEV with Different Masonry Option for a Single Storied House Per Sqm of Plinth Area			
S No	**Masonry**	**EEV(MJ)/Sqm of Plinth Area**	**% Saving**
1	Brick	4257.000	0
2	Hollow	2646.000	38%
3	AAC/CLC	2676.000	37%
4	FalG	2808.550	34%
5	HF Flyash	2881.050	32%
6	HF SEB	3010.650	29%
7	Solid Concrete	3063.550	28%
8	Rat-Trap	4077.000	4%

It is seen that the masonry alone constitute about 50 % of EEV and if this can be replaced by LOW EMBODIED ENERGY materials, there will be tremendous saving in the EEV of the Houses.

Table 15 :(Year 2007 Prices in Indian Rupees)

Building Material	**Size (in cm)**	**Wall Thickness**	**No of units/sqm**	**Cost/Cu.m (Rs)**	**Cost/Sq.m (Rs)**
		(in cm)		**(Approx)**	**(Approx)**
AAC	40 x 20 x 20	20	13	3600	720
Brick	22.9 x 11.4 x 7.6	23	116	2400	552
Hollow Concrete Blocks	40 x 20 x 20	20	13	2700	540
HF (fly ash block)	23 x 22 x 11.5	23	40	2800	616
HF(soil-cement block)	23 x 22 x 11.5	23	40	2800	616

Table 16:
Materials input per sqm of walling with 1:6 Cement Sand mortar

Building Material	Wall Thickness	Units of Blocks Required	Cement (kg)	Sand (cum)	Plaster	EEV of Blocks	EEV of Cement
Brick	(9" thick wall)	116	14.5	0.06	Required	521.7	97.2
Hollow Concrete Block	(8" thick wall)	13	7.5	0.03	Required	137.5	50.3
AAC	(8" thick wall)	13	7.25	0.029	Required	143.8	48.6
HF (fly ash block)	(9" thick wall)	40	1	0.029	Optional	208.9	6.7
HF(soil-cement block)	(9" thick wall)	40	1	0.029	Optional	240.6	6.7

5. Maintenance/Operational energy:

In these residential units, only energy required are Electrical energy and cooking gas energy. The Electrical energy can be quantified as per electrical load sanctioned in these types of houses with most efficient electrical appliances used in the houses, like CFL 15 W for lights-7 nos, 2- 60 W fans, 1-Refrigrator 75 W, 1-TV, 1-Room cooler 100 W and power points.
Average running of the appliance per month is as per following table 17:
Total electrical consumption (units per hours) – (Source BSES Energy Bill -Delhi)
Total 3.30 Units per day or 99 Units per months in winters i.e. 356.40 MJ
Total 6.18 Units per day or 185.40 units per Month in Summers i.e. 667.44 MJ.

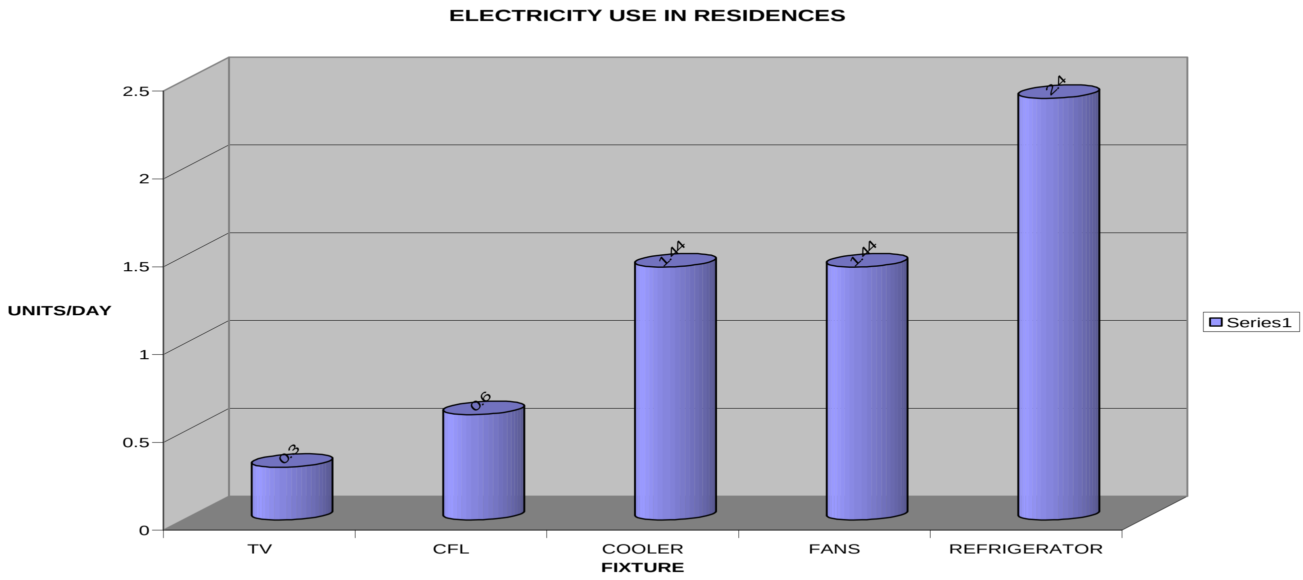
Table 17:

FIXTURE	WATTS	NUMBERS	UNITS/HOUR	DURATION/DAY (IN HOURS)	UNITS/DAY
CFL	15	7	0.15	4	0.6
ROOM COOLER	100	1	0.12	12	1.44
FANS	60	2	0.12	12	1.44
REFRIGERATOR	75	1	0.1	24	2.4
POWER POINTS	100	2			
TV	40	1	.05	6	.30
TOTAL(SUMMER)					**6.18**
TOTAL(WINTER)					**3.3**

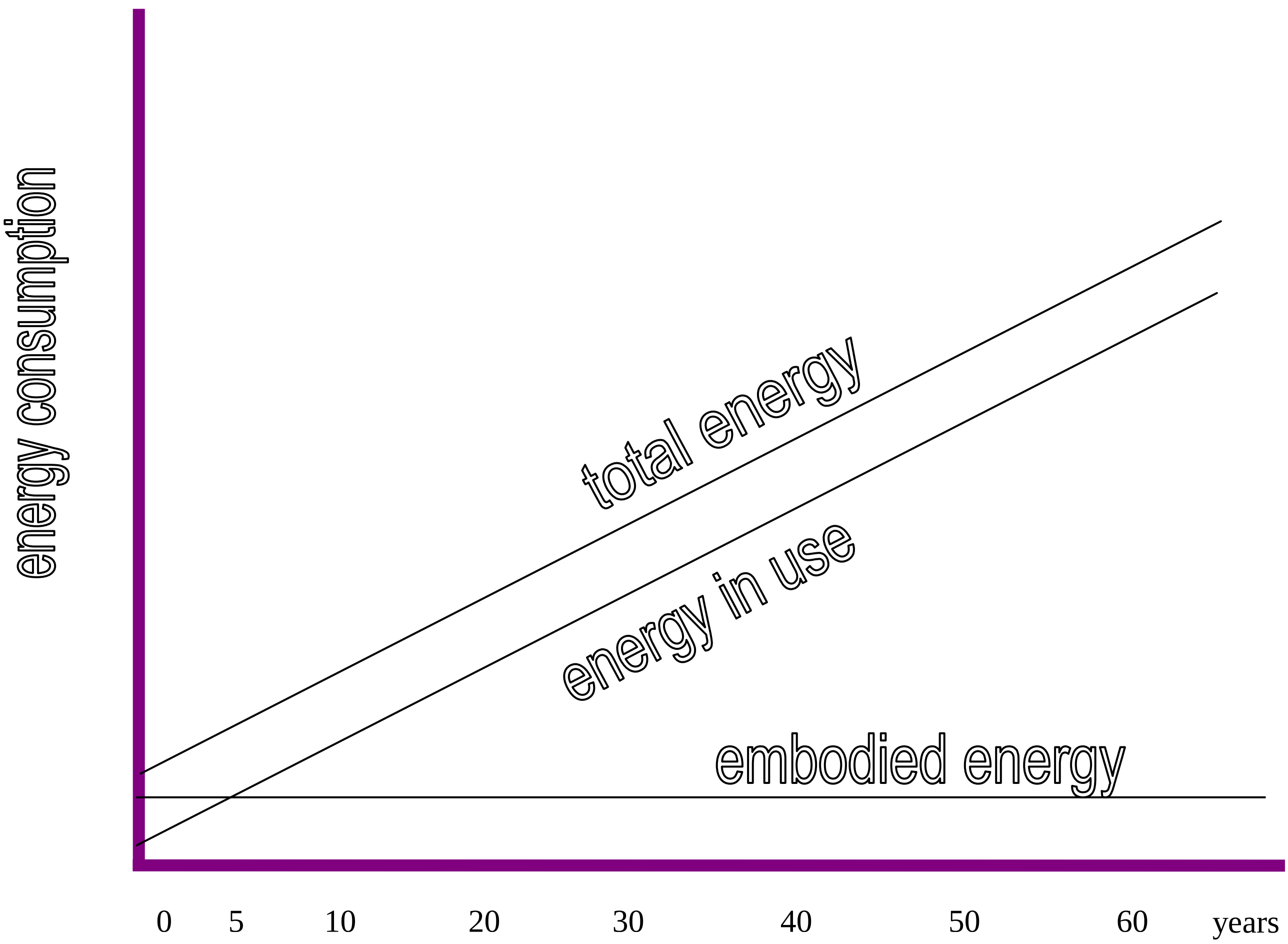
The Plinth Area of the Unit at Plate 2 is 29 sqm, EEV =29*4257MJ (Table 6) = 123.45 GJ

Maintenance/operational Energy per year= 99*3+185.4*9 (assuming 3 month winter & 9 month summer) = 1965.40 Units/Years or 7076.16 MJ/ year= 7.076 GJ/ year

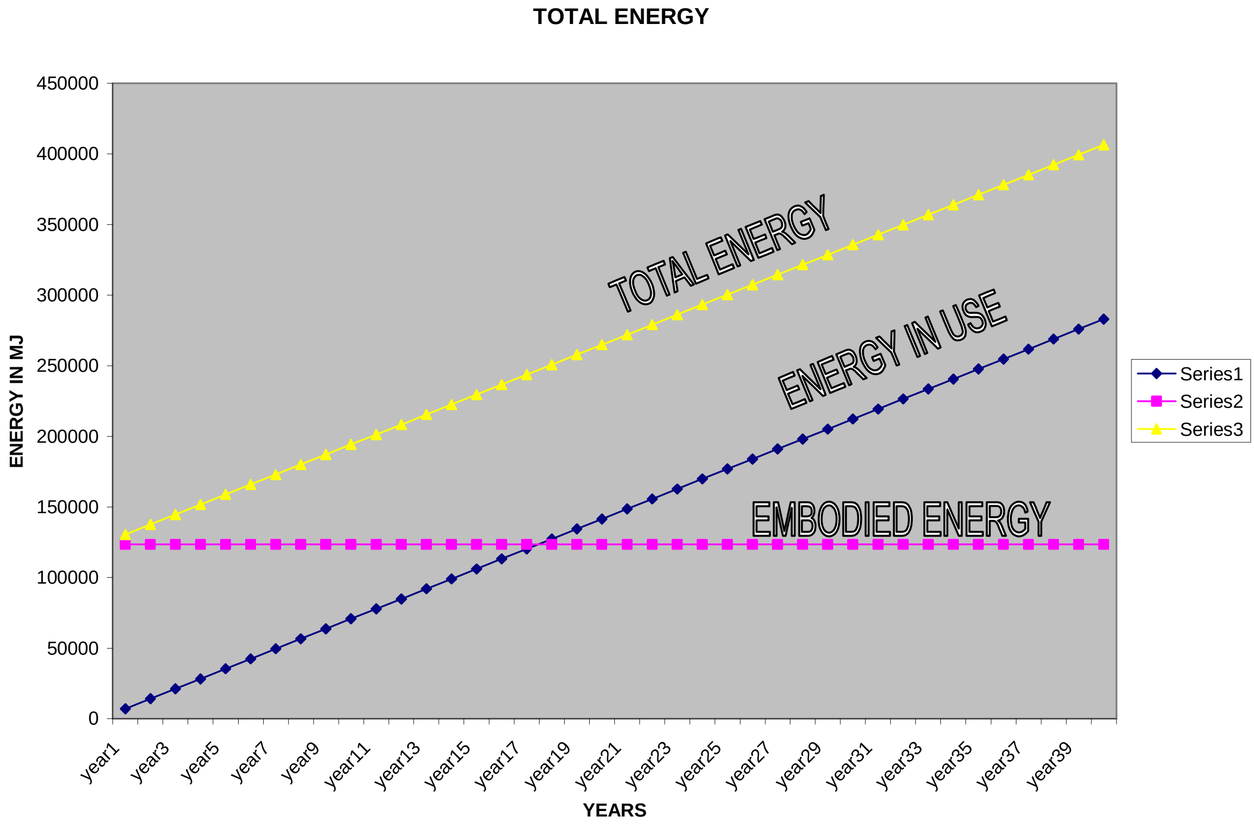
The Electrical energy in use overtakes the embodied energy of the unit after 17.4 years.



NORMAL HOUSE IN UK – GREEN HOMES (5)



COST EFFECTIVE HOUSE IN INDIA- ENERGY AUDIT



6. Conclusion:

In India Ratio of EEV to Maintenance Energy of the Residential units is much higher compared to UK houses and this should be main criteria for judging the Energy efficiency of the houses considering the fact that life of a house is about 50-60 years in India.
As a thumb rule the energy in use overtakes the embodied energy of the houses in five years of use of a residential building in UK and in India this takes about 20 Years. Hence, over the lifetime of a building it is definitely very useful to take steps to reduce the energy in use through active and passive solar/ climatic design measures, but it more important to use the building materials with low EEV and quantities of the materials must be minimized, especially masonry.
This is also clear that the masonry by burnt Bricks, takes very high amount of EEV, and the materials like Hollow Cement Concrete blocks, AAC/CLC etc requires much less EEV.
Hence Planning as well as Intervention by Cost effective building materials, can significantly bring down the EEV of the houses.

*** These are the views expressed by the researchers, based on their own study. Hudco may or may not have the same view on this finding.

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GROUND-COUPLED COOLING IN HANOI

Lu Aye*, Tuan Ngo, Tshewang Lhendup & Bob Fuller

Renewable Energy and Energy Efficiency Group
Department of Civil and Environmental Engineering
Melbourne School of Engineering
The University of Melbourne
Victoria 3010
AUSTRALIA
*Corresponding author: lua@unimelb.edu.au
Tel: +61 3 8344 6879, Fax: +61 3 8344 6215

Abstract
The energy required for space cooling could be reduced by using ground as a heat sink depending on the weather conditions and the ground characteristics. In this paper the theoretical performance of a closed loop ground coupled cooling system for a commercial building in Hanoi has been investigated as an alternative to the conventional air-to-air cooling system. A theoretical computational model for the prediction of the cooling system performance has been developed. It was found that the ground-coupled cooling system performs better (approximately 30% energy saving) than a conventional air-to-air cooling system.

Keywords: Ground-coupled cooling; heat pump; simulation; Hanoi
Theme: Energy efficiency and advance simulations

1 Introduction

The energy required for space cooling could be reduced by using the ground as a heat sink depending on the weather conditions and the ground characteristics. The ground temperature is relatively constant at few metres below the surface. Yasukawa *et al*. (2009) conducted extensive groundwater temperature surveys in the Red-river plain, Vietnam (Fig. 1). They reported that the temperature of the ground remains fairly constant below summer outside air temperatures in the Hanoi (Fig. 2). A recent study by Tuyen *et al*. (2010) concluded that there is a good prospect for extensive use of the ground-coupled cooling system for space air conditioning in the Hanoi region.

The conducive conditions are: the long time duration of hot weather in summer, large power consumption for air conditioning, and the electricity shortages in summer.

A ground-coupled cooling system uses the ground as a heat sink for reducing the energy consumption of the cooling system. Closed loop ground-coupled cooling systems comprise of either a series of parallel pipes laid out horizontally in the trenches a few metres below the surface or vertically suspended in the boreholes back filled with a good thermal conductance material. A closed loop ground-coupled cooling system uses refrigerant, water or a water-antifreeze solution as a heat transfer fluid in the ground loops. The ground loops reject heat to the earth. Figure 2 shows a schematic of closed loop ground-coupled cooling system with horizontally laid ground heat exchanger loops. The system works with the air handler drawing in hot air from the room.

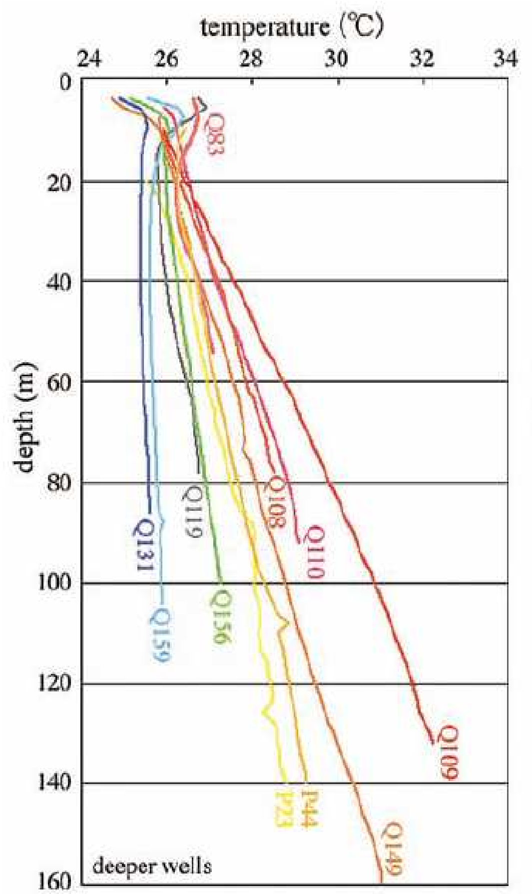

Fig. 1. Temperature profiles of the wells around Hanoi are
(Source: Yasukawa *et al*. 2009)

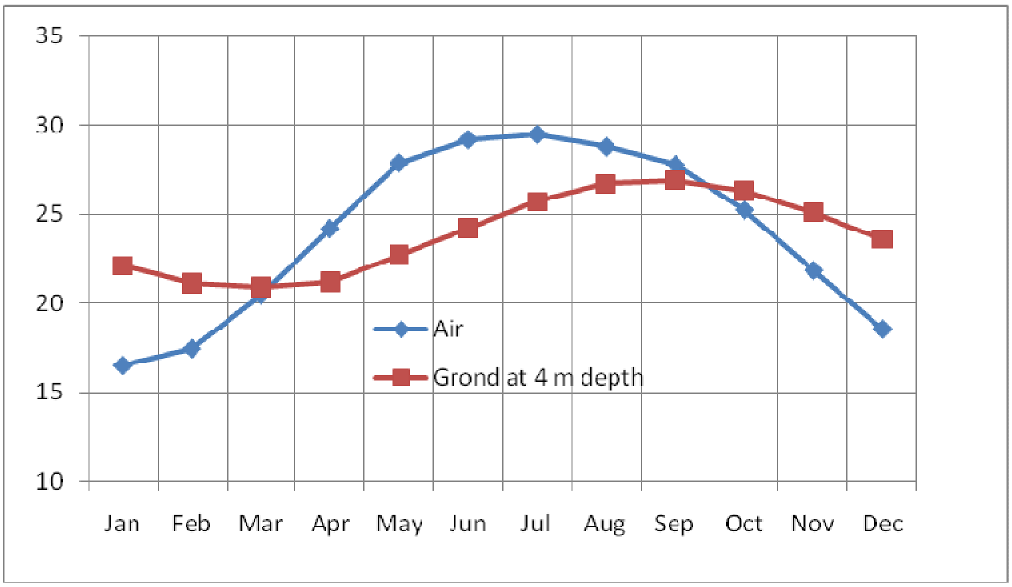

Fig. 2. Monthly calculated "undisturbed" ground temperatures at 4m depth and ambient air temperatures (°C)
(Source: http://apps1.eere.energy.gov/buildings/energyplus/weatherdata)

As the hot air passes over the evaporator coil, the refrigerant being colder than the air, heat flows to the refrigerant thus evaporates the refrigerant. The air becomes colder and flows into the conditioned space. The refrigerant in vapour form flows into the compressor which further increases the temperature and pressure of the vapour refrigerant. As the high pressure and temperature refrigerant arrives at the condenser, the heat from the refrigerant is transferred to the water due to which the refrigerant temperature drops and condenses to liquid. The liquid refrigerant is further cooled as it passes through an expansion valve decreasing both the pressure. This low temperature and pressure refrigerant flows to the evaporator and the cycle is repeated.

In this paper the theoretical performance of a closed loop ground-coupled cooling system for a commercial building in Hanoi was investigated as an alternative to a conventional air-to-air cooling system. The aim of this paper is to compare the effectiveness of ground-coupled cooling system and conventional air-to-air system for commercial building applications in Hanoi. A theoretical computational model for the prediction of the system performance was developed for the vapour compression cooling. Then this was used to compare the performance of ground-coupled cooling system and conventional air-to-air cooling system in Hanoi. The model developed and the results obtained for Hanoi are presented and discussed.

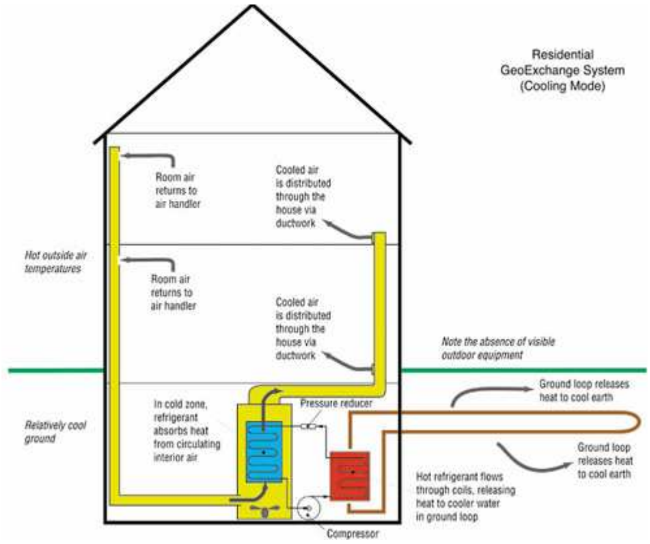

Fig. 2. Schematic of a close loop ground-coupled cooling system
(Source: http://www.greenbuilder.com/sourcebook/groundsource/images/geoexchange_cooling.jpg)

2 Cooling and Heating Requirements in Hanoi

Hanoi is the capital and second largest city of Vietnam. It is located on the bank of the Red River. The climatological information of Hanoi, based on monthly averages for the 93-year period (1898-1990), is shown in Table 1. The Hanoi region is hot in summer and warm in winter. There are over 200 days of cooling demand in a year (Tuyen *et al*. 2010).

It should noted that annual average number of rain days is approximately 150, which make the ground high moisture content and high thermal conductivity and favouring good ground heat exchange.

Figure 3 shows the annual heating and cooling energy requirement for one floor of a typical 50-storey concrete building in Hanoi. The annual heating and cooling are estimated to be 14 MWh and 117 MWh respectively. The peak heating load is 33 kW and the peak cooling load is 64 kW. The heating and cooling demand of this office building was estimated by running a TRNSYS model with Hanoi hourly climatic data provided by Building Technologies Program, US DOE (2010).

The following assumptions were used:

- number of occupants – 54 based on 25 m^2 per person,
- about one third of the occupants have computers,
- the building is occupied from 9 am to 5 pm on weekdays,
- the building is unoccupied at weekends.

Table 1. Climatological information of Hanoi
(Source: World Weather Information Service 2010)

Month	Mean Temperature (C) Daily Minimum	Mean Temperature (C) Daily Maximum	Mean Total Rainfall (mm)	Mean Number of Rain (Days)
Jan	13.7	19.3	18.6	8.4
Feb	15.0	19.9	26.2	11.3
Mar	18.1	22.8	43.8	15.0
Apr	21.4	27.0	90.1	13.3
May	24.3	31.5	188.5	14.2
Jun	25.8	32.6	239.9	14.7
Jul	26.1	32.9	288.2	15.7
Aug	25.7	31.9	318.0	16.7
Sep	24.7	30.9	265.4	13.7
Oct	21.9	28.6	130.7	9.0
Nov	18.5	25.2	43.4	6.5
Dec	15.3	21.8	23.4	6.0

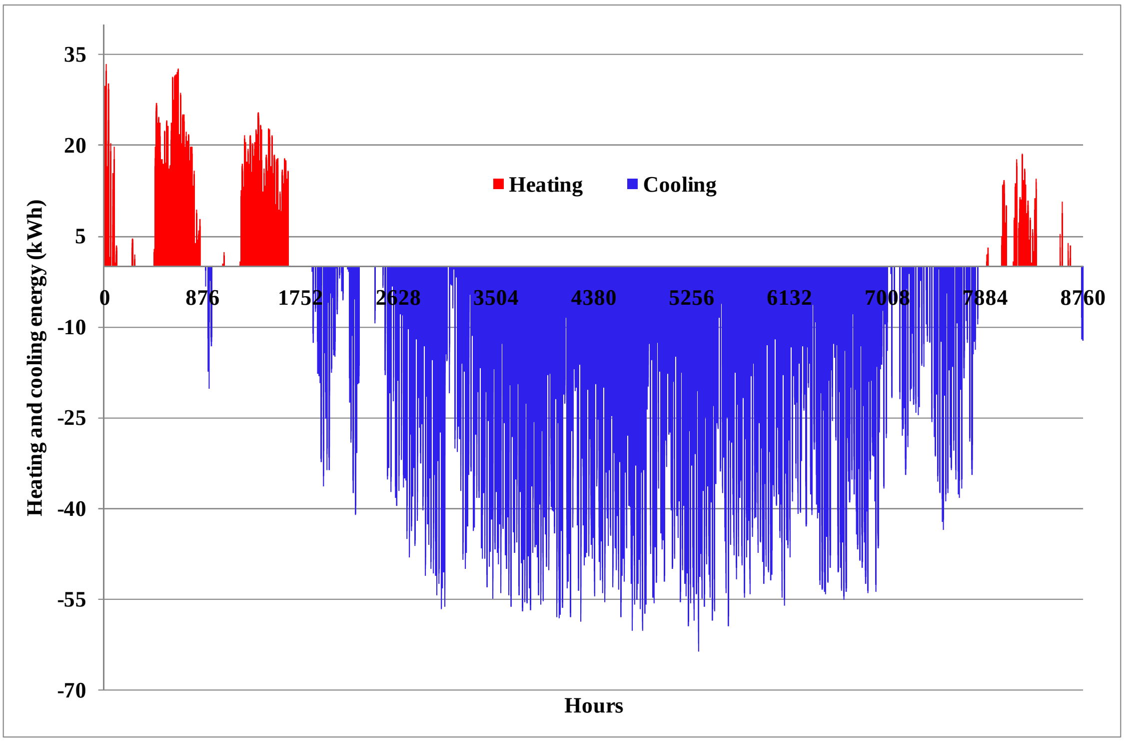

Fig. 3. Heating and cooling demand

3 Performance of the Vapour Compression Cooling System

The cooling coefficient of performance (*CCOP*) of the cooling system is a performance parameter which is defined as the ratio between cooling output (Q_L) and the work input (W) (Fig. 4).

$$CCOP = \frac{Q_L}{W} = \frac{Q_L}{Q_H - Q_L} \quad (Eq.1)$$

$$Q_L = \frac{UA_e \times (T_{ei} - t_e + T_{eo} - t_e)}{\ln[(t_e - T_{ei})/(t_e - T_{eo})]} \quad (Eq.2)$$

$$Q_H = \frac{UA_c \times (t_c - T_{ci} + t_c - T_{co})}{\ln[(t_c - T_{ci})/(t_c - T_{co})]} \quad (Eq.3)$$

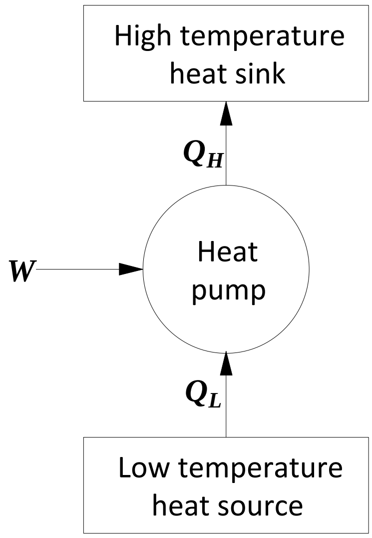

Fig. 4. Energy flow of a cooling system (Source: Lu Aye 2010)

Where *UA* = overall heat transfer coefficient (kW/C)
t_e = refrigerant evaporating temperature (C)
t_c = refrigerant condensing temperature (C)
T_{ei} = evaporator side entering air temperature (C)
T_{eo} = evaporator side leaving air temperature (C)
T_{ci} = condenser side entering air or fluid temperature (C)
T_{co} = condenser side leaving air or fluid temperature (C)

4 Results and Discussion

By using the set of equations (Eq. 1 to 3) (performance prediction model) presented in the previous section and using the following assumptions, the average *CCOP*s for the conventional air-to-air cooling system and the ground coupled cooling system were estimated (Table 2).

- Overall heat transfer coefficient ratio between condenser and evaporator is 1.5 [-]
- Average ambient air temperature is 29.5°C
- Average ground temperature is 26°C
- The evaporating temperature is 10°C
- The supply air temperature is 14°C
- The condensing temperature for the conventional air-to-air cooling system is 50°C
- The condensing temperature for the ground-coupled cooling system is 47°C

Table 2. Performance comparison

	Air-to-air system	Ground coupled system
CCOP	3.2	4.5
Annual electricity used for cooling	37 MWh	26 MWh
Energy saving (%)	-	~30%

By using the performance prediction model it is estimated that for the same air supply and evaporating temperatures the ground-coupled cooling system has about 3°C lower condensing temperatures. This makes the energy efficiency of the ground coupled system better than the conventional air-to-air cooling system.

5 Conclusion

A theoretical computational model for the prediction of the system performance of vapour compression cooling systems has been presented. The model has been used to compare the performance of a conventional air-to-air cooling system and a ground-coupled cooling system for a commercial building in Hanoi. It was found that the potential electricity saving of the ground-coupled cooling system is about 30 % compared to the conventional system. It is recommended that further detailed transient analysis should be done to confirm this finding.

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SOLAR AND WIND POWERED HYBRID AIR-CONDITIONER FOR A RAINFOREST ECO LODGE

[1] Palayangoda L.I, [2] Fernando P.

[1] Postgraduate Student, ICBT Campus, 31 & 33, Hotel Road, Mount Lavinia, Sri Lanka;
[1]Telephone: +94-34-2225265; +94-77-3643151
E-mail: lipa@kth.se; lpalayangoda@gmail.com

[2] Senior Lecturer, University of Peradeniya, 20400, Sri Lanka;
[1]Telephone: +94-81-2393608; Fax: + 94-81-2393611
E-mail: primal@kth.se

ABSTRACT

Eco lodges in Sri Lanka are becoming increasingly popular among foreign and domestic tourists to spend their leisure in a way which is much closer to the nature. The Eco lodges in the rainforest areas are more popular than the others but the high humidity level courses discomfort to the nature lovers. According to climate experts, the temperature of rainforest regions varies from 15 ^oC - 30 ^oC throughout the year and the relative humidity varies 70% to 95% which is on the higher side. The high humidity disrupts the ability of the human body to cool itself, which may lead to a heat stroke. Exceptionally high humidity can also trigger asthma symptoms. Another unpleasant effect of high humidity is the appearance of mold. High humidity makes it easier for molds to reproduce, and they can appear virtually anywhere, damaging whatever they grow upon. This research project has begun with the objective of converting the Lodge area into a comfort zone for the nature lovers. In addition this project will deliver a green lodge by introducing green energy to power up the system. Green energy sources like wind and solar have not been introduced to Eco Lodge sector so far and as a result, this sector contribute significant role damaging to the surrounding environment by generating CO_2 as well as noise by their power up systems. Further it will help the upcoming eco lodges as the base line study for high RH regions and the benefits of green energy.

KEYWORDS

Eco Lodge, Humidity, Green Energy, CO_2

ABBREVIATIONS

AC	-	Alternating current	**L**	-	Length
Ah	-	Ampere hour	**LED**	-	Light Emitting Diode
CEB	-	Ceylon Electricity Board	**Ltd**	-	Limited
CO_2	-	Carbon dioxide	**m/s**	-	Meters per Second
CO	-	Carbon monoxide	**MAX**	-	Maximum
COP	-	Coefficient of performance	**MIN**	-	Minimum
DC	-	Direct current	**MSL**	-	Mean Sea Level
EE	-	Energy efficiency	**Pvt**	-	Private
ESCO	-	Energy service company	**PWM**	-	Pulse-width modulation
GHG	-	Greenhouse gas (climate)	**PVC**	-	Poly Vinyl Chlorite
GMT	-	Greenwich Mean Time	**RH**	-	Relative Humidity
HRC	-	High Rupture Capacity	**SPV**	-	Solar Photo Voltaic
Kmph	-	Kilo meters per hour	**V**	-	Voltage
kW	-	Kilowatt	**W**	-	Width
kWh	-	Kilowatt Hour	**WP**	-	Peak Watts

1 INTRODUCTION

Eco lodges in Sri Lanka are very popular among foreign and local nature lovers. The eco lodges in rainforest areas are more popular than others. However, the high humidity levels of the locations are not comfortable to the nature lovers.

An Eco Lodge at Deniyaya area in Sri Lanka is under the management of Rainforest Ecologic (Pvt) Ltd. According to climate experts the temperature and relative humidity in this region varies from 15 - 30°C and 70 to 95% respectively. This location has an average wind speed of about 5.5 m/s and altitude about 800m above MSL. This land is adjacent to the Sinharaja Rain Forest which is a world heritage site.

Rainforest Ecologic (Pvt) Ltd is facing several problems in the implementation of the Eco Lodge. The problems are,

1. High Relative humidity during the year

   RH is considered as high if its level goes above 60%. The high humidity disrupts the body's ability to cool itself, which may also lead to a heat stroke. Exceptionally high humidity can also trigger asthma symptoms. Therefore, people with heart problems or asthma are advised to be extremely careful during such conditions. Another unpleasant effect of high humidity is the appearance of mold. High humidity makes it easier for molds to grow, and they can appear virtually everywhere, damaging whatever on they are growing up. Mold spores also pose a threat for allergy and asthma sufferers. Dust mites also thrive when the humidity is high. Present in almost every home, these tiny pests are yet another nuisance for people with allergies and asthma.

2. Tough environmental barriers imposed by the Central Environmental Authority of Sri Lanka due to the adjacent Siharaja Evergreen Forest ecosystem

   Sinharaja Rain Forest is a world heritage site and major eco tourism destination that describes as a Tropical Lowland Rainforest or Tropical Wet Evergreen Forest. This forest covers an extent of approximately 11187 ha. From east to west the length of the forest is about 21 km and its width from north to south is about 3.7 km. The Sinharaja forest was initially declared a Man and Biosphere Reserve (MAB) in 1978, as representative of Tropical Humid Evergreen Forest ecosystem in Sri Lanka has been recognized by UNESCO as part of its International Network of Biosphere Reserves.

3. To be a green Eco Lodge

   This Lodge is powered by an alternative energy source as a pilot project. The world's trend is more towards the renewable energy usage as the alternative. Wind and solar energy is more environmentally benign than many alternatives. [1], [2]

The project will implement by ISB Technical Services Limited, the Engineering arm of Industrial Services Bureau (ISB) Kurunegala, Sri Lanka (www.isb.lk). ISB Technical Services Limited (ISBTS) of North Western Province has been active in the field of energy and environment since 1993.Its main strength in the energy field has been the energy management and conservation and introducing new technologies to improve the energy efficiency. As the world's trend is more towards the renewable energy usage, ISBTS is actively involved in introducing renewable energy to the industry as well as for rural electrification.

2 OBJECTIVES

This project has begun with the following objectives.

1. Convert the Lodge area into a comfort zone for living
2. Introduction of green energy to power up the system
3. To be a base line study for high RH regions
4. Promoting green energy among other Eco Lodges in Sri Lanka

3 METHODOLOGY

The methodology consists of nine steps and first five steps will be completed at the first stage and the rest is more likely to complete at the second stage. The first five steps are,

1. Monitoring the variation of RH and temperature in the Lodge area

Table 1. *Measured RH and temperature of the area in the year 2009~2010*

Month	**RH (%)**	**Temperature (^{o}C)**
January ‘10	89.30	21.50
February ‘10	88.70	23.00
March ‘10	86.10	23.80
April ‘10	85.50	24.90
May ‘10	87.10	22.90
June ‘10	87.60	22.50
July ‘09	87.40	23.30
August ‘09	87.10	22.80
September ‘09	87.00	22.60
October ‘09	87.80	21.80
November ‘09	89.10	21.60
December ‘09	89.70	20.90
Average	87.70	22.63

Measurements been taken by Testo 175-H2 humidity data logger. The logger was placed at a room in the lodge and the readings taken per 6 hour basis and taken the average valve for each month. The measurement was carried out during the year 2010. Data for 2009 was taken from ISB Technical Services Ltd, Kurunegala. The recorded maximum relative humidity of the above time period is 96.80%. According to Table 1 the average humidity remains around 87.7 % and the average temperature lays around 22.63 ^{o}C

2. Designing an air-conditioning system

The main purpose of the air conditioner is to reduce the humidity up to 50% which is expected by the eco lodge management.

Table 2: *existing and the desired condition*

Existing Condition		Desired Level	
RH (%)	^{o}C	RH (%)	^{o}C
87.7 (96.8 max)	22.63	50%	As it is

Details of the lodge area needed to condition.

Table 3: *Details of Double rooms available*

	Area/ Capacity	Nos
Double Room	10' x 10'	2
Occupancy		2 people
Family Rooms	15' x 15'	1
Doors	3' x 7'	2
Windows	2' x 4'	6
No of lamps (CFL)	20 W	4
No of Lamps (incandescent)	40W	2

Total load = 36000 btu/ hr (10.56 kW)

Assumed efficiency of the system (COP) = 3.00

Total electricity consumption = 3.52 kWh

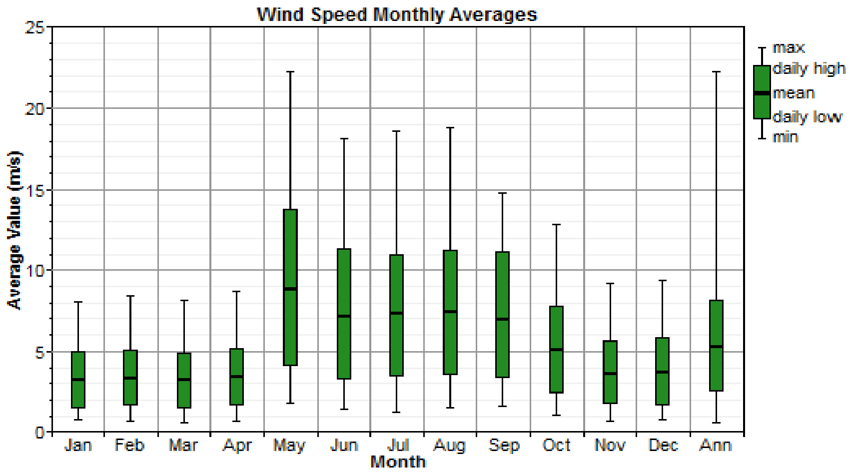
3. Collection of the past wind data measurement for the lodge area

Table 4: *Monthly wind data*

Month	**Avg. wind speed (m/s)**
January	3.20
February	3.35
March	3.21
April	3.45
May	8.83
June	7.16
July	7.35
August	7.45
September	6.99
October	5.07
November	3.63
December	3.70
Average	5.28

Source: Deniyaya CEB wind mast

The Ceylon electricity board (CEB) is monitoring the wind direction and wind speed in Deniyaya area from their wind mast. The data is taken from the CEB by ISB technical Services for the implementation of this project. The wind mast is continuously monitoring the wind data and updating its (



[2] , [7]

Figure 1: *Wind pattern according to HOMMER*

The data taken from CEB is been analyzed by HOMMER and obtained the figure 1. Highest wind speed (8.33 m/s) observed in May. According to figure 1 May to October is the wind season for the lodge area giving an average wind speed of 7.14 m/s. The Rest of the period of the year the average wind speed is 3.2 m/s, which is marginal to run the wind turbine.

4. Collection of the past solar data measurement for the lodge area

Table 5: *Monthly solar resource for the lodge area for year 2009*

Month	Clearness Index	Daily Radiation ($kWh/m^2/d$)
January	0.502	4.715
February	0.581	5.767
March	0.553	5.737
April	0.497	5.187
May	0.468	4.747
June	0.445	4.412
July	0.464	4.629
August	0.432	4.432
September	0.471	4.868
October	0.455	4.555
November	0.484	4.588
December	0.494	4.528

Source: Homer Version 12.19 (14 June 2005) (www.nrel/gov/homer)

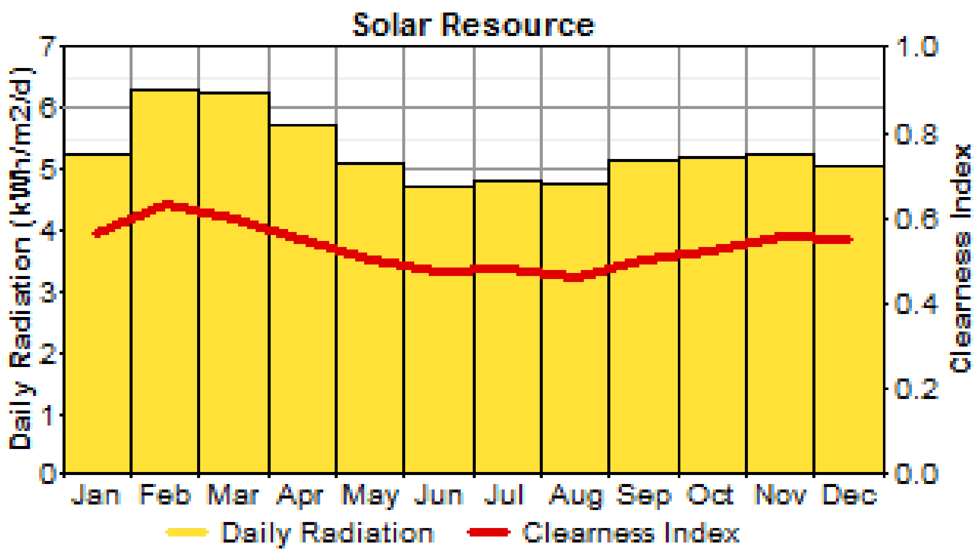


Figure 2: *Solar resource pattern according to HOMMER*

The solar resource is taken for the lodge location (latitude 6^{o} 15' north and longitude 80^{o} 15' West) from the HOMMER software for time zone GMT 6.00, Sri Lanka. According to Table 4 the solar radiation and the cleanness index remains same for the entire annum. February, March and April are the peak months for solar radiation and cleanness index for the lodge area.

[1], [6]

5. Designing of the wind-solar hybrid system

Table 6: *Expected power output from the hybrid system*

Month	Avg. wind speed (m/s)	Daily output from 1 X Whisper200 (kWh)	Daily output from 750W SPV (kWh)	Total (kWh)
January	3.20	1.70	3.0	4.70
February	3.35	2.00	3.0	5.00
March	3.21	1.80	3.0	4.80
April	3.45	2.10	3.0	5.10
May	8.83	12.50	3.0	15.50
June	7.16	12.30	2.1	14.40
July	7.35	12.35	2.1	14.45
August	7.45	12.40	2.1	14.50
September	6.99	10.83	2.1	12.93
October	5.07	5.50	3.0	8.50
November	3.63	2.20	3.0	5.20
December	3.70	2.23	3.0	5.23

[1], [2] , [6] , [7],[9]

Complete System Configuration

Table 7: *Aero generator Whisper 200: 1 nos. including external wind turbine controller (Housed inside master control unit)*

Model	Whisper 200
Rotor Diameter	9 feet
Weight	30 kg
Mount	2.5 inch
Start-Up Wind Speed	3.1 m/s
Voltage	48V
Peak / Rated Power	1000 watts @ 11.6 m/sec
Number of Blades	3
Material of Blades	Polypropylene & Carbon Fiber Composite
Material of Body	Cast Aluminium (Corrosion proof)
Survival Wind Speed	55 m/s
Over-speed Protection	Angle governor & dump load
Controller	External regulator
Bearings	Low Friction, totally enclosed & self lubricated
Controller output	48V DC

Wind Controller:

(a) Capacity = 1KW/48V
(b) Modular construction for easy replacement.
(c) All control cards are easy replaceable plug-in type.
(d) Operating ambient up to 52^{0}C
(e) Painting : Powder coated
(f) Built-in Battery reverse current flowing blocking diode
(g) Short circuit protection.
(h) Overcharge protection by load diversion to dump load.

Tower for Whisper 200 Wind Turbine

Table 8: *Tower details for the wind Turbine*

Construction Material	Mild Steel
Protection from corrosion	Hot Dip galvanized
Type	Tilt up tubular 2.5” tower, 60ft. with guy support.

SPV modules: 1 Complete Set

Table 9: *Details of the SPV modules*

Capacity	750 WP 48V nominal (750 WP x 2)
Make	MNRE approved
Peak power per module	75 Watt peak
Dimension & Weight	1 Set : 1206mmLX530mmW & 7.63 Kg
Total area required	20 sq meters
Temperature	- 40 to 90^0 C
Wind Load	Up to 200 kmph
Humidity	0 to 100 %
Voc of each module	19 volts
Type of cell	mono/poly crystalline silicon
Efficiency of cell	13% to 15 %
Lamination type	Vacuum laminated Glass to tedler

Module mounting structure: 2 set

Table 10: *Details of the module mounting structure*

Construction Material	Mild Steel (angle / flat)
Protection from corrosion	Hot Dip galvanized

Battery: 1 Set

Table 11: *Details of the battery bank*

Estimated Units generated from system	**5 to 10 units/day**
Voltage configuration	48V
D.O.D allowed tubular battery	80 % max
Battery charge Efficiency	85 %
Autonomy considered	**1.8 days**
Battery capacity required	48V 600 Ah (2X48V 300Ah)
Type	Tubular lead acid flooded electrolyte
Positive plate	Tubular
Negative plate	Pasted Flat
Voltage of each cell	2 volts nominal
Electrolyte	Dilute Sulphuric Acid

Solar charge Controller (housed inside master controller)

(a) Capacity = 2.5 kW/48V (60 amps max) (This single module used for 1kW & 2kW SPV. For 3kW to 5kW 2 modules used. For 6kW to 7.5kW 3 modules used)
(b) Modular Construction for easy replacement
(c) Built-in Amp and Volt meter
(d) All status monitoring indicating LED and control switches are available on the front
(e) All control cards are easy replaceable plug-in type.
(f) Operating ambient upto 52^{0}C
(g) Painting : Powder coated
(h) Built-in Battery reverse current flowing blocking diode
(i) Battery line short-circuit protected HRC Fuse with carrier
(j) PWM based overcharge protection/load diversion

Inverter: 1 Set

Table 10: Details of the inverter set

Max Load	**2.5 kVA**
Power factor	85 %
Inverter Efficiency	90 %
Inverter capacity required	2.5 kVA 48 V DC
Efficiency	**90 % +**
Duty	Continuous
Waveform	**Pure Sine wave**
Ambient	60 deg. C.
Protection	IP under voltage, IP over voltage, OP overload, OP short – circuit
Output regulation	+/- 2.5 %
Input DC bus voltage variation	+/- 25 %
Relative Humidity	98 %
Instrumentation	Output Voltage, current & kWh.
Power Device	MOSFET / IGBT
Control	Pulse Width Modulation
Power Factor	0.85

Master Controller

Solar charge controller for 750WP SPV systems	1 no. Built in
Wind charge controller with dump load for 1no. 48V 1.0 kW wind turbine	1 no. Built in
Metering :- a) Solar generation in Amps. b) Wind generation in Amps. c) Total generation in Amps. d) Grid based AC/DC charger supply in Amps. e) System voltage (D.C) f) Load supply in Amps.	Built in
100 Amps heavy duty battery C.C charger AC 240 volts input from AC Mains to 48 Volts DC output for battery charging	Built in
AC mains /battery AC/DC charger auto start / stop automatic switch.	Built in

Copper Cables: As required

Type	Copper cables of 1.1kV grade PVC insulated

Connection Diagram of the System

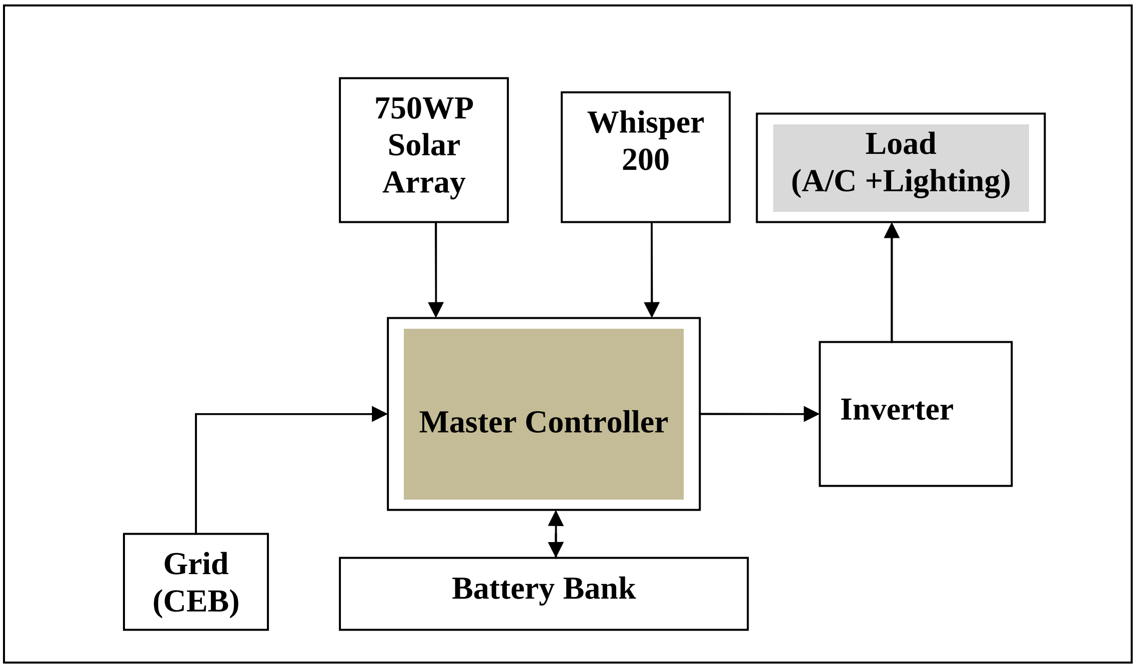


Figure 3: ***Connection Diagram of the System***

[2] , [9]

4 RESULTS AND DISCUSSION

An air conditioner is more likely functioning as a de-humidifier as it maintains indoor temperature of the lodge area around 22 ^{o}C. The energy consumption of the air conditioning system is approximately 3.52 kWh. The minimum power delivered by the wind solar hybrid system is 4.7 kWh and the system has an excess of 1.2 kWh to cater the lighting load of the rooms.

When a wind turbine and an SPV system are interfaced, the power generation from these two is mutually supplemented, and the resultant hybrid system offers a reliable and cost-effective electricity supply in a decentralized mode. The major advantage of solar-wind hybrid system is the enhanced reliability when solar and wind power production is used together. Additionally, the size of battery storage can be reduced slightly as there is less reliance on either method of power production. Often, when there is no sun, there will be plenty of wind. Wind speeds are often low in periods, when the sun resources are there at best. On the other hand, the wind is often stronger in seasons, there are less sun resources. Even during the same day, in many regions worldwide or in some periods of the year, there are different and opposite patterns in terms of wind and solar resources. Such different patterns can make the hybrid system the best option in electricity production. A hybrid wind-solar electric system demands a higher initial investment than single larger systems.

Advantages through green power for an eco lodge:

- Door opening for green certifications and carbon credits
- No net emission of greenhouse gases

- Renewable sources of energy are available in most locations
- Generally, technologies are proven, reliable and require minimal maintenance
- Systems do not require a constant input of consumables (no motoroil, filters, etc)
- Life cycle costs of wind and solar can be very attractive
- Long service life (15-25 years)

General drawbacks of a green energy system in Sri Lanka are as follows.

- High initial cost (especially for photovoltaic systems)
- Lack of soft loan schemes to tackle the high initial cost
- High-tech components in renewable electricity generation systems
- Low power output (except in the areas with large wind and hydropower resources)
- Often needs some type of a non-renewable energy backup system
- High cost and unavailability of high-efficiency appliances in local markets

[3], [6] , [7] , [8]

5 CONCLUSIONS

As it was presented in this paper, practical aspects for hybrid (wind and solar)systems are very feasible application for eco lodges or small hotels located in remote location of Sri Lanka whose location’s characteristics give it ability to exploit, not one but two alternative energy sources for the generation of a large part of the necessary energy. Of course, before the installation of a hybrid system, a very good study of the climatic characteristics such as the wind potential and the solar radiation must certainly be one. So, the energy demand for hotels and lodges in the country can be partially or fully covered with the use of wind and solar hybrid system as standalone or grid-connected systems when location characteristics and areas’ wind energy potential is good.

This eco lodge will be a model for green energy adopted lodge and it will deliver the message of green concept towards the Sri Lankan hotel industry.

ACKNOWLEDGMENT

We wish to express our sincere thanks to ISB Technical Services (ISBTS), Kurunegala and the Rain Forest Eco Lodge Pvt Limited, Colombo for the given opportunity to carry out the project.

We take this opportunity to express our sincere thanks to the Manager, the Eco Lodge, Deniyaya for accommodating and guiding us to carry out the activity. We also thank other Officials and Personals for their assistance extended to us in various ways for this activity. We would like to thank ISBTS for giving their assistance by providing Equipment and providing the historical wind and solar data for the eco lodge area.

ABOUT THE AUTHORS

L. I. PALAYANGODA, B.Sc. Hons. Ruhuna, AME (SL) is currently reading for his MSc. at the Department of Energy Technology, Royal Institute of Technology (Kungl Tekniska Högskolan -KTH), Stockholm, Sweden through ICBT Campus, Mount Lavinia, Sri Lanka. His research interests are in the areas of Sustainable Energy Engineering.

P. FERNANDO, B.Sc. Hons. University of Moratuwa, Sri Lanka., M.Sc. KTH Sweden, Ph.D. KTH Sweden., is a Senior Lecturer at the Department of Mechanical Engineering, University of Peradeniya, Sri Lanka. His research interests are in the areas of Sustainable Energy Engineering.

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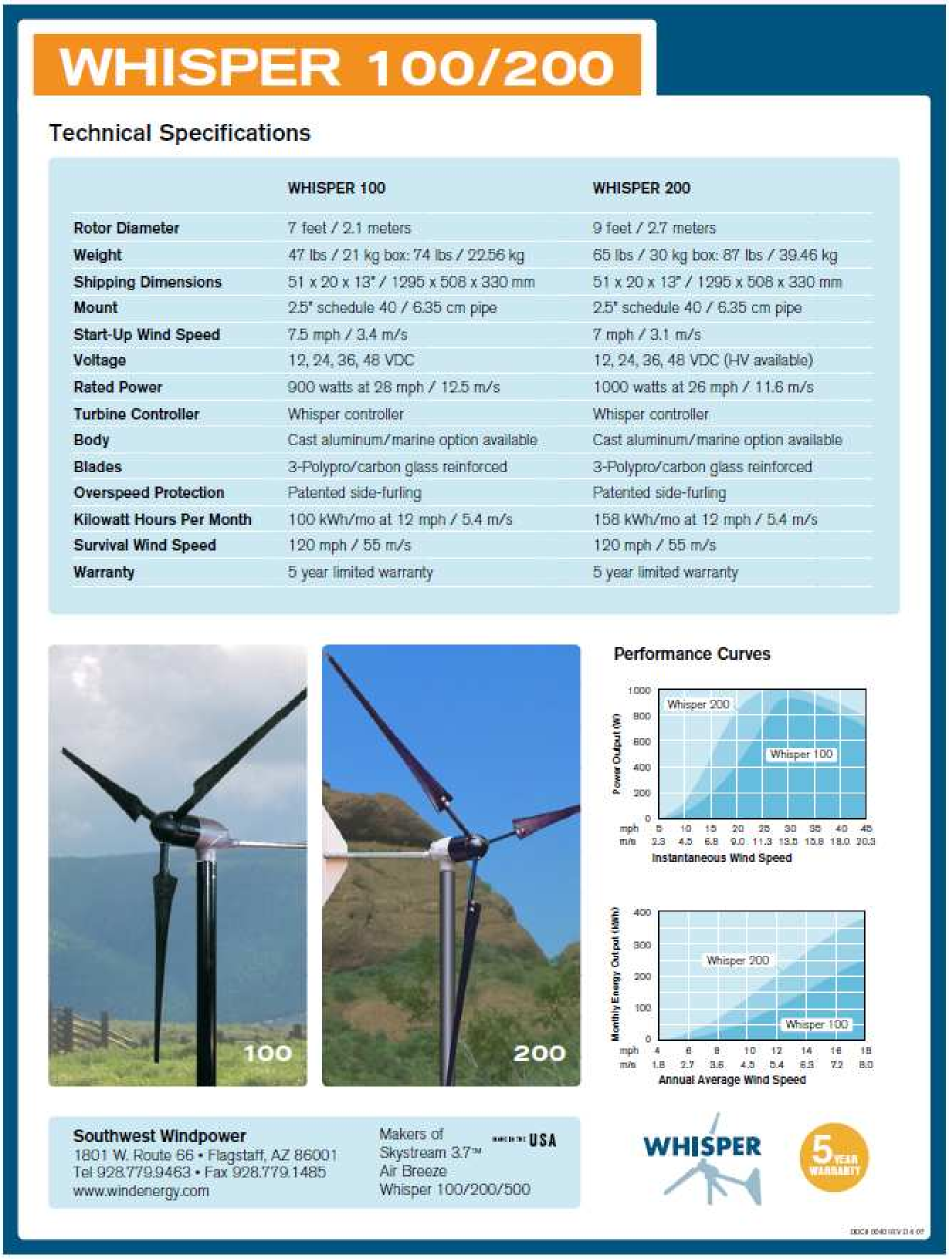
ANNEX



WHISPER 100/200

Technical Specifications

	WHISPER 100	WHISPER 200
Rotor Diameter	7 feet / 2.1 meters	9 feet / 2.7 meters
Weight	47 lbs / 21 kg box: 74 lbs / 22.56 kg	65 lbs / 30 kg box: 87 lbs / 39.46 kg
Shipping Dimensions	51 x 20 x 13" / 1295 x 508 x 330 mm	51 x 20 x 13" / 1295 x 508 x 330 mm
Mount	2.5" schedule 40 / 6.35 cm pipe	2.5" schedule 40 / 6.35 cm pipe
Start-Up Wind Speed	7.5 mph / 3.4 m/s	7 mph / 3.1 m/s
Voltage	12, 24, 36, 48 VDC	12, 24, 36, 48 VDC (HV available)
Rated Power	900 watts at 28 mph / 12.5 m/s	1000 watts at 26 mph / 11.6 m/s
Turbine Controller	Whisper controller	Whisper controller
Body	Cast aluminum/marine option available	Cast aluminum/marine option available
Blades	3-Polypro/carbon glass reinforced	3-Polypro/carbon glass reinforced
Overspeed Protection	Patented side-furling	Patented side-furling
Kilowatt Hours Per Month	100 kWh/mo at 12 mph / 5.4 m/s	158 kWh/mo at 12 mph / 5.4 m/s
Survival Wind Speed	120 mph / 55 m/s	120 mph / 55 m/s
Warranty	5 year limited warranty	5 year limited warranty

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A Bed Room Similar to Lodges'

TRIGENERATION - A GREEN APPROACH FOR MEETING TOTAL BUILDING SERVICE REQUIREMENT

Chamila Ranasighe[1], Suranjith Thennakoon[2] and Leelananda Rajapaksha[3]

[1,2] Department of Mechanical Engineering, University of Peradeniya, Sri Lanka
(chamilaranasinghe@gmail.com,thennakoonsuranjith@yahoo.com)

[3] Associate Professor, Department of Mechanical Engineering, University of Peradeniya, Sri Lanka.
E-mail: lraja@ pdn.ac.lk
Telephone: +94-81-2393606 Fax: + 94-81-2388158

Abstract: Tourism industry in the country is presently seeing a considerable growth, with demand for hotel and accommodation industry to keep pace. In expanding this sector, provision of electricity and other utilities is essential. In this regard, the availability of electricity supply and the quality of available power are serious concerns when it comes to expanding the facility to remote but attractive areas. As a means of handling this situation, in this paper the possibility of using stand alone trigeneration to meet heating, cooling and electricity demand of such facilities is explored and compared with the present system of grid connected power and diesel based heat generation. A comparison of the two systems; trigeneration and present base case scenario, reveals that there will be over 50% reduction in CO_2 emission and about 41% saving in energy bills over those of base case when a trigeneration system is sized based on the criterion of meeting total cooling demand using vapour absorption refrigeration. A simple payback estimates shows that recovery of investment for trigeneration system is well under two years.

Keywords: cogeneration, trigeneration, absorption refrigeration, air-conditioning, CO_2 emission

1. Introduction

Growth in tourism industry demands improvement in necessary infrastructure facilities such as accommodation, transportation. Accommodation with standard facilities such as restaurants and other recreation aspects is one important factor that affects the visitor numbers. When trying to expand and improve accommodation facilities in remote but attractive areas, availability and quality of electricity supply are serious concerns where preference in many new cases to have an independent source of power supply such as a diesel generator.

In general, hotel facilities that plan to attract visitors need maintaining certain level facilities where making available of utilities such as lighting, air conditioning, hot water and low-pressure steam (< 5 bar) are essential. In the present practice, majority of existing service providers use power from national grid to run vapour compression chillers to produce required comfort air conditioning, and operate low pressure boilers to meet the steam and hot water demand. In doing so, these facilities use two sources of energy; electricity for running air conditioning machinery and another fuel (diesel, gas or fuel oil, biomass etc.) to run the boilers.

The above indicated end use of energy provides an opportunity for integration of the mechanisms to obtain the required utility services (cooling, heating and power) based on a single source of energy. This paves the way for cogeneration, trigeneration or polygeneration depending on how the scheme of energy use are planned and how the waste energy from one step is utilized in another. Such planning of the use energy resources certainly contributes to cutting down the carbon dioxide emission and reduced energy bills for the industry concern. Based on above background of timely need in the country, this paper evaluates the energy cost and benefits of trigeneration using the latest technologies of steam turbines, boilers and LiBr+H_2O absorption chiller.

2. Theoretical background

2.1 Steam power plant

Steam turbines or gas turbines are the usual prime movers in fossil fuel or biomass based power plants. In cyclic operation, these systems generally rejects considerable amounts of thermal energy in the form of latent heat of steam being condensed or thermal energy in high temperature combustion products. Fig 1 shows a schematic diagram of main hardware assembly of a stem power plant system and Rankine cycle (with superheat), which represents the thermodynamic cycle for a conventional steam power plant on a property plane.

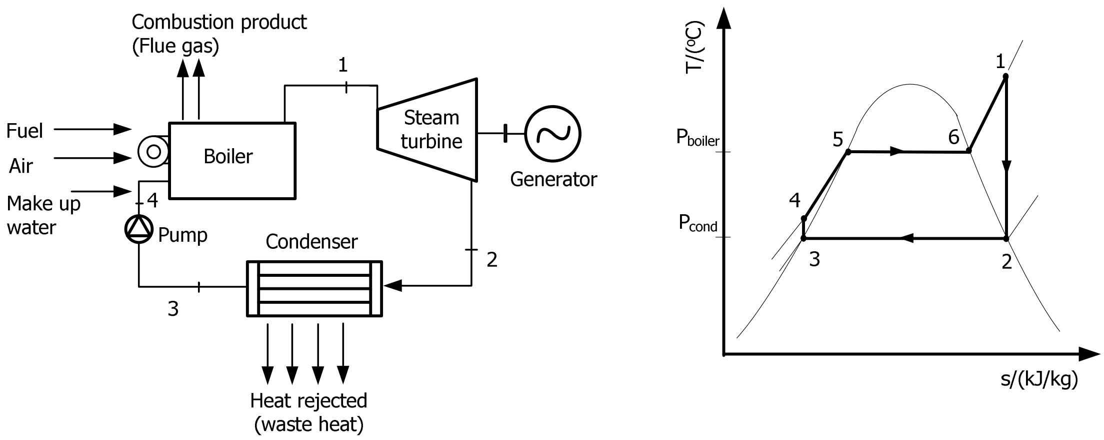


Fig 1 – Schematic diagram of hardware arrangement of a steam power plant (left) and Rankine cycle (with superheat) on temperature - entropy (T-*s*) plane (right)

In Fig 1, process 1-2 produce useful work that drives an electricity generator to produce electric power, whereas process 2-3 is where the waste heat from the system is dumped to the environment. The amount of heat or work interactions of these processes could be estimated using the first law of thermodynamics. In a cogeneration perspective, the waste heat at the condenser is the useful product. In general, the efficiency of steam power plants vary in the range 25 % to 45%, which to a greater extent depend on the operating parameters of the power plant, number of turbine stages, the capacity and the fuel etc [1, 2]. To elaborate on this aspect, Fig 2 presents a general picture of how the selection of operating parameters of the steam cycle affect heat output and efficiency for a unit work output, where Pc is the condenser pressure .

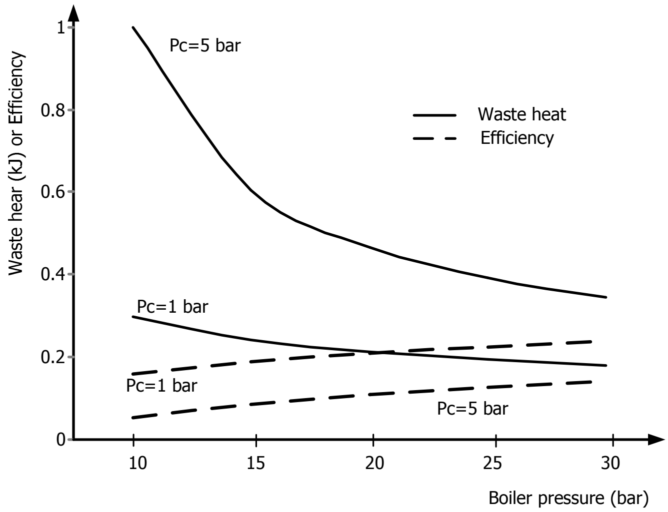


Fig 2 - Effect of main operating pressures on the heat output and efficiency of steam power plants (generated using REFPROP [3] based on Rankine cycle with superheat)

2.2 Heat driven refrigeration

Absorption refrigeration cycle is similar to the vapour compression refrigeration cycle in certain respects. However, when compared with vapour compression cycle, the absence of an electrically driven compressor and the presence of two fluids (refrigerant and absorber) in place of a single refrigerant are the main differences of absorption refrigeration cycle. Fig 3 shows the essential components of a vapour absorption refrigeration system, where the heat supplied to the boiler drives out the refrigerant from refrigerant and absorber mixture. Since absorption cycle is a heat activated thermal cycle, it exchange heat with surroundings with no appreciable input of mechanical energy, and any source of heat, including low-temperature waste heat can be used as the source of thermal energy.

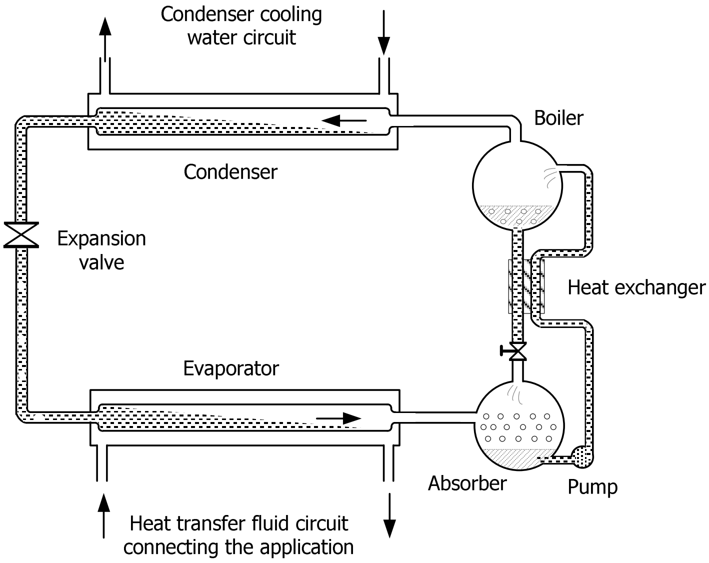


Fig 3 – The essential components of the vapour absorption system [4]

The working fluids in absorption systems achieve their intended cycle functions by undergoing phase change. For known combinations of refrigerants and absorbers, over pressure ranges of interest, $Q_{evp} \approx Q_{cond}$ and $Q_{boiler} \approx Q_{absorber}$ as the latent heat of evaporation and condensation are relatively constant when operating further away from the critical point [5]. This refers to relatively lower cooling coefficient of performance (COP) of absorption system (COP = Q_{evp}/Q_{boiler}) in comparison with vapour compression systems, where Q_{evp} is the amount of heat absorbed at the evaporator and Q_{boiler} is the amount of heat added at the boiler.

Two common refrigerant, absorber combinations in common use are water and lithium bromide (LiBr), and ammonia and water. In the case of LiBr and water mixture, water is the refrigerant and this combination is commonly used for producing chilled water at temperatures above 3 ^{o}C [6], mainly for air conditioning applications. In the case of water and ammonia combination, ammonia being the refrigerant, it is possible to achieve temperatures below 0 ^{o}C for industrial applications such as making block ice, where temperatures below – 10 ^{o}C is the requirement.

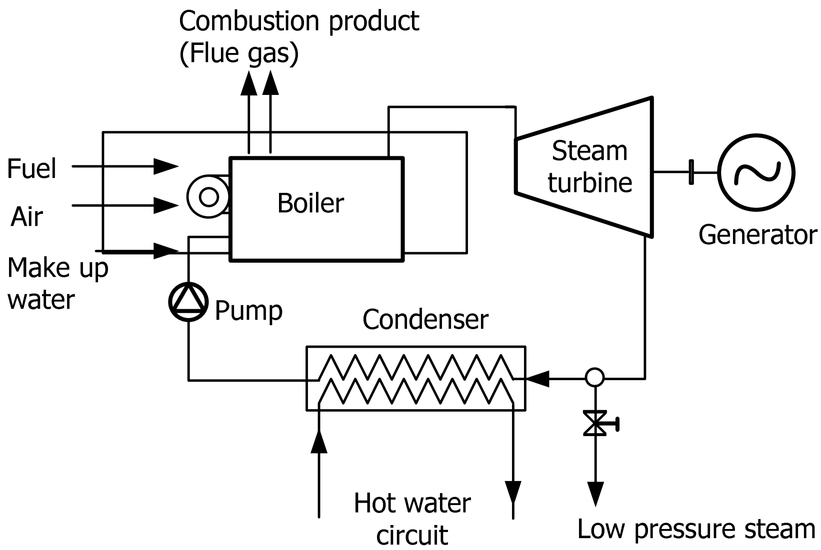
In the context of the theme of this article, the LiBr and water combination is the suitable working fluids for absorption systems to be used in utilizing waste heat to produce comfort air conditioning in industrial applications.

2.3 Cogeneration or Trigeneration to meet end user demand

In many cases of power plant system, in a carefully thought, well planned energy application, at least part of the thermal energy in the waste stream could be recovered for useful purposes. The present day technologies allow generating power as well as use of waste heat in meeting heating demands of the end users, enhancing the overall energy conversion efficiency by a considerable degree. Main benefits of cogeneration are lower consumption of primary energy and associated low energy bills, no or reduced transmission and distribution losses, less burden on national grid and less environmental pollution [7]. The content of this section mainly discuss steam power plants associated cogeneration concepts.

When considering the end user's requirements, for the selected industry concerned, it makes more sense to use cogeneration or trigeneration plants to meet the service requirements of electricity, cooling and heating. When the waste heat from steam power plant is available in a temperature range 150 ~ 200 ^{o}C, for an application that need specified amounts of electricity, air conditioning and heating (hot water and steam), a cogeneration system or a trigeneration system could be planned to completely meet one, two or all three aspects above.

Fig 4 presents possible cogeneration and trigeneration schemes that suits an application requiring power, cooling and heating.



a) Cogeneration system – Power and heat

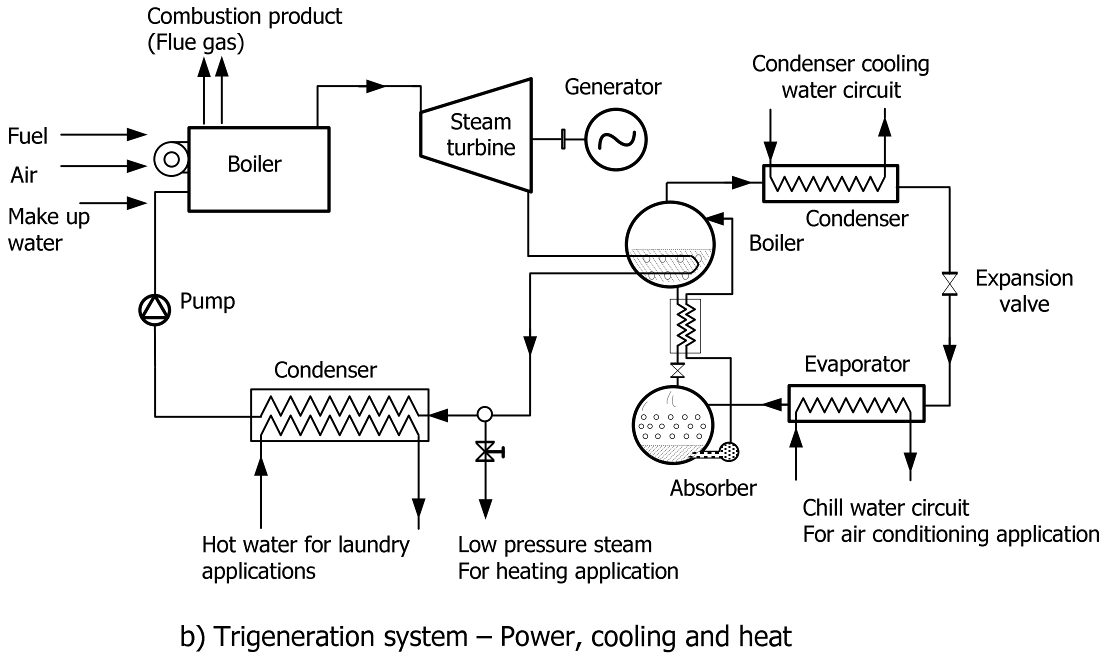


Fig 4 – Schematic diagrams of cogeneration and trigeneration options

2.4 Carbon dioxide emissions from heat and power generation

The saving in emissions and resulting reductions in environmental pollutions is based on the concept of reduced use of fossil fuel when implementing cogeneration concept for an application which use different sources of energy otherwise to meet the heating/cooling and power demands. Table 1 presents emission factors from heat and power generation based on standard power generation technologies. However, these values also depend on the overall efficiency of the plant.

Table 1 – CO_2 emission from electricity and heat generation for different fuels [1, 8, 9]

Fuel	g CO_2/kWh(elect)	g CO_2/kWh(thermal)
Coal	1260	370
Diesel	810	240
Liquefied petroleum gas	650	240
Residual fuel oil	635	260
Natural gas	370	230

2.5 Technology options and costs

Steam turbine technology is a time tested and well established area of power generation field so that manufacturers of steam turbines are found all over the world, covering capacities from few kilowatts to thousands of megawatts. this invariably means one tend to get a number of cost options depending on quality, country of origin, brand and manufacturer etc. Absorption refrigeration technology on the other hand, though time tested and reliable technology, has not developed to a level of global manufacturing as steam turbine industry. Very few manufacturers produce these systems and mostly use manufacture specific component level designs. Table 2 presents present days cost of the two machinery items obtained from manufacturers.

Table 2 - Cost of steam power plants and absorption refrigeration systems

Item	Price ($/kW)	Remarks
Steam power plant	200 ~ 300	Prices include turbine, generator, controls and tools, Not included installation technical assistance charges, steam piping and insulations, and local installation charges, taxes, VAT etc.
Absorption refrigeration system ($LiBr+H_2O$)	125 ~ 175	

3. Results and discussion

3.1 Cases compared and discussed

In order to quantify the cost and benefits of trigeneration, a real life application need to be used in estimates and planning so that the situation concerning power consumptions and energy bills could be compared. For this purpose, we consider a hotel of general description given in Table 3, located in a remote part of the country where stand-alone power supply is preferred due to issues of local power quality, interruptions and availability of required capacity.

Lets consider the base case of obtaining power from national grid for running the air conditioners (based on vapour compression chillers) and all other electrical loads including lighting, and diesel fired boiler is used for generating low pressure steam and hot water. Then, as an alternative, lets consider a trigeneration system that has been planned and sized to provide the required air conditioning based on vapour absorption refrigeration using LiBr and water combination.

Table 3 – Details of the selected case for comparison of benefits of trigeneration vs conventional use of energy

Parameter	Value/units	Remarks
Number of rooms	100	Average floor area of a room 500 ft^2
Common areas	30% total room area	Lighting intensity, 1.2 W/ft^2
Outside temperatures		Dry bulb and wet bulb
Day time AC demand	100%	
Night time AC demand	50% of day time demand	
Lighting intensity	1.0 W/ft^2	Based on recommendations of US dept of energy and ASHRAE 90.1-2007 [10]
Kitchen area	5% total room area	
Kitchen lighting	1.2 W/ft^2	Based on recommendations of US dept of energy and ASHRAE 90.1-2007 [10]
Kitchen equipment power	25% of total lighting	
Hot water consumption per room, including laundry needs	50 liter per 24 h	Water at 50 ~ 60 ^oC, obtained through hot water boiler presently
Steam generation	1000 kg/h for 6 ~ 8 hours per day (24 hour)	Based on usage of existing facilities, total steam generation for steam bath, sauna etc, presently generated using diesel, at 7 bar
Boiler house power consumption	40 kW	

The operating pressure of the boiler is selected to be 40 bar, which produce a steam flow rate sufficient to run the absorption chiller after expansion through the turbine where electricity is generated. The steam is first fed to steam turbine and the turbine exit is set to provide sufficient pressure for the chiller as given in Fig 4b. Used steam leaving the chiller is condensed (if necessary) and sent back to boiler. If steam is bled off as low pressure steam, make up water will have to be provided. Table 4 summarizes the two cases

Table 4 – Main operating parameters of the base case and estimated operating parameters of trigeneration system

Parameter	Value	Base case	Trigeneration (Fig 4b)
Air conditioning cooling load	1170 kW	Use vapour compression chillers, COP of the plant 2.25, ancillary loads 10% of compressor load (total estimated load: 580 kW)	Use vapour absorption chillers, steam demand 1255 kg/h, cooling capacity 1163 kW$^{\$}$
Lighting and other electrical loads	150 kW	Obtained from the grid at Rs. 9.30 per kWh	Obtained through generated power from steam turbine, which is estimated to be 176 kW
Low pressure steam and hot water	1000 kg/h steam	Use a diesel fired boiler of steam generating capacity 1000 kg/h. Diesel cost Rs. 75 per liter	Use condenser heat and exit steam from chiller for hot water and low pressure steam

$^{\$}$ – Obtained from details of commercially available absorption chiller units, ancillary electricity demand of the chiller is 30 kW

In this instance, the sizing criterion of the trigeneration plant is such that it has been planned to meet the complete air conditioning load. The boiler has been selected to provide required steam flow rate for the chiller, and the power generation corresponds to this mass flow rate of steam which meet the heat requirement of the absorption chiller. The power generated in the steam power plant can be used in house or exported to national grid. In this case the amount of power generated is too small for exportation, however, sufficient to meet electricity demand in house.

3.2 Benefit of trigeneration system

When using the trigeneration system configuration in Fig 4, and emission factors given in Table 2, the estimated fuel cost and carbon emissions from the two power supply schemes are given in Table 5. In arriving at these results, the night time air conditioning demand is assumed to be 50% of that during the day, and the boiler runs at 50% of its capacity during the night.

Table 5 – Daily use of energy of trigeneration system and base case

Parameter	Units	Base case	Trigeneration $$
Total electricity consumption	kWh/24 h	13,031	2,591
Total heat generated, kWh/24 h	kWh/24 h	11,440	17,921
Total fuel used for heat generation	Litre/24 h	532.2	1,888

$$ - Boiler combustion efficiency: 92%, heavy fuel oil fired boiler, density and calorific of fuel oil 950 kg/m3 and 39.2 MJ/kg respectively,

Table 5 presents estimated daily energy use of the two cases considered in the study. In the case of trigeneration, there is only one source of energy; fuel oil, that fuel both modes of energy. This is the very feature of cogeneration which is explored in this example to obtain all three utilities; cooling, heat and electricity, for the facility concern.

Table 6 presents estimated cost of energy for two cases considered as well as the reduction of carbon dioxide emissions when a new facility is planned to take the advantages of trigeneration to meet all the utility services in house. The total energy bill of Rs 4.8 million per month of the base case has reduced to little under Rs 2.0 million per month when trigeneration is used with a sizing criterion of meeting the total air conditioning demand in this example. The other benefit of this attempt is the amount of carbon released to the atmosphere reduces to a half of that of the base case which is a very significant achievement when considered with the saving in energy bills.

Table 6 – Energy cost and carbon emissions of trigeneration system and base case

Parameter	Units	Base case	Trigeneration $$
Total electricity bill	Rs/month	3,635,660	Use in house power
Total oil bill (for heat generation)	Rs/month	1,204,211	1,956,846 £
Amount of CO_2 released	kg/year	5,026,219 (1)	2,516,165 (2)

$$- Price of fuel oil Rs 50 per liter, price of electricity Rs 9.30/kWh, electricity generated in-house used for meeting lighting and other electrical loads

£ - After deducting equivalent electricity cost of lighting and other electrical loads at Rs 9.30 per kWh.

(1) – For diesel based power generation, with CO_2 emission of 810 g/kWh (elect)

(2) – For fuel oil based heat generation, with CO_2 emission of 260 g/kWh (thermal)

4. Concluding remarks

The benefits of using trigeneration in a potential local industry was discussed with estimates based on information obtained from relevant literature and justifiable assumptions. As shown in the results, the concept, if implemented with due planning and equipment sizing in the development of tourism industry or any other similar application, could bring in substantial environmental and financial benefits while reducing the amount of raw energy imported for thermal power generation.

A quick estimate based on 250 $/kW for steam plant and 150 $/kW refrigeration system, and when local tax and installation cost included, the above trigeneration system has an attractive pay back duration of under 2 years depending on the other uncertainties associated.

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About the Authors

Chamila Ranasighe, BScEng (Hons, First class) Peradeniya and **Suranjith Thennakoon** BScEng (Hons, Second class upper division) Peradeniya, are both young members of the academic staff of dept of mechanical engineering, University of Peradeniya

Leelananda Rajapaksha, PhD, CEng, MIMechE, is an Associate professor attached to the dept of mechanical engineering, University of Peradeniya

USE OF GREEN FACADES IN SUSTAINABLE BUILDING ENVIRONMENTS: QUANTIFYING THE UPTAKE RATES OF AIR POLLUTANTS BY FACADES DRAPED WITH TROPICAL CREEPERS.

V. Jacob Thottathil[1], C. Balamuralikrishna[2], S.Ghosh[3]

[1] PhD student, School of Mechanical and Building Sciences, VIT University, Vellore.
[1]E-mail: jacobtvarghese@rediffmail.com, Telephone: +919500363276

[2] Professor, School of Mechanical and Building Sciences, VIT University, Vellore.
[2]E-mail: cbalamuralikrishna@vit.ac.in, Telephone: +918124845155

[3]Senior Professor, School of Mechanical and Building Sciences, VIT University, Vellore
[3]Associate, School of Earth and Environment, University of Leeds, United Kingdom
[3]E-mail: satyajitg@vit.ac.in, Telephone: +91 416 2202207

Abstract: Green facades are an attractive option for sustainable built environments. Their use is well documented for much of the developed world. It is an irony that although tropical creepers are ever green, quick growing and lush, their use as facades in developing nations is limited. In this study, we present results from two facades. The first a controlled façade draped with Passion flower creepers is exposed to clean unpolluted air within the School of Mechanical and Building sciences, VIT University . The second identical façade, also draped with the same creeper, is exposed to varying levels of SOx, NOx and Suspended Particulate Matter at a busy traffic junction on Katpadi road Vellore , In Tamilnadu, India . Sensitivity studies shall be presented based on the deposition pattern of pollutants on these two facades. Scanning Electron Micrographs will elucidate the mechanistic details of the deposition patterns of suspended particulate matter, whilst liquid extraction methods coupled with Spectrophotometric analyses will reveal the concentrations of sulphates and nitrates assimilated by the stoma and the mesophilic tissues from street pollution. Finally, results from a computational model will be presented enabling one to calculate the deposition velocities of SOx and NOx on to the facades.

Keywords: *Green facades, Deposition Velocity, Oxides of Sulphur and Nitrogen, Passion Flower.*

1. Introduction

Green facades are a cladding system for buildings in which climbing plants, or in some cases trained shrubs, cover the surface of a building (see Figure1). Traditionally, self-clinging climbers have been used because they require no supporting network of wires or trellis. Modern facade greening, however, favors the use of climbers supported by steel cables or trellis, holding the plants away from the building surface- we have used such Facades (see Figures 2 and 3) for our experimental and theoretical analyses where we draped them with Passion Flower Creeper *(passiflora caerulea)*

The coastal district of Tamil Nadu experiences a hot humid climate, which is particularly favourable for the rapid growth of shrubs and creepers. In this study, the suitability of creepers on green facades is examined. These are not just aesthetically appealing; they are efficient removers of air pollutants. Green facades are an attractive option to reduce energy requirements in residential as well as office buildings.

Climbers can dramatically reduce the maximum temperatures of a building by shading walls from the sun. They can reduce the daily temperature fluctuation by as much as 50 percent, a fact of great importance in warm-summer climate zones. There is thus a contribution to the reduction of the heat island effect and dust generation. Evergreen climbers provide winter insulation, not only by maintaining a pillow of air between the plant and the wall, but by reducing wind chill on the wall surface. The use of evergreen climbers on walls that do not receive sunlight, on the other hand, helps reduce heat loss in winter. One third of a house's demand for winter heating is generated by wind chill, at least in climates where cold-season winds are a regular feature. Reducing wind chill by 75 percent, facades draped with evergreen climbers reduce heating demand by 25 percent

Climbers on buildings can help protect the surface of the building from damage from very heavy rainfall and hail and may play some role in intercepting and temporarily holding water during rainstorms, in the way that green roofs do. Climbers absorb pollutant gases and filter air. They also help to shield the surface from acid rain, which might be an important consideration for certain traditional as well as modern cladding materials.

Figure1. Green Facades in VIT University, Vellore, Tamil Nadu

Rai *et al.* (2009) studied that the plant foliar surface is the most important receptor of atmospheric pollutants. It undergoes several structural and functional changes when particle-laden air strikes it. Remarkable differences in the growth parameters and micro-morphological features (like wax, cuticle, epidermis, stomata and trichomes) were recorded in the dust-treated plants when compared to the respective controls. The reduction in growth parameters, the size of epidermal cells, and stomata were reduced and cuticle damage was also observed. The relative proportion of fine particles, which play a major role in hampering the overall growth of a plant, was higher in comparison to ultra-fine and coarse particles. It was observed that the particles larger than the stomata opening generally pile up on the pore, while fine particles clog the stomata, affecting the gaseous exchange process. Ranjit et al. (2008) studied a parameterization method based on meteorological parameters for calculation of dry deposition of S and N compounds on natural surfaces (leaf of Cassia siamea) at Dayalbagh, Agra. Dunnett *et al*. (2004) studied that climbers and urban trees are highly effective at trapping dust and concentrating certain dust-derived pollutants in their tissues, particularly in those tissues that are then discarded. Climbers absorb pollutant gases and filter air. In a study of *Parthenocissus tricuspidata*, lead and cadmium concentrations were shown to be highest in dead leaves and dead wood. These heavy metals are thus taken out of the atmosphere and concentrated in a form that falls to the ground.

In this paper, we shall first describe the experimental procedures for the design of the facades, their portability and use. We shall also describe observational results for two existing full scale facades within the VIT campus located just across an underpass connecting the administrative blocks to the hostels. This underpass is a specially designed to facilitate the pass of goods carrying heavy vehicles resulting in substantial amounts of air pollution. Since a full scale controlled façade was not available for this underpass, we had to fabricate two other portable facades draped with another popular creeper (see figures 2 and 3) to study the impact of diesel powered IC engines. The amounts of sulphates and nitrates removed by these portable façade are presented along with scanning electron microscopic analyses.

Figure 2 Controlled façade draped with *(passiflora caerulea)*.

Figure 3. Exposed façade draped with *(passiflora caerulea)*

2. Meteorology of Vellore

Vellore is a highly populated industrial area, where this study is done. Vehicular emissions near traffic junctions are high during the daytime. So the atmosphere of Vellore is polluted with gases like SO_2, and NO_2. These gases are advected by the prevailing winds and the turbulence in the atmospheric boundary layer diffuses the pollutants. It is therefore essential to analyse the prevailing meteorology.

Vellore is a district of Tamil Nadu. It is located at 12.93° Northern Latitude and 79.13° Eastern Longitude. The city is situated 122.3 km South West of Chennai and 166.7 km East of Bangalore (www.distancefromto.net). Vellore is on the plains surrounded by low, rocky hills. Temperature ranges from as low as 10°C in the winter months of December-February to even 43°C in the summer months of April-June. It has essentially a dry climate for the non-monsoon months. Precipitation usually happens during the months of June-August and October-December (www.fallingrain.com).

The seasonal maximum and minimum solar insolation and average temperature variations over Vellore are shown in table 1.

Table 1 Solar Insolation and Temperature variation over Vellore

Season	Solar Insolation(W/m^2)[a] at 2:30 p.m.		Temperature(°C)[b]	
	Min.	Max.	Avg. high	Avg. low
Midsummer(April)	107.9	664	36	26
Autumn(August)	234.6	602.4	35	26
Late Autumn(October)	146.7	564	32	24
Winter(December)	78.3	537.6	29	22
Spring(February)	164.7	616	31	22

[a]*Source:* Weather data 2008-09
[b]*Source:* www.weather.msn.com

The location of Vellore is such that the boundary layer is strongly influenced by (i) the diurnal variation of solar insolation within the tropical belt, (ii) wind flow patterns owing to its proximity to the Bay of Bengal and (iii) the effect of added roughness elements in the surface boundary layer due to the green facade. The deposition velocities of sulphur dioxide for summer and winter clearly imply that it is maximum in summer and minimum in winter (see figure 5).

A wind rose for April'09 shows that the winds are more frequent from the southwest compared to the north throughout the month of April. The mean monthly wind speeds recorded at Vellore, TN are between 1 m s^{-1} and 1.8 m s^{-1}.

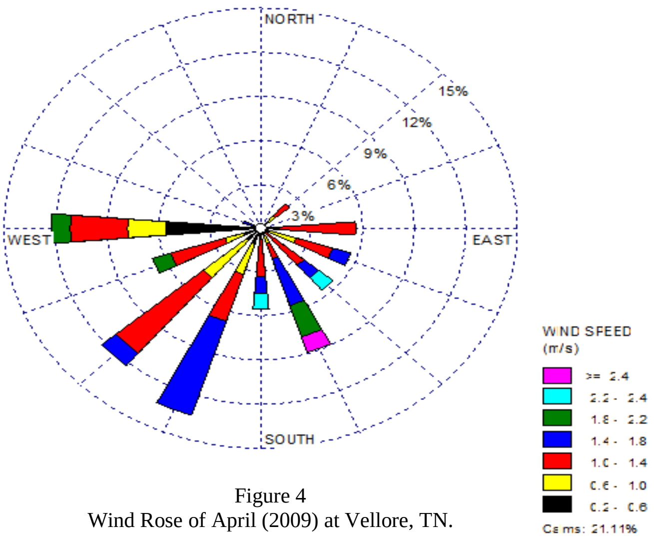

Figure 4
Wind Rose of April (2009) at Vellore, TN.

The deposition of sulphur dioxide on tropical foliage typical to Tamil Nadu was estimated by Patra and Ghosh (2010) and Seth et al. (2010) using the temperature, solar insolation and the wind data shown above. This involved a detailed calculation of the binary diffusion coefficient of sulphur dioxide in air using Lennard – Jones model and will not be described here. However we discuss below the theory of resistances used in the present study.

3. Theory of Resistances

In the universally used formulation for dry deposition, it is assumed that the deposition flux is directly proportional to the local concentration C of the depositing species, at some reference height above the surface (e.g., 10 m or less):

$$F=-v_dC \quad (1)$$

F represents the vertical dry deposition flux, the amount of material depositing to a unit surface area per unit time. The proportionality constant between flux and concentration, v_d, has units of length per unit time and is known as *deposition velocity*. By convention, a downward flux is negative, so that v_d is positive for a depositing substance (Seinfeld *et al*., 1997)

The process of dry deposition of gases and particles generally consists of Aerodynamic, Quasi-laminar and Canopy resistances.

3.1 Aerodynamic Resistance

Turbulent transport is the mechanism that brings material from the bulk atmosphere down to the surface and therefore determines the aerodynamic resistance. The aerodynamic resistance (R_a) for gases (Voldner *et al*., 1986) in neutral and stable conditions is calculated by

$$r_a = 4(u\sigma_\theta^2)^{-1} \quad (2)$$

and in unstable conditions by

$$r_a=9(u\sigma_\theta^2)^{-1} \quad (3)$$

where, u= mean wind speed and σ_θ = standard deviation of wind direction.

It was assumed that the atmospheric was stable in winter and unstable during the summer and the monsoon.

3.2 Quasi Laminar Resistance

Adjacent to the surface there exists a quasi-laminar layer, across which the resistance to transfer depends on the molecular properties of the substance and the surface characteristics. The quasi-laminar layer resistance for a gas in terms of the Schmidt number is given by (Wesely, 1989),

$$r_b = \frac{5S_c^{2/3}}{u_*} \quad (4)$$

where, the dimensionless Schmidt number, $Sc = v/D$ (v is the kinematic viscosity of air and D is the molecular diffusivity of the species) and u_* is the friction velocity.

3.3 Canopy or Surface Resistance

The surface or canopy resistance r_c poses the greatest computational complexity. r_c is assumed to be zero for particles and thus in developing a model for r_c we need consider only gases. The approach adopted here is based primarily on the methodology developed by Wesely (1989) for regional-scale modelling over a range of species, land-use types, and seasons. The surface resistance is calculated from individual resistances by the following equation:

$$r_c= \left(\frac{1}{r_{st}+r_m}+\frac{1}{r_{lu}}+\frac{1}{r_{dc}+r_{cl}}+\frac{1}{r_{ac}+r_{gs}}\right)^{-1} \quad (5)$$

where the first term includes the leaf stomatal (r_{st}) and mesophyllic (r_m) resistances, the second term is the outer surface resistance in the upper canopy (r_{lu}), which includes the leaf cuticular resistance in healthy vegetation and other outer surface resistances; the third term is the resistance in the lower canopy, which includes the resistance to transfer by buoyant convection (r_{dc}) and the resistance to uptake by leaves, twigs, and other exposed surfaces (r_{cl}); and the fourth term is the resistance at the ground, which includes a transfer resistance
(r_{ac}) for processes that depend only on the canopy height and a resistance for uptake by the soil, leaf litter, and so on at the ground surface (r_{gs}).

The bulk canopy stomatal resistance (for a Passion flower type foliage) is calculated from tabulated values of r_j (where r_j is the minimum bulk canopy stomatal resistance for water vapour), the solar radiation (G in W m^{-2}), and surface air temperature
(T_s in ^oC between 0 and 40 ^oC) using

$$r_{st} = r_j \left[1 + \left(\frac{200}{G+0.1}\right)^2 \left(\frac{400}{T_s(40-T_s)}\right) \right] \tag{6}$$

Outside this range, the stomata are assumed to be closed and r_{st} is set to a large value. The combined minimum stomatal and mesophyllic resistance is calculated from

$$r_{sm}^i = r_{st}^i + r_m^i = r_{st}\left(\frac{D_{H_2O}}{D_i}\right) + \frac{1}{3.3\times10^{-4}\,H_i^* + 100 f_0^i} \tag{7}$$

where D_{H_2O}/D_i is the ratio of the molecular diffusivity of water to that of the specific gas, H_i^* is the effective Henry's law constant for gas (for SO_2, H_i^* = 1 X 10^5 M atm^{-1}), and f_0^i is a normalized reactivity factor (between 0 and 1) for the dissolved gas. The second term on the R.H.S of (14) is the mesophyllic resistance for the gas.
The resistance of the outer surfaces in the upper canopy for a specific gas is computed from

$$r_{lu}^i = r_{lu}\left(\frac{1}{10^{-5}\,H_i^* + f_0^i}\right) \tag{8}$$

where r_{lu} is tabulated for midsummer and winter conditions from Wesley (1989).
The resistance r_{dc} is determined by the effects of mixing forced by buoyant convection and is estimated from

$$r_{dc} = 100\left(1 + \frac{1000}{G+10}\right)\left(\frac{1}{1+1000\theta}\right) \tag{9}$$

where θ is the slope of the local terrain in radians. The resistance of the exposed surfaces in the lower portions of structures (canopies or buildings) is computed from

$$r_{cl}^i = \left(\frac{10^{-5}H_i^*}{r_{clS}} + \frac{f_0^i}{r_{clO}}\right)^{-1} \tag{10}$$

Similarly, at the ground, the resistances are computed from

$$r_{gs}^i = \left(\frac{10^{-5}H_i^*}{r_{gsS}} + \frac{f_0^i}{r_{gsO}}\right)^{-1} \tag{11}$$

4. Estimation of Deposition Velocity

Having obtained the values for all the resistances, we now proceed to the quantification of the dry deposition velocity of SO_2 on vegetation draped surface. The total resistance, r_t, to deposition of a gaseous species is the sum of the three individual resistances described earlier and is, by definition, the inverse of the deposition velocity:

$$v_d^{-1} = r_t = r_a + r_b + r_c \tag{12}$$

The most important factors that modulate values of the deposition velocity are the wind velocity and the solar insolation (see figure 5) which is the highest during April–a reflection on higher SO_2 uptake rates by the profusion of green leaves during summer. Though both solar insolation and wind velocity influence the value of deposition velocity, the v_d values are most influenced by solar insolation and we show the solar insolation dependent v_d. variability (see figure 5)

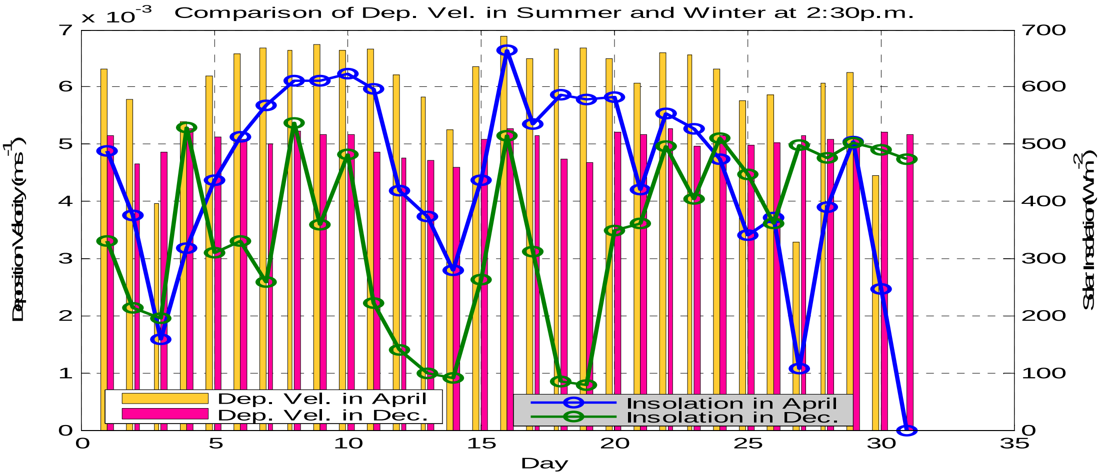

Figure 5
Seasonal Comparison of Deposition Velocity of SO_2 vis-à-vis solar insolation

5. Experimental analyses

In this section, we shall describe our main experimental results, contrasting changes induced in the exposed façade (see figure 3) vis-à-vis the controlled façade (see figure 2). In order to quantify the cleansing effect of our experimental façade, we exposed it to the emissions from the IC engine (see figure 6). The experimental façade was kept at a distance of 1.22 meters away from the exhaust – this distance was chosen so that the passion flower foliage received diffused emissions rather than direct emissions. The IC engine (Kirloskar, 4.4kw, 6bhp, 1500rpm – see figure 7) was operated with minimum load conditions for 90 minutes-this was done because exposed facades on buildings and walls (see figure 1) are generally exposed to low to moderate levels of air pollution-only buildings close to heavy traffic laded streets receive high levels of pollution. We can simulate the latter situation by running our IC engine at a moderate and maximum load condition.

Figure 6
Experimental facades exposed to emissions from IC Engine

Figure 7
IC Engine used for experimental analyses

Samples were collected from the experimental façade in intervals of 15 minutes to study the emission effects. Altogether 6 sets of leaf samples were taken plucked from random locations on the façade. Each sample has 4 leaves - 2 leaves for SEM analysis and the other 2 for chemical analysis. Also one set of sample was collected from the controlled façade to compare the results from the experimental façade.

To quantify the concentrations of sulphates and nitrates assimilated by the Stoma and the Mesophilic tissues, the samples collected were dried in an oven for 24 hours at $70^0 – 80^0$C. The dried samples were then powdered and stored separately. 250 milligrams of each sample was soaked in distilled water for 24 hours. It was then filtered and then made up to 50 milliliters. 10 milliliter from this mixture was further made up to 100 milliliters for analyzing sulphates. For analyzing nitrates, 5 milliliters was taken from the remaining 40 milliliters and made up to 50 milliliters. The concentrations of sulphates and nitrates were finally determined by UV Spectrophotometer absorption method.

The samples reserved for SEM analyses were dried properly to eliminate the moisture –this is essential for the free movement of electrons. An Auto Fine Coater (JEOL JFC-1600) prepared the specimens for SEM observation (see figure 8). It can efficiently coat biological and other nonconductive specimens with metals in a short time duration resulting an accurate film thickness.

The SEM which photographed the images has the following specifications: Model JEOL 6390 magnification 5 – 300000 x, electron gun with accelerating voltage 0.5 to 30 kv, admissible specimen size 8 – 150mm, movement in XYZ, tilt -10^0 to $+90^0$ and rotation 360^0 endless. (see Figure 9).

Figure 8
Auto Fine Coater

Figure 9
Scanning Electron Microscope

6. Results and discussions

The results obtained from the chemical analysis of the collected samples reveal that sulphates and nitrates are efficiently removed by facades draped with Passion flower. Although the initial sulphate concentration slightly increases in the range of 80 – 90 milligram per litre, the concentration of nitrates shows a steep increase from 40 – 100 milligram per liter. This directly implies that the removal rate of oxides of nitrates by *passiflora caerulea* is more efficient than that of oxides of sulphates. In Table 2 the six samples corresponded to six levels of exposure times with 25 increments reaching up to 90 minutes of exposure.

As expected, sample collected from the controlled façade has lower concentrations of sulphates and nitrates. It clearly indicates the presence of oxides of sulphates and nitrates in the emissions from the IC Engine which are further trapped and removed by the *passiflora caerulea* creeper. The variations of pH, sulphates and nitrates discussed are tabulated (see table 2)

Table 2: Variations of pH, Nitrates and Sulphates

	pH of the Samples	Concentration of Sulphates (mg/L)	Concentration of Nitrates (mg/L)
Sample-1	6.05	81	41
Sample-2	7.66	80	40
Sample-3	7.48	90	50
Sample-4	7.64	85	70
Sample-5	7.68	87	80
Sample-6	7.68	80	100
Controlled	7.81	70	30

Having established the fact that sulphates and nitrates were indeed found in the experimental samples, we now proceed to discuss the results of our SEM analysis.

Exposing the façade to emissions for the entire 90 minutes resulted in clogged stoma – note the white patch in centre of the stomata opening (see figure 10a). In contrast in the controlled sample we see the stomatal opening free of any obstructions (see figure 10b). In subsequent studies we shall quantify the extend of stomatal blockages as a function of time of exposure and solar insolation variability.

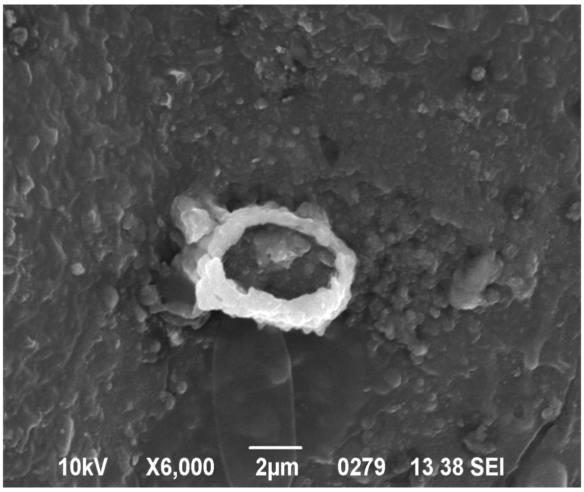

Figure 10a
Sample-6 collected after

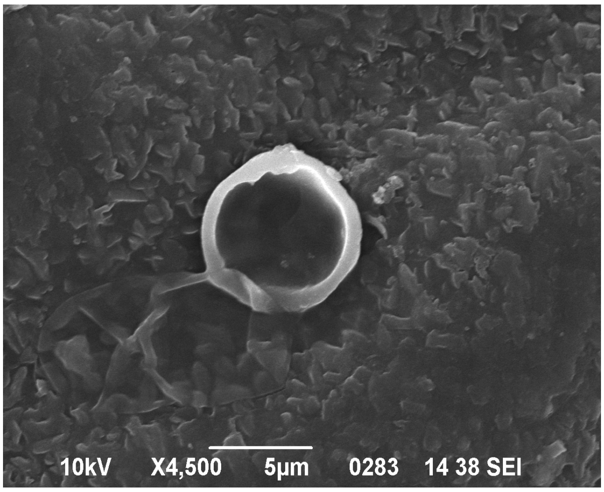

Figure 10b
Sample collected from the controlled facade

7. Concluding Remarks.

Green facades are an attractive option to cleanse building environments from the effects of air pollution. In addition, they considerably cool the buildings thereby minimizing energy requirements. In hot tropical climes, there are a variety of creepers that can drape facades rapidly. The prolonged sun shine hours not only promote the lush growth of creepers, they also ensure that the stomatal pores remain wide open during most of a working day. In this paper we have shown results from a movable vertical façade draped with *passiflora caerulea*. Chemical analyses as well as scanning electron microscopic analyses reveal that even for a modest exposure time of 90 minutes, gaseous as well as particle air pollutants are trapped by the leaves of the façade. Indeed, stomatal pore openings with a diameter of 2.43 - 4.34 micrometers can trap sub micron pollutant particle. We propose to undertake further studies to yield parametric relationships between the surface deposition fluxes, the stomatal diameters and the solar insolation variability.

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About the Authors

V. JACOB THOTTATHIL, B.E Industrial and Production, Mangalore University, Karnataka, M.Tech.(Honours) Manufacturing Technology, N.I.T. Calicut, Kerala, currently a research scholar at the School of Mechanical and Building Sciences, VIT University, Vellore, Tamil Nadu, India.

C. BALAMURALIKRISHNA, Dr. C. Balamuralikrishna, PhD. N.I.T Surathkal, Karnataka, is currently a professor at the School of Mechanical and Building Sciences, VIT University. He has more than 20 years of experience and serves as a leading consultant in the field of Environmental Engineering.

S. GHOSH, Dr. Ghosh is currently a Senior Professor at the School of Mechanical and Building Sciences, VIT University. He is also an Associate member of the School of Earth and Environment, University of Leeds, U.K. He has spent about 20 years in the U.K. including 9 years at Cambridge University. His research areas include Cloud Microphysics, Aerosol Science, Two phase flows, Large Eddy Simulations. He is currently an Editor of 'Atmospheric Science Letters' published by the Royal Meteorological Society, U.K.

ACHIEVING ENERGY EFFICIENCY IN BUILDINGS DESIGN THROUGH INNOVATIVE PLANNING AND DESIGN SOLUTIONS

[1]Shailendra Kumar Mandal, [2]Kamini Sinha

[1]Department of Architecture,
National Institute of Technology Patna, Patna-800 005, India
[1]E-mail: vastumandala@yahoo.com

[2]Department of Architecture,
National Institute of Technology Patna, Patna-800 005, India
[2]E-mail: kamini_nitp@yahoo.com

Abstract

The idea that the, earth is a finite planet in the mathematical sense is commonly accepted today. In 1972, both the Club of Rome, in its report, (Meadows et al, 1972), and the first Earth summit in Stockholm expressed this idea. This amounts to all its resources including fossil energy being limited. Renewable resources, already partially tapped, will only replace a portion of this energy. This substitution will be difficult to accomplish since the level of predation of human beings on the planet is causing serious malfunctioning of the biosphere.
The building sector does not escape from today's spotlight on the environmental impacts of human activities. Buildings are major energy consumer during both construction and 'usage, and also generate large quantities of waste. Built up structures are consumers of 40% of the global primary energy and generator of 24% co_2 emission. As such, criticality of buildings and their role in minimizing energy consumption and promoting sustainability of human habitat assumes importance. The options relating to building materials, building technologies, landscaping, heating and cooling system etc have already been explored.
It is logical to apply the principles of energy costing to building prospects and look for ways to minimize energy consumption during their entire lifetime. Accordingly, the paper focuses on design strategies for making building highly energy efficient and sustainable in terms of site planning, macro and micro climatic conditions, landscaping, orientation, fenestration and shading and building materials. It discusses two case studies from India incorporating innovative design solutions to achieve energy efficiency.

Key Words- *Building Design, Site and Building Orientation, Internal Layout, Ventilation, Courtyards*

1 Introduction

Considering the role and importance of energy as the major driver of economic growth and physical development coupled with limited availability of conventional and non-renewal sources of energy and ever rising demand and spiraling market prices, issues related energy consumption, energy conservation and promoting non-conventional and alternate sources of energy have assumed global concerns. Considering the fact that existing built up structures account for 40% of the global primary energy consumption and generator of 24% of CO_2 emission, criticality of buildings and their role in minimizing energy consumption and promoting sustainability of human habitat assumes importance. With rapid urbanization and growth of population, more and more buildings would be required to be constructed to meet the increasing demand of shelter, trade and commerce, industries, entertainment, institutions etc. and accordingly level of energy consumption are likely to rise on a compounded pattern. Making buildings energy efficient have distinct advantages in terms of not only saving money on energy costs but also reduction of adverse impact on the environment through the reduced use of fossil fuels, increased comfort levels achieved through effective use of natural light and ventilation.

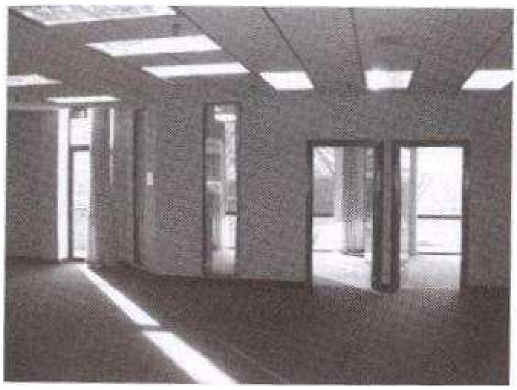

Fig. 1: *Optimum use of natural light reduce energy consumption*

2 Building Design

Buildings as they are designed, constructed and used have enormous energy implications. With number of people and institutions rushing towards urban centres, energy requirements of cities due to buildings is going to rise sharply in future. Looking at the high degree of energy consumption by built environment, which has been placed at 300 Kwh for every square meter on annual basis, there appears to be enough options to bring it down to the level of 140 Kwh with proper design. Thus built environment is the sector which would require close scrutiny and monitoring for effecting overall economy in the levels of energy consumption. Experience has shown that buildings can be designed to meet the occupant's needs for thermal comfort at reduced level of energy consumption by adopting an integrated approach to building design. The integrated approach could include orientation, shape and size of the building, built form, surface to volume ratio, building efficiency, ratio between length and depth of the building, using simple techniques of building structure, efficient structural design, principles of solar passive techniques in building design, using energy efficient equipment, control and operation strategies for lighting, heating, ventilation etc. using solar energy for meeting the energy needs of buildings, replacing energy intensive materials with low energy components etc.
Main features of energy efficient buildings would essentially revolve around

- Site and Building Orientation
- Internal Layout of Buildings
- Window placement, sizing and shading
- Insulation
- Ventilation
- Courtyard
- Landscaping
- Building Materials
- Use of energy efficient appliances

2.1 Site and Building Orientation

Orientation has the greatest impact on the energy consumption by buildings. The issue of energy in context of building, has to be viewed in the dual context of planning of plots/sites and the actual designing of the buildings. To make sure that the building makes best use of the solar and wind energy, it would be essential that the majority of the buildings should have the site advantage. Accordingly town planners have important role cast for themselves while preparing the layout plan, so that maximum number of plots have best orientation. Once this is ensured at the planning level, it would be much easier for the Architects to evolve a design which would be energy efficient.

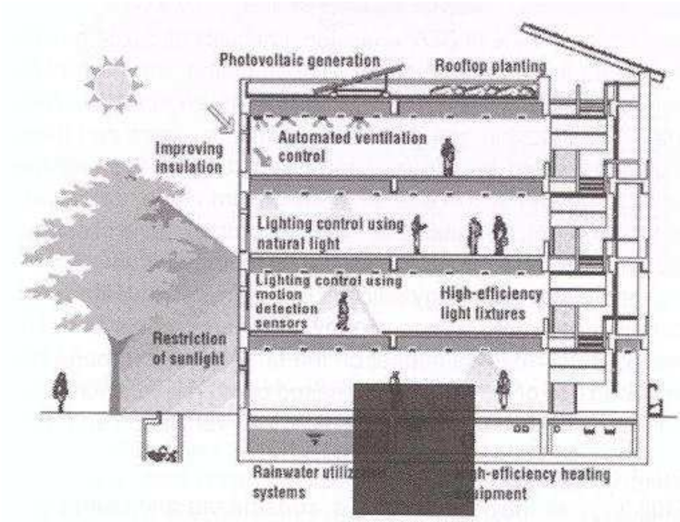

Fig. 2: *A few features of energy efficient building.*

Further the planners should ensure that ratio of plot width and depth is fixed in a manner that the entire depth of built up area allowed on a plot should have access to natural light, minimizing the requirement of artificial lighting. This would be particularly important in case of row housing where the plots have the option to draw light from front and rear only.

Orientation needs to be effectively used in order to evolve energy efficient building design by making use of solar radiation and the wind. However requirements of building design would vary from region to region, state to state and within regions in states. Accordingly, buildings with regards to sun and wind will have to be oriented differently in different regions. Architects can ensure high degree of energy efficiency both in construction and operation by critically studying the macro and micro climate, applying bio-climatic architectural principles and making optimum use of desirable conditions.

Artificial heating and cooling are biggest consumers of energy in buildings, which is placed at 26% of average household energy consumption. Accordingly, it will be critical to minimize the energy load on building due to heating and cooling. A major principle of energy efficient building design would be to orient the building in a manner that maximizes sun heat gain into building during winter while excluding it during the hot long days of summer. This can be made possible due to the fact that the angle of sun changes from season to season. In summer, sun rises early in North-East and climbs high in the South before setting in North-West and heat gain is mainly through roof, east and west windows of buildings. In winter sun rises later in South of East, stay low in South before setting in South of West. South windows and walls receive the maximum winter sun and warmth. To achieve the design goal of optimum energy efficiency, basic rule of a building would be to have North and South facing walls 1.5 to 2.00 times the length of East and West facing walls in order to allow reasonable access to morning and late afternoon sun on the East and West whereas maximum sun during winter through the southern walls. However small projection on the southern wall will help in cutting the vertical sun and avoiding the heat trickling into the buildings during summers.

Fig. 3: *A perfect orientation for summer and winter.*

2.2 Internal Layout of Buildings

Not only placing of external walls is critical but also rational allocation of internal spaces in the buildings for achieving the desired level of energy efficiency. Spaces can be classified and grouped considering their energy requirements. Space requiring ambience temperatures should be grouped and planned in the best orientation whereas spaces having little relevance to living, etc. can be placed in directions considered as adverse from the point of view of orientation. Accordingly, indoor living and entertainment spaces can be placed on the Eastern and Southern sides for having bright and warm areas during winters and cool spaces in the summers. Thus placing bedrooms on the East and South will be more comfortable for sleeping during both summers and winters. Mechanism of grouping rooms with similar uses together for creating different zones and using doors to separate them would help in evolving appropriate design solutions. Closed design offers distinct advantages in terms of energy efficiency as compared to open designs. However, where ever the specific needs of planning re-quires open plans then use of glass doors would be appropriate to define different zones. Kitchen, laundry, bathrooms can be grouped to minimize the need for long hot water pipes. Garages and stores can be kept on the western side of the house so as to seal the living areas from the intense summer heat.

2.3 Windows Placement, Sizing and Shading

For evolving energy efficient design solutions appropriate placement, sizing and shading of buildings would be critical. Windows have multiple functions to perform including solar collectors trapping heat from the sun which is useful in winter but not during summers, acting as ventilators during summers., funneling cool late afternoon and night time breeze to reduce heat accumulated during the day and to let in daylight to make spaces bright. A balance need to be made between controlling sun access, allowing adequate cross-ventilation and natural light to enter. Accordingly 1/3rd to ½ of the southern face of the building can be put under glass for trapping winter sun and shading from summer sun with correctly designed eaves. An overhang of 0.4 times the distance from eaves to bottom of windows will be sufficient to save it from the heat of the sun during summers. Use of solar pergola can also be made to regulate the impact of the sun in the building. However, it would be critical that shading devices do not block the sun's access to the interior of the building during winter. Eastern and Western windows provide warmth in winter from early morning and afternoon sun, but they pose difficulty from sun in summer. It makes rooms on East and West comfortably warm, in particular those on the West. Accordingly it would be critical to keep area of Eastern and Western windows minimum and wherever provided should have vertical screens, louvers, blinds, shutters to block the sun. North facing windows can be made large to facilitate good ventilation and light without losing much of heat.

Tinted glass and reflective films absorb and reflect heat leading to reduction of heat and light. They can be used on East and West where glazing is unavoidable due to design/site reasons. Double glazing can be used to reduce winter heat loss. However, during summer, they require full shading minimize heat gains/transfer.

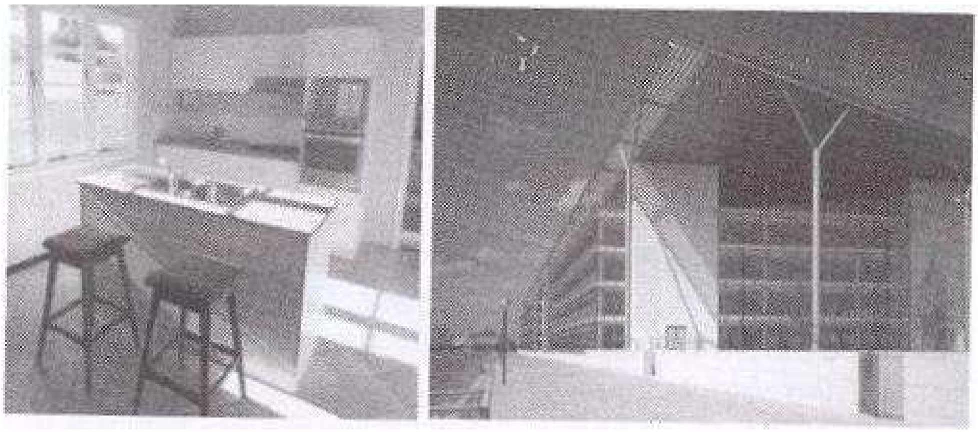

Fig. 4: *Orientation of windows and use of solar pergola*
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2.4 Internal Windows

The internal windows are important for reducing winter heat loss. Windows can lose heat five to ten times faster than an equivalent area of wall. Heat loss can be minimized by keeping warm air inside the room away from cold windows, closed curtains covering entire window up to ground made of heavy fabrics are considered effective in minimizing heat loss.

2.5 Insulation

Acting as barriers, insulation makes spaces more comfortable by reducing the heat loss in winter and heat gain in summer. Insulation of ceilings, roof, external walls and air gaps would be critical to achieve the desired objectives of energy efficiency. Bulk and reflective are two major kinds of insulations used. Bulk insulation works by trapping small cells or layers of air within the insulating materials which are effective in retarding heat transfer whereas in case of effective insulation, reflection of light and heat are used as mechanism to reduce the heat transfer.
Effective use of thermal insulation, treating roof for regulating solar radiation, using cavity walls, locating, sizing and detailing properly windows and shading devices help in evolving design solutions which are bio-climatic and ultimately help in reducing energy requirement of the building. However advanced techniques of passive heating and ventilation like trombe wall, water wall, roof based air heating system, wind towers, courtyard, earth-air tunnels, evaporating, cooling, etc. can be effectively used for evolving low energy building designs.

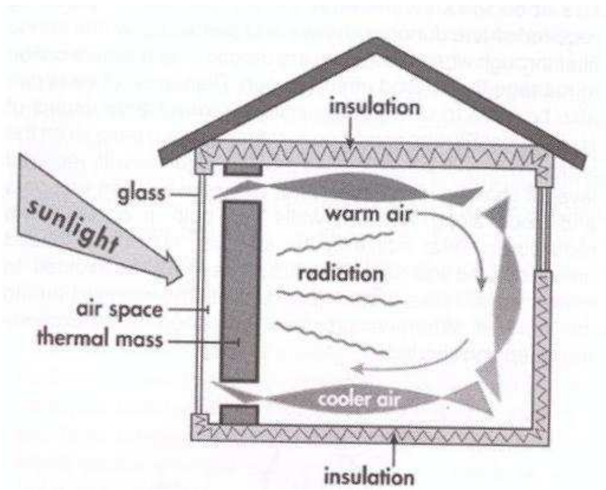

Fig. 5: *Illustration of insulation.*

2.6 Ventilation

Natural ventilation is considered an essential part of energy efficient structures. For achieving cross ventilation correct positioning of the doors and windows would be critical. A large opening on the leeward side of the building will maximise the air flow through room's facilitating the removal of heat accumulated during the day. Ventilation is very critical in hot and humid zones for creating climate sensitive design solutions for buildings.

2.7 Courtyards

Courtyards have been considered critical in promoting energy efficiency in buildings. They facilitate not only natural air and light into inner areas but also high degree of cross ventilation. Courtyards make buildings safe from large heat intake and glare. Acting as large evaporator, cooler during summers, courtyards promote enormous cooling without mechanical aids. Landscaped courts are great moderators of micro-climate within buildings. Acting as great heat dump, courtyards minimize heat loss during winters.

Courtyards with water column fountains have been considered as great environmental moderators. Looking at the distinct advantages, courtyards should be considered as valued partners in building designs. Building and zoning laws should leverage courtyards for evolving energy efficient building solutions.

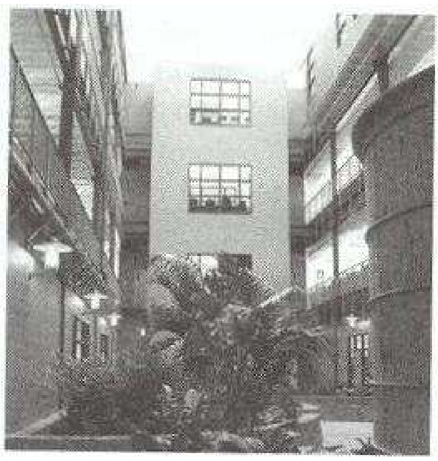

Fig. 6: *Use of courtyards.*

2.8 Landscaping

Effective use of landscaping as part of building design can help considerably in lowering energy consumptions in the buildings. Gardens can act as significant climate moderators. Use of deciduous trees / vines in West can help in providing required shade during summers and permit winter sun to filter through when the leaves are dropped, as a simple option to manage the good effects of the sun. Plantation of trees can also be used to shield the buildings from the adverse impact of some trees. Shrubs or creepers grown on an open pergola on the southern face of a building can provide windows with required level of shading in summers. Use of evergreen creepers and trees along western walls can help in considerable reduction of heat intake in the summer. Use of unshaded paving on the South and Western sides should be avoided to minimize the intake of heat reflected into the windows during the summer. Wherever provided they need to be properly designed and shaded.

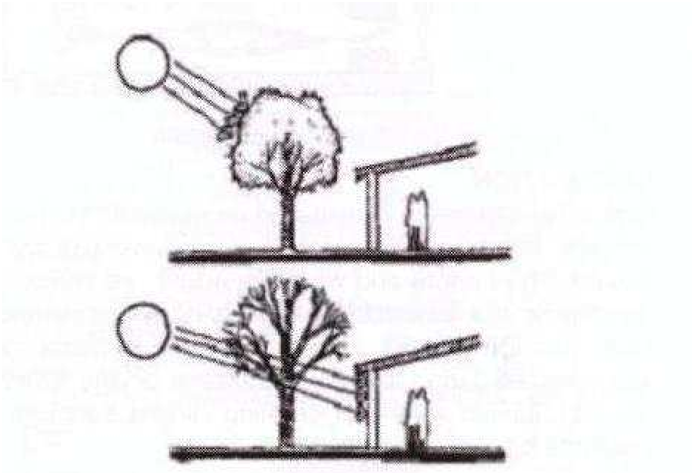

Fig. 7: *Deciduous trees provide shade from summer sun and allow winter sun.*

2.9 Building Materials

Choice of building materials has important bearing on energy consumption level of the buildings. Using low energy materials, efficient structural design and reduction of energy used for transportation can help in achieving high degree of energy efficiency. Choice of locally available materials and innovative construction techniques has clearly demonstrated their usefulness in reduction of energy consumed by the buildings during construction and operation. In addition, using dense materials such as brick, stone, concrete and rammed earth which heat up and cool down slowly (having high thermal mass) are critical for adequately storing winter daylight warmth and gradually releasing at night. In summers thermal mass can also help in keeping buildings cooler during the day when provided with proper ventilation, allowing the night air to cool down the inside mass resulting in more comfortable conditions next day.

For evolving energy efficient buildings, role of building industry would be critical. Government should encourage the industry which produces energy efficient materials. Industry must evaluate and monitor every material in terms of energy. Industry should be encouraged to promote R&D with support from building material agencies to ensure that energy requirement of each material is thoroughly studied before the material is allowed to be marketed. Specification should also include the energy component in order to make Architects/builders understand the energy implication of material being used.

Fig. 8: *A Small structure created using locally available material-saves cost.*

2.10 Colours of External Building Finishes

External finishes of buildings need careful choice in order to regulate the heat gain/loss by walls/roof. As a general rule, light colours have a tendency to reflect sun's heat, while darker colours absorb it. This fact can be made use of for identifying colours for roofs and walls. During summer's, choice of light colour would be critical to minimize heat gain and to keep inside spaces cooler by reflecting heat from the sun and for heat gain during winter, the colours will have to be darker. This would call for repeatedly changing the colours of walls and roof. Better option will be to keep the building properly insulated which is an effective mechanism of controlling and regulating heat transfer.

2.11 Use of Energy Efficient Appliances

80% of the energy consumed by any building over its entire life cycle is during the operational phase of the building whereas 20% energy is consumed during construction. Accordingly, it would be critical that energy consumed after the buildings are made operational should be minimized. Substantial portion of energy is consumed by the electrical gadgets which are used for lighting, heating and cooling which can be reduced considerably provided energy efficient appliances are used. CFL lights, evaporative coolers, rated kitchen gadgets, etc. would be critical to minimize the energy consumption. With careful design and planning only low energy light fittings should be used along with dimmers placed strategically to have maximum impact.

Roof areas can be used for installing series of photo-voltaic solar panels which can generate enough electricity to cater to the entire electricity needs of the building making it a zero energy building. Excess power, if available can be returned to the main power grid. Buildings have large facades can be used for installing solar panels to generate electricity to met its energy requirements.

Fig. 9: *Solar water heater, wind mill, evaporative coolers, solar lighting and C.F.Lamps.*

3 Case Studies in India

3.1 *Himuruja Office Building,Shimla*

Fig. 10: *Himruja office building, Shimla*

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Design Features

- Air heating panels designed as an integral part of the south wall provide effective heat gain. Distribution of heat gain in the building through a connective loop that utilizes the stairwell as a means of distributing heated air
- Double-glazed windows with proper sealing to minimize infiltration
- Insulated RCC diaphragm walls on the north to prevent heat loss
- Solar chimney
- Specially designed solarium on south for heat gain
- Careful integration of windows and light shelves ensures effective daylight distribution
- Solar water heating system and solar photovoltaic system

3.2 *Retreat, Resource efficient TERI retreat for environmental awareness and Training, Gurgaon*

Fig. 11: *RETREAT; Resource efficient TERI retreat for environmental awareness and training, Gurgaon*

Design Features:

- Wall and roof insulation
- Building oriented to maximise winter gains; summer gains offset using shading
- East and west walls devoid of openings and are shaded
- Earth air tunnel for rooms – four tunnels of 70-m length and 70-cm diameter each laid at a depth of 4 m below the ground to supply conditioned air to the rooms
- Four fans of 2 hp each force the air in and solar chimneys force the air out of rooms
- Ammonia absorption chillers for the conference block
- Hybrid system with 50 kW biomass gasifier and 10.7 kW solar photovoltaic with Inverter and battery backup to power the building
- 2000 lpd building integrated solar water heating system
- Energy-efficient lighting provided by compact fluorescent lamp, high efficiency fluorescent tubes with electronic chokes.
- Day lighting and lighting controls to reduce consumption
- Waste water management by root zone system
- Building monitoring and management

4 Conclusion

Looking at the existing scenario, accelerated urbanization in the Indian context imposes immense pressure on the dwindling energy resources. However, the resource crunch confronting the energy sector can be effectively alleviated if we plan, design and develop human settlements and buildings by using appropriate strategies and incorporating sound concepts of energy efficiency and sustainability.
Appropriate knowledge and technology is available for creating energy efficient buildings but behavioral, organizational and financial barriers would require immediate demolition for achieving the desired results. Adopting holistic and integrated approach, shared accountability and responsibility towards improved energy performance, making energy more valued by educating and motivating professionals involved in building industry would be critical to promote energy efficient buildings. Efficiency gains in buildings are likely to provide the greatest energy reduction globally. It is estimated that demand reduction measures could almost halve expected growth in global electricity demand and CO_2 emissions from building energy use can be reduced by 29% at no net cost by 2020. however, creating energy efficient buildings would involve design community producing energy efficient building designs, financial community supporting investment in energy efficiency, building industry offering product and services for supporting designs economically, building owners and operators supporting designs financially and utilities supporting intelligent distribution and sustainable content of energy to and from building. Making all stakeholders work together would require effective policies and programme to be put in place on priority.

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THERMAL COMFORT WITH EVAPORATIVE COOLING FOR TROPICAL CLIMATES

K.P. Arandara[1], R. A. Attalage[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 Mechanical Engineering, University of Moratuwa, Moratuwa, Sri Lanka.
[2]E-mail: dinu@mech.mrt.ac.lk
[2]Telephone: +94-11-2640463 (Ext.4012)

[3]Professor, Department of Civil Engineering, University of Moratuwa, Moratuwa, Sri Lanka.
[3]E-mail: thishan@civil.mrt.ac.lk ; mtrjayasinghe@yahoo.com
[3]Telephone: +94-11-2650567-8 (Ext.2006 - 2001); Fax: + 94-11-2651216

Abstract: the tropical climatic conditions that prevail in the countries located close to the equator generally has warm humid climatic conditions. This high humidity generally results due to the relatively high moisture content in the atmosphere. It is shown that warm humid climatic conditions can still offer a good possibility for the use of evaporative cooling when the beneficial effect of evaporative cooling is coupled with physiology cooling effect that is available with enhanced indoor air qualities.

Key words: Thermal comfort, cooling systems, relative humidity, air velocity

1. Introduction

Human thermal comfort is defined as the conditions in which a person would prefer neither warmer nor cooler surroundings. According to British Standard BS EN ISO 7730, thermal comfort is “the condition of mind which expresses satisfaction with the thermal environment”. Simply, thermal comfort describes a person’s psychological state of mind and is usually referred to whether someone is feeling too hot or too cold. In other words, the human body is like a complex internal combustion engine. To achieve thermal comfort, the body must balance its heat gains and losses by properly adjusting its functions (i.e. perspiration), while also responding to the prevailing environmental conditions (i.e. temperature and humidity). Under good thermal conditions, the human body can function at optimum levels, thus maintaining a good productivity [1].

In different areas of the world, thermal comfort needs may vary based on the climate. Much of the equatorial belt within the tropical climate zone experiences hot and humid weather. Because a substantial part of the Sun’s heat is used up in evaporation and rain formation, temperatures in the tropics rarely exceed 35°C; a daytime maximum of 32°C is more common. This can be coupled with relatively high humidity levels. Therefore, special attention is needed to ensure thermal comfort within the buildings located in tropical climates.

Due to the severity of the prevailing conditions in tropical climates, there are times, however, that comfort cannot be achieved by the functions of the body itself. Under such circumstances it is necessary to provide some assistance, either by natural, hybrid or mechanical means. It is important though, for rational use of available energy resources, to first exhaust all means of achieving comfort by natural or hybrid techniques before resorting to energy consuming mechanical systems.

Researches have indicated that the building sector consumes approximately 40% of the world’s energy [2, 3] and as such is a major player in the energy agenda in tropical climates. Furthermore, a recent survey has pointed out that the three main sources of energy waste in a standard office building would be air-conditioning (50 % of the power bill), artificial lighting (20 %) and computers (20 %) for a standard office building [4]. The inefficiency of existing buildings in tropical countries especially in Sri Lanka means that large amounts of energy would be required to maintain comfortable temperatures especially with future scenarios like global warming. In this context, evaporative cooling could offer many advantages as an effective low energy consuming means for providing thermal comfort.

2. Typical tropical climatic conditions

Typical climatic conditions in a tropical country could be found by using a real example. Sri Lanka, which is located with 5^0 55” to 9^0 50” N latitude and 79^0 42”- 81^0 51” E longitude is a country that is influenced by various climate based events of the world. For example, the western and southern part would get high rainfall primarily due to thunder storms and the south western monsoons. The rainfall can be as high as 2,000 - 2,500 mm per annum in many places and hence categorized as a wet zone.

The central hills located at this centre of the island makes the climate of the rest of the country consisting of east and north along with north central parts of the country to receive not much rainfall from south western monsoons. This means that the annual rainfall of these areas would be about 1,500 mm to 2,000 mm. These parts are preliminary served by north eastern monsoon which brings in less moisture and hence less rainfall. Thus, these areas are identified as dry zones. There could be an intermediate zone also as shown in Figure 1. The typical climatic data for Colombo is given in Table 1.

3. Comfort zones for Sri Lanka

It is relatively straightforward to indicate the comfort zone on a psychometric chart. For this, it is important to know the neutrality temperature. It was shown by Jayasinghe and Attalage [5] that for both wet and dry zones of Sri Lanka, the use of a single neutrality temperature of 26^0C is sufficiently accurate for developing comfort zones. It was shown by Szokolay [6] that for conditioned environments, a band of 2^0C can be used about the centre point. The recent research by De Dear and Brager [7] has indicated the use of a band width of 3.5^0C about the neutrality temperature. It was also recommended by Jayasinghe and Attalage [5] that for tropical climate conditions, it is desirable to use an upper boundary of 0.015 for humidity ratio.

Since evaporative cooling has the potential to maintain sufficient ventilation with indoor air velocities of 0.4 to 0.6m/s, especially with conditioned air delivered using ducts, the comfort zone can be further extended considering the physiolocal effects of cooling. The comfort zones developed incorporating all these features are shown in Figures 2 and 3. Figures 2 indicates that in a day with a humidity ratio of about 0.016, thermally comfortable conditions would prevail even at about 30^0C. Figure 3 indicates that even 32^0C can be tolerated when the indoor air velocity is about 0.5 - 0.6 ms^{-1}. These two figures indicate that climatic acclimatization coupled with physiology cooling effect could provide thermally comfortable conditions over a wide range of temperatures such as 26^0C to about 32^0C. this temperature range obtained by using theoretical indications have been validated by the comfort surveys carried out in Sri Lanka [8].

4. The feasibility of evaporative cooling

Evaporative cooling is particularly suitable for warm dry climates. However, certain features of warm humid climates also make it suitable especially when coupled with physiolocal cooling effect that can be obtained with enhanced indoor air velocities. People who have lived long in tropical climates have adopted them well to the various seasonal variations either with the clothes that they ware or with their expectations. This can be broadly identified as acclimatization to a particular climate.

This acclimatization is clearly indicated on the comfort zone where the center point identified as the neutrality temperature is affected by the average diurnal temperatures. Hence, the neutrality temperature for a tropical climate would be 26^0C where as it would be about 24^0C or less in a temperate climate. One feature of tropical climates is the high humidity in the early morning coupled with low outdoor temperatures such as 23^0C-25^0C. During the day, the outdoor temperatures could rise to about 30^0C-32^0C. Since, the humidity ratio of a particular day is generally constant, this rise in temperature will drop the relative humidity to about 60%.

This means that the day time could become a good candidate for evaporative cooling. The modern evaporative cooler can give a reasonable drop in temperature such as 3^0C to 4^0C with a marginal rise in the humidity ratio. Since people are generally tolerant to high humidities such as 70-75% when the physiological cooling effect is also available, evaporative cooling becomes a very good strategy even in warm humid tropical climate such as that prevails in both wet and dry zones of Sri Lanka.

5. Applications

Evaporative cooling could be a very good candidate to mitigate the adverse effects of heat islands and global warnings. Figure 4 shows an application of evaporative cooling in a factory building. Figure 5 indicates its application in a conference room. The use of ducts with properly designed and installed diffusers will allow a reasonably uniform air velocity distribution particularly at the level of the occupants. Such forced evaporation with air at 27^0C – 29^0C would have the potential to remove sweat from the skin while ensuring a pleasing air stream preventing the occurrence of dry skin. When the air supply is coupled with a suitable system of extract fans, it would be possible to ensure a continuous supply of fresh air with acceptable relative humidity.

Such a system can be considered as one with zero recycling of air. Thus, it has the potential to act as one that will not promote any diseases causing viruses or bacteria.

6. ASHRAE guidelines on envelope

Another key requirement for conditioned environments given in ASHRAE 90.1 is the requirements pertaining to the building envelop. However, such requirements would be less stringent when the energy consumption for heating and cooling is less than $15W/m^2$. In this context, the adoption of evaporative cooling offers a key advantage since the envelope could be constructed with locally available building materials without adopting the recommendation of Table 5.1 of ASHRAE 90.1

7. Conclusion

It is shown with the extended comfort zones developed for typical climatic conditions of tropical climates that evaporative cooling has the potential to become a low energy consuming active means. One of the key features is its ability to keep the energy consumption at a value less than $15W/m^2$. That will be a major achievement with respect to energy efficiency in buildings intended for factories or offices.

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About the Authors

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.

R. A. ATTALAGE, B.Sc. Hons. Moratuwa., Ph.D. France, MIE (SL), C.Eng. is a Professor at the Department of Mechanical Engineering, University of Moratuwa. His research interests are in the areas of Energy Sources and Technologies, Transfer Processes Energy, Conservation and Management Solar Energy and Building Energy Conservation.

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 a MOS Executive in MAS Intimates Thurulie (Pvt) Ltd., in Thulhiriya.

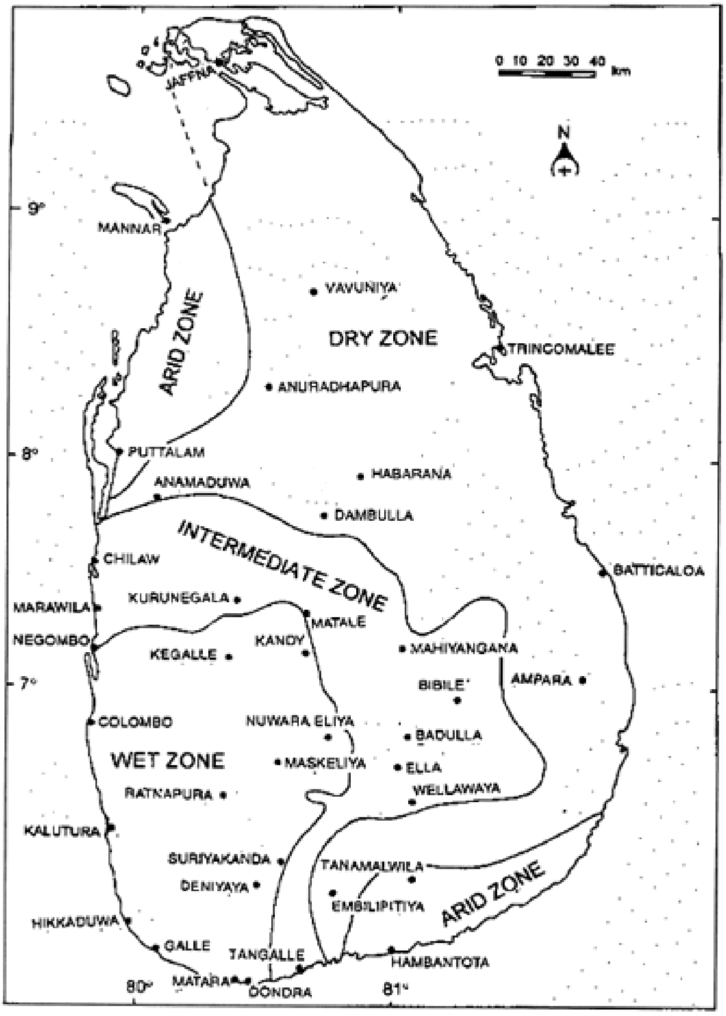

Figure 1: Climatic zones of Sri Lanka

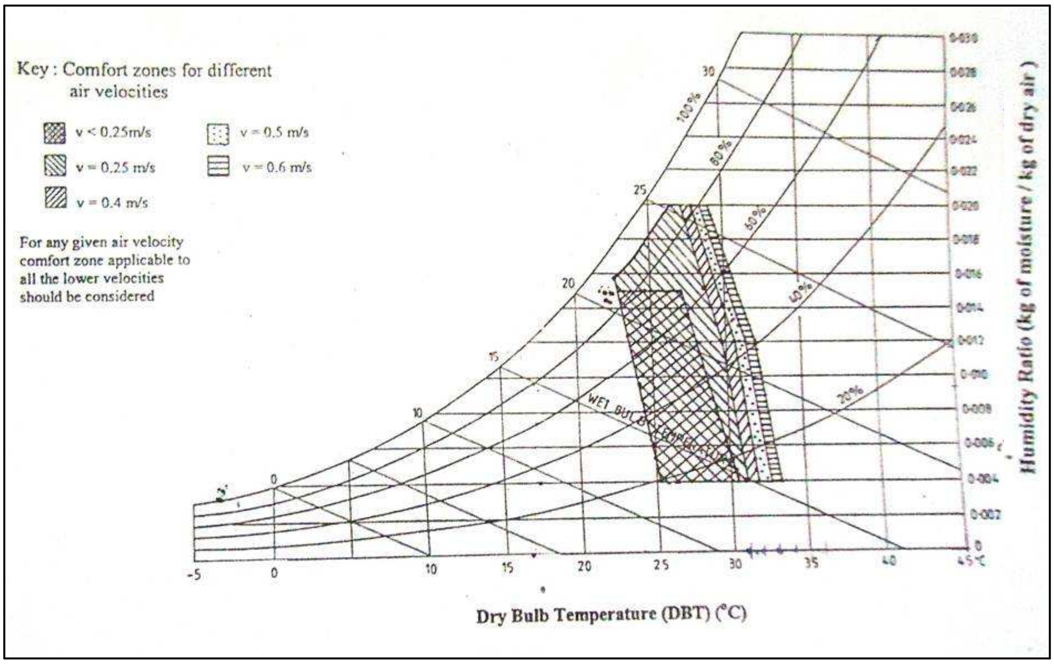

Figure 2: Psychometrics chart with extended comfort zone for Colombo in Sri Lanka

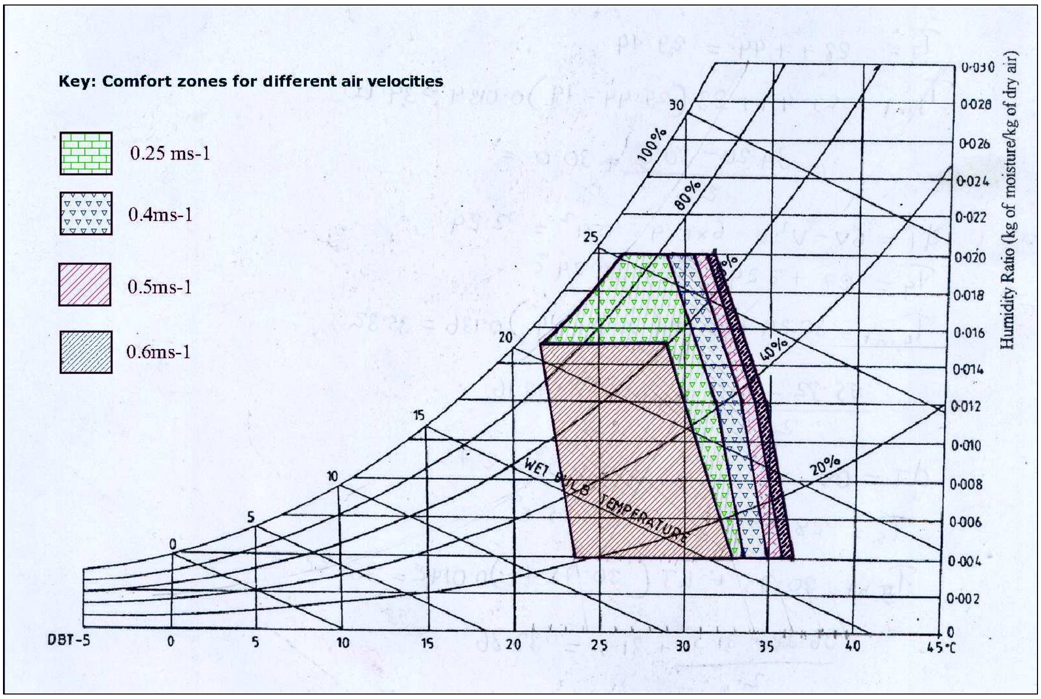

Figure 3: Psychometrics chart with extended comfort zone for Colombo in Sri Lanka

Figure 4: An evaporative coolers installed in a factory building

Figure 5: Conference room cooled by an evaporative cooling

Table 1: Average climatic data applicable to Colombo, Sri Lanka

Month	Sunshine (Hours per day)	Average rainfall (mm/month)	Mean daily temperature (^{0}C)		Minimum & maximum relative humidity (%)	
			Max (around 14.00hours)	Min (around 6.00hours)	Min (around 14.00 hours)	Max (around 6.00 hours)
Jan	7.5	87.9	30.3	22.2	58	90
Feb	8.2	96.0	30.6	22.3	59	92
Mar	8.8	117.6	31.0	23.3	64	94
Apr	7.9	259.8	31.1	24.3	68	95
May	6.2	352.6	30.6	25.3	72	92
Jun	6.6	211.6	29.6	25.2	73	93
Jul	6.1	139.7	29.3	24.9	70	90
Aug	6.5	123.7	29.4	25.0	65	90
Sep	6.4	153.4	29.6	24.7	67	91
Oct	6.2	354.1	29.4	23.8	70	92
Nov	6.8	324.4	29.6	22.9	67	93
Dec	6.9	174.8	29.8	22.4	61	91

IDENTIFYING THE ADAPTIVE OPPORTUNITIES IN FACTORY ENVIRONMENTS FOR A BETTER THERMAL COMFORT - PRELIMINARY STUDIES

Samantha Wijewardane, [2]M.T.R. Jayasinghe, Yogi D Goswami

[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: Achieving thermal comfort by active means has become one of the largest energy intensive activities in factory environments, which directly influence the manufacturing cost. As a result searching for new methodologies that could improve the thermal comfort with minimum usage of energy has become vital. In the scope of adaptive thermal comfort model, improving the adaptive opportunities is a potential passive technique that can be used to minimize the energy requirement. However not many researches were conducted especially in tropical climates to investigate the ways of expanding adaptive opportunities. The research presented was conducted to understand the existing adaptive opportunities and to explore and identify new methods that could improve thermal comfort in factory environments.

Key words: *Thermal comfort, adaptive model, Adaptive opportunity*

1. Introduction

The rising energy costs and also associated reduction in energy supplies has created many opportunities for optimization of energy usage. One of the possibilities that offer energy saving options in factory buildings is the optimum use of climatic acclimatization and adaptive opportunities to ensure thermal comfort with minimum use of active means. For this, countries close to the equator with tropical climatic conditions could become a good candidate. On one hand, there is a significant manufacturing base located within factory buildings. Also the optimization of energy needed for thermal comfort could make a reasonable impact on the manufacturing costs. The factory workers are generally inhabitants of tropical climates. Hence, they may be tolerant to the climate conditions. This can be attributed to climatic acclimatization and the clothing worn. Hence, it offers good thermal comfort exploitations even with simple means such as enhanced air movements when the surrounding temperatures rise to higher levels during the day times.

Therefore with the perception that there could be many adaptive opportunities in which when implemented could save much energy, a preliminary investigation was done by questionnaire surveys and field surveys in selected factories in low altitudes of Sri Lanka. These surveys were designed and implemented to understand the existing adaptive opportunities in naturally ventilated factory buildings mainly with respect to behavioral adjustments. Further, this preliminary research can be considered as an attempt to explore and identify the components of “behavioral adjustments”, which have higher energy saving potential, for further detailed studies.

2. The Background

Sri Lanka, located close to the equator with 5° to 9° latitude, is a good example for warm humid tropical climatic conditions where many light-work manufacturing factories are located. Because of the warm humid condition, achieving thermal comfort without active means is a challenging problem in most of the factories operated in low altitudes. As there are many evidences supporting the fact that thermal comfort influences the productivity of workers [1-5], operating a fine line between reductions of energy for thermal comfort and obtaining the maximum productivity is a challenging problem. Therefore, the ability to maintain a good harmony between optimizing energy needed for thermal comfort and productivity has become a key factor that influences the competitiveness between different manufacturing environments. As a result, searching for new methodologies that could improve thermal comfort with minimum usage of energy has become a crucial task.

“Adaptive opportunity” is one of the main *concepts* in adaptive hypothesis. It is defined as the varying degree of opportunity or scope that building provides their occupants to adjust the internal environment (and themselves) to achieve thermal comfort [6]. Enhancing the adaptive opportunities in built environments allow adapting to the situation without searching for alternative active means. Sealed, centrally air conditioned buildings provide minimal adaptive opportunity, while naturally ventilated buildings with operable windows and ceiling fans typically afford high degrees of adaptive opportunity [7]. There are many benefits to be gained from an improved understanding of the influence of adaptation on thermal comfort in the built environment [8]. In the scope of adaptive model, improving the adaptive opportunities is a potential passive technique that can be used to minimize the energy requirement for thermal comfort.

There are three modes of adaptation [6] (1) Behavioral adjustment (Personal, environmental, technological or cultural), (2) Physiological (genetic adaptation or acclimatization) and (3) psychological (habituation or expectation). Behavioral adjustment includes all modifications a person might consciously, or unconsciously make, which in turn modify heat and mass fluxes governing the body’s thermal balance. The behavioral adjustment was defined in terms of three subcategories [8]: (I) Personal adjustment: adjusting to the surroundings by changing personal variables such as adjusting clothing, activity, posture, eating/drinking hot/cold food or beverages, or moving to a different location; (2) Technological or environmental adjustment: modifying the surroundings themselves, when control is available, such as opening/ closing windows or shades, turning on fans or heating, blocking air diffusers or operating other HVAC controls, etc.; and (3) Cultural adjustments, including scheduling activities, siestas, adapting dress codes, etc.

3. The Worker and Factory Environment

Factory worker is Classified here as an employee who is directly involved in the production related activities of the factory establishment excluding any working supervisory personal and administrative personal. Majority of the factory staff consisted of factory workers that have comparatively least authority to adjust the thermal environment [9].

The pattern of occupancy in factories is different from other buildings. In factories, there is a specified time to arrive and leave and very few enter the environment in between, contrast to office buildings where people visit in different times. Because of the uncertainty of number of people in and out from office buildings$_1$ the thermal load of office buildings vary compared to factory buildings. Thus, factors can encourage thermal adaptation easily than office buildings. Often factory workers have a certain dress code that used by all, thus simplified the clothing effect on individual thermal comfort. Further it stimulates the adaption because of using the same form of dress over a long period. The rules and regulations practiced within factory premises are also different from other building types. In addition factory workers could be identified as different respect to the non thermal issues that affect the thermal preferences [10]. These factors, which illustrate the uniqueness of factory environment and workers, encourage the research on adaptive opportunities within factory buildings.

4. The Methodology

In a factory environment, the need for quality assurance of manufacturing activities will create many specific constraints to adaptive opportunities available for workers. Therefore, adequate information has to be collected from the management on the existing scenarios and preferences. It was also necessary to check the actual possibilities prior to formulating the experiments. They were obtained with observation made with factory visits and also conducting questionnaire surveys among the workers.

4.1 Questionnaire Survey for Factory authorities and workers

A questionnaire survey was carried out to find with factory authorities to obtain the following information.

(a) The preference ideas and basic knowledge of factory authorities regarding adaptive opportunities and related issues,
(b) Existing “adaptive opportunities” within the factory environments and to explore and identify the potential “adaptive opportunities” that can be implemented within the working environments.

A survey form was created as shown in appendix A, in Microsoft word format and distributed among factory authorities mainly by e-mails. They have the convenience of filling the survey form and return back via c-mails. About 10 factories that engage in light to medium work were also visited to acquire knowledge on thermal comfort issues and to investigate the rules and regulations practiced within the factories that influence the adaptive opportunity thus thermal comfort. A similar questionnaire survey was carried out with a sample of worker as well. Few preliminary field surveys were designed and carried out in a steel fabrication factory and in a plywood factory located low altitudes of Sri Lanka. The main purposes were to find the effectiveness of some behavioral adjustments on thermal comfort in warm humid conditions and to perceive the potential of some methodologies that arise as a result of questionnaire survey to use as “adaptive opportunities” in warm humid conditions.

5. Results and Discussion

5.1 Identification of Adaptive Opportunities

A good understanding of available behavioral adjustments (behavioral opportunities) and the behavioral opportunities that can be implemented for better thermal comfort within the factories were gained from the questionnaire survey. Further, the activities that would enhance the adaptive opportunities if implemented at right time were identified.

From the field surveys and questionnaire surveys it is found out that for a better adaptation not only workers but also supervisors and factory planners or designers have a role to play. Potential activities that affect the behavioral adaptation can be classified regarding different parties involved as follows:

1. Worker’s control: Adjusting clothing, posture, eating/drinking hot/cold food or beverages, or moving to a different location if permissible; Technological adjustments such as opening/ closing windows or shades, turning on fans or heating when control is available.

2.Supervisory restrains: Selecting time and time interval for lunch breaks, tea intervals, water breaks, etc. selecting the operating time and intensity of electrical appliances such as fans, control over changing places of workers, etc.

3. Planners and designers tasks: Determining the sizes and orientation of openings in the building envelope, selecting the positioning of fans and blowers, designing of production lines taking the thermal issue as a constraint, initiating thermally favorable dress code for workers, etc.

5.2 Possible Adaptive Measures.

There are many adaptive measures that can be used within factory buildings. They are the following.

***Moving**: -* The adaptive opportunity “moving” is mainly considered as the spatial shift seeking for a better comfort. Examples include moving away from a sun-patch or moving into an air-stream where thermal comfort is better than the initial position. But the fixed workplace layouts and set team structures designed for a particular process flow, which are a common observe in the bulk of factories provide little room to employ this option for thermal comfort. The economic pressure combined with relative low attention on thermal comfort would make it unfeasible to consider the spatial adaptation even in future factory designs. But it was noticed that the workers were allowed to walk away and comeback for personal requirements from their working positions time-to-time depending on the activities they are involved. Observed time gaps that workers have a chance to initiate a short walk were ranged from 10 minutes to an hour. This opportunity for a short walk could be used effectively to improve the thermal comfort.

Altering the clothes: - The clothing effect on thermal comfort was studied extensively in laboratories [15] and in office environments [16]. But not much information is available in tropical climates and in places like factories where clothing is prescribed. Most of the factory workers have an imposed dress code depending on the factory they are working. Usually a long sleeve shirt was used for upper body and only alternations allowed were scrolling up the sleeve and unbutton more often the top button of the shirt. In hot climates where upper temperature limit is more significant the clothing effect gets more complicated due to two effects called "pumping" effect and "wick" effect [17]. The pumping effect is due to the air-movement between clothing and skin and wick effect is due to the evaporative cooling enhanced by certain types of clothing fibers. It is showed that clothing insulation values could even become negative due to these effects, thus highlighted the significance of clothing effect on adaptation.

***Changing posture and activity**: -* By changing the posture the effective surface area of the body can be increased facilitating the heat loss by evaporation. The effective surface area can be increased by about 20% just by holding the arms further away and sitting with legs further apart than normal [17]. Not only the posture but also the postural changes affect the body temperature and heat balance and therefore the thermal comfort [18]. But in the factory work the posture is associated with a particular endeavor, which a worker is performing, and a good body weight balance determined by the posture is important to carry out the task effectively. Even though there might be few opportunities for changing the posture in favor of thermal comfort it's consequences on the body such as additional muscle strain and on the working efficiency is questionable and complicated therefore need well-directed extensive research to understand the problem, which is beyond the scope of this research. .

The "activity" is directly related to the thermal comfort as the metabolic rate, which influences the heat balance of the body, depends on the nature of the activity. The metabolic rates given in the literature [20] are average values and in reality these rates differ from a person to a person [18]. Also an individual can carry out the same activity for a range of metabolic rates by changing the rate or rapidness of the work. In this scenario the metabolic rate can be considered as an adaptive opportunity. A worker has the option to self-pace the rate of work depending on the environment and the perceived heat strain. But in factories in which the workers are paid for "piece-rate" this option hardly exists due to the monetary pressure. Workers tend to keep the same working tempo even in harsh warm instances to complete more work within the time available. The gross mechanical efficiency of human work, i.e. useful work out per metabolic energy production varies with type of work but for many activities it is order of 20% for an average person [19]. The other 80% dissipate as heat through the body to the environment. But for a well-adapted skillful worker this efficiency can be quite high causing less body heat production and he could dissipate this less amount of heat much more easily to the surrounding. Therefore he could feel a better comfort than an average worker doing the same work.

***Eating/drinking hot/cold food or beverages**:* A study done on a group of Australian Shearers [21] revealed how adequate meals; tea and water breaks prevent them from dehydration and maintain the same productivity level even in harsh hot conditions. Although Eating/drinking were not allowed at the working premises in bulk of the factories due to quality controlling regulations workers can be encouraged to drink more water regularly by facilitating easy access. Also lunch, tea and water breaks can be scheduled properly such that workers get more breaks when temperature is high and a fewer number of breaks at moderate comfort temperatures.

Apart from the personal adjustments that described so far, controlling the ventilation and solar glare are two popular technical adjustments that could be done to adapt to the thermal environment. The significance of the effect of ventilation and solar radiation respect to warm and humid climates was highlighted in many publications [22].

Most of the naturally ventilated factories use electrical fans especially ceiling fans to provide the adequate airflow. Many field surveys [23] have showed that people prefer variable fluctuating air-movements rather than uniform and monotonous airflows. Also the higher fluctuations of natural wind have close relationship with people's pleasure [24]. The fluctuation of electrical fan airflow is different from the fluctuation characteristics of natural wind. Therefore for a better thermal comfort and thermal adaptation it is required to increase the variability of air-movements as much as close to the variability of natural airflow. This scenario motivated to compare two basic types of fans i.e. ceiling and pedestal fans on the basis of the variability of airflow.

6. Conclusion

Useful information about the adaptive opportunities the potential to implement adaptive opportunities and also the difficulties arising when implementing the adaptive concepts were gathered. Also it is required to implement some experiments to understand especially regarding the effects of posture and activities to thermal comfort and productivity.

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Appendix A

Questionnaire Survey

1. Please comment on the following
   - Freedom of the worker’s to control electrical appliances and windows, shadings, etc
   - About the uniform of the workers
   - The frequency of having a short break

2. Describe the satisfactory/unsatisfactory of
   - Positioning of the windows, shades, fans, blowers, etc
   - The orientation of the production line(s) relative to the building envelope

3. Comment on your overall satisfaction about the thermal comfort of the factory

4. Your ideas of improving the thermal comfort of the factory

EFFECT OF BUILDING VENTILATION ON INDOOR ENVIRONMENT

Batagoda J.H.[a], Jayasinghe C. [b]

[a]Research Assistant, Department of Civil Engineering, University of Moratuwa, Sri Lanka
E-mail: janithahb@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: chintha@civil.mrt.ac.lk
Tel: +94 11 2650568(Ext 2111) Fax: +94 11 2651216

Abstract: Planning healthy indoor environments is a main part of sustainable design. In order to provide better indoor comfort for the occupants, modern building planning and operational practices should be either modified or improved. This study was aimed at building planning aspects and ventilation rates resulting in indoor environments. Since indoor CO_2 concentration can reflect the ventilation rate of built environments, it was monitored in a sample of residential buildings by varying several aspects related to air quality. This includes CO_2 levels with void to wall ratio of activity spaces in buildings, operating patterns of windows, and the wind speed inside the building together with the effect of micro-climate. Also the ventilation rates of an air conditioned spaces were monitored with a survey of occupant comfort levels .It was revealed that good ventilation rates by having higher void to wall ratio, higher indoor wind speed and proper window operating schedule can lower the indoor CO_2 levels and improve the comfort conditions. Therefore, complying with proper building planning practices such as selecting proper orientations, capturing the wind direction, practicing proper operating schedules for the provided openings and creating better micro-climate would result healthier built environments.

Key Words: *Ventilation, CO_2, Wind speed, Microclimate, Building Planning*

1. Introduction

It has become common knowledge that health hazards are developing due to various outdoor pollutant sources. However, most of the people spend about 80% - 90% of their time indoors. Hence, it is important to create better indoor environments for healthy living. There are a number of indoor air pollutants such as CO, CO_2, NO_2, SO_2, PM_{10}, $PM_{2.5}$ (particulate matter) and VOCs (volatile organic compounds). Each type of pollutant can cause different levels of health hazards for the occupants. The acceptable concentrations of each of the indoor pollutant have been specified in US Environmental Protection Agency (USEPA) and World Health Organization (WHO) guidelines.

When residential buildings are constructed, various activities can contribute to lower the quality of indoor air. The stages can include; during construction, building materials used, cooking, burning garbage, vehicle emissions from nearby roads, chemicals used in maintaining the houses etc, Building ventilation system plays a very significant role in maintaining a good indoor environment with better air quality. CO_2 concentrations, although it is non toxic, can be a direct indication of the ventilation system of the building. Higher concentrations of CO_2 also cause discomfort for the occupants.

The research presented in this paper includes the effects of void/wall ratio, window operating schedule, wind speed and microclimate on indoor CO_2 levels in free running residential buildings. Also this paper includes a case study carried out in an air-conditional environment on CO_2 levels and indoor comfort levels. The study was carried out in Sri Lanka where tropical climatic conditions prerail.

2. Building planning and indoor environment

In tropical climatic regions, the buildings with more enclosed spaces would need active means of lighting and ventilation for thermal and visual comfort. However, active means such as air conditioning would need significant amount of energy which would make the buildings unsustainable. Therefore, creating buildings as free running which rely on natural light and ventilation would be more desirable for the tropical climates. Especially in a developing country like Sri Lanka, there would be many free running buildings designed with passive features to maintain indoor thermal comfort.

When the built environment is designed as free running in tropical climates, the designers expect the external air to penetrate indoors through the openings provided, creating natural ventilation. There is also a need to maintain proper wind speeds in the indoors which enhances the comfort levels. However, most of the openings of residential buildings are kept closed during day time since majority of the household occupants are at the workplaces or at school due to prevailing social setup. By the time the occupants return home, the building is heated up and the indoor air is stuffy. If they do not open the windows in the night time as well, owing to their busy schedules, it might pollute the indoor environment further. The stagnant air collected over a period of time would create more and more indoor pollutants and long term exposure would create health problems to the occupants

Several methods have been developed in evaluating and relating air quality and ventilation in buildings. One of the techniques is analyzing CO_2 concentrations in the building, although CO_2 cannot be an indicator of overall indoor air quality [1, 2].

There are two concepts that can be used in defining relationships in indoor air quality and ventilation by using CO_2 concentration indoors. The first is that the amount of CO_2 generated by one person depends on the size and their level of physical activity where it is discussed in ASHRAE fundamentals handbook [3]. The second is to use CO_2 as a tracer gas to study the building when the CO_2 concentrations are elevated than the outdoors. Indoor CO_2 is sometimes referred to as an indicator of indoor air quality without describing a specific association between CO_2 and air quality, and number of relationships are available including the health effect of elevated CO_2 concentrations. Carbon dioxide at levels that are unusually high within indoors may cause occupants to grow drowsiness, get headaches, or function at lower productivity levels. Humans are the main indoor source of carbon dioxide. Indoor CO_2 level is an indicator of the adequacy of fresh air recharge relative to the indoor occupant density and metabolic activity.

The outdoor acceptable levels of CO_2 are ranging from 300 ppm – 500 ppm [4]. When the CO_2 levels are over 500 ppm, it indicates that the outdoor air is containing combustion or other contaminant sources. An exposure level of 700 ppm – 1000 ppm is taken as acceptable levels for indoors but the occupants will experience the stuffiness and odours [4].

Since the indoor CO_2 level is very important parameter, it would be useful to determine the way that indoor CO_2 levels could be influenced by the indoor operating conditions like availability of natural or forced ventilation. It would also be useful to determine the effect that a micro-climate created with a lot of trees could affect it. This paper deals with these aspects.

3. Experimental program and results

The study covered in this paper included the following experimental components:

- Indoor comfort levels were monitored in a randomly selected sample of residential buildings mainly located in urban areas. All the buildings are free running and relying on natural light and ventilation.
- A case study was conducted in an air conditioned environment to investigate the effect of controlled environment on indoor comfort levels. This space is a computer room in the Department of Civil Engineering, University of Moratuwa, Sri Lanka with a floor area equivalent to 125 m^2 in which, 40 computers have been kept.

Measurements

- This study was mainly focused on the indoor environment related to indoor CO_2 levels, indoor temperature, relative humidity and wind speed. Planning aspects of the buildings such as area of openings, locations of openings, orientation were also observed. A questionnaire was conducted among the occupants of the houses in the sample and in the air conditioned space considered in the case study in order to investigate whether they have any health problems related to the prevailing indoor environment.
- For each location, spot measurements were taken for all the parameters (CO_2 using ZG106 CO_2 and temperature monitor, temperature and humidity using a dry and wet bulb thermometer, wind speed) in every 15 minutes for a 3 hours period. All the measurements were taken during the day time. The spot measurements were also taken for CO (Carbon Monoxide) content of the air conditioned environment in 10 minute time intervals for a period of 8 hours.
- Measurements were taken by keeping the windows open and closed positions for free running residential buildings.
- Measurements were also taken in residential buildings in the same vicinity, with varying conditions for microclimate around the houses. Here, the micro climates were categorized as “poor” and “good” depending on the tree cover and presence of plants which are observed as more than 2m tall to act as a barrier to outdoor pollutants. The condition of microclimate was judged as poor or good by observation and not by measurement of coverage of trees, due to practical limitations of the study.

4. Results and discussion

4.1 Variation of CO_2 levels with Void/Wall ratio

Figure 1 shows the variation of the ratio Indoor/Outdoor CO_2 level with Void/Wall (V/W) ratio. This clearly shows that the CO_2 concentration inside the building would be very close to the outdoor concentration where the amount of active voids (Windows) is high, confirming the findings stated in literature [5, 6].

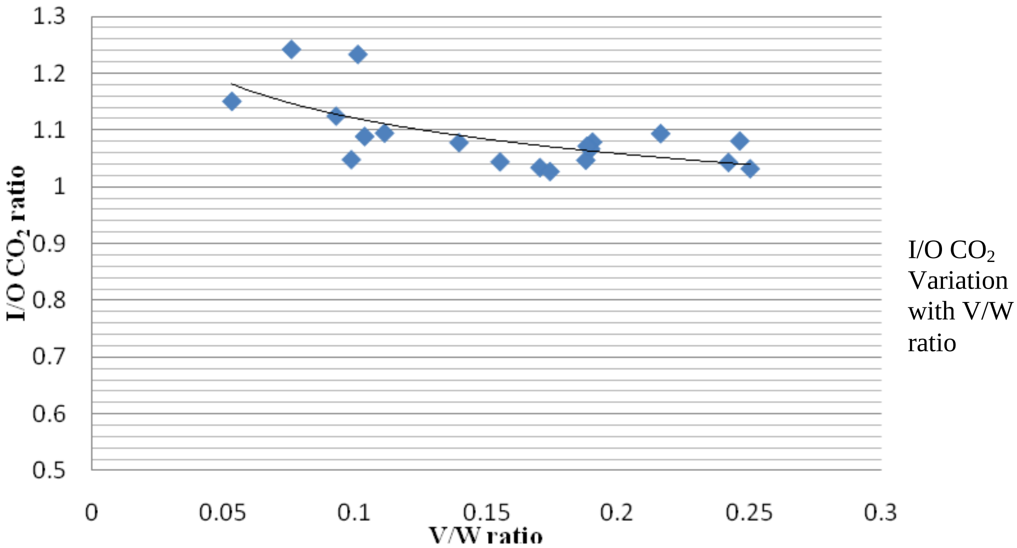

Figure 1: I/O average CO_2 variation with wall to void ratio

If the building designer can provide adequate number of windows by complying with the building regulations (in excess of the minimum recommended) prevail in the country, a reasonable level of indoor CO_2 can be maintained. The minimum is 1/7 of the floor area of any habitable room in Sri Lanka [7]. In order to maintain a reasonable indoor CO_2 level, the minimum void to wall ratio of an activity space (room) is proposed to be in the range of 0.15. It is preferable to have these openings in two different walls in order to facilitate cross ventilation.

4.2 CO_2 levels with operating practices of ventilation system

Figure 2 presents the importance of operating the ventilation system properly. The windows provided by the designer, should be opened and allow natural ventilation to happen during the operating cycle of the building. CO_2 measurements were taken over a period of five hours at each house in the sample by keeping the windows opened and closed. The I/O ratios for CO_2 concentration were evaluated for both conditions for all the locations and the average values are graphically represented in Figure 2. In Figure 2, it is clearly seen that indoor CO_2 level can go up when the windows are kept closed.

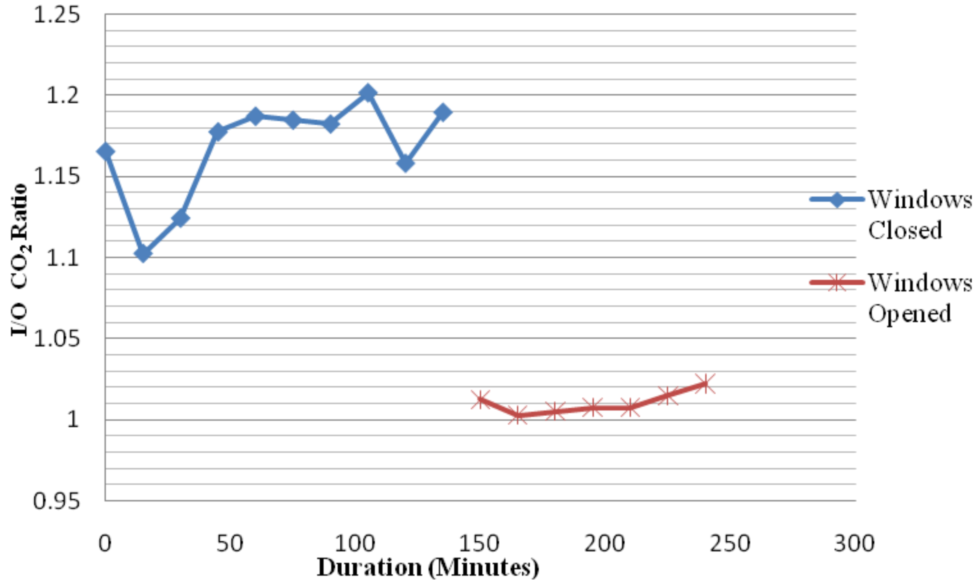

Figure 2: CO_2 variation windows opened and closed

It is very close to the outdoor CO_2 concentration when the windows are properly operated. Therefore, it is very important to provide and operate the means of natural ventilation over the entire life span of the building.

4.3 CO_2 variation with wind speed

The study conducted by Heidari [8], has shown that the air movement could affect the human comfort and the human comfort is the main point in building design. It was found that preferred speed at 28^0C is 1.0 ms^{-1}, at 29.6^0C, it is 1.2 ms^{-1} and at 31.3^0C, 1.6 ms^{-1}.

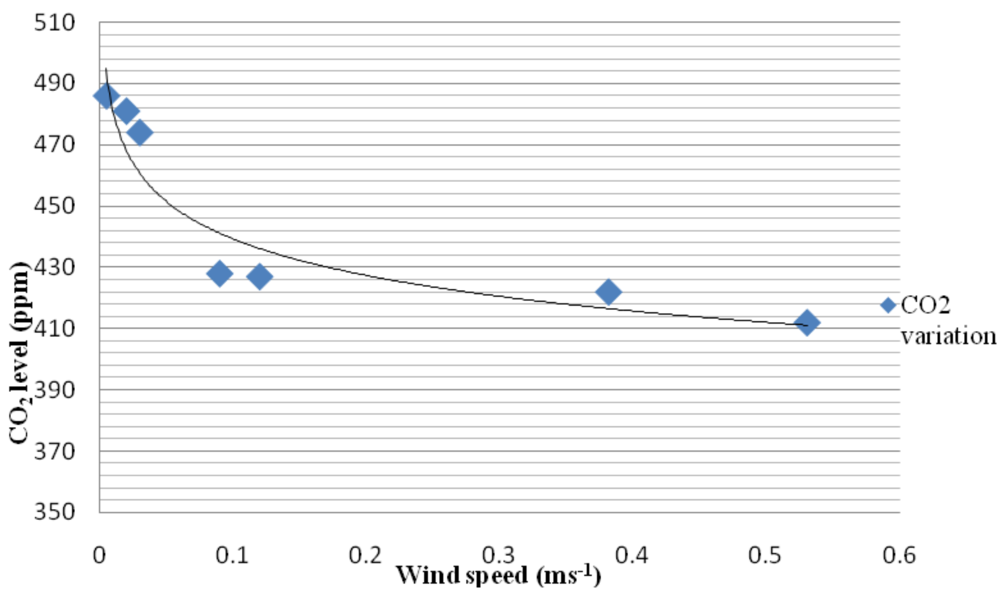

Figure 3: CO_2 variation with wind speed in the urban setting

Figure 3 shows the importance of providing adequate cross ventilation and air movement inside the activity spaces. This allows dilution of air pollutants collected indoors. It can be seen in Figure 3 that average indoor CO_2 levels decrease with the wind speed.

Therefore providing adequate cross ventilation is a very important factor in the design process of activity spaces. Better air circulation in all activity spaces of a building can be facilitated by implementing proper building planning practices such as selecting correct orientation by considering wind direction, paths of solar radiation, creating microclimate etc. Air movement in the night can be facilitated by providing strategically located court-yards inside the houses. This can also improve the thermal comfort inside the house.

4.4 *Effect of micro climate on indoor environment*

The building orientation and the microclimate play a key role in the thermal comfort of a building. Measurements were taken in the residential buildings considered in a sample for, with and without microclimate around different activity spaces. One such example is shown in Figure 4 where the TV lobby is surrounded by a good microclimate whereas the living room has a poor microclimate around it. Similar cases were monitored in the study and the results for the example considered are complied in Figure 5.

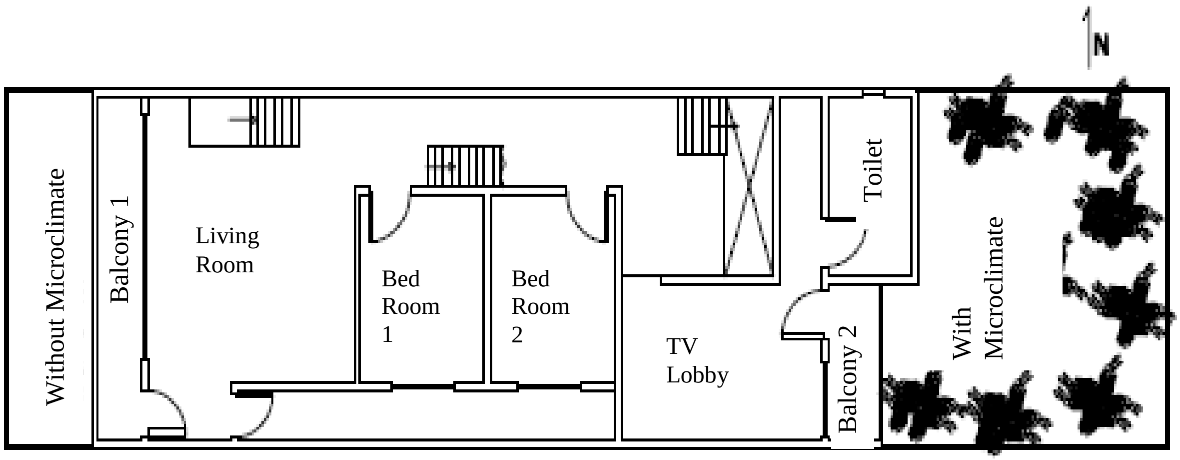

Figure 4: First Floor Plan of the house

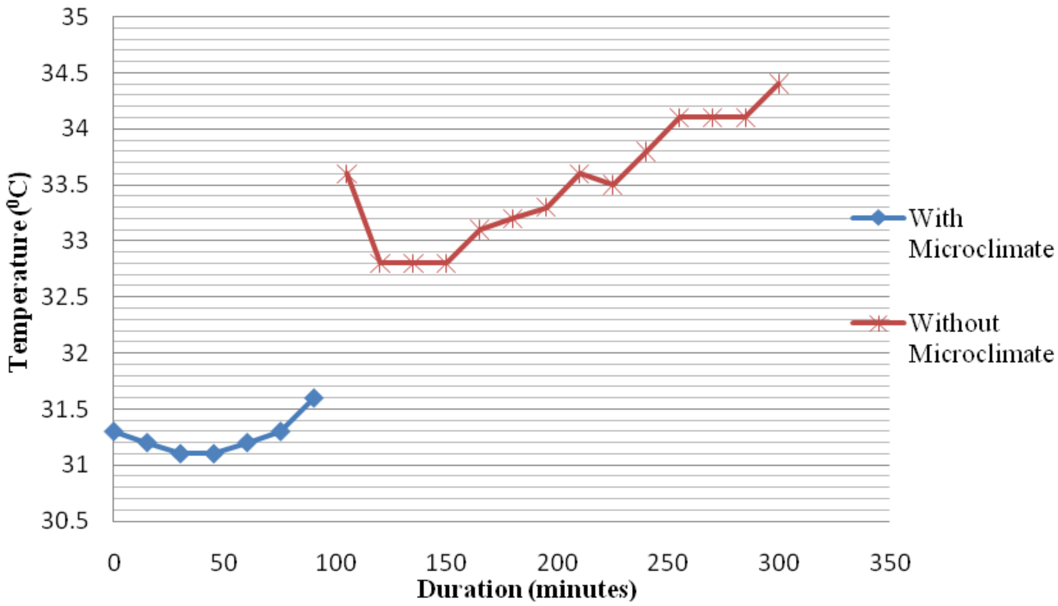

Figure 5: The temperature variation with and without Microclimate

The effect of the microclimate can be clearly seen in Figure 5. The indoor temperature is reduced by 2^0 – 3^0C when there is proper microclimate around the houses. The orientation of the house is important as well and the placing of windows that will affect the thermal comfort of the building [8].

In order to provide better outdoor comfort levels in tropical cities, it was proposed to include shading in street canyons, covered walkways and tree plantation, since microclimate plays a very important role in providing comfort conditions [9].

4.5 A case study in an air conditioned environment

Ventilation and climate control refers to the provision of clean outdoor air and properly conditioned supply of air into the occupiable spaces of a building. Outdoor air is provided as a mean of diluting occupant generated bio effluents and other indoor contaminants, and conditioned air is provided to maintain occupant comfort. Outdoor air can be provided either mechanically or via openable windows or vents.

A case study was carried out in a room with a floor area of 125 m^2 which is entirely run on active means of ventilation. This is the main computer room of Department of Civil Engineering, University of Moratuwa, Sri Lanka.

The room has about 50 computers, three laser printers, four line printers and three severs. Usually it is occupied by 40 students and five staff members, at a given time. A questionnaire survey was conducted in order to investigate whether the occupants have any sickness or discomfort related to the indoor environment.

It was found that the occupants who spend around six hours in this room have sicknesses such as head ache, drowsiness and lethargy, mainly in the afternoon.

The room was fitted with three split type air conditioners which are in full operation during the day time. The air conditioners are located in the places indicated in Figure 6.

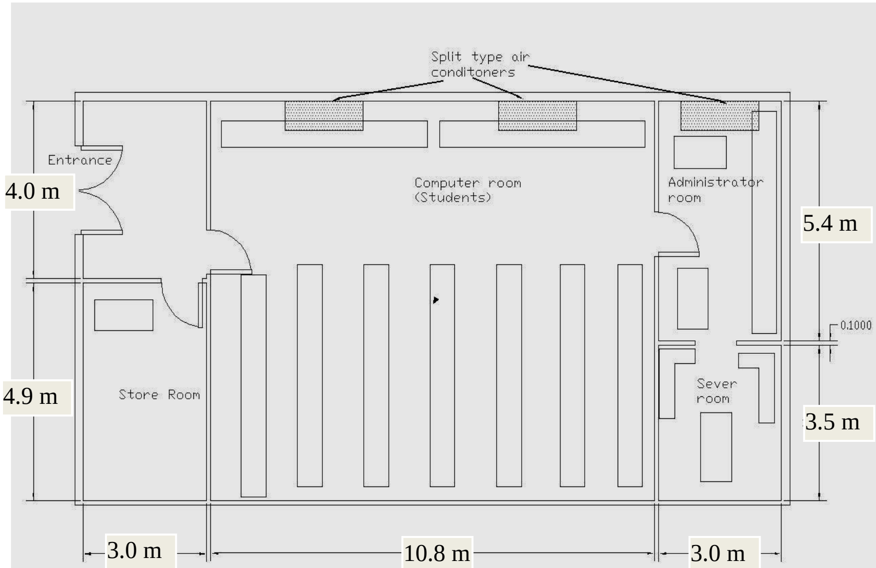

Figure 6: Plan view of the computer room

The levels of CO_2, CO, SO_2 and NO_2 were measured inside the room together with temperature and relative humidity. It was found that only CO_2 levels are relatively high and CO, SO_2 and NO_2 are negligible. Measurements were taken in every 15 minutes for a period of 3 days from 09:00 to 13:00 hrs.

The observations revealed that the CO_2 levels are higher than the recommended ASHRAE standards for an indoor environment.

Due to these findings the room was fitted with two exhaust fans with a discharge rate of 180 cfm bringing in the fresh air from the outdoors to the indoors. A same set of measurements were taken in the computer room after improving the ventilation system.

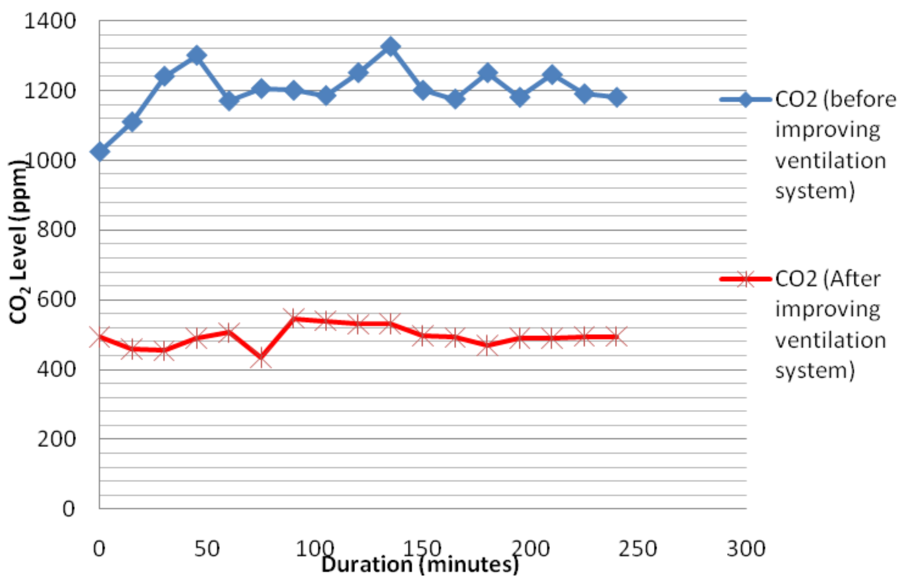

Figure 7: The CO_2 variation with time

The CO_2 levels before and after improving the ventilation system is shown in Figure 7 and it can be clearly seen CO_2 levels are higher than 1000ppm recommended by ASHRAE before the improvement. After the improvements it had come down to about 400 – 500 ppm.

5. Conclusion

The study covered in the paper was aimed at determining the effect of void to wall ratio, indoor wind speed, operating schedule of openings on the ventilation rates and the occupant comfort. It was revealed that the indoor CO_2 levels can drastically go up with low void to wall ratio. Therefore it is recommended to provide window area at least equal to the minimum provided in the building regulations.

The orientation of the building should be selected by considering the direct solar radiation and the wind direction. It could be clearly seen that CO_2 levels inside the building goes down with higher wind speeds and would approach the outdoor levels

The operating schedule of the openings in a naturally ventilated building has been identified as another important parameter which contributes immensely towards the indoor comfort. Although the adequate number of windows is provided in the design, the expected comfort levels cannot be achieved unless the proper operating schedules are maintained.

In order to provide better indoor thermal comfort, creating a good microclimate around the house can greatly contribute. It was found in the study that there is a 2^0C to 3^0C reduction in the indoor temperature with better microclimate around the house. Therefore the planners are encouraged to design the landscaping of the built environment to achieve better indoor thermal comfort.

When considering artificially ventilated spaces, the acceptable comfort levels can be maintained with better rates of ventilation with recharge of fresh air. With a better ventilation system, is was able to achieve lower CO_2 levels around 500ppm and better occupant comfort with no complaints about sicknesses related to indoor air quality. Therefore, even if the building is air conditioned, it is essential to check the adequacy of ventilation rates which could be linked with the indoor CO_2 levels.

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DEMAND CONTROL VENTILATION AND SRI LANKAN APPLICATION – A CASE STUDY

Jayasinghe C. [(a)], Batagoda J.H.[(b)]

[(a)]Professor, Department of Civil Engineering, University of Moratuwa, Sri Lanka
E-mail: chintha@civil.mrt.ac.lk
Tel: +94 11 2650568(Ext 2111) Fax: +94 11 2651216
[(b)]Research Assistant, Department of Civil Engineering, University of Moratuwa, Sri Lanka
E-mail: janithahb@civil.mrt.ac.lk
Tel: +94 11 2650568 (Ext 2021) Fax: +94 11 2651216

Abstract: Using Carbon Dioxide (CO_2) has been oftenly used as a indicator of indoor air quality, thus now is been practice all over the world. Since advance technologies for gas sensing have been developed to regulate air handling systems while continuously monitoring the occupied building area. Either too little or too much fresh air can be a problem to the building functionality, where over ventilation results higher energy usage and cost than appropriate ventilation while potentially increasing indoor air quality problems in warm, humid climates. To ensure adequate ventilation to the buildings, the American Society of Heating, Refrigerating and Air Conditioning Engineers (ASHRAE) had recommended ventilation rates in standard 62. Further, to coincide with the standards, many ventilation systems are designed to admit maximum level whenever a building is occupied, making the presumption that every area of the building is occupied, which leads towards over ventilated buildings. Adopting demand control ventilation (DCV) to a building will prevent over ventilation and secure the indoor from poor ventilation. In applications of CO_2 based demand control ventilation, minimum base ventilation should be provided, and system is much appropriate for the areas where human activities are held. The potential amount of energy saved due to the implementation of a DCV system will be momentous in large commercial buildings. Adopting this system will reduce the amount of energy that a ventilation system requires to accumulate in a given time that depends on the occupancy level.

Key Words - Carbon Dioxide (CO_2), Demand control ventilation, ventilation systems

1.0 Introduction

When HVAC is considered, the energy efficient buildings are making a rapid contribution to the building industry, where the necessity of energy conservation is a must in a modern building. In the design of ventilation practices, influence of energy conservation is conquering the conventional practices.

There are many ventilation requirements and recommendations in the form of outdoor air flow rates per person. To achieve such requirements, ventilation systems are designed to provide the minimum levels of outdoor air based on the amount of people and the floor area.

Carbon Dioxide (CO_2) is an achromatic and odorless gas. Building occupants are the main indoor emitters of CO_2 therefore; the indoor generation of CO_2 is dependent on the number of occupants and their level of physical activity level. In addition to simple respiration, smoking also creates CO_2 in large amounts. Higher than the normal levels of CO_2 in indoors may cause occupants to grow drowsy, get headaches, or function at lower activity levels. Indoor CO_2 levels could be an indicator of the adequacy of outdoor air ventilation relative to indoor occupant density and metabolic activity of those occupants.

2.0 Background

Demand control ventilation attempts to achieve acceptable *indoor air quality (IAQ)* at a reduced energy cost by controlling the outdoor airflow rate of an HVAC system, based on certain measured parameters. For example, indoor pollutant concentration or measures of building occupancy using different means of sensors that exemplify the building occupancy level can be used as parameters on which to base the rates of ventilation. Considering a CO_2 based demand control ventilation system, CO_2 is used as an indicator of the occupancy and the amount of ventilation achieved by the system. A sensor measuring the CO_2 concentration is used to control the ventilation system providing necessary ventilation to the building.

The potential advantage of a demand control ventilation system is that, it will make sure the space will have exactly the right amount of ventilation necessary thus, whether the occupancy level is high or low, the ventilation system automatically maintains IAQ of the building at the appropriate level. This process of automatic ventilation controlling will not only increase the indoor air quality but will also help to bring down the energy consumption because the system only runs at the capacity accommodate the current demand and never higher.

The demand control ventilation using CO_2 sensing is a combination of two technologies, the CO_2 sensors and the air handling system that uses the data from the sensor to regulate the ventilation system to provide necessary outdoor air to the indoors. Demand control ventilation systems operate on the premise that basing the amount of ventilation air on the fluctuating needs of the building occupants, rather than a pre-set formula. This will save energy and at the same time will help to maintain a healthy indoor air quality levels.

Controlling outdoor air intake rates using CO_2 *demand controlled ventilation* (DCV) offers the possibility of reducing the energy price of over-ventilation during periods of low occupancy, while still ensuring adequate levels of outdoor air ventilation. As discussed later in this report, depending on climate and occupancy patterns, CO_2 DCV may provide significant energy savings in commercial and institutional buildings. While a number of studies have suggested that extent of such savings via field studies and computer simulations, need additional work to better define the magnitude of energy savings possible and the dependence of these savings on climate, building and system type, control approach, and occupancy patterns. In addition, important issues remain to be resolved in the application of CO_2 DCV including how best to apply the control approach, including issues such as which control approach to use in a given building, sensor location, sensor maintenance and calibration, and the amount of baseline ventilation required to control contaminant sources that do not depend on the number of occupants.

While it is not critical to the application of CO_2 DCV, the emission rate of occupant generated CO_2 is certainly a relevant issue in this discussion. This section discusses the rate at which people generate CO_2. People consume oxygen and generate CO_2, at a rate that depends primarily on their body size and their level of physical activity. The relationship between activity level and the rates of oxygen consumption and carbon dioxide generation is discussed in the ASHRAE Fundamentals Handbook (ASHRAE 1997). The rate of oxygen consumption V_{O_2}, in *l/s*, of a person is given by the following equation;

$$V_{O2} = \frac{0.00276 A_D M}{(0.23RQ + 0.77)} \qquad \text{Eq (1)}$$

where *RQ* is the respiratory proportion, i.e., the relative volumetric rates of carbon dioxide produced to oxygen consumed. *M* is the level of physical activity, or the metabolic rate per unit of surface area, in mets ($1\ met = 58.2\ W/m^2$). A_D is the DuBois surface area in m^2, which can be estimated by the following equation;

$$A_D = 0.203 H^{0.725} W^{0.425} \qquad \text{Eq (2)}$$

Where H is the body height in *m* and *W* is the body mass in kg. For an average size adult, A_D equals about 1.8 m^2. Additional information on body surface area is available in the EPA Exposure Factors Handbook (EPA 1997). The value of RQ depends on diet, the level of physical activity and the physical condition of the person. It is equal to 0.83 for an average size adult engaged in light or deskbound activities. RQ increases to a value of about 1 for heavy physical activity, about 5 *met*. Given the expected range of RQ, it has only a secondary effect on carbon dioxide generation rates.

Table 1: Typical Met levels for various activities (ASHRAE 1997)

Activity	**Met**
Seated, quiet	1.0
Reading and writing, seated	1.0
Typing	1.1

Filing, seated	1.2
Filing, standing	1.4
Walking at 0.9 m/s (2 mph)	2.0
House cleaning	2.0-3.4
Exercise	3.0-4.0

Emmerich et al. (1994) applied the model developed by Knoespel et al. (1991) to examine the performance of DCV systems under less favorable conditions and to study the impact on non-occupant generated pollutants. Emmerich used the same building, Madison location, and the HVAC systems described above but varied the simulated conditions to include pollutant removal effectiveness as low as 0.5 and an occupant density up to 50 % greater than design. For all cases examined, the DCV system reduced the annual cooling and heating loads from 4 % to 41 % while maintaining acceptable CO_2 concentrations. In addition to requiring more energy use, the constant outdoor airflow strategy resulted in CO_2 levels above 600 ppm for more than half of occupied hours for cases with poor pollutant removal effectiveness.

3.0 Case study

In the Sri Lankan context, adaptation of demand control ventilation will provide much more energy efficient buildings and will provide much better indoor air quality for the occupants in houses and also for other buildings as well. The most common type of air conditioners available in Sri Lanka is the split type air conditioner where the system has less recharge to the building from the outdoor which leaves the indoor environment is lacking adequate ventilation.

Ventilation and climate control refers to the provision of clean outdoor air and properly conditioned supply of air into the occupiable spaces of a building. Outdoor air is provided as a mean of diluting occupant generated bio effluents and other indoor contaminants, and conditioned air is provided to maintain occupant comfort. Outdoor air can be provided either mechanically or via openable windows or vents.

A case study was carried out in the main computer room of Department of Civil Engineering, University of Moratuwa, Sri Lanka, with a floor area of 125 m^2 which is entirely run on active means of ventilation.

The room has about 50 computers, three laser printers, four line printers and three severs. Usually it is occupied by 40 students and five staff members, at a given time. A questionnaire survey was conducted in order to investigate whether the occupants have any sickness or discomfort related to the indoor environment.

It was found that the occupants who spend around six hours in this room have sicknesses such as head ache, drowsiness and lethargy, mainly in the afternoon.

The room was fitted with three split type air conditioners which are in full operation during the day time. The air conditioners are located in the places indicated in Figure 1.

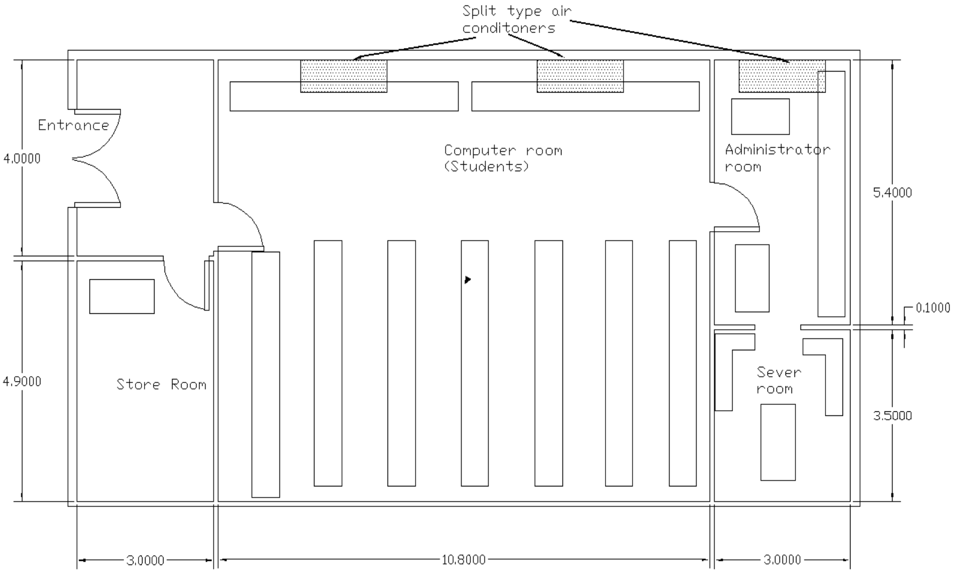

Figure 1: Plan view of the computer room

The levels of CO_2, CO, SO_2 and NO_2 were measured inside the room together with temperature and relative humidity. It was found that only CO_2 levels are relatively high and CO, SO_2 and NO_2 are negligible. Measurements were taken in every 15 minutes for a period of 3 days.

The observations revealed that the CO_2 levels are higher than the recommended ASHRAE standards for an indoor environment.

Due to these findings the room was fitted with two exhaust fans with a discharge rate of 180 cfm bringing in the fresh air from the outdoors to the indoors. A similar set of measurements were taken in the computer room after improving the ventilation system.

The outdoor air flow rate of the space without the exhaust fans;

O_2 consumption per person = $V_{O2} = \dfrac{0.00276 A_D M}{(0.23RQ+0.77)}$

Where, A_D = 1.8 m^2 (for an average person)
M = 1.1 (a person Typing, seated)
RQ= 0.83 (for an average person)

Using the formula $V_{O2} = \dfrac{0.00276\times1.8\times1.1}{(0.23\times0.83+0.77)}$
$= 5.69\times10^{-3}\ l/s$

The CO_2 generation per person $V_{CO2} = 0.83\times5.69\times10^{-3}\ l/s$
$= 4.73\times10^{-3}\ l/s$

Using the formula $Q_0 = \dfrac{1.8\times10^6 G}{(C_{in,eq}-C_{out})}$

The average indoor CO_2 level is 1201 ppm (2161.31 mg/m^3), and the outdoor concentration of CO_2 is 410 ppm (737.83 mg/m^3)

$$Q_0 = \frac{1.8\times10^6\times4.73\times10^{-3}}{(2161.31-737.83)}$$
= 5.98 *l/s* (per person)

In ASRAE standards 62.1-2007, it is sated that the outdoor air flow rate per person in a computer lab in an educational facility should be 5 *l/s* (Table 6-1 of ASRAE standards 62.1-2007), where it is not valid in isolation. But considering the single space, the breath zone outdoor air flow rate using ASHRAE standards 62.1-2007, (Equation 2, Equation 6-1 of ASRAE standards 62.1-2007),

$$V_{bz} = R_p.P_z + R_a.A_z \qquad \text{Eq(3)}$$

Considering the lab to be fully occupied at its maximum capacity of 45 occupants,

R_p = 5 *l/s* (Table 6-1 of ASRAE standards 62.1-2007)
P_z = 45
R_a = 7.4 (Table 6-1 of ASRAE standards 62.1-2007)
A_z = 125 m^2
$V_{bz} = R_p.P_z + R_a.A_z$
= 5×45 + 7.4×125
= 1150 *l/s*

The total ventilation rate per person = 25.56 *l/s*

It is clear that the ventilation rate required per person is not met in the computer lab. Due to the inadequate ventilation rates, the space always has a high CO_2 concentration and there is evidence that occupants spending more than 4 hours inside have had symptoms of breathing unhealthy indoor air.

Figure 2 shows the ventilation requirement to the single space with the occupant density per single person according to the ASHRAE standards.

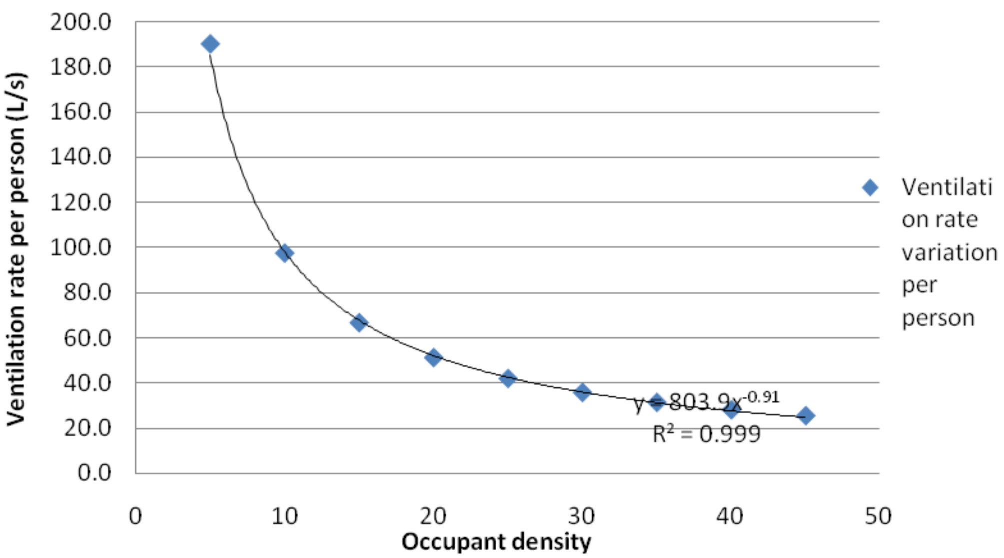

Figure 2: Variation of outdoor air flow per person with the variation of occupancy density in the activity space (computer lab)

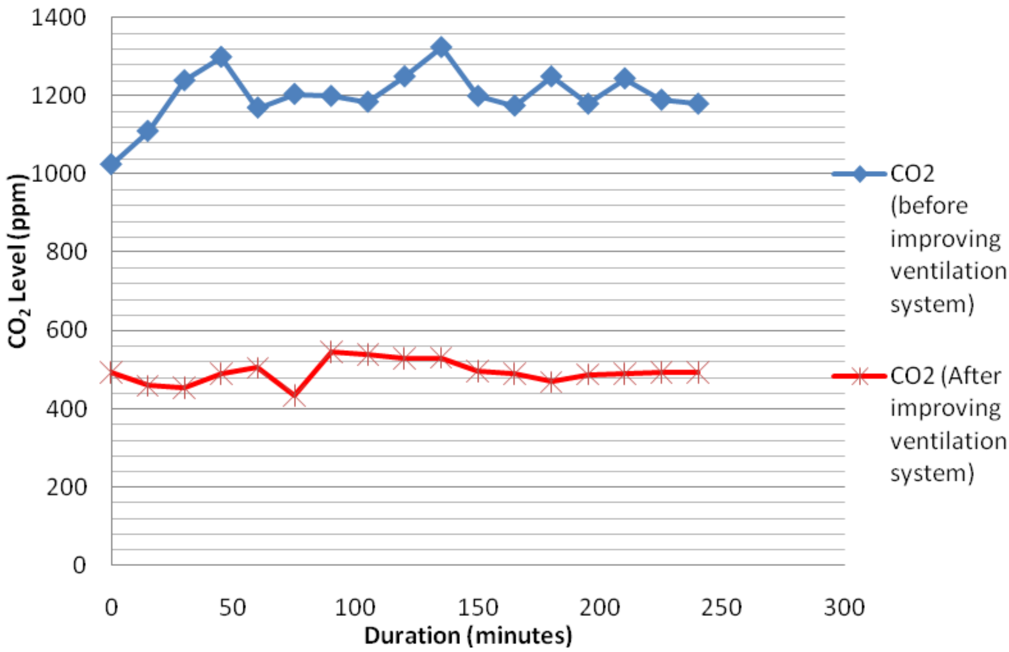

Figure 3: The CO_2 variation with time

The CO_2 levels before and after improving the ventilation system is shown in Figure 5.3 and it can be clearly seen CO_2 levels are higher than 1000ppm recommended by ASHRAE before the improvement. After the improvements it had come down to about 400 – 500 ppm.

4.0 DCV for Sri Lanka

The use of demand control ventilation to Sri Lankan buildings will help to develop healthy environments in the buildings while making the system energy efficient. Currently the amount of buildings that used this system is very low where they tend to use the split type air conditioners or central system. When considering the split type air conditioners the amount of air recharge to the building is very low, and it is clearly shown by the case study. However in the case of central air conditioning systems the indoor air tends to be recharged continuously which increases the amount of energy consumed by the system even when the occupancy levels are low.

Considering the previous case study in an air conditioned space it is clear that when the space is recharged with outdoor air the CO_2 levels comes down, but when the occupancy levels is low the amount of air recharge needed is less. A system which can identify the occupancy level in a given space will be much more efficient at recharging the indoor with the necessary outdoor air to build up a good indoor environment. Furthermore, recycling the indoor air up to a level that the occupants can tolerate will minimize the amount of energy spent on cooling the air brought from the outdoors.

There are very few studies which have been done on the use of demand control ventilation in tropical climates, but with advent of “green concepts” and “sustainable development”, it is necessary to conserve energy consumption in buildings. Developing algorithms to control the ventilation process will provide the means to control the ventilation system when the occupancy density is varying with time. Three different occupancy estimation algorithms are available to consider when adopting the demand control ventilation to a building, which are; steady state, approximate dynamic detection, and exact dynamic detection.

Steady-state CO_2 concentrations can be determined for a given ventilation rate based on a single-zone mass balance analysis. Assuming that the CO_2 concentration in the building or space of interest can be characterized by a single value C, the mass balance of CO_2 can be expressed as follows:

$$V\frac{dC}{dt} = G + QC_{out} - QC \qquad \text{.............................. Eq(4)}$$

Where:

- V = building or space volume (mass) in m^3 (mg)
- C = indoor CO_2 concentration in mg/m^3 (ppm)
- C_{out} = outdoor CO_2 concentration in mg/m^3 (ppm)
- t = time in s
- G = indoor CO_2 generation rate in mg/s (m^3/s)
- Q = building or space ventilation rate in mg/s (m^3/s)

O_2 consumption per person = $V_{O2} = \frac{0.00276 A_D M}{(0.23RQ + 0.77)}$

Where A_D = 1.8 m^2 (for an average person)
M = 1.1 (a person Typing, seated)
RQ = 0.83 (for an average person)

Using the formula $V_{O2} = \frac{0.00276 \times 1.8 \times 1.1}{(0.23 \times 0.83 + 0.77)}$
$= 5.69\times10^{-3}$ L/s

The CO_2 generation per person $V_{CO2} = 0.83\times5.69\times10^{-3}$ L/s
$= 4.73\times10^{-3}$ L/s

Using the formula $Q_0 = \frac{1.8\times10^6 G}{(C_{in,eq} - C_{out})}$

Assuming the indoor CO_2 level is at a constant of 500 ppm (899.79 mg/m^3), and the outdoor concentration of CO_2 as 410 ppm (737.83 mg/m^3)

$$Q_0 = \frac{1.8\times10^6\times4.73\times10^{-3}}{(889.79-737.83)}$$
= 56.03 L/s (per person)

When the computer lab occupancy is at full capacity the maximum capacity total outdoor air flow rate } 56.03 × 45 = 2521.35 L/s

When the computer lab occupancy is at half the capacity the maximum capacity, total outdoor air flow rate } 56.03 × 22= 1400.75 L/s

Assuming the outdoor air temperature to be 30^0C and the indoor temperature is maintained at 27^0C

The heat gain due to the ventilation $= q_v = 1200\times V_r$
Where V_r is 2521.35 L/s for maximum
$= 1200\times2.52135$
$= 3025.62$ W/K

The total energy gain $= q_v\times\Delta T$
$= 3025.62\times3$
$= 9076.86$ W

The total energy gain is the equivalent of the total energy needed for the air conditioner to cool the system. Hence, when the occupancy level comes down the total energy requirement comes down proportionally.

5.0 Conclusion

Applying the demand control ventilation system to the computer lab will make sure that the amount of energy required for the building will go down relevant to the occupant density. Demand Controlled Ventilation has been seen to offer new technical and administrative tools for the operation of complex buildings. In particular, the energy budget for cooling and ventilation may typically be halved by careful design, thus considerably lowering the running expenses. However, the improvements in indoor air quality that follow as a free bonus of DCV are equally important, since air quality is measured and controlled against set standards wherever and whenever people are present. Health personnel concerned about bad indoor climate for the workforce need not any longer argue against the perceived high cost of securing much better indoor air quality. DCV strategy calls for less outdoor air over the course of the cooling seasons than does a regulatory ventilation strategy, then the annual energy required to heat or cool the outdoor air decreases. In addition, lower outdoor air requirements decrease the fan energy expended to introduce and expel the air from the building. It is widely believed that actual occupancy levels in U.S. buildings are significantly lower than the design occupancy levels that conventional ventilation systems are set to handle. Buildings and spaces with large swings in occupancy, e.g., movie theatres and conference rooms, tend to realize the largest savings. DCV reduces peak electricity demand when actual occupancy levels fall below design occupancy levels during peak demand periods. Lower levels of outdoor air translate into decreased cooling loads and, therefore, air-conditioning power draw. In some cases, DCV may allow building operators to close fresh air dampers for short periods during the hottest hours typically coinciding with peak electric load. In general, peak reductions vary from building to building, depending on occupancy patterns. Consequently, the average peak demand reduction likely will mirror the cooling energy savings potential.

Acknowledgements

The authors wish to express their sincere thanks to the National Research Council Sri Lanka for providing financial support for the equipment used in the study. The research grant provided by the Senate Research Committee, University of Moratuwa, Sri Lanka has helped immensely to employ the research assistants for the study.

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EFFECT OF GREEN WORKPLACE ENVIRONMENT ON EMPLOYEE PERFORMANCE

Samaranayake, S.U.[1] and De Silva, S.[2]

[1] Dept. of Agricultural Extension, University of Peradeniya, Peradeniya, Sri Lanka.
E-mail: ssuari.uop@gmail.com
Telephone: +94-11-2650567-8 (Ext. 2006 - 2001); Fax: + 94-11-2651216

[2] Senior Lecturer, Dept. of Agricultural Extension, University of Peradeniya, Peradeniya, Sri Lanka.
E-mail: saliyades@pdn.ac.lk
Telephone: +94-81-2395522; Fax: + 94-81-2395521

Abstract: Today, to minimize the negative environmental impacts establishment of green buildings has become a worldwide trend. Many industries are adopting Leadership in Energy and Environmental Design (LEED) certification, as a global benchmark for high performing green buildings. This concept is new to Sri Lanka and the impact of LEED on employee performance and satisfaction has not been studied yet. Therefore, this study attempts to examine the perceptions of employees regarding their green workplace environment and its impact on their performance. The study was conducted with randomly selected, 30 factory staff members and 30 factory workers in an export apparel company that has won a platinum award for LEED. Majority of employees (68.9%) mainly factory staff members, had a good understanding about the LEED practices. Employees believe that introduction of LEED created a good impact on their work lives. About 86% of employees perceive that their performance has improved after establishing the green building. The green building has rewarded the company by improving its employee performance, saving energy and resources, maintaining the market, and creating a better public image.

Keywords: green buildings, employee performance, LEED, sustainable development

1 Introduction

The textile and apparel sector continues to maintain its dominant position in the industrial sector of Sri Lanka, while contributing around 44% of industrial production and 49% of the country's total export earnings with a value of US $2,809 million in 2009 (Central Bank, 2009). There are 830 garment factories in Sir Lanka, of which 157 are small, 438 are medium, and 235 are large. The industry produces around 500 million units of garments per annum of which woven garments account for 55% and knitwear 45 % (Saheed, 2005). The present status of the apparel industry in Sri Lanka represents dynamic, ever-changing conditions in import dependence for fabrics and other raw materials, tax and fiscal incentives/barriers including GSP+, labor costs, and skilled and trainable labor. In addition to the above, other conditions such as health and safety, environmental issues, quality issues, eco-labeling, formation of economic and trade groups, currency fluctuation, etc., also have an effect on the apparel industry (Saheed, 2005).

Environmental responsibility is fast becoming a major determinant of sustainability of any business in the world. As David Birnbaum (2010) states end-consumers begin to see that global warming and other environmental problems that directly affect the quality of their lives and those of their families; they will shun products which are perceived to have been made in conditions that pollute the planet. Therefore, green buildings will become a necessary for the survival of apparel industry. In this respect, establishment of green factories has become imperative to reduce environmental burden imposed by the use of the raw material and energy resources, waste generation and chemical emission. Many organizations are adopting Leadership in Energy and Environmental Design (LEED) certification[20], as a global benchmark for high performing green buildings.

[20] LEED is an internationally recognized rating system that acts as third party verification for green building certification (USGBC, 2002).

Although this concept is new to Sri Lanka, responding to demands of the buyers, three green factories have already been established in apparel industry with LEED standards. Brandix Green Factory at Seeduwa was the first apparel manufacturing facility in the world to be rated Platinum certification under LEED. MAS Holding's *Thuruli* factory in Thulhiriya, and CKT apparel of Hidramani Group are the other green factories in Sri Lanka. It is important for these organizations i.e. top management and other interested parties, to understand the benefits of green building environment from the employees' perspective. According to The Royal Institution of Chartered Surveyors (2005), the green building benefits relate to increased occupant productivity and satisfaction, exceeding even the projected environmental benefits. However, the lack of worker awareness, communication problems, and the lack of supporting research about reported employee benefits, may reduce expected benefits. Therefore, this study was carried out to examine the perceptions of employees regarding their green environment and its impact on their performance in one of the green factories in export apparel industry.

2 Material and Methodology

2.1 Towards Green Buildings

The green building movement as a sustainable development strategy is fast becoming a necessity (Prakash, 2005). Kibert (2007) defines Green Building as a healthy facility, built in a resource efficient manner using ecologically based principles. According to LEED-EB Reference Guide (2006), "Green" has become a shorthand term applied in building construction industry to denote high performance buildings innovated with the objective of to be environmentally responsible, economically profitable and healthy place to work and live.

According to a study done in the United States, buildings annually consume more than 30 percent of the total energy and more than 60 percent of the total electricity. Green building practice can substantially reduce negative environmental impacts through high performance, energy saving, and market leading design, construction and operations practices. The added benefits of green operations and management include reduced operating costs, enhanced building marketability, increased workers' productivity, and reduced potential liability resulting from indoor air quality problems (LEED-EB Reference Guide, 2006).

2.2 Green Building Assessment Schemes

When attention and awareness regarding development of the sustainable constructions is increased, it was very important to have an assessment system for green buildings. The most often building environment assessment schemes that are used today include Building Research Establishment Environmental Assessment Method – BREEAM, Comprehensive Assessment System for Building Environment Efficiency – CASBEE, Green Star, and Leadership in Energy and Environmental Design – LEED (Prakash, 2005).

BREEAM scheme is the most widely used building environmental rating scheme in the UK, which was voluntarily started in 1988. It assesses the building impact on the environment including management, health and wellbeing, energy, transport, water, materials, waste, land use, ecology, and pollution and gives credits up to maximum of 102 under each category (Roderick, *et al*, 2010). CASBEE was started in 2001 that can be applied for many types of buildings, such as offices, schools, retail stores, restaurants, halls, hospitals, hotels and apartments under various categories such as planning, design, completion, operation and renovation (Endo *et al*, 2007). Green Star is the most followed voluntary building environmental assessment scheme in Australia. It was developed to accommodate the need of buildings in hot climates where cooling systems and solar shading are of major importance (Roderick, *et al*, 2010). The Green Star rates a building with corresponding to its management, the health and wellbeing of its occupants, accessibility to public transport, water use, energy consumption, the embodied energy of its materials, land use and pollution (GREEN BIM, 2007)

LEED is an internationally accepted benchmark for the design, construction and operation of high-performance green buildings. According to LEED-EB Reference Guide (2006) LEED-EB refers to LEED Certification for Existing Buildings that cover building operation and system upgrades in existing buildings where the majority of interior and exterior surfaces remain unchanged. This certification process envelops whole-building cleaning and maintenance issues including chemical use, indoor air quality, energy efficiency, water efficiency, recycling programs, exterior maintenance programs, and system upgrades to meet green building energy, water, air, and lighting performance standards. It aims to maximize the operational efficiency while minimizing the environment impacts.

LEED promotes a sustainable approach by considering the performance of green building on five key areas of human and environmental health: sustainable site development, water savings, energy and atmosphere, materials and resources, and indoor environmental quality. Innovations in operations were also added recently as a key area to the system. This sixth category tries to cover the sustainable building expertise as well as design measures not covered under the five initial environmental categories (USGBC, 2002; LEED Reference Guide, 2006).

2.3 Research Methodology

A survey in the selected organization, *viz.* an export apparel factory that has won a platinum award for LEED, was carried out to achieve the objectives of this research. It was conducted among randomly selected, 30 factory staff members and 30 factory workers. Employee perceptions were obtained through a structured questionnaire, which consisted of employee performance as dependant variable and attributes of the indoor and outdoor environment as the independent variables. Mostly descriptive statistical techniques were used to analyze data through the Statistical Package for Social Science (SPSS) software. The findings were verified through interviews, informal discussions, and participant observations.

3 Results and Discussion

3.1 Characteristics of the Employees

The response rates of factory workers and factory staff members were 100% and 93.33% respectively. Normally the mainstream of the labor force in garment industry is females, thus the majority of the respondents were females (67.8%). Most of the respondents were Machine Operators (43.1%) and a greater part of the factory staff that responded were Production Supervisors (53.6%). Employees, who had less than five years experience in the factory made up 75.9% of the respondents. Around 71% of the employees were at the mid age group (25-35 years old) and approximately similar proportions were belong to under 25 years old (13.8%) and above 35 years old (15.5%) categories respectively. While majority of factory staff members (60.7%) were married, majority of factory workers were unmarried. Considering the group of factory staff, greater part (89.3%) was educated up to G.C.E Advance Level (A/L). Among factory workers that much education qualification was not observed as the majority of them (73.3%) were in the up to G.C.E. Ordinary Level (O/L) category.

3.2 Employee Awareness about LEED

Majority of factory staff members (57%) perceive that they have very good understanding about LEED standards practiced in the factory. In contrast, only 13% factory workers believe that they have very good understanding about LEED. The χ^2 test ($p = 0.004$) shows a significant relationship between employee category and understanding about LEED. This may be due to the presence of close relationship between factory staff members and the key personnel who are responsible for the green project (green project team). There was not such kind of visible relationship between factory workers and green project team.

The results obtained for understanding about the environmental impacts of practicing LEED is somewhat similar to above result. Most of factory staff members (57%) compared to only 14% of factory workers believe that they have very good understanding about environment impacts of practicing LEED. A significant relationship between employee category and understanding about the environment impacts of practicing LEED was seen in the χ^2 test ($p = 0.002$), perhaps again due to the close relationship between factory staff members and green project team. Nevertheless, of the factory workers 69% believed that they have a fair understanding about the environment impacts of LEED. This may due to regularly updates about the factory commitment to protection of environment through the internal communication unit which is operated by the Human Resources Department. None of the employees perceived that they have poor understanding about the LEED standards and their environmental impacts. The management takes a keen interest to update the employees, about their commitment to reduce negative environment impacts through practicing LEED guidelines.

There was also a relationship between level of education and awareness of LEED standards and their impact on environment. When the educational level of the employees increases their perceived understanding regarding LEED also increases. Among the respondents who have studied up to A/L, 52% believed that they have very good understanding about the LEED standards, compared to only 20% of employees who have studied up to only O/L. There was also a relationship between gender and awareness of LEED standards and their impact on environment. A strong rationalization for this relationship could not be found except for the fact that the majority of the factory workers, who had lower education level and contact with green project team, were females.

3.3 Perception on LEED / Green Workplace Environment

3.3.1 Indoor Environment Quality

Since people spend approximately 90% of their time indoors, quality of the indoor environment plays a critical role in people's comfort, health, and work performance. Research suggests that Indoor Environmental Quality (IEQ) improvements can increase worker productivity by as much as 16%, resulting in rapid payback for IEQ capital investments (LEED- EB Reference Guide, 2006).

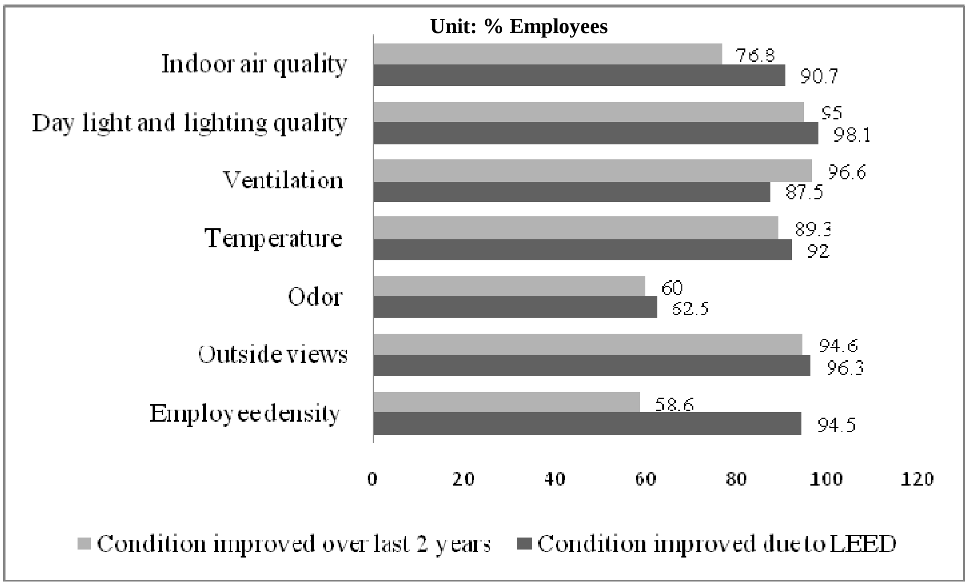

Figure 1: Perception on Indoor Environment Quality (IEQ)

Figure 1 presents the percentage of employees (both factory workers and factory staff members) who perceive that the selected parameters of indoor environment quality (IEQ) improved over the last two years and the percentage of employees who attribute this improvement to the introduction of LEED standards. Heerwagen (2000) stated that there is a direct effect of indoor air quality on performance.

As perceived by the majority of employees, indoor air quality has improved, compared to the condition in the factory before the introduction of the LEED standards (i.e. green workplace environment) in the factory. Out of them, 91% of the employees believe that improved indoor air quality was due to introduction of green workplace environment. The informal discussions with the factory employees also revealed that they are really satisfied about the air quality of the factory. However, some complained about a dustiness of the indoor environment. Dust is normally generated by the garments and there is a probability of accumulation of dust inside the factory due to less number of exhaust fans used. Management has provided masks for the employees, but those were not sufficient sometimes according to the factory workers.

A study done by Fisk (2002) found that improvements in lighting and thermal conditions may lead to additional and even larger productivity gains. Day lighting emerged as an important IEQ factor, which is naturally affected by material and colour selections, which affected employee's perception of performance and productivity (Prakash, 2005). Most of the employees (95%) perceive that there is an improvement in indoor environment due to higher day light usage and lighting quality. Of them almost all (98%) attribute this improvement to introduction of LEED standards. The natural light enables keeping of more plants inside the plants. Employees were highly satisfied not only due to increased quality and quantity of light but also due reduced stress levels as a result of more greenery views in the indoor environment. Majority of employees (95%) believe that there is more possibility to see outside gardens after the factory was converted to a green workplace. In the past employees worked in a more covered or closed environment. Now they can see outside green environment, which has helped them to recover from fatigue and tiredness of their eyes, which the employees appreciate very much. Employees suggested naturalizing the indoor environment by arranging more green plants inside the factory too.

A study conducted by Kumar, et *al*, (2002) found that inadequate ventilation systems negatively affect employee health, well-being, and productivity. Performance (speed and accuracy) of typical office tasks improves with increased ventilation rate (Lawrence Berkeley National Laboratory, 2009). Most of the employees (96.6%) perceive that ventilation of the indoor environment is better than previous level of ventilation thanks to the introduction of LEED standards (88%). The company has installed sensors in every occupied area to monitor the CO_2 concentration and to operate fresh air dampers to maintain at 900 ppm with aim of maintaining proper ventilation inside the plants. Employees also believe that there is an improvement in the thermal comfort after the introduction of green workplace environment.

Majority of employees, but lower percentages compared other IEQ parameters perceive that odor noticed in the factory and density of employees within a plant were decreased after the introduction of LEED. Most of the employees are now enjoying more space for each individual than past.

3.3.2 Outdoor Environment Quality

As mentioned in the LEED-EB manual for operations and maintenance (2008), sustainable sites, water efficiency, energy and atmosphere, and resource and material use are considered as criteria of measuring the outdoor environment quality. The percentages of employees who perceived that the selected parameters of outdoor environment quality (OEQ) improved over the last two years and the percentage of employees who attribute this improvement to the introduction of LEED standards are presented in Figure 2. When considering minimization of water wastage and practices of water recycling, approximately all the employees agreed those were improved mainly because of the LEED standards. The company has been able to save water up to 58.3% after the introduction of LEED in the factory through increasing water use efficiency, water recycling and rain water harvesting.

Similarly, majority of employees perceive that there is an improvement in energy efficient operating strategies within the factory and most of them think that this was happened due to introduction of green workplace environment in the factory. The company replaced the air conditioning system with an energy efficient system and installed skylights to reduce artificial light requirements. It has also replaced two vans operated on auto diesel to transport cut fabric pieces and daily office use by two electric vehicles.

Reduction of contributions to air pollution and to global warming is also believed improved after the introduction of green workplace environment. The factory has successfully reduced the emission of CO_2, SO_2, NO_X gases to the atmosphere by 78.6%, 71.2%, and 92.2% respectively. Employees' positive response regarding reduction of harmful chemical and toxins usage, environmentally sensitive buildings, sustainable landscape management and condition of outdoor environment of the factory were also very high. Most of them agree that this improvement is a result of implementation of green project.

When comes to the solid wastage management and recycling, greater parts of the employees believe that that factory has improved after the green project. The factory was able to reduce the waste generation by 100% either by recycling or reuse. They are not only focused on typical wastes management techniques but also on processing wastes to come up with various products. Employees suggested improving the outdoor environment by increasing greenery area, planting medicinal plants and renew the garden time to time.

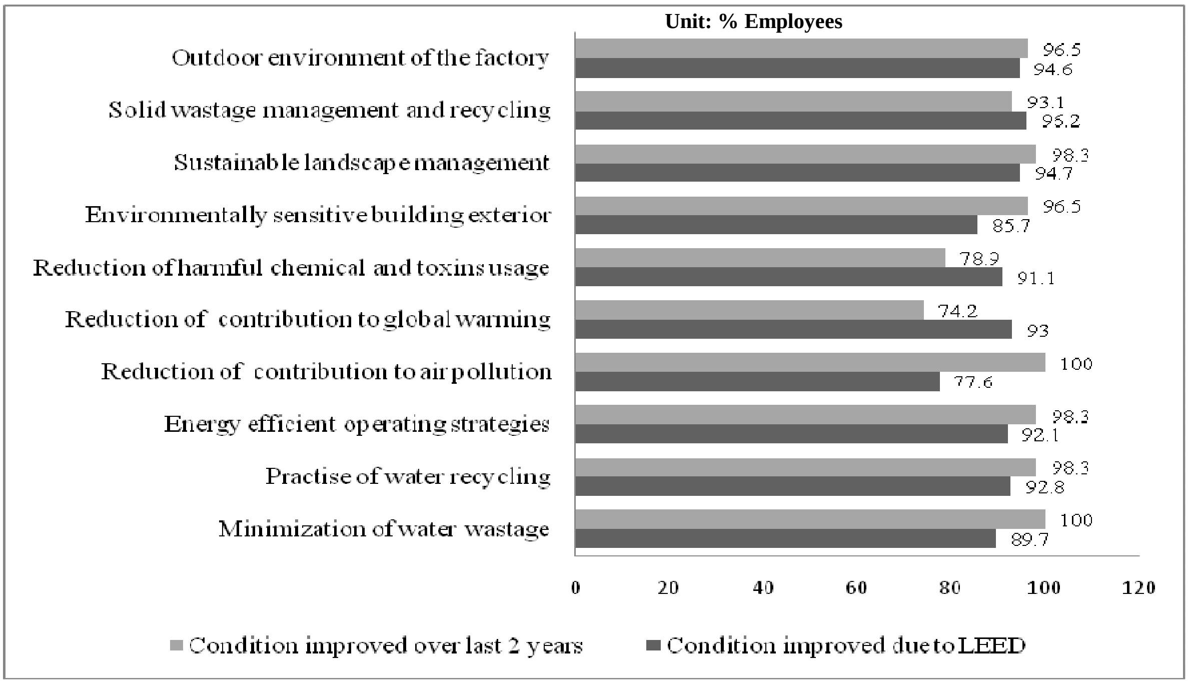

Figure 2: Perception on Outdoor Environment Quality

3.4 Impact on Work Life

A green workplace environment benefits different stakeholders of an organization. There are many financial outcomes resulted by the green workplace environment through reducing resource utilization, maintenance costs, risks and health hazards, absenteeism, and turnover while increasing the overall productivity. It improves the process innovation and increase the work process efficiency. From the shareholders point of view, it may help to improve public image, enable to sell to pro-environmental customers and attract high quality workers, and improve ability to work with community stakeholders. Not only that but it may also help employees to improve quality of work life, personal productivity, and well being (Heerwagen, 2000; Soundarapandian, 2007).

Figure 3 shows the specific impacts of green workplace environment on the performance of employees. All the respondents perceive that complaints regarding environment issues were reduced after the implementation of green workplace environment. The greater part of them (88.7%) believes the green project as the main reason for this change. Many health issues often arise due to poor indoor and outdoor environments of the workplaces. Poor IEQ has been related with sick building syndrome (SBS) symptoms, respiratory illnesses, sick leave, and loss in productivity (Seppanen and Fisk, 2006). According to the Department of Labour of Sri Lanka, Rs. 65.9 million has been spent as compensation to settle 265 industrial accidents in 2008. However, in this factory 97% of the employees perceive that there was a reduction of their health care cost after the introduction of green workplace. Majority of employees believe that the main reason for reduction of health care cost was the safe, hygienic and comfortable environment gifted by the green project.

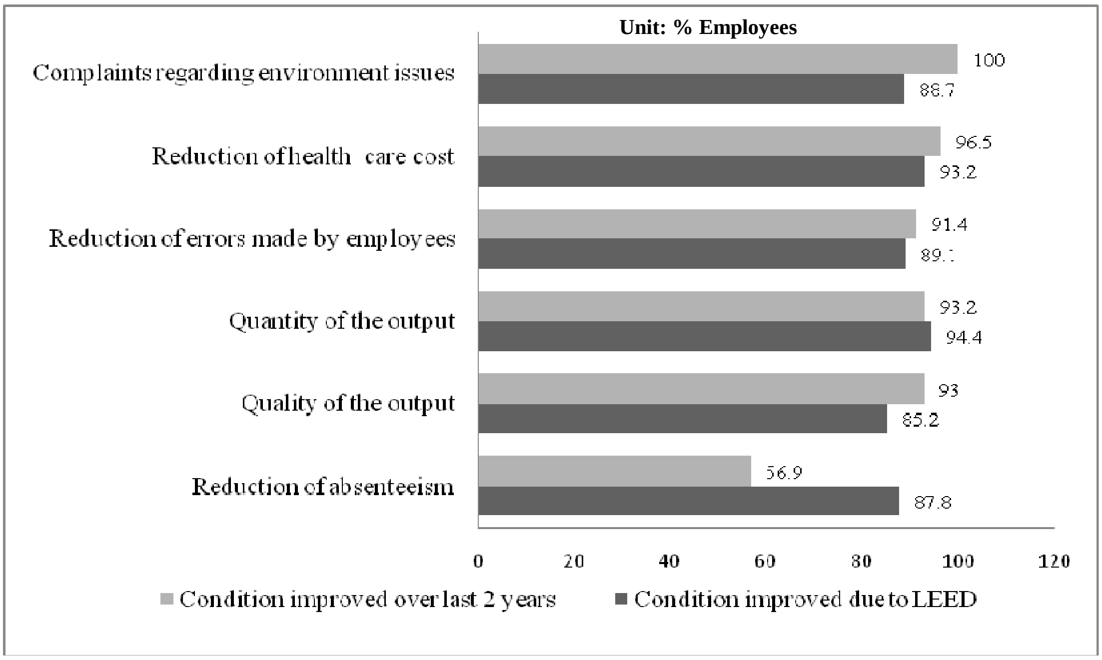

Figure 3: Performances of the Employees

Errors made by employees were reduced as perceived by the employees themselves after the implementation of the green workplace environment. Many employees stated that their mental satisfaction, clear mind and reduced stress improved their concentration on work, consequently reduced errors made by them. When considering the quality and quantity of the output, approximately 93% of employees from each employee category believe their performances were improved after the implementation of green workplace environment. More than half of the employees perceived that their absenteeism rate was also reduced than past. However, there was a significant difference between employee categories and absenteeism rates as 71% of factory staff members perceive reduction in absenteeism is due to introduction of green environment, whereas only 43% of factory workers perceive the same way.

Both factory workers and factory staff members perceive that there is an improvement in comfort of the working environment to perform better in day to day work. When considering the overall perception about performance of employees, majority of employees (86.2%) believe that their performances at workplace have improved after the introduction of green workplace environment to the factory. Furthermore, 89% of the factory staff members perceive that performance of employees under their supervision has improved significantly after LEED (green project).

When considering the job satisfaction, greater part of the respondents (around 90%) believe that their job satisfaction increased after the implementation of LEED standards. There was not a significant difference between employee categories on perception of comfort of the working environment or perceived job satisfaction. As indicated by the employees, in addition to the physical benefits, they were highly satisfied with the psychological benefits of green workplace environment. This supports the findings of Hikari Kato *et al*. (2009) of green workplace offers greater psychological benefits, mental and job satisfaction to employees in addition to physical improvements, health and productivity gains. Vicki Heath (2006) has also stated that there is a heavy influence of employee's workplace environment on their error rate, level of innovation and collaboration with other employees, absenteeism and, ultimately, how long they stay in the job.

Having a green workplace is not only healthy for the environment, but it will contribute to the general wellbeing of employees and they will feel more inspired and motivated, leading to improved productivity while creating a perception on employee to be more environmentally friendly and to be greener at home. Past studied have reveled that office workers believe they would be 21% more productive if given a better working environment. Nearly 90% of senior executives, feel that a better physical working environment would have a positive impact on their company's bottom line. Over 90% say that the quality of their working environment affects their mood and attitude about their work. Almost as many (89%) believe that the quality of their working environment is very important to their sense of job satisfaction (Kirsten, 2007, Kato *et al.,* 2009)

4 Conclusion

The green workplace environment (adoption of LEED standards) has rewarded the apparel company by improving their employee performance and job satisfaction, creating a better public image and maintaining the market, while helping to minimize the environmental damage. Both factory staff members and factory workers believe that introduction of green workplace environment created a good impact on their work lives. Subsequently, this positive attitude regarding green workplace environment, in future would helpful to attract talent laborers and reducing labor turnover, a serious problem in the apparel industry.

The results were also useful to managers and interested parties by highlighting areas of perceived deficiency in green workplaces and ensuring a more targeted effort in meeting the needs and expectations of employees. It was found that factory staff members are more aware about the LEED and the environmental impacts of practicing LEED than factory workers, perhaps due to their differences with relationship with the personnel implementing the green project and education levels. Employees perceive that there is a deficiency in attitudes regarding benefits of the green workplace environment, thus there should be an attitude change of employees and update employee knowledge. The turnover of the employees, particularly the factory workers should be addressed to make the green workplace more successful in this company.

Finally, the researchers believe that these results helped identifying strengths and weaknesses of establishing green workplace in factories in employees point of view, a relatively new concept in Sri Lanka. Further research on green workplaces in Sri Lanka in comparison to traditional workplaces would enable generalization of these findings.

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IMPROVING THE SAFETY OF BUILDINGS THROUGH AN INNOVATIVE SUSTAINABLE FAÇADE SYSTEM

Damith Mohotti, Raymond Lunmantara, Tuan Ngo, Priyan Mendis

The University of Melbourne, Victoria 3010, Australia.
E-mail: pushpaj@civenv.unimelb.edu.au Tel: (+61) 42 5531977

Abstract: A building's façade system is the outer layer of a structure that is designed to provide protection to building occupants and contents from external hazards with varying intensity. In the modern world, many structures undergo different types of dynamic loadings such as blast and ballistics, earthquakes, high winds, hurricanes, tsunamis etc. It is a prime importance of the modern structures to sustain those dynamic loadings without excessive damage. Due to the recent trend towards sustainable development, there are more prevalent uses of innovative systems such as the double skin façade systems, which lead to new challenges in assessing the performance of these façade systems under extreme loadings. This paper presents a review of innovative double layer skin façade system with some finite element modeling to assess the behaviour.

Keywords: Double skin façade (DSF) system, finite element (FE) modelling

1. Introduction

Sustainable development has become an increasing priority for building projects worldwide. However, threats of terrorist attacks around the world have also caused building owners and occupants to pay attention to building safety issues.

In recent years, terrorist attacks and natural disasters have increasingly occurred around the world. There are large number of explosions occur within or close to main cities of many countries. These cities are mainly congested with buildings with glazed façade systems. The percentage of injuries caused by the blast is mainly due to the impact of flying fragments. This amount could be as high as 80-90 percent. An example of the magnitude of damage caused by flying fragments is the attack on the Central Bank, Colombo, Sri Lanka in January, 1996. The building was surrounded by few other high-rise buildings with glazed façade systems and more than 90 percent of casualties were due to the impact of flying fragments of the broken glass panels.

Both sustainability and safety measures must be considered within the overall project context, including impacts on occupants and the environment, regardless of the level of protection deemed appropriate. This project aims to develop a secure and sustainable facade system for buildings which will have a significant enhancement over other conventional facades in terms of blast and impact protection and life cycle energy performance. New protective technologies combined with day lighting and climate control systems of building façade will be investigated in this project to: 1) improve the impact and blast resistance of the façade; 2) improve the comfort and performance of building occupants; and 3) reduce greenhouse gas emissions that contribute to global warming. The case study presented in this paper is an attempt to establish the performance characteristics of glazing façade panels in the form of pressure impulse curves. This work is part of an ongoing research project which investigates the behavior and performance of innovative sustainable double skin façades.

2. Background

2.1 Theoretical Blast Wave Parameters

The pressure-time curve of a blast wave is characterized by an abrupt pressure rise when the wave arrives at the target and the following exponential decay into a negative pressure phase. Usually, for windows or façade systems overpressure, P_{so}-P_0 is not the most important criterion [1]. The impulse $i_{s,}$ which is the area under the pressure-time curve is$_s$ of equal importance if not the governing parameter. A typical pressure-time curve of a blast wave is shown in figure 1.

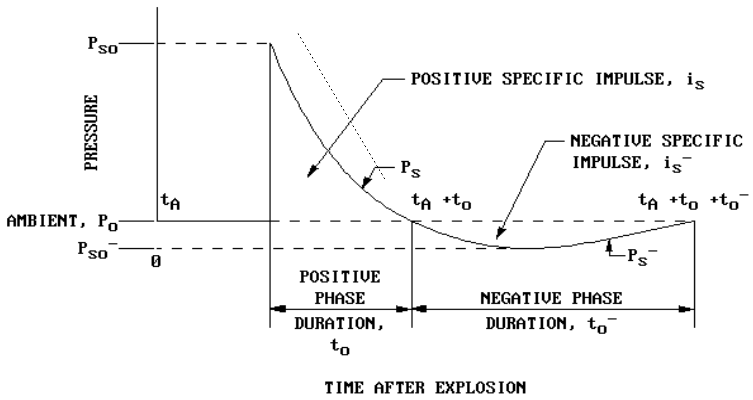
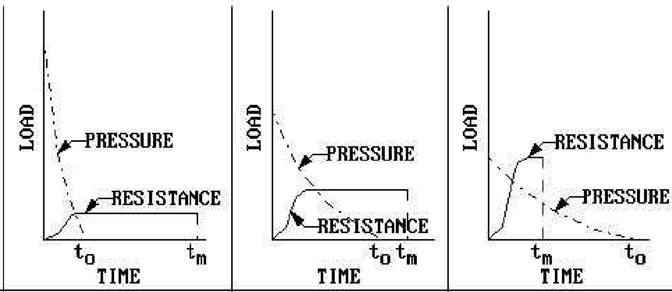


Figure 1 Typical peruse-time variation [2]

Usually, only the positive pressure phase is considered in analysis and the positive pressure phase is idealized as a triangular pressure time history. The negative phase is mostly unimportant in relation to flying debris towards to the protection area. For windows which have to prevent debris on both sides of the construction (e.g. courtyards, buildings close to highly frequented traffic areas, overhead glazing), the negative phase is also important. In addition to the longer duration than the positive phase, the interaction of the negative phase with the pre-damaged structure could be critical in establishing the component performance. In some instances where the structure has a long natural period, the negative phase may decrease the maximal structural deflection. For comparison of simulation with test results, it is necessary to establish the actual time history of the blast pressure including the negative pressure phase.[1]

2.2 Pressure-Impulse curves (iso-damaged curves)

An iso-damage (pressure impulse) curve is a characteristic curve that represents a certain damaged state of an element. A wide range of applications, such as assessment of structural damages and assessment of human survival under blast load pressures, demonstrate the curves' versatility. In general, when subjected to a varying pressure and impulse combination, the response of a structure is governed by the natural period (t_m) of the structure and blast load duration (t_0). Hence, there are three possible scenarios that could occur in the blast event as given below in figure 2.



PRESSURE DESIGN RANGE	HIGH	LOW	VERY LOW
DESIGN LOAD	IMPULSE	PRESSURE-TIME	PRESSURE
INCIDENT PRESSURE	>> 100 psi	< 100 psi	< 10 psi
PRESSURE DURATION	SHORT	INTERMEDIATE	LONG
RESPONSE TIME	LONG	INTERMEDIATE	SHORT
t_m / t_o	$t_m / t_o > 3$	$3 > t_m / t_o > 0.1$	$t_m / t_o < 0.1$

Figure 2 Parameters defining pressure design ranges[2]

Adopting the SDOF approach, the damage or no damage state can be determined based on the maximum displacement criteria. Thus, knowing the maximum allowable displacement, the impulsive and quasi-static asymptote on the pressure impulse curves can be quickly established by applying simple energy conservation principles. Readers can refer to Smith and Hetherington,1994[3] for more details of the development of pressure-impulsive curves. The generic non-dimensional pressure impulse curve is as shown in figure 3.

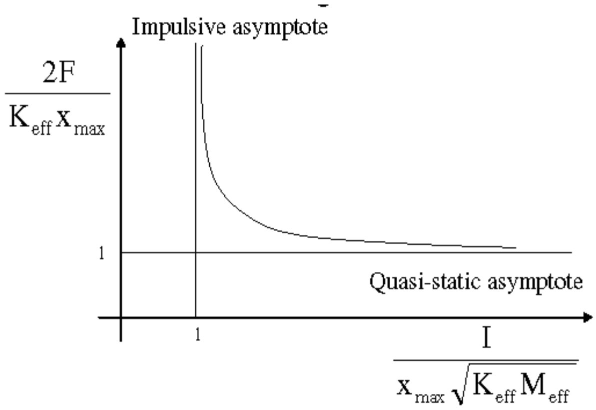


Figure 3 Generic non dimensional pressure-impulse curves.

For each building or construction, it is necessary to specify a permitted hazard level, depending on what the acceptable damage level after an explosion. Standard ISO/DIS 16933 contains hazards rating system from level A- "no hazards" up to level F-"high hazard" as summarized in table 1. The extent of the numerical evaluations shall be based on rating level B to C. For typical windows at level B to C, the following failure modes are common [1]:

- Fracture of glazing
- Crack of PVB interlayer
- Separation of splinters from the rear side of the window
- Pullout from the edges of the frame or failure of structural sealant glazing
- Failure of fittings
- Composite failure in thermally insulated profiles, crack or fracture of fiber-reinforced plastic connections
- Collapse of profile connections
- Local crack or buckling of aluminum profiles due to high plastic strains
- Anchorage failure

Table 1 hazard levels- ISO/DIS 16933

Hazard rating	Hazard rating description	Definition
A	No Break	The glazing is observed not to fracture and there is no visible damage to the glazing system. Calculations via equivalent static loads by diagrams and tables may be sufficient in simple cases.
B	No Hazards	The glazing is observed fracture but is fully retained in the facility test frame or glazing system frame with no breach and no material is lost from the interior surface. Numerical evaluations using nonlinear material laws and plastic deformation capability is possible. Equivalent static systems are not suitable.
C	Minimal hazards	The glazing system is observed to fracture and the total length of tears in the glazing plus the total length of pullout from the edge of the frame is less than 20 percent of the glazing sight perimeter. Also there are no more than 3 perforations or indents anywhere in the vertical witness panel and any fragments on the floor between 1m and 3m from the interior face of the specimen have a sum of total united dimension of 250mm or less. Numerical evaluations using nonlinear material laws and plastic deformation capability is possible. Equivalent static systems are not suitable.
D	Very low	The glazing is observed to fracture is located 1m behind the original location. There

	Hazards	are no more than 3 perforations or indents anywhere in the vertical witness panel and fragments on the floor between 1m and 3m from the interior face of the specimen have a sum of total united dimension of 250mm or less. Exact modeling of failure criteria's of all parts and connections with details are necessary. Extreme fine mesh is required
E	Low hazards	The glazing is observed to be fracture but glazing fragments falls beyond 1m and up to 3m behind the interior face of the specimen and not more than 0.5m above the floor at the vertical witness panel. Also there are 10 or fewer perforations I the area of the vertical witness panel and higher than 0.5m above the floor and none of the perforations penetrate more than 12mmthrough the thickness of the foil backed insulation board layer of the witness panel. Exact modeling of failure criteria's of all parts and connections with details are necessary. Extreme fine mesh is required
F	High Hazards	Glazing is observed to fracture and there are more than 10 perforations in the area of the vertical witness panel and higher than 0.5m above the floor or there are one or more perforations in the same witness panel area with fragments penetration more than 12mm through the thickness of the foil backed insulation board layer of the witness panel. Exact modeling of failure criteria's of all parts and connections with details are necessary. Extreme fine mesh is required.

3. Proposed Double Skin Façade System (DSF)

The DSF system proposed in this project is shown in figure 4. The system consists of the following components:

1) External facade: is a single-sheet laminated glass
2) Shading system: is a venetian blind system, which is normally used for sun-light control. In the project, it is proposed that the shading system will have a dual function for improving both sustainability and safety. Firstly, it will be coated with amorphous silicon photovoltaic material to become a renewable energy source. Secondly, the venetian blinds will also be designed as a cable catcher for catching glass fragments from the external facade.
3) Internal façade: normal glazed windows.
4) Ventilation system: the ventilation system will regulate the air movement in the cavity using solar energy generated from the PV blind system.
5) Climate sensing and control system: Automatic control of the ventilation and opening of the shading system will be done based on the sensor system which can monitor temperature and solar radiation as well as track sun position.

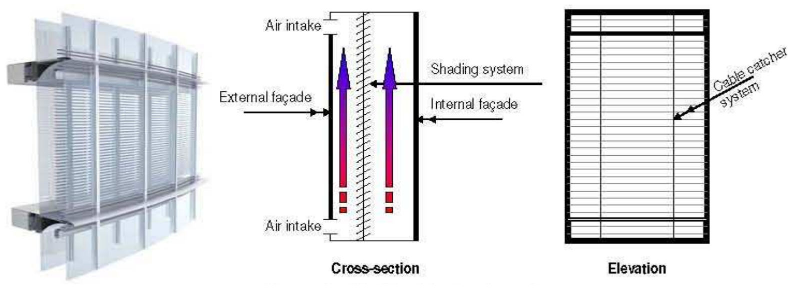


Figure 4 Double skin facade systems

3.1 Energy performance and life cycle energy of DSF systems

The aesthetic desire for fully glazed building envelopes poses serious challenges to building designers. Considerable research has been conducted into the thermal behaviour of double skin facades in the past decade. A search of just one leading international journal on building performance shows more than 20 papers investigating this topic in the last decade. The overwhelming emphasis of this research has been directed to reducing energy consumption. The variations in double skin façade designs (box, shaft, corridor and multi-storey) [4] means that comprehensive and reliable design guidelines and software which can be used by building designers to evaluate options are still not available. The embodied energy implications of a DSF also need to be balanced against any cooling, heating or lighting energy savings that it provides. Further research is required to provide these tools and life cycle energy analysis of this complex facade system.

3.2 Energy performance and life cycle energy of DSF systems

The main challenges in blast protection of DSF systems are: 1) how to dissipate as much energy of the blast wave as possible after the failure of the external facade; 2) how to stop the flying fragmentation from the breakage of the external facade. The research team at the University of Melbourne has been involved in full scale blast trials in Woomera from 2002-2007 [5]. It was observed in those trials that the ultimate failure mechanism of glass is not well understood particularly at the edges. Recent testing (Woomera) has shown several mechanisms including:

1. PVB tearing at the glass-frame edge
2. PVB pull-out between the laminated glasses leafs at the glass-frame edge.
3. Through thickness cracking of the inner leaf of the glass at the glass edge.

As these mechanisms are still unpredictable, where glass is designed to its limits, there is a high degree of risk that the glass at its ultimate failure displacement may not perform as designed. Under blast pressures, it is likely that the panels would be dislodged as a whole and propelled into the structure as shown in figure 2. Laminated glass is commonly used as the external skin of the façade system, which underlines the importance of establishing the projectile borne out of the external skin of the DSF system.

4. Analysis Procedure & FE Modeling

4.1 Analysis Procedure

The FE modelling approach is used to develop the P-I curves of both the internal and external layers of the façade system. Once the P-I curves of both external and internal layers are obtained, the P-I curves of both the external layer and the curtain wall could be used as a failure criteria in computational fluid dynamic (CFD) modelling. The detailed simultaneous analysis process is presented in [6].

In this exercise, two models were built and analysed with the LS-DYNA FE code. The DSF system consists of one layer of an internal skin and one layer of an external skin. For the purpose of this preliminary study, shading system was excluded and the external layer of the glazing system is limited to non-laminated glass panel. The internal skin of the case study structure consists of one single framed glazing unit that is embedded into the ceiling and floor of a retail atrium. A glass panel with typical dimensions of 3 m tall, 1.2 m wide and 8 mm thick was selected. The frame units are typically bolted to the support structure at intervals of approximately 900 mm. Thus, the translational degrees of freedom of the models were constrained at the bolt locations, as illustrated in Figure 8(a). Meanwhile, the external layer covering the vision panel of the system is typically a 2.4 m tall, 1.2 m wide and 10 mm thick glass panel. The external skin façade system is typically supported at four locations with bolt-like devices, which allow rotation but restrict the translational movement of the glass panel. A simplified schematic of the external skin of the DSF system is shown in Figure 8(b).

Shell and eight-node solid elements were used to model the glass panels and framing system, respectively. The models were built as quarter models with two axes of symmetry. The framing system of a window glass unit normally involves a complex interaction between the head-subhead, sill-subsill and the actual panel itself. However, the internal skin of the DSF system model simplifies this interaction into three elements: the aluminium frame, the sealant material, and the glass panel itself. The translational degrees of freedom of the frame were restrained only at the likely bolt locations. The simplifications were made based on preliminary parametric studies and comparison between the responses of the typical glazing unit model and the simplified glazing unit model. The simplifications incorporated were necessary due to the high computational demand of the typical full-blown model [6].

(a) (b)

Figure 5 FE model of external skin of the DSFS

4.2 Material Constitutive Models

In the analytical model, three different material types need to be defined. The aluminium frame in the glazing unit exhibits linear elastic with ductile post yield behaviour. An isotropic elastic plastic material model, which is capable of modelling material plasticity, is used. An elastic material model was used for the structural sealant model. The cost-effective elastic material model is used to simulate the glass panel behaviour in this exercise.

4.3 Blast Load Application

In the FE analysis phase, the in-built CONWEP function in LS-DYNA was used to randomise the blast pressure-impulse. The blast pressures, computed using the CONWEP function, were applied as shell surface pressures. The empirical modelling approach, CONWEP[7], can provide a blast pressure estimate with a reasonable degree of accuracy[5]. However, it must be noted that CONWEP could not simulate the negative phase of the blast pressures. It was acknowledged that the negative phase of the blast pressures might have an influence on the glass panel response. Thus, the effect of the negative phase on the glass panel response is subject to further research in the project.

4.4 Pressure-Impulse Curves

The characteristic P-I curves of the internal skin and the external skin of the façade system are shown in Figure 9.

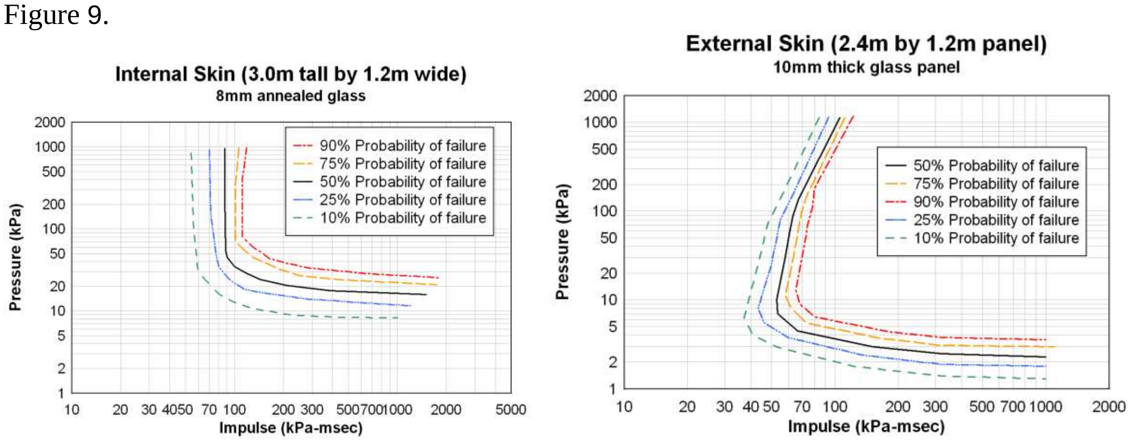


Figure 6 P-I curves for internal and external skin façade in DSFS

4.5 Computational Fluid Dynamics Phase

In this phase, the CFD code, Air3D [8] which is capable of modelling the blast wave-structure interaction to a significant degree of accuracy[9], was utilised to derive the overall performance of the façade system. In this analysis, the failure criterion of the overall DSF system was defined as the breach of the internal skin of the façade system.

After obtaining the P-I curves representing the failure criteria of the internal skin and the external skin of the façade system, the failure criteria of the external skin and the internal skin were used in the CFD analysis to establish the combined fragility of the DSF system. The façade layers are modelled as frangible panels with the P-I curve as a failure criterion in the CFD model. The CFD model was built to simulate a condition similar to a blast trial environment, whereby both layers of the façade system are embedded into two rectangular test modules.
The CFD analysis approach is capable of tracking the blast pressure and blast impulse applied onto the façade layer. Hence, the response of DSF system can be defined in three stages. These are:

- Stage 1 – Blast pressure arrival at the external façade surface.
- Stage 2 – External façade response. In this stage, the failure or non-failure of the external façade layer is determined by comparing the P-I values imparted on the external façade system against the P-I curve as a failure criterion. If failure occurs, the blast wave will propagate into the structure, leading to a Stage 3 response.
- Stage 3 – Internal façade response. In this stage, the failure or non-failure of the internal facade layer is determined by comparing the P-I values imparted on the internal façade against the internal façade's P-I curve as a failure criterion.

One fragility curve only exhibits the vulnerability of the façade system to one particular charge weight. Thus, several sets of analysis need to be carried out to assess the vulnerability of the same façade system subjected to different threat charges. In a set of analysis, the blast charge weight is kept constant throughout, whilst the stand-off distance is varied. For example, if a charge weight of 25 kg TNT with a 24 m stand-off distance is required to induce a 50% failure probability on the curtain wall layer, a point with an abscissa of 24 m and an ordinate of 50% can be recorded on the fragility chart. The 25 kg charge weight fragility curve is developed by repeating the analysis process to obtain the stand-off distances required to induce a 10%, 25%, 50%, 75% and 90% probability of failure criteria.

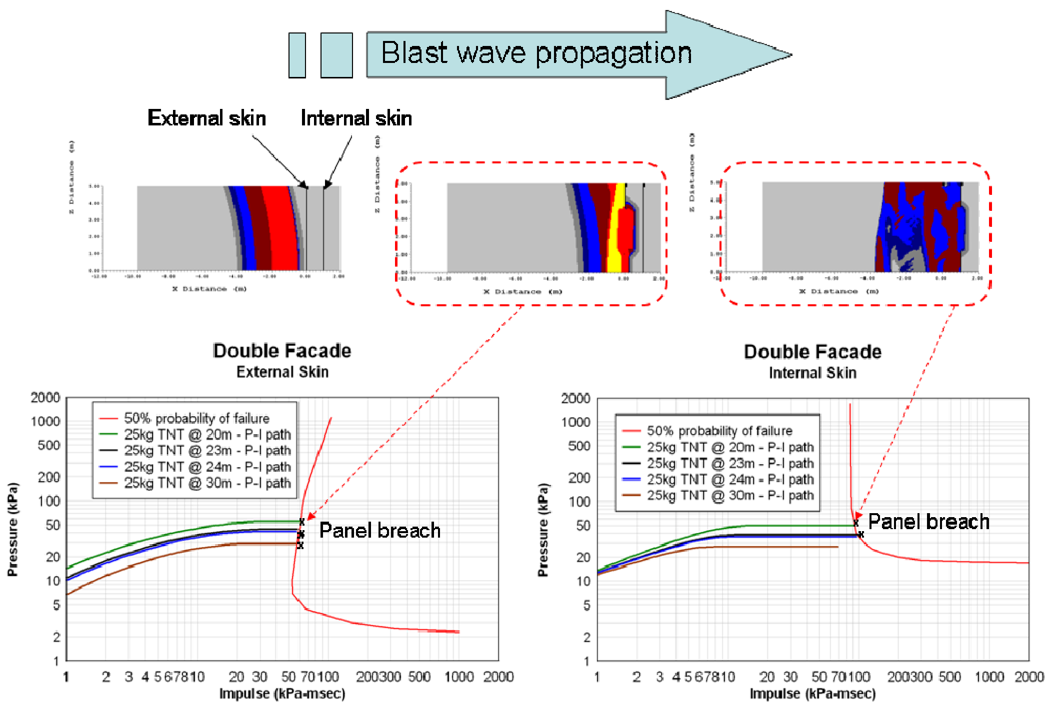
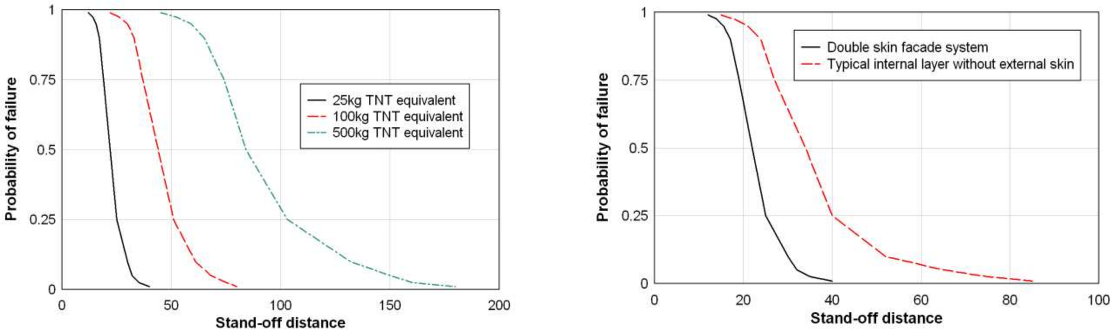


Figure 7 Double skin façade response

5. Results and Discussion

The fragility of the DSF system is shown in figure 11(a). In addition to providing an early indication of the performance of the DSF system, the analysis results indicate a marked improvement of façade system blast performance when a sacrificial external layer is used in the system (i.e. the DSF system). The blast performance improvement is illustrated in figure 11(b).



(a) (b)

Figure 8 (a) Fragility curves for DSF system, (b) Comparison between façade systems with external layer and without external layer for 25 kg TNT equivalent charge

6. Concluding Remarks

The Doble Skin Faceade system is envisaged to be a very popular system in the future due to the trend in the construction industry towards sustainable design and construction. This paper presents an attempt to quantify the implications of future adoption of this DSF system on the blast performance of the overall façade system. Performance indicators for preliminary DSF system, in the form of P-I and fragility curves, were developed in this exercise. A particular failure criterion, tensile fracture of the glass panel, was adopted in this analysis. However, the framework developed in this exercise can be used in conjunction with different failure criteria. The fragility curves developed for the DSF system indicate that the sacrificial external skin would contribute towards increasing the overall façade performance. The findings from this exercise also indicate that the performance of the overall system can be improved by adopting an external skin layer that is capable of dissipating a significant proportion of the blast energy. This preliminary analysis is based on the assumption that the internal skin failure is governed by the blast pressure propagation. It must be noted that two components, namely, the PVB laminates and the cable catcher system were left out in the analysis. Further studies to establish the effect of the PVB laminates and the shading system in the DSF system is currently under way.

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HIGH STRENGTH CONCRETE FOR SUSTAINABLE CONSTRUCTION

Wu, D.[1], Sofi, M.[1] Mendis, P.[1]

[1]Department of Civil and Environmental Engineering, University of Melbourne, Victoria, Australia.
E-mail: d.wu11@ugrad.unimelb.edu.au
Telephone: +61 3 83447244; Fax: +61 3 83444616

Abstract: The use of HSC for construction, especially for multi-story buildings, has become very common in industrialized and developing countries. In view of gaining popularity as a construction material, high strength concrete (HSC) properties are discussed in this paper. A brief review of literature is presented. The necessary ingredients of HSC such as fly ash, slag and silica fume which are mostly industrial byproducts make the product environmentally friendly. The main engineering properties of the HSC are also reviewed.

Keywords: High Strength Concrete, Engineering Properties, Environment

1. Introduction

During the past few years, high-strength concrete (HSC) has been generating increased interest amongst civil and structural engineers. The expanding commercial use of this relatively new construction material can be explained partially by the life cycle cost-performance ratio it offers, as well as its outstanding engineering properties, such as higher compressive and tensile strengths, higher stiffness and better durability, when compared to the conventional normal strength concrete (NSC). From a historical point of view, in the middle of the 20th century concrete with characteristic strength (f'_c) of 25MPa was considered high-strength. In the 1980s, 50MPa concrete was considered high-strength. About two decades ago, HSC was mostly specified for projects as an alternate design. But today, HSC is being specified in the preliminary design stage as a sensible solution for concrete construction. Nowadays, technology for producing HSC has sufficiently advanced such that concretes with compressive strengths of up to about 120MPa are commercially available, and strengths much higher than that can be produced in the laboratories. The significant economic advantages of HSC are very well-documented, and evident from the number of recent construction projects where HSC has been used successfully [1].

The use of HSC for construction, especially for multi-story buildings, has become very common in industrialized and developing countries. In Australia, where the majority of buildings are concrete structures, almost all concrete high-rise and medium-rise building projects utilize HSC. Australia has taken the advantage of the benefits of high-strength concrete [2, 3] through its widespread use on buildings such as 120 Collins Street, Melbourne Central, the Rialto project in Melbourne, the 43-storey high Casselden Place project in Melbourne. In Seattle USA, the strength of concrete used on the Pacific First Centre was about 125MPa [4]. The Freedom Tower in New York City, which will be one of the world's tallest superstructures, is projected to be completed in 2010. The structure consists of a robust high-strength concrete core paired with a highly redundant perimeter steel moment resisting frame. Most experience on HSC in Europe has been gathered in Norway, with that country's development in offshore platforms, bridges, and highway pavements [5]. In Germany, HSC was first utilized in a high-rise building in Frankfurt, completed in 1992. HSC with a mean strength of 100 MPa was used in the Petronas Towers in Kuala Lumpur in 1998. The Eureka Tower, which is one of the tallest buildings in Australia was completed in 2006 has utilized HSC up to 100 MPa.

In general, concrete is not considered as a sustainable construction material in terms of large consumption of raw materials, contribution to greenhouse gas emissions from cement and low durability. The ruling argument is that the production of 1kg of cement which, is the main ingredient of concrete, generates 0.8 - 0.9kg of CO_2 emission [6]. It is commonly argued that with a high growth rate, the demand for concrete consumption will substantially increase in the near future imposing a heavy burden on the ecological system. The CO_2 emission related to concrete production, inclusive of cement production is between 0.1 and 0.2tonne per 1tonne of produced concrete [7]. The environmental impact of concrete as a construction product is questionable with studies demonstrating concrete products requiring much less energy with a lower net environmental impact when compared to other construction materials such as steel [8]. Regardless of the impact to the environment, it is a true fact that currently urbanization worldwide relies heavily on the concrete industry. Worldwide, some 6 billion tons of concrete is produced per year, making concrete one of the world's most popular construction materials. On the economical front, this represents about 13 to 14 trillion USD world trade dealing [9].

In view of the importance of climate change, sustainability has become the main concern for the concrete industry. In order to assess the environmental impact of concrete, a multi-dimensional, life-cylcle approach is adopted [9, 10]. The findings show that, when considering the life-cycle stages of the product into consideration in order to carry a comprehensive and impartial assessment, concrete can be considered as a material that burdens the environment least [11]. It is further reported that using high performance concrete has multiple environmental benefits. For instance, it is possible to build a durable structure with minimum maintenance that lead to a reduction in the consumption of raw materials and greenhouse gas emissions, and reclaiming industrial waste products and using them as effective construction materials [11].

In spite of this, steps have been taken in place to reduce the CO_2 emissions into the atmosphere. In order to limit the usage of Portland cement, the concrete industry has been increasingly inclined towards substituting Portland cement with fly ash, slag and micro silica fume all which are industrial byproducts. It is believed that this substitution can be increased without impairing the performance of the concrete grades [10].

This paper presents literature review of current studies undertaken on HSC. The review is divided into two parts. In the first part, the main constituents of HSC are presented. The main ingredients which influence the performance of HSC are discussed (Section 2). The addition of mineral admixtures which are mostly industrial by-products is common in the production of HSC. The effects of these admixtures on the concrete properties are also discussed. On the second part, the engineering properties of HSC concrete are presented (Section 3). For the purposes of this paper, HSC is defined as concrete with compressive strength, f'_c, in the range of 50 - 100 MPa. NSC is concrete with $f'_c < 50$ MPa.

2. High strength concrete constituents

The sustainable high performance concrete does not contain any special or unusual ingredients. A common mix includes Portland cement, super plasticizers, silica fume, fly ash and slag, with relatively large amount of cementitious by-products for cement replacement. The significance of each material in producing high strength concrete is discussed in this section.

2.1. Water/binder (w/b) ratio and cement content

HSC usually contains one or two mineral additives which are used as partial replacement for cement. Therefore, the term water/cement (w/c) ratio used in reference to normal strength concrete (NSC) is replaced by w/b ratio, where the binder is the total weight of the cementitious materials (cement + additives). The minimum w/b ratio for full hydration of cement pastes is approximately 0.36 [12]. For NSC this limit is usually exceeded for workability requirements. However, in the case of HSC, complete hydration is not essential for full strength to be attained and therefore it can be made with w/b ratios less than 0.36. HSC's have been made with w/b ratios as low as 0.2. However, high dosages of superplasticizers are required to maintain workability [13]. Patnaikuni and Patnaik [12] suggests that a w/b ratio of 0.23 is an optimum value for maximum compressive strength of very high-strength concrete mixes.

The incorporation of mineral admixtures such as silica fume, fly ash, slag or rice-husk ash is common in production of HSC concrete. These cementations by-products facilitate the manufacture of high-strength concrete.

2.2. Mineral admixtures

Silica fume

Silica fume is a by-product of the manufacturing process of silicon and ferrosilicon alloys and is in a form of glass which is highly reactive. The small size of particles will accelerate the reactions with calcium hydroxide which enables silica fume to replace Portland cement for a small proportion. The major purpose of introducing silica fume to the concrete mix is to achieve high strength and durability. The presence of silica fume also enhances the effectiveness of superplasticizer, which consequently reduces w/b ratio required to achieve a certain level of workability [14]. Normally, 3 to 10% of silica fume is used for high performance concrete. Behnood & Ziaria [15] found that the silica fume has more pronounced effects on compressive strength than a decrease in w/b ratio. The optimal value for silica fume and w/b were estimated to be 6% and 0.35 respectively. In an experimental study, Ting et al. [16] concluded that about 10% replacement of cement by silica fume is the optimum dosage.

Fly ash

Fly ash is a by-product of the combustion of pulverised coal in thermal power plants. It is removed as a fine dust by mechanical extractors, electrostatic precipitators or fabric filters. Fly ash can be included into concrete either blended with cement or directly introduced as an additional cementitious material at the concrete mixing plant. Typical applications are in pumped or in superplasticised concretes, particularly where heat of hydration is considered to be a problem.
The introduction of fly ash has effects on many properties such as workability, hydration, strength development shrinkage, heat evolution and durability. The inclusion of fly ash in the concrete mix reduces the water content required to produce a certain level of workability. Experimental studies by Jiang and Malhotra [17] have found the reduction of water content can be as high as 20%, if high quality of fly ash were used.

Fly ash generally has adverse effects on concrete strength, especially at the early age. However, fly ash may have better performance when the w/b ratio is low. It has been demonstrated that at w/b=0.5, a 45% fly ash resulted in about 30% reduction in 28-days strength, but at w/b ratio=0.3, the reduction in strength is reduced to 17% [18]. So far, it is possible to add 50% or more fly ash to replace Portland Cement to achieve sustainable high strength concrete with less than 130 kg/m^3 water content and 200kg/m^3 cement content.

Slag

Slag is a by-product material obtained from pig iron in the blast furnance and is formed by the combination of earthy constituents of iron ore with limestone flux. The presence of slag develops the workability and strength of concrete. Generally, the dosage rate of slag is between 15% and 30% of the cementitious material [19]. In fresh concrete, slag tends to improve the workability of the concrete due to their angular shape and smooth surface texture. Consequently, the required amount of superplasticizer could also be reduced by increasing slag content [20]. It has been shown that the compressive strength of concrete increases with increasing amount of copper slag when superplasticizer is not applied [21].

2.3. Coarse Aggregates

As compared to NSC, the mineralogy and the crushing strength of coarse aggregates has a significant effect on the strength of a HSC mix. According to Setunge [22], higher strength aggregates do not necessarily produce higher-strength concretes. A more desirable property is the compatibility of the stiffness of the aggregates and the mortar. The ideal material will be crushed rock with low stiffness and high strength. It is generally observed that smaller aggregates are desirable to produce high strength concrete due to reduction in the water accumulating near the coarse aggregates and larger available surface area for bonding with cement matrix. For commercial applications, taking into

account the economy of production, workability and shrinkage and creep, well graded aggregates of 14-20mm size are recommended.
Aitcin [23], for the purposes of discussion divided high-strength concrete and very high-strength concrete into five categories and discussed the relative importance of various factors on the strength of concrete. The first three categories are summarized in Table 1.

Table 1: Categories of HSC [23]

Category	Strength	
Category I	50–75MPa	Manufactured using good quality, but generally used materials, existing production technology and w/b ratio of about 0.4. Mineral admixtures are not required. Superplasticizers may be used to achieve the required workability,
Category II	75–100MPa	High quality, generally used materials are required. Due to the very low w/b ratio of about 0.25 - 0.30, superplasticizers are required to achieve adequate workability. The use of mineral admixtures is also strongly recommended. The coarse aggregates must be round or cubic in shape.
Category III	100-125MPa	Very high quality materials, efficient mixing techniques and stringent quality control needed. The w/b ratio must be lowered to 0.22-0.25. High dosages of superplasticizers are essential in conjunction with silica fume.

The other two categories refer to 125 MPa and beyond and will not be discussed here as they are not commonly used and difficult to achieve in the field.

2.4. Superplaticizers

Superplasticizers are essential to produce good workable high-strength concrete. There are, basically three principal types of superplasticizers:
(i) lignosulfonate-based (ii) melamine sulfonate and (iii) naphthalene sulfonate.
In general, a combination of the above types is used for high-strength concrete. The amount of superplasticizer to be added to a mix is governed by the required workability.

2.5. Curing

De Larrad [25] reports that self-desiccation is probable in HSC and hence specimens cured in water will absorb water, thus increasing the strength of the concrete. An opposite view is expressed by some who argue that water evaporation from a NSC cylinder is greater than that from a HSC cylinder. Therefore, the strength development of a NSC cylinder will be more affected by deficient curing than the strength development of a HSC cylinder.

Studies by Aitcin [26] on curing of HSC show that HSC members have a delayed response to strength gain. Aitcin suggests that due to the low permeability of high-strength concrete it takes considerable time for water to penetrate the concrete and contribute to the hydration process, hence longer periods of moist curing of HSC specimens is recommended.

3. Engineering properties of HSC

The aim of this section is to provide an overview of the structural engineering properties and characteristics of HSC, in light of the recent experimental and theoretical research and published results. There are other non-structural benefits of using HSC, for example the improved durability of the material which is a result of reduced porosity and the use of high-quality materials. However, the topic of durability is not discussed here in detail.

3.1. Compressive strength

Enhanced compressive strength is the most important of HSC's functional properties. Admixtures such as silica fume or fly ash are not essential to the manufacture of high-strength concrete with compressive strengths closer to 50 MPa. However, the incorporation of these mineral admixtures, particularly silica fume does facilitate the process and silica fume is also essential to produce very high-strength concrete. The main reason for the spectacular increase in concrete strength in silica fume concrete is the creation of a dense concrete matrix enabled by the uniformly distributed fine silica fume particles in between larger cement particles. The use of superplasticizers and good compaction by vibration aids in the densification process lead to the higher strength. According to de Larrad and Malier [26], the microstructure of high-strength concrete is very dense and amorphous and contains very little free water. It has a very low-porosity and lacks the accumulation of lime crystals, as in the case of NSC.

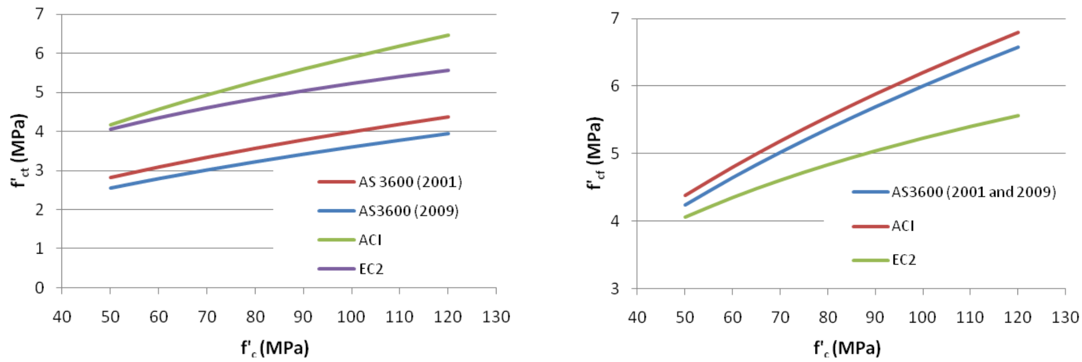
3.2. Characteristic Principal Tensile strength

The tensile strength of HSC is significantly greater than that of NSC, though to a lesser extent than the compressive strength. Fracture surfaces are smooth, indicating the homogeneity of the material. The densification of the concrete matrix and the aggregate-matrix transition zone explains the improvement of the tensile strength.

The characteristic flexural tensile strength, f'_{cf}, and the characteristic principal tensile strength, f'_{ct} in accordance to various standard procedures are summarized in Table 2.The recommended values are plotted against f'_c in Figure 1.

Table 2: The characteristic flexural tensile strength (f'_{cf}) and the characteristic principal tensile strength (f'_{ct}) according to various standards

Standard	**Recommendations for f'_{cf} and f'_{ct}**
AS3600-2001 [27]	$f'_{cf} = 0.6\sqrt{f'_c}$ (1) $f'_{ct} = 0.4\sqrt{f'_c}$ (2)
AS3600-2009 [28]	$f'_{cf} = 0.6\sqrt{f'_c}$ (3) $f'_{ct} = 0.36\sqrt{f'_c}$ (4)
ACI 318-2005 [29] ACI 363R [30]	$f'_{cf} = 0.62\sqrt{f'_c}$ (5) $f'_{ct} = 0.59\sqrt{f'_c}$ (6)
Eurocode EC2-2004 [31]	$f'_{ct} = 2.12\ln\left[1+\frac{f'_c+8}{10}\right]$ (7) f'_{cf} = max [(1.6-h/1000)f'_{ct}; f'_{ct}] (8) Where h is the depth of the cross section



(a) f'_{cf} (b) f'_{ct}

Fig.1: The characteristic flexural tensile strength (f'_{cf}) and the characteristic principal tensile strength (f'_{ct}) according to various standards

3.3. Modulus of Elasticity

The modulus of elasticity (E_c) of HSC is dependent on parameters such as the volume of aggregates, the modulus of the paste and the modulus of the aggregates. The recommendations for E_c values according to various standard procedures and researchers are summarized in Table 3 and plotted in Figure 2.

Setunge [32] has shown that the existing AS3600-2001 formula (Eq (9)) has the tendency to overestimate the elastic modulus of HSC. The new AS3600-2009 recommends Eq. (9) to estimate the modulus of elasticity of concrete up to 40 MPa. However, for concrete strength greater than 40 MPa, the new code recommends Eq. (10) to predict the elastic modulus of concrete.

Table 3: The modulus of elasticity according to various standards and researchers

Standard	**Recommendations for E_c***
AS3600-2001 [27]	$E_c = 0.043\rho^{1.5}\sqrt{f'_c} \pm 20\%$ (9)
AS3600-2009 [28]	$E_c = \rho^{1.5}(0.024\sqrt{f'_c} + 0.12) \pm 20\%$ (10)
ACI 318-2005 [29]	$E_c = \left(3.32\sqrt{f'_c} + 6895\right)\left(\frac{\rho}{2320}\right)^{1.5}$ (11)
Eurocode EC2-2004 [31]	$E_c = 22 \times 10^3 \left(\frac{f'_c}{10}\right)^{0.3}$ (12)
Mendis et al. [33]	$E_c = 0.43\eta\rho^{1.5}\sqrt{f'_c} \pm 20\%$ where, $\eta = 1.1\text{-}0.002f'_c \leq 1.0$, (13)
Carrasquillo et al. [32]	$E_c = \left(3320\sqrt{f'_c} + 6900\right)\left(\frac{\rho_c}{2320}\right)^{1.5}$ (14)

* The nominal density of normal weight HSC ρ = 2400 kg/m^3

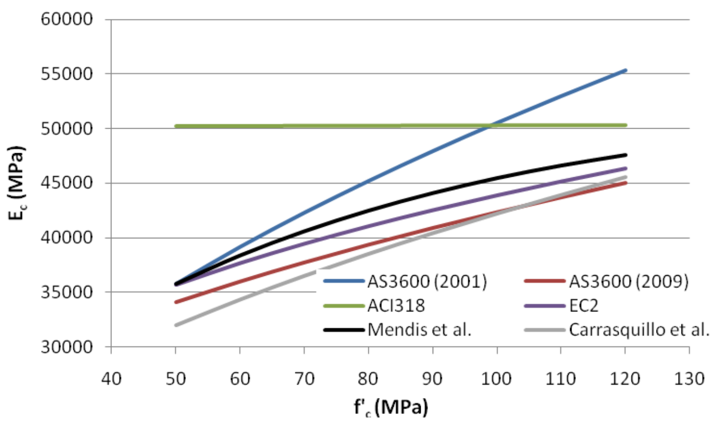


Fig. 2: Modulus of elasticity (E_c) values according to various standards and researcher

3.4. Required cover for durability

It is noted in the commentary of AS3600 that carbonation and ionisation (an increase in the reactive ion concentration such as chloride) are two factors that will influence durability in terms of corrosion of steel. HSC consists of a more uniform microstructure and lower porosity compared to NSC. This indicates a higher resistance to penetration of CO_2 and Cl ions into the concrete, thus reducing the corrosion of steel reinforcement. However, during production of HSC, macrocracking due to plastic shrinkage, microcracking due to self-dessication and thermal cracking may represent potential problems from a corrosion protection point of view. HSC's are also being specified for a range of critical civil engineering structures where the durability properties of the concrete are of paramount consideration and where design life requirements of over 100 years are needed. It is therefore prudent to also adopt the cover values specified in AS3600 for 50 MPa concrete for concrete strengths higher than 50 MPa. However the designers may reduce the cover values according to Table 4. It must be noted that for f'_c< 70 MPa, the same values as given in AS3600 [28] are recommended.

Table 4: Required Cover Values

Exposure Classification	Standard Formwork and Compaction are used (Table 4.10.3.2 of AS3600[28])		Rigid Formwork and Intense Compaction are used (Table 4.10.3.4 of AS3600[28])		Spun or Rolled Members (Table 4.10.3.5 of AS3600[28])	
	$f'_c <$ 70 MPa	$f'_c \geq$ 70 MPa	$f'_c <$ 70 MPa	$f'_c \geq$ 70 MPa	$f'_c <$ 70 MPa	$f'_c \geq$ 70 MPa
A1	20	20	15	15	10	10
A2	20	20	15	15	10	10
B1	25	20	20	15	15	10
B2	35	30	25	20	20	15
C	50	45	40	35	25	20

3.5. Fire resistance of HSC

Fire resistance of concrete members is normally accomplished by structural adequacy and insulation for a specified fire resistance period. Several researchers have concluded that with the exception of spalling, which is defined as the detachment of pieces of concrete when a concrete member is exposed to fire. there is no apparent reason to treat high-strength concrete differently from lower strength concrete.

Spalling can take place over the whole surface area of a member or in localised areas [38]. The risk of spalling is higher in high-strength concrete due to the following reasons:

1. Low permeability of HSC retains the moisture inside the concrete resulting in a high moisture content being present for prolonged periods.
2. Low porosity of HSC creates higher pore pressure.
3. HSC tends to be subject to higher compressive stresses than lower strength concrete.

In 1996, a comprehensive investigation was conducted at the National Institute of Standards and Technology, on experimental and analytical studies on fire performance of HSC. The key findings and a literature review are given by Phan [35]. It was observed that concrete with dense pastes resulting from the addition of silica fume are more susceptible to explosive spalling. HSC made with lightweight aggregate appears to be more prone to explosive spalling than HSC made of normal weight aggregate concretes. Chan et al. [36] showed that moisture content and strength are the two main factors governing explosive thermal spalling of concrete. Also, HSC specimens heated at higher heating rates, such as hydrocarbon fire which occurred in WTC collapse on September 11, and larger specimens are more prone to spalling than smaller specimens heated at lower rates.

According to the review by Phan [35], the material properties of HSC vary differently with temperature as compared to those of NSC. The differences are more pronounced in the temperature range of between 25°C to about 400°C, where higher strength concretes have higher rates of strength loss than lower strength concretes. These differences become less significant at temperatures above 400°C. Compressive strengths of HSC at 800°C decrease to about 30% of the original room temperature strengths. The tensile strength versus temperature relationships decreases similarly and almost linearly with temperature for HSC and NSC. HSC mixtures with silica fume have higher strength loss with increasing temperatures than HSC mixtures without silica fume.

There are only a few studies reported recently on the structural behaviour of HSC members subjected to fire. Meda et al. [37] studied the ultimate behaviour of HSC sections at high temperature and after cooling subjected to several fire durations. They concluded that HSC sections are more temperature-sensitive than NSC sections. However, the difference is not significant. Kodur [38] recommended design guidelines for mitigating spalling and enhancing fire endurance of HSC columns. In a recently concluded project at the University of Melbourne, Ta [39] found that HSC will go through more pronounced explosive spalling under hydro-carbon fire compared to standard fire.

4. Concluding Remarks

The constituents of HSC have been discussed. It is noted that the necessary ingredients making the concrete such as fly ash, slag and silica fume are mostly industrial byproducts which are otherwise wasted in landfills. This should be considered towards recognition of HSC as an environmentally friendly material. The main engineering properties of HSC are reviewed in the paper.

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MONITORING OF A TALL BUILDING TO DEVELOP AXIAL SHORTENING MODELS INCORPORATING HIGH STRENGTH CONCRETE

[1]Baidya N., [2]Mendis P.

University of Melbourne, Australia
[2]pamendis@unimelb.edu.au

Fragomeni S.

Victoria University, Australia

Abstract: This paper addresses axial shortening prediction of the vertical concrete elements of tall buildings with a particular focus on developing a reliable model for high strength concrete (HSC). An established reinforced concrete column shortening model used for normal strength concrete (NSC) is modified to predict axial shortening in vertical elements made of HSC. To compare with the theoretical model, the axial shortening measurements taken from the 83 storey World Tower Building, Sydney (WTS), obtained during the construction period, are used. The theoretical model having the best match with the actual measurements are recommended for predicting axial shortening of vertical elements using HSC.

Keywords: Axial shortening, column, high strength concrete, monitoring column, tall building

1. Introduction

There are two basic types of shortening of columns which affect the behaviour and functioning of tall buildings; (i) axial shortening and (ii) differential shortening. Axial shortening is the total cumulative shortening, which occurs due to elastic, shrinkage and creep deformations. Differential shortening is the difference between two axial shortening results at the same level. Axial shortening can potentially cause problems to infrastructure of buildings including ventilation, water and sewerage pipes, and heating systems, in addition to potential structural problems to facades, beams and slabs joining the columns. A major consequence of significant differential shortening of vertical elements (cores, columns and walls) of a building is slab tilt, which in turn rotates and distorts non-structural partitions.

It is essential that the problem of axial and differential shortening of vertical elements should be considered when building layout is designed to minimize the effect of this problem. For this, reliable methods are needed to accurately quantify the values of axial shortening. By judicially selecting appropriate column sizes, reinforcement percentages and concrete strengths, the problems of axial and differential shortening can be minimised. With the advent of advanced building technologies, use of high strength concrete (HSC) is becoming more common in the construction of vertical elements of tall buildings. Compared to conventional normal strength concrete (NSC), HSC offers significantly better structural engineering properties, such as higher compressive and tensile strengths, higher stiffness and better durability. For prediction of axial shortening in tall concrete buildings with HSC, equations of elastic modulus, shrinkage and creep based on HSC data are mentioned here. These include those proposed by Ahmad & Shah (1985), Carrasquillo et al. (1981), Gilbert (2002), Huo et al. (2001), McDonald & Roper (1993), Mendis et al. (1997) and Mokhtarzadeh & French (2000).

The idealisation of a building structure representing the complete sequential construction cycle, including the differential loading rates between adjacent structural elements, is essential. This enables an accurate definition of the loading history of each adjacent element to be employed in the prediction of differential behaviour. Thus, the final outcomes of a cumulative (axial) and differential shortening analysis of the columns depend mainly on two criteria: the idealisation of the building with its idealised construction cycle and the analytical column model containing the concrete and steel properties.

The aim of this paper is to conduct an analytical study of axial shortening of the selected vertical members in the World Tower Building, Sydney (WTS) with a view to develop a more reliable model for axial shortening. This analysis utilises the most reliable HSC equations of elastic modulus, shrinkage strain and creep coefficient to calculate the axial shortening of columns and cores.

2. Normal and high strength concrete models

NSC and HSC models for elastic modulus, shrinkage and creep prediction have been extensively reviewed by Baidya et al. (2010). A summary of the most applicable models for both NSC and HSC are given in Tables 1 and 2.

Table 1: *Normal strength concrete models.*

	Equation	**Reference**	**Notation**
Concrete compressive strength $f_c'(t)$	$f_c'(t) = \frac{t}{\alpha + \beta t} f_c'(28)$	ACI Committee 209 AS 3600 (2001)	t = age of concrete (days) α, β = constant used for compressive strength
Elastic modulus $E_c(t)$	$E_c(t) = 0.043\rho^{1.5}\sqrt{f_c'(t)}$	ACI Committee 318 (2008) AS 3600 (2001) Pauw (1960)	ρ = density of concrete (kg/m^3)
Creep coefficient $\phi(t,\tau)$	$\phi(t,\tau) = \frac{(t-\tau)^{0.6}}{10+(t-\tau)^{0.6}}\phi^*(\tau)$	ACI 209R-92	τ = age of concrete (days) at loading $\phi^*(\tau)$ = final creep coefficient
	$\phi(t,\tau) = k_2\, k_3\, \phi_{cc.b}$	AS 3600 (2001)	k_2 , k_3 = modification factor $\phi_{cc.b}$ = basic creep
Shrinkage strain $\varepsilon_{sh}(t)$	$\varepsilon_{sh}(t) = \frac{t}{35 + t}\varepsilon_{sh}^*$	ACI 209R-92	ε^*_{sh} = final shrinkage strain at time infinity
	$\varepsilon_{sh}(t) = 0.00085\, k_1$	AS 3600 (2001)	k_1= modification factor

Table 2: *High strength concrete models.*

	Modified equations related to	**Equation**	**Reference**	**Comments/Notation**
Elastic modulus E_c	ACI	$E_c = 3320\sqrt{f_c'} + 6900$	Carrasquillo et al. (1981)	$21 < f_c' < 83$ MPa
		$E_c = 3.385 \times 10^{-5}\rho^{2.5}(f_c')^{0.325}$	Ahmad & Shah (1985)	f_c'= compressive strength at 28-days (MPa)
	AS 3600	$E_c = 0.043\eta\rho^{1.5}\sqrt{f_c'} \pm 20\%$	Mendis et al. (1997)	η = coefficient for modulus of elasticity $\eta = 1.1 - 0.002 f'_c \le 1$
		$E_c = \rho^{1.5}(0.024\sqrt{f_{cm}} + 0.12)$	Gilbert (2002) AS 3600 (2009)	f_{cm}= mean compressive strength of concrete at 28 days (MPa) $f_{cm} \le 100$ MPa

Creep coefficient v_t φ_{cc}	ACI	$v_t = (v)_u \dfrac{t^{0.6}}{K_c + t^{0.6}}$	Huo et al. (2001)	(v_u) = ultimate creep coefficient K_c = adjustment for early age creep coefficient
	AS 3600	$\varphi_{cc} = k_2\, k_3\, k_4\, k_5\, \varphi_{cc.b}$	Gilbert (2002) AS 3600 (2009)	k_2, k_3, k_4, k_5 = modification factor $\varphi_{cc.b}$ = basic creep coefficient
Shrinkage strain $\varepsilon_{sh}(t)$ ε_{sh} ε_r ε_{cs}	ACI	$\varepsilon_{sh}(t) = \dfrac{t}{45 + t} (\varepsilon_{sh})_u$	Mokhtarzadeh & French (2000)	$(\varepsilon_{sh})_u$ = ultimate shrinkage strain $(\varepsilon_{sh})_u$ = 530 microstrain
		$\varepsilon_{sh} = (\varepsilon_{sh})_u \dfrac{t}{(K_s + t)}$	Huo et al. (2001)	K_s = adjustment for early age shrinkage
	AS 3600	$\varepsilon_r = \varepsilon_b\, k_e\, k_h$	McDonald & Roper (1993)	ε_b = basic shrinkage strain k_e, k_h = shrinkage strain coefficient
		$\varepsilon_{cs} = \varepsilon_{cse} + \varepsilon_{csd}$	Gilbert (2002) AS 3600 (2009)	ε_{cse} = endogenous shrinkage strain ε_{csd} = drying shrinkage strain

3. Reinforced concrete column HSC model

In this paper, a shortening model suitable for calculating long term vertical deformations of HSC columns obtained by modifying the existing NSC model is presented. A constant load P is applied to a short symmetrically reinforced concrete column at time *t*. For equilibrium, the load P must be resisted by internal forces as given in Equation (1):

$$P = N_c(t) + N_s(t) \tag{1}$$

where $N_c(t)$ is the internal force in concrete and $N_s(t)$ is the internal force in steel.

Using the age-adjusted effective modulus method (AEMM) proposed by Bazant (1972), total axial shortening strain of the column for *k* loadings is given by:

$$\delta_{total} = \delta_{elastic} + \delta_{creep} + \delta_{shrinkage} + \delta_{reinforcement} \tag{2}$$

In Equation (2) the elastic, creep and shrinkage strains can be estimated using corresponding ACI and AS equations (or other models) as given in Section 2. The full formulations of each component in Equation (2) are given by Koutsoukis & Beasley (1994).

For the j^th storey, the final cumulative column shortening is as follows:

$$\delta_{\text{cumulative}} = \sum_{i=1}^{j} \delta_{total\ \ for\ \mathrm{i}^{\mathrm{th}}\ storey} \tag{3}$$

Equation (3) summarises the application of the AEMM to obtain the final axial shortening predictions, for which the individual components (i.e. elastic, creep, shrinkage and reinforcement) are accumulated over the entire height of the column.

4. Measurement of world tower building, Sydney

The WTS building (230 m high and 83 storey) constructed in 2004, is one of the tallest buildings in Australia. This building provided an opportunity to monitor and record the axial shortening history of selected columns. At different levels of the WTS building, 28-day concrete strength varies from 32 to 60 MPa for the core and from 40 to 90 MPa for columns. Strain gauge points were installed and monitored mechanically on three columns TC1, TC4 and TC9 from levels 14 to 39. These gauge points were located in three stations on the face of the column at the base, at the top and in the middle. The data was collected using a demountable mechanical gauge system (Demec gauge), with some columns monitored for almost 200 days. Data were collected for columns with concrete strengths of 50, 60, 80 and 90 MPa, giving useful results for HSC columns. Also, internal strain gauges were installed on various floors to measure the transfer of stress from the concrete to the steel reinforcement. The extensive field measurement results were presented by Baidya (2005) and Bursle (2006).

5. Comparison of observed and predicted axial shortening

The cumulative strain data of the WTS building were used for comparison with theoretical results. A combination of three equations - one for each of elastic modulus, creep coefficient and shrinkage strain - is required to calculate column axial shortening model as shown in Equation (2). Thus, a number of empirical equations of elastic modulus, creep coefficient and shrinkage strain previously derived for HSC and NSC were combined randomly to form six AS and eight ACI models to calculate axial shortening of column. The combinations are given in Tables 3 and 4. In this study, only readings from level L26 to L33 of column TC1 were selected for the analysis because adequate experimental data were not as extensive for other levels and columns. The exterior column TC1 (corner column with two interior faces) was selected for monitoring based on the symmetry of the building. Following data were taken for all levels of the column TC1 (from L26 to L33): (i) 28-days concrete strength 80 MPa, (ii) cross-sectional area 1 m^2, (iii) perimeter 4 m, (iv) reinforcement 1.2%, (v) basic shrinkage strain 550 microstrain and (vi) average humidity 62%. It should be noted that the equations for elastic modulus, creep and shrinkage proposed by Gilbert (2002) are now incorporated in the recently released Australian standard AS 3600 (2009).

Table 3: *Different AS model combinations.*

Model	Elastic Modulus	Creep Coefficient	Shrinkage Strain
1	Gilbert, 2002	Gilbert, 2002	Gilbert, 2002
2	Gilbert, 2002	Gilbert, 2002	MacDonald and Roper, 1993
3	Mendis et al., 1997	AS 3600 (2001)	AS 3600 (2001)
4	Ahmad and Shah, 1985	AS 3600 (2001)	MacDonald and Roper, 1993
5	Carrasquillo et al., 1981	Gilbert, 2002	MacDonald and Roper, 1993
6	Pauw, 1960	AS 3600 (2001)	AS 3600 (2001)

Table 4: *Different ACI model combinations.*

Model	Elastic Modulus	Creep Coefficient	Shrinkage Strain
1	Gilbert, 2002	Huo et al., 2001	Huo et al., 2001
2	Mendis et al., 1997	Huo et al., 2001	Mokhtarzadeh and French, 2000
3	Pauw, 1960	Huo et al., 2001	ACI 209R-92
4	Carrasquillo et al., 1981	Huo et al., 2001	Huo et al., 2001
5	Ahmad and Shah, 1985	Huo et al., 2001	Huo et al., 2001
6	Mendis et al., 1997	Huo et al., 2001	Huo et al., 2001
7	Pauw, 1960	ACI 209R-92	ACI 209R-92
8	Carrasquillo et al., 1981	ACI 209R-92	ACI 209R-92

Model predictions were compared with the field data of the WTS building considering two different cases: a) Case 1 - comparison of all fourteen models (six models from AS and eight models from ACI) at level L33 for column TC1, and b) Case 2 - comparison of six selected models (the best three from AS and the best three from ACI) at levels L26 to L33 for column TC1.

Case 1 was used to select the six best models for further comparison in case 2. For Case1, column shortening predictions from the six AS models and eight ACI models were compared with the observed values of column TC1 at level L33 using time-trend, prediction error and observed-versus-predicted (scatter plots). From the analysis of the prediction errors Models 1, 3 and 5 from the AS group and Models 8, 4 and 2 from ACI group (see Table 3 and 4) were selected as the best models for further analysis for all levels L26 to L33. The full details are given by Baidya (2005).

For Case 2, in order to pinpoint the best model among the selected six models from the AS and ACI groups for column TC1, more analysis of the observed and predicted cumulative strains (predicted minus observed) were performed on a (i) level by level basis from L26 to L33 using time-trend and prediction error (see Figure 1 for L26) (ii) model by model basis from L26 to L33 and (iii) statistics of overall prediction error from L26 to L33 (see Figure 2).

The observed and predicted time-trends of cumulative strains for six models and their prediction errors for level L26 are shown in Figure 1. The observed values covered a period from 70 to 177 days after casting of concrete with observations made at irregular time intervals. Figure 1 shows that the AS (Models 1, 3 and 5) and ACI (Models 2, 4 and 8) group of models over-predict cumulative strain at early days (until ~150 days for AS Model 1, Models 3 and 5, and ~160 days for all ACI models – termed here as a "threshold period"), but after this threshold period, these models tend to under-predict cumulative strain. Even though this general trend is observed for all models, it is interesting to note that the predicted and observed trend lines do produce similar strains at various times.

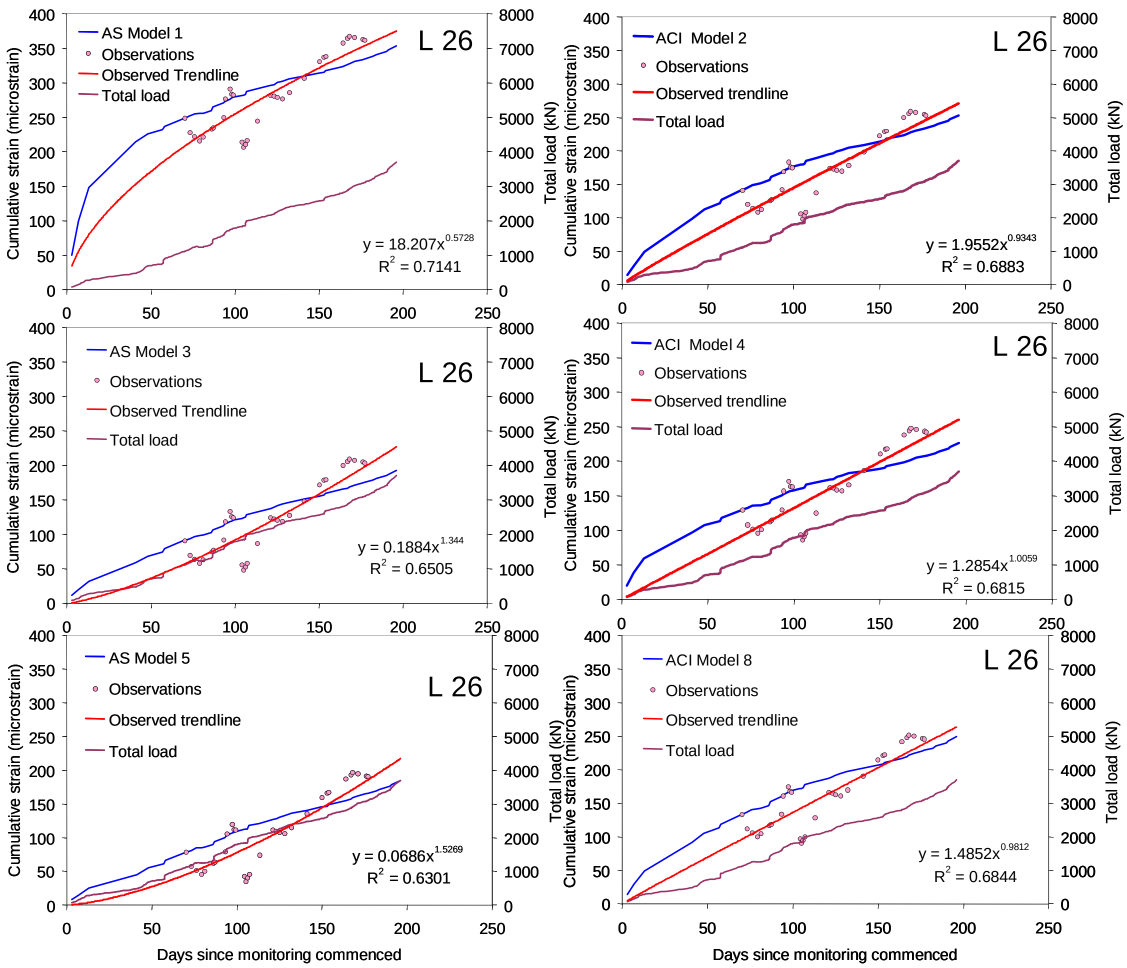

Figure 1: *Time-trend between the observed and predicted cumulative strains – Level L26. For AS and ACI model details, see Table 3 and 4.*

In the model wise analysis Baidya (2005) reported that all three AS models (1, 3 and 5) perform better in levels L29 and L31, and all ACI models (2, 4 and 8) are better in levels L30 and L31. It is noted that these models have high prediction errors in themselves and are considered relatively better or worse in this study from a subjective judgement. One model predicting better than other models in one level but performing poorly in other levels indicates that there is no universal model that is better and suitable for all levels.

When overall error statistics (mean, standard deviation and coefficient of variation of all levels from L26 to L33 of column TC1 combined together) are compared, some models do stand out from others in terms of their prediction ability (see Figure 2). Figure 2 provides error bands (mean ± std dev) for the six selected models, indicating that ACI Model 2 has the lowest overall mean error when prediction errors from all floors are put together. In contrast, AS Model 1 has the highest overall mean error.

Considering the ACI group Model 2, Model 4 and Model 8, and in the AS group Model 5, Model 3 and Model 1, are the best to worst models, respectively. If both ACI and AS groups are considered together then ACI Model 2 gives the best prediction. It is noted that these observations are based on comparing overall statistics of these selected six models only. Full details are given by Baidya (2005).

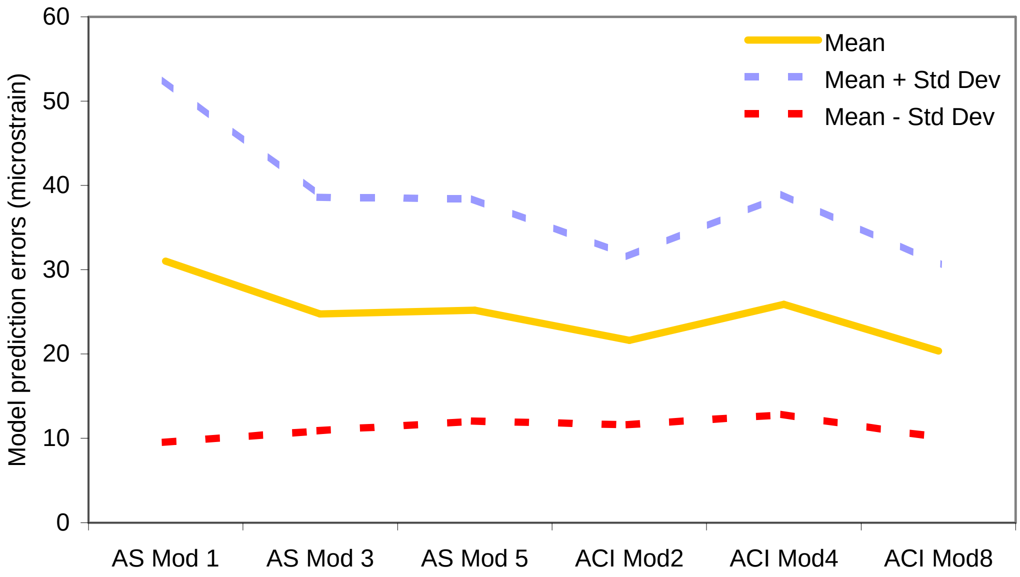

Figure 2: *Overall model prediction error: Overall mean ± Std Deviation.*

6 Conclusions

In addition to an analysis of prediction error values (level wise and model wise), overall error statistics (mean, standard deviation and coefficient of variation) are analysed to find out the best performing model for each of the selected six models of the AS and ACI groups. From this analysis, it can be concluded that ACI Model 2 and AS Model 5 are the best models (Figure 2). It is interesting to note that elastic modulus (Mendis et al., 1997 for AS and Carrasquillo et al., 1981 for ACI), creep coefficient (Gilbert, 2002 for AS and Huo et al., 2001 for ACI) and shrinkage strain (MacDonald & Roper, 1993 for AS and Mokhtarzadeh & French, 2000 for ACI) equations for these two best models are taken from the equations applicable to HSC. This indicates that equations applicable to HSC should be utilised when available.

Comparisons with field observations also show that development of accurate prediction methods that will have general relevance to deformations in buildings is complicated. This is partly due to the fact that the factors that influence the creep and shrinkage deformations of insitu concrete are very complex and highly variable and therefore it is difficult to specify them accurately. More field investigations generating quality data may help improve the prediction of elastic modulus, creep coefficients and shrinkage strains for HSC and NSC, and thereby the prediction of axial and differential shortening in vertical concrete members.

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CONSIDERATION OF DAMAGES OF A FLOATING ROOF-TYPE OIL STORAGE TANK DUE TO THERMAL STRESS

Yoshihiro Hirokawa[1], Minoru Yamada[1], Haruki Nishi[1],
Shinsaku Zama[1], and Ken Hatayama[1]

[1]Research and Development Division, National Research Institute of Fire and Disaster
4-35-3 Jindaiji-Higashimachi, Chofu-city, Tokyo 182-8508, Japan

Abstract
Several cracks were found on some actual floating roofs of the crude oil tanks in a southern Japan refinery. It was assumed that one of the causes is due to thermal stress during the day. In order to figure out whether the thermal stress could cause damage on the floating roof, strain and temperature were measured on the actual floating roof by using optical fiber gauges. Furthermore, thermal stress analysis and fracture possibility estimation were also carried out as additional analysis. As a result, thermal stress on the floating roof turned to be relatively small and could not cause the initial crack. However, the temperature fluctuation in a day could affect the crack propagation.

Key words: *Oil storage tank, Floating roof, Measurement, Optical fiber gauge, Thermal stress analysis, FEM, Fracture probability, Crack*

1. Introduction

An oil storage tank equipped with a floating roof is called a floating roof tank [1]. The floating roof covers the oil surface in order to prevent volatilization and contamination. The single-deck type floating roof is mainly composed of a thin deck and a pontoon maintaining buoyancy. The deck thickness does not depend on tank size and approximately 5mm thick steel plate is used even if the diameter is as much as 100m. Therefore, the floating roof is vulnerable to the external load such as earthquake motion. For example, the Tokachi-oki earthquake ground motion hit Tomakomai city in Hokkaido in Japan in 2003 and around 200 tanks had the damage in Hokaido at that time. One of them in Tomakomai city suffered serious damages on the floating roof. The roof lost the buoyancy from the pontoon and sank completely. As a result, the tank was on a whole surface fire for 44 hours [2].

According to the recent investigation conducted by the authors, several cracks were found on the floating roof of the crude oil tank in a southern Japan refinery [3]. The tank has 100,000 kL capacity, 80m diameter, and 22m height. The geographical features are that the refinery is located in the area where several big typhoons tend to pass every year and that the temperature difference between day and night is relatively large. In order to prevent the potential serious incident, it is necessary to investigate causes of the cracks on the floating roof.

In this study, it was assumed that one of the causes was due to thermal stress on the floating roof tank during the day. Strain and temperature on the floating roof were measured by using an optical fiber gauge in order to comprehend the floating roof during the day. Thermal stress analysis was also conducted by using the finite element method (FEM) in order to compare with the measurement results on the actual floating roof. In addition to that, the fracture possibility of the deck plate due to cyclic thermal stress was estimated by means of probabilistic fracture mechanics approach.

2. Measurement on the Actual Floating Roof

In order to figure out whether the thermal stress could cause damage on the floating roof, strain and temperature were measured on the actual floating roof by using optical fiber gauges. The measurements were conducted on March, June and August in 2008 and on August in 2009.

2.1 Sensor layout

The floating roof on which several cracks were found is the single-deck type floating roof with an additional center pontoon. The tank has a diameter of 80m and height of 22m with a 100,000 kL capacity. The thickness of the deck plate is 4.8mm. The tank was built in 1980.

In order to measure thermal stress amplitude during the day, optical fiber gauges equipping with the explosion-proof were adhered to the roof surface. Figure 1 and Figure 2 show the actual floating roof and layout optical fiber gauges to measure strain and temperature amplitude. “STR#” in

Table 1 shows the distance of strain gauges from the wall of the center pontoon. Temperature gauges were set in between STR2 and STR3 beside the line of the strain gauge sensor.

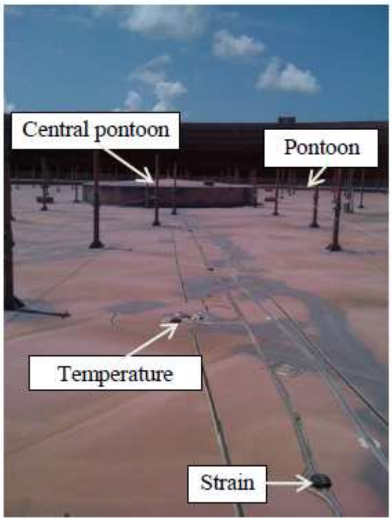

Figure 1 Actual floating roof equipped with pontoon and center pontoon.

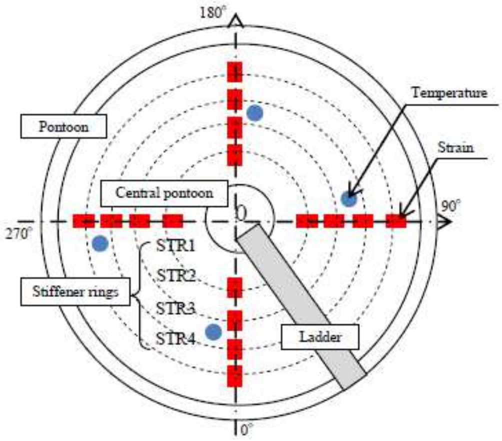

Figure 2 Layout of optical fiber gauges adhered to the floating roof.

Table 1 Sensor location on the floating roof.

	Distance from the wall of the center pontoon in each direction (m)			
	0°	*90°*	*180°*	*270°*
STR1	11.8	12.1	12.0	12.0
STR2	18.0	18.0	18.0	18.0
STR3	24.1	24.0	24.0	24.0
STR4	29.5	29.5	29.5	29.5

2.2 Measurement results

Figure 3 and Figure 4 show strain and temperature amplitude history in 0^{o} direction during the day respectively. The origin of the time in both figures means 12A.M. Figure 3 shows that 412μStrain amplitude occurred in 12 hours around at STR 1 when temperature amplitude was 56 degree Celsius. If Young's modulus is 206GPa, 83MPa stress amplitude will occur. This value is less than that of yield stress of general structure steel (245MPa) [4]. As for the result of STR4 in Fig. 3, the buckling could occur in 9 hours. This will be discussed in the section 3.3.

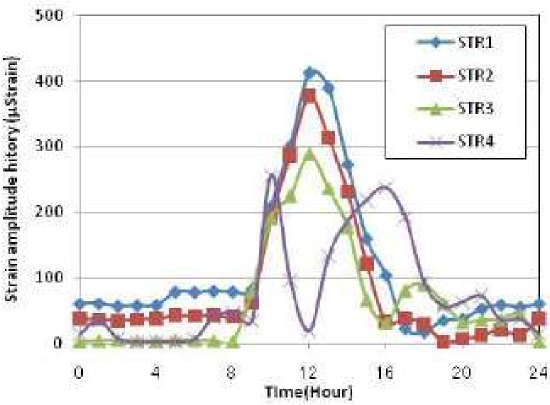

Figure 3 Strain amplitude history in 0° direction.

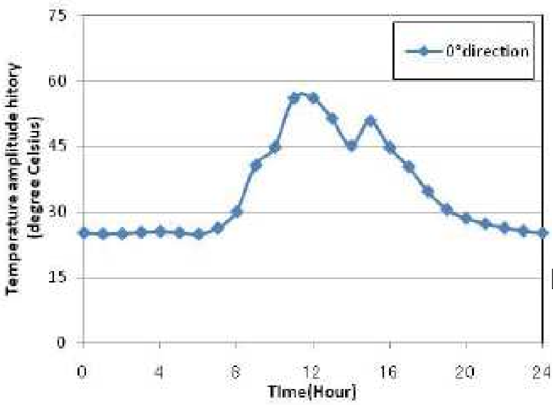

Figure 4 Temperature amplitude history in 0° direction.

thickness of each part was equivalent to the design thickness. The number of nodes and elements was 348 and 60 respectively. Table 2 shows the material constants. The material was supposed to be the general structural steel, SS400 [4].

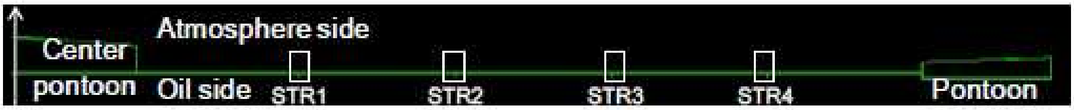

Figure 5 Two dimensional axial model.

Table 2 Material constants

Young's modulus	2.1×10^{11} (N/m²)
Poisson's ratio	0.3 (-)
Thermal expansion	1.13×10^{-5} (1/°C)
Thermal conductivity	51.6 (W/m°C)
Heat transfer coefficient of the wall inside the pontoon	10 (W/m²°C)
Heat transfer coefficient of the back side of the deck plate	35 (W/m²°C)

3.2 External load and constrain condition

Figure 6 shows the numerical temperature data based on measurement results and Fig. 7 shows temperature and constrain conditions. Temperature of the deck surface around the stiffener ring was around 10 degree Celsius smaller than that of other area because stiffener rings worked as a heat release fin. Even in the pontoon, the heat was likely to release from the inside wall of the pontoon. Therefore, temperature amplitude histories were given as two types of curves shown in Fig. 6. The circled numbers 1 and 2 in Fig. 7 correspond to the numbers in Fig. 6. This means that the temperature data of the circle number 1 in Fig. 6 was given to the deck surface excluding the surface above the stiffener rings. Also, the temperature data of the circle number 2 in Fig.6 was given to the pontoon surface and the surface above the stiffener rings. As for the inside wall of the pontoon shown as the line of the circle number 3 in Fig. 7, the initial temperature amplitude which was 32 degree Celsius was given. Regarding the constrain conditions, the horizontal translation was fixed on the center line and the vertical on the outer pontoon as shown in Fig. 7.

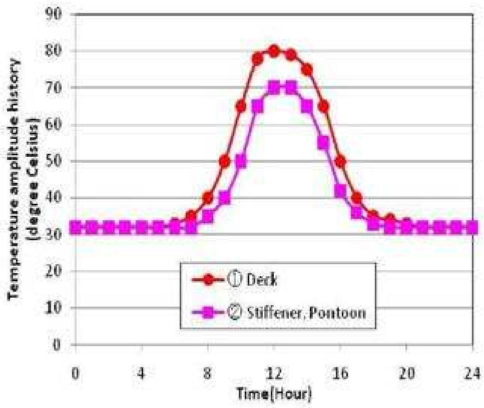

Figure 6 Numerical data of temperature amplitude history.

Figure 7 Temperature and constrain conditions on the floating roof.

3.3 Numerical results

Figure 8 shows the strain amplitude history obtained by the numerical simulation. According to Fig. 8, the strain amplitude history was subject to the temperature history and that the maximum strain amplitude occurred at around maximum temperature amplitude. Compared with Fig. 3, corresponding curves in Fig. 8 show almost the same amplitude except for STR 4. Regarding STR 4, the measurement result could show Snap-back. It is considered that imperfections in the actual floating roof affected the deformation. The numerical model is the ideal model without imperfections, while the actual floating deck involves welded parts and additional pieces attached to the deck surface, which could be considered the origin of the buckling [5].

Figure 9 shows the effective stress history. Curves fluctuated within 8 to 18 hours. The maximum stress occurred at STR 4 in 10 hours. The maximum stress was at most 27.4MPa. In comparison with tensile strength of the general structure steel (400MPa) [4], the value of the maximum stress was fairly small. Furthermore, the maximum stress was less than the fatigue limit that is around 100MPa in the case of the general structure steel [6]. From these results, thermal stress on the floating roof was unlikely to lead to the fracture on the actual floating roof.

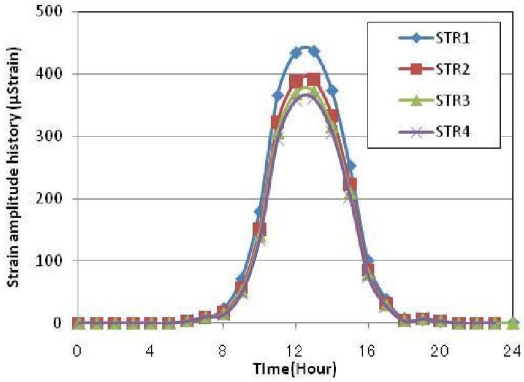

Figure 8 Strain amplitude history of the numerical simulation.

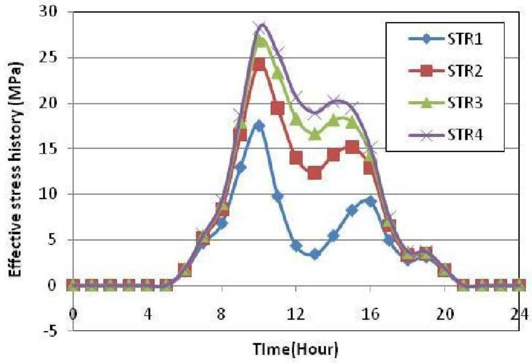

Figure 9 Effective stress amplitude history of the numerical simulation.

4. Fracture Possibility due to Thermal Stress

From the numerical results, cracks were unlikely to occur on the intact floating roof due to thermal stress. Next, initial cracks which occurred from any cause were discussed as to whether thermal stress allowed cracks to propagate or not. However, the crack occurrence involves a variety of uncertainties. Therefore, the fracture probability due to the thermal stress was estimated by using probabilistic fracture mechanics approach [7].

4.1 Fracture probability

The fracture probability P_f is shown in Eq.(1). In this study, P_f was calculated by means of Monte Carlo Method that is the way to estimate the probability of a certain event subject to the probability density function [8]. Mf and N_t describe the total number of the fracture and the event respectively. The event was randomly given according to the random function [9].

$$P_f = M_f/N_t \tag{1}$$

4.2 Flow chart to estimate fracture probability

Figure 10 shows the flow chart to estimate the fracture probability. As for an initial crack a_0, it shall be defined as a two dimensional surface crack shown in Fig. 11. The crack length a and thickness of the deck plate h have a dimension in mm. Details of the flow chart are described as follows:

1) *Initial crack*: An initial crack a_0 shall be subject to the probability density function p in Eq.(2). μ is the mean of the crack length in mm. Three cases (μ=h/3=1.6mm, h/4=1.2mm, and h/5=0.96mm) were considered. The number of the initial cracks given to the calculation was 1,000 in each case. All of the cracks were passed through the inspection process. The relatively large cracks that could be detected by the inspection were rejected and replaced. The probability function B(a), which describes that the inspection fails to detect the cracks, is given in Eq.(3) [10]. The coefficients α and β are 0.113 and 0.005 respectively.

2) *Crack propagation*: As for the crack propagation criteria, Paris law given in Eq.(4) was applied [7]. ΔK is stress intensity factor range in $MPa(m)^{1/2}$. Constant numbers C and m are 5.21×10^{-13} and 3.0 respectively [11]. The crack shall propagate in case that ΔK is larger than threshold ΔKth equal to

2MPa(m)$^{1/2}$ [11]. The stress intensity ΔK is given as difference between maximum stress intensity factor Kmax and minimum intensity factor Kmin in Eq.(5). The stress intensity factor K is determined by using the cubic stress function shown in Eq.(6) [12]. The variable ξ is shown in Fig. 12 as the ratio of the crack length a to the deck thickness h. Coefficients Ai in Eq.(6) were decided to fit the stress distribution. According to Fig. 9, maximum effective stress was 27.4MP on the deck surface attached with the stiffener ring #1. Also, the effective stress on the opposed side of the deck surface was 13.2MPa. Here, the stress function was simply defined as the linear function showing in Fig. 12. The stress intensity factor K is given as Eq.(7) based on the principle of superposition. The coefficient F0 and F1 are give as Eq.(8) and Eq.(9) according to the ratio a/t [12, 13]. Also, the periodic inspection subject to the probability function shown in Eq.(3) shall be conducted every several years. In this simulation, the four cases (no periodic inspection, every 5 years, every 8 years and every 10 years) were tested.

3) *Fracture probability*: Fracture probability was calculated by Eq.(1). The fracture condition refers to the status that the initial crack length a_0 becomes equivalent to the plate thickness h.

$$p=(1/\mu)\exp(-a_0/\mu) \qquad (2)$$

$$B(a)=\beta+(1-\beta)\exp(-\alpha a) \qquad (3)$$

$$da/dN=C(\Delta K)^m,\ \Delta K_{th}<\Delta K \qquad (4)$$

$$\Delta K=K_{max}-K_{min} \qquad (5)$$

$$\sigma(\xi) = A0+ A_1\xi + A_2\xi^2 + A_3\xi^3 \qquad (6)$$

$$K=(\pi a)^{1/2}(A_0F_0+2\xi A_1F_1/\pi) \qquad (7)$$

$$\left.\begin{array}{l}\text{At } a/t<=0.7\\ F_0=0.6820\xi^4-1.8283\xi^3+3.4051\xi^2+0.0209\xi+1.1215\\ F_1=1.2402\xi^4-2.2730\xi^3+2.5718\xi^2-0.0578\xi+1.0727\end{array}\right\} (8)$$

$$\left.\begin{array}{l}\text{At } a/t>0.7\\ F_0=4729.3333-17919.6364+27100.4530-20442.3042+7692.5106-1153.4967\\ F_1=2688.0000-10106.3636+15174.8303-11369.3741+4250.9351-633.0513\end{array}\right\} (9)$$

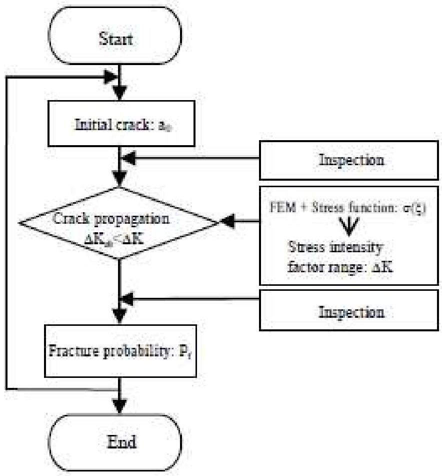

Figure 10 Flow chart for estimating fracture probability.

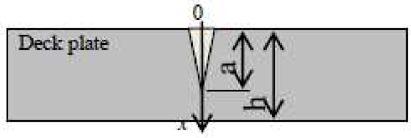

Figure 11 Two dimensional surface crack.

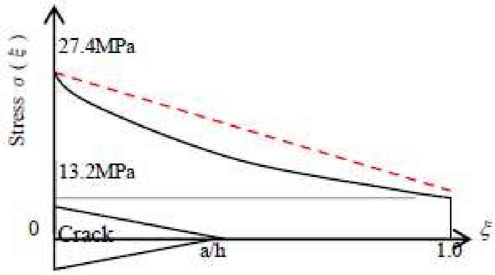

Figure 12 Coordinate system and stress function along the crack.

5. Fracture Probability Results

Figure 13 shows the fracture probability in case that average crack lengths are h/3, h/4, and h/5 without any of the initial inspection or the periodic. According to Fig. 13, the larger the average crack length is, the earlier the fracture begins to occur. In the case of h/3, the fracture begins to cause in 7.2 years and the fracture probability could attain to 0.45 in 30 years. Similarly, in the case of h/4, fracture would begin to occur in 12.8 years and the fracture probability could attain to 0.39 in 30 years. In the case of h/5, the fracture probability could attain to 0.21 within 18.8 years to 30 years.

Next, Figure 14 shows the fracture probability considering the periodic inspection (every 5, 8 and 10 years) in case of h/3. Figure 14 illustrates that the fracture probability can be approximately zero right after the periodic inspection. This result does not depend on the period of the inspection. The shorter the interval of the inspection is, the lower the maximum value of the fracture probability becomes.

The actual tank shall be inspected every 8 years. Based on the period of service (about 30 years), the fracture probability was supposed to be 4.3e-2 when the cracks on the floating roof were detected.

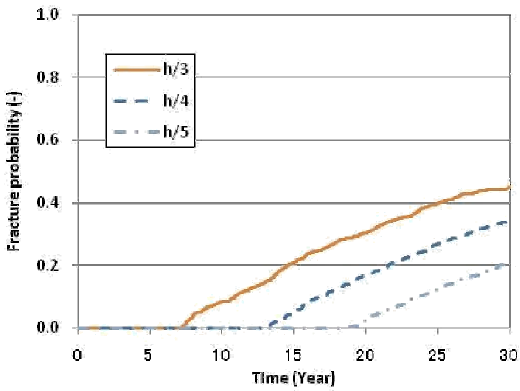

Figure 13 Fracture probability (average crack length: h/3, h/4, h/5 without any of inspections).

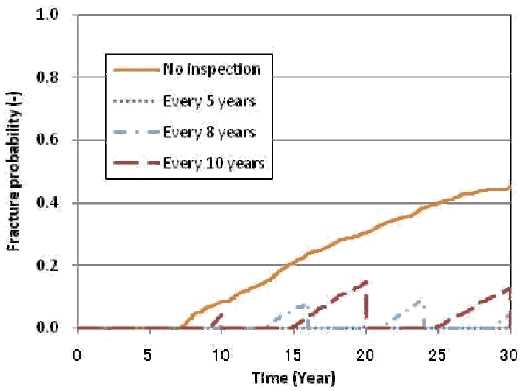

Figure 14 Fracture probability considering the periodic inspection (average crack length: h/3)

6. Conclusion

In order to investigate the cause of the crack occurrence on the floating roof, the measurement on the actual floating roof, the numerical simulation, and the fracture probabilistic estimation were carried out. Conclusions are shown as follows:

(1) Strain and temperature amplitude history on the floating roof were measured by using optical fiber gauges. The maximum strain around the stiffener rings was around 400μStrain. However, the value of the estimated maximum stress was much smaller than the value of tensile strength of the general structure steel.

(2) Thermal stress analysis was conducted by using the two dimensional axisymmetric model. Compared with the measurement results, the numerical results showed adequate value. The value of effective stress around the stiffener rings was at most 30MPa, which was less than the fatigue limit and the tensile strength of the general structure steel.

(3) The initial cracks which occurred from any cause were discussed by probabilistic fracture mechanics approach. It is considered that the initial crack could propagate and penetrate by cyclic thermal stress over time, while the cracks unlikely occur on the intact deck due to thermal stress.

(4) Considering the case in this study, the fracture probability can be estimated to be 4.3e-2. Generally, this probability would be small, while the results depend on the probability function. Therefore, other factors such as Typhoon should be also discussed in the future.

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EXTRACTION OF STRUT AND TIE MODEL FROM 3D SOLID ELEMENT MESH ANALYSIS

[1]Dammika A.J., [2]Naveed Anwar

[1]"Mahanuwara Gedara" Pallegama, Hungama. Sri Lanka.
dammikaaj@gmail.com M Eng, Structural Engineering, Asian Institute of Technology, Thailand

[2]School of Engineering and Technology, Asian Institute of Technology,
P.O Box 4,Klong Luang, Pathumthani,12120, Thailand.
nanwar@ait.ac.th Associate Director, ACECOMS

ABSTRACT

Strut-and-tie model (STM) method is a lower bound method based on the theory of plasticity, which can be used especially for the design of structural concrete members in D- Region. An approach for automatically finding an appropriate STM for structural concrete members modeled and analyzed by using solid element mesh is introduced in this study. The finite element analysis can be performed in any FEA computer program that has the solid element meshing capability. The solid element principle stress trajectories of the concrete member are obtained and struts and ties are extracted based on the direction cosines and magnitude and graphically displayed. The algorithm includes two main important features: (1) to extract and display an appropriate STM from the output of FEA; and (2) to refine, analyze and design the extracted appropriate STM. As a sample application, concrete pile cap configuration is used in demonstrating the capability of the proposed method in finding an appropriate strut-and-tie model and compared with the previous theoretical and experimental studies that deal with STM for the purpose of verification of the results.

Keywords: Strut and Tie Model, Stress Trajectories, Solid Element, Principal Stress, Finite Element Analysis

1. Introduction

The strut and tie model (shortened as STM) approach which is originated from truss analogy concept has become more rational to use as a tool for designing of structure's disturbed or discontinuity regions (D-region) of concrete structures. So, it fulfills the void in many design codes and guidelines by providing reasonable approach to design structure's D–regions instead of using rule of thumbs and experience to design in such structural components. The structural member is idealized as a truss by introducing uniaxial compressive struts and tension ties in this model which represent the actual load transfer mechanism of the particular member under applied loads and given support conditions. In the concept of truss analogy, truss mechanism is defined for fully cracked reinforced concrete section and concrete is considered to be as no more tensile strength. The truss action is produced by diagonally cracked web concrete struts while longitudinal and transverse reinforcement are act as horizontal and vertical ties. The locations where struts and ties intersect are called as nodes and these nodal zones represent biaxial or tri axial stress fields depending on two dimensional or three dimensional truss configurations of particular member. This particular study is focused on three dimensional strut and tie model which cannot be idealized from two dimensional strut and tie model as usual practice for many three dimensional structural components.

According to the experience of many researchers, there is no single or unique STM is available for any particular structural component. So, engineering judgments are helpful in this occasion to find out correct truss model for particular member which is defined based on stress trajectories obtained from elastic analysis or load path approach as described by Schlaich (1987).

Since STM becomes rational design approach for designing structure D-regions in which no sufficient guide lines are provided in many of design code of practices. Many of the structures including joints, brackets, openings, corbels, deep beams, pile caps contains D-regions in which geometric or static or both discontinuities are available. In fact proper design guide lines are necessary to overcome problems and failures concentrated on those D-regions due to empirical design provisions and detailing practices. So, finding of accurate truss idealization for such structural components is essential for economical and safest design practices. Unfortunately finding of necessary STM for structural members has to undergo so many barriers although that is the most suitable approach

available to identify the load transfer mechanism of such components. As understood from the literature, available tools to find STM are iterative and more time consuming. Although FEM approach provide clear picture of stress distribution of any structural configurations, in many instants, even imagination of STM for a particular situation is difficult as their complexities in stress distribution in many structures D-regions. On the other hand difficulties occur in confirming the adequacy of identified STMs and selecting better model from available many options. So, it is necessary to pay attention on those unresolved problems associated with STM design approach.
Obtaining of elastic stress distribution for complex structural configurations can be easily achieved by employing finite element method (FEM). FEM is the available most powerful tool which is more popular among designers to analyze complex structural configurations to find out its response for applied loading conditions. For the present study, FEM is the basic tool which is going to be used to find the hidden truss mechanism in three dimensional structural components according to its elastic stress distribution. In this study, it is proposed to use three dimensional solid element mesh analysis of a pile cap in order to extract its strut and tie mechanism use in load transfer.

2. Material and Methods

History

The concept of truss analogy in structural design spreads over hundreds years of history (Ritter, 1899; Mörsch, 1902)(Schlaich (1987).According to the past records, although certain part of the structures are designed with almost required accuracy, some other parts are designed using rule of thumb or judgment based on past experience. In that case most of the researchers are contribute their effort to apply concept of truss analogy to find solutions for designing of such irregular reinforced concrete members.
As a further development of the approach, the application of strut and tie model for design of structure D-regions as well as for B regions are present by Schlaich (1987) in well described manner. After that the concept of strut and tie approach becomes rational in D-region design and is included in many design codes as guide line to design of such deep and irregular members.
In the strut and tie model proposed for reinforced concrete structures, loads are carried through set of compressive fields (strut) and these are interconnected by tension ties which is usually reinforcement bars, pre-stressed tendons or concrete stress fields. These compression strut and tension ties are interconnected at nodes. Once a suitable truss model is identified, the forces of the strut and ties are calculated satisfying equilibrium between applied loads and inner forces in order to sizing of them. Especially this method implies that the structure is designed according to lower bound theorem of the theory of plasticity.
As it can propose many strut and tie models for a structural member, it is necessary to find out optimized model for particular member. Since loads try to use the path with the least forces and deformations, the models with the least and shortest ties are best because of reinforced ties are much more deformable than concrete struts. This simple criterion is formulated as follows by Schlaich. (1987)

$$\sum F_i l_i \varepsilon_{mi} = Minimum$$

Where

$$F_i = force\ in\ strut\ or\ tie\ i$$
$$\varepsilon_{mi} = mean\ strain\ of\ member\ i$$
$$l_i = length\ of\ member\ i$$

There are basically three types of struts and ties are identified. Those are Concrete strut in compression (C_c), Concrete ties in tension without reinforcement (T_c) and Ties in tension with reinforcement (T_s). The nodes can recognize as CCC-node, CCT- node, CTT-node or TTT-node depending on the combination of above mentioned strut and ties. The principle remains same for the nodes which combine more than three strut and ties too.
As mentioned earlier T_s is one dimensional element between two nodes which is essentially longitudinal reinforcement bars or pre-stressing tendons as well as stirrups. In the case of longitudinal reinforcement adequate anchorages need be provided in order to avoid brittle anchorage failures at load below the ultimate capacity. Whereas C_c and T_c are two dimensional or three dimensional stress fields which spread between two adjacent nodes.

The successes full usage of strut and tie model is depends on understanding of basic member behavior and engineering judgment and it is basically a design tool. The process of developing a strut and tie model for particular member is an iterative and graphical procedure and it can be done using few approaches as identified. This can be done either by using stress trajectories based on Elastic Analysis or load path approach or by using standard models.

The strut and tie model is based on the cracked section and it gives lower bound capacities according to the lower bound theorem of plasticity which is the theoretical basis of truss analogy. It is assumed that crushing of concrete (struts and nodes) does not occur prior to the yielding of reinforcement (ties or stirrups) for this is to be true.

Finite Element Analysis

In the case of finding appropriate STM for a structural concrete member, as it is understood from the literature, identification of the flow of internal forces that is stress trajectories of that particular member is the main focus. In that case, FEM is the available more reliable tool which can be used to analyze any complex structural components by meshing it in to small elements. Nowadays it is more popular among engineers because of its compatibility with modern digital personal computers even for analysis of much complex problems.

As it is clear from the literature, FEM is used by many researchers as their tool to find the stress trajectories of structural components. As a similar approach, in proposed study, Finding of stress trajectories of a pile cap is going to be done employing linear elastic finite element analysis using solid element mesh with aid of digital computer.

Computational Approaches for Developing Necessary Strut and Tie Models

As experienced by many researchers and designers, the traditional methods which are used to find STM for particular structural concrete member is much time consuming and most of the time it would be an iterative procedure based on designer’s intuition and previous experience. Also it is a difficult task for designers to find correct strut and tie model for members with complicated geometry and loading conditions. So, many researchers focused on their studies to find out necessary STM for structural concrete members using computational approaches. Most of such approaches are related to the automatic generation of STM using computer.

As a computational approach, Liang et al.,(2000) proposed a performance based evolutionary topology optimization method for automatic generation of optimal strut-and-tie models for reinforced concrete structures based on evolutionary structural optimization method (ESO).. In that approach, the element virtual strain energy is calculated for element removal, while a performance index is used to monitor the evolutionary optimization process. In this method, the load transfer mechanism of the structural member is gradually characterized from remaining elements of the discritized concrete member after systematic removal of elements that have the least contribution to the stiffness.

Furthermore, Liang et al. (2001) again introduced the method for automatic generation of STM for prestressed concrete beams by using the performance based optimization (PBO) technique.. In PBO, the performance objective of topology optimization is the minimizing of weight of the continuum structure while maintaining deformations within acceptable limits.

Ali and White (2001) also introduced a computer aided approach for designing of D-regions of concrete structures using optimization approach to define the topology of the equivalent truss structure. In their optimization algorithm, two new features are included in order to avoid the generation of impractical reinforcement layout and to account stress redistribution.

The basic idea of the ESO method is also used by Kwak (2006) to determine more rational strut-and-tie models. In their method, the ESO method using truss elements are effectively used in finding the best strut-and-tie models in RC structures. Brick element composed of six truss elements is used as a basic structural element unit in order to prevent the structural instability that may occur during the evolutionary optimization process. Systematic removal of each brick element that has the least virtual strain energy is used to find the optimal load transfer mechanism of an RC structure, which is equivalent to the optimal topology of the strut-and-tie model. The optimization criterion of minimizing the total elastic strain energy of the structure is applied in the method.

All of the attempts discussed from the literature are focused on two dimensional STM. Liang-Jenq leu et al. (2006) discussed about STM methodology for three dimensional RC structures and they proposed refined ESO method (RESO) to develop STM in three dimensional space. This method also

utilizes the finite element model with given loading and support conditions and optimum topology is found from removal of ineffective materials which is determined from strain energy density of each element.
Although many attempt have been taken place in finding of necessary strut and tie model for D-regions of structural concrete members, most of the time they were end up with inefficient conclusions. On the other hand, the usage of finite element output is not completely used by any researcher although they get the help of FEM. In many cases only ESO method is and its modifications are governed the finding of necessary STM. As described by Schlaich et al. (1987) stress trajectories are straight forward to use to find STM. Although stress trajectories are used to identify STM, it is not easy to use for any cases especially when structure is too complicated. On the other hand it is not an easy task to identify the relevant STM through stress trajectories in three dimensional stress states as far as internal element stress trajectories cannot be visualized.
As a new approach, considering the FE analysis results, modified space truss model is proposed by Kanok-Nukulchai et al. (1996) especially for pile caps which bare even larger size of columns and any number of piles under pile cap. This modified truss model gives more realistic results taking in to account of column size and its location, pile stiffness and dimensions of the pile cap. In this model, the column axial loads and moments are assumed to be transferred to the pile cap at the corners of the equivalent rectangular column section which is bounded my main reinforcement cage. These column nodes is used to apply the equivalent loads determined from axial loads, moments and shear forces act at column base which is main advantage over simple truss models proposed by previous researches.

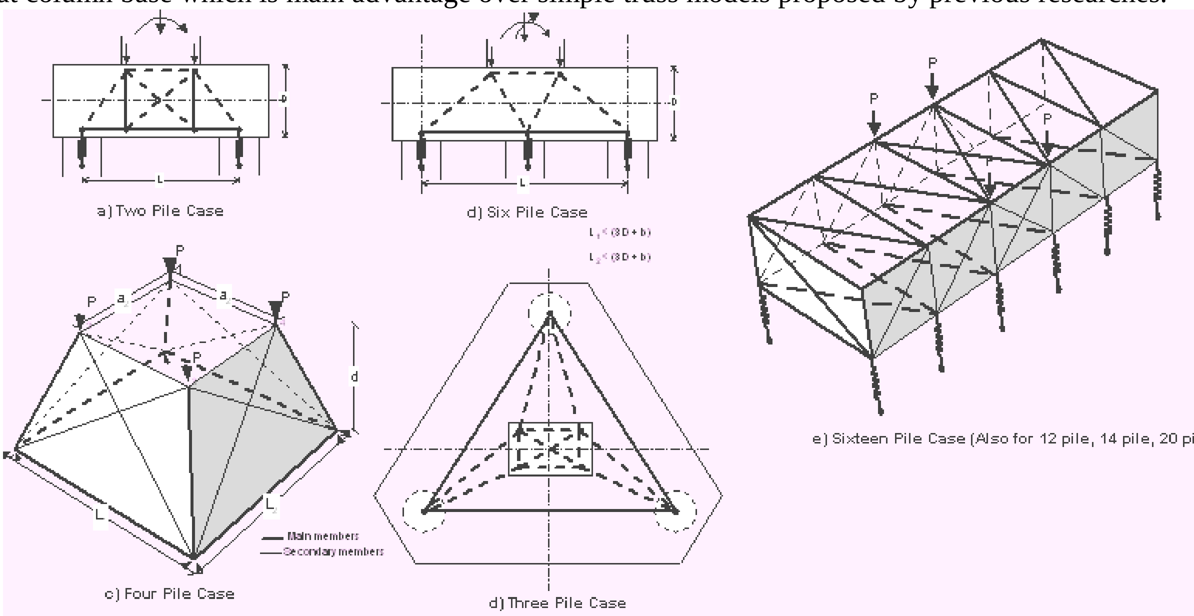


Figure 2.1: *Modified 3- D Truss Model for Pile Caps (Nukulchai & Anwar, 1996)*

With full usage of finite element mesh analysis results, Jason (2008) prepared the computer programming platform to extract STM in two dimensional structural members using stress trajectories obtained from shell element mesh finite element analysis. His study is limited to shell element mesh of shear walls and deep beams. As an extension of that research, present study is focused on to develop computer programming platform to extract STM for three dimensional structural components using 3D solid mesh FE analysis. Pile cap is the three dimensional structural D-region concrete member focused on this study which is analyzed using solid element mesh in order to extract necessary strut and tie model.

Present Methodology

The methodology followed by this study is mainly comprised of two major steps; Linear elastic analysis of selected structural component and developing of Visual Basic programming platform to extract the STM. Steps of the research methodology are adapted in schematic diagram below.

FINITE ELEMENT ANALYSIS OF STRUCTURAL MEMBERS

(1) Linear elastic analysis of selected structural members using solid element mesh in SAP 2000 V12

(2) Obtain the following data from FEA output results
(i) Solid element stresses at corner joints
(ii) Direction cosines of principal stresses at solid element corner joints
(iii) Solid element corner joints and centroid coordinates
(iv) Solid element joint connectivity
(v) Solid Element Properties

STM EXTRACTION PROCESS IN VISUAL BASIC PROGRAMINGPLATFORM

(3) Import and store data obtained from step 2

(4) Calculations
(i) Calculation of average principal stresses at solid elements centroids and corner joints.
(ii) Calculation of direction cosines of average principal stresses in each element

(5) Groupings
(i) Solid element grouping with nearly equal principal stress directions
(ii) Solid element grouping with nearly equal principal stress values which having nearly equal direction for strut and tie layout

(6) Display
(i) Averaged principal stress vectors of the structural member
(ii) Primary strut and tie layout

(7) Refining of primary strut and tie model based on
(i) Limit of stress magnitude(stresses less than specified percentage of maximum stress available are ignored)
(ii) Number of divisions in direction cosines variation
(iii) Number of divisions in stress range variation in particular direction
(iv) Size of the strut or tie

Refined strut and tie layout – Figures/Captions

3. Computational Aspects

Solid Element Stress Vectors Grouping with Nearly Equal Directions

After the averaged maximum absolute principal stress of each nodes and element centroids are extracted together with their direction cosines, the model becomes system of points which represent the solid element joints and solid element centroids with having a principal stress vector at each point. The next step is the grouping of stress vectors with nearly equal directions. Separate module is written to do the screening of all principal stress vectors according to their direction cosines. Since the directions of each and every stress vector are specified through three direction cosines, screening will follow one after another direction cosine values which range from -1 to +1. For simplicity, direction cosine values with respect to each global axis are divided in to number of equal groups which are ranged from -1 to +1. Once this step of screening is completed, the output is stress vectors and those are stored in separate cells which represent particular direction. The cells having no stress vectors are eliminated.

Solid Element Grouping with Nearly Equal Principal Stresses Magnitudes

Once the stress fields which are having nearly equal directions are grouped together, the next step is to identify the stress fields which are having nearly equal magnitude from previously screened nearly equal direction groups and grouped them together. The number of stress value groups belongs to one directional set can vary from one to any reasonable value in order to extract better strut and tie layout. Once the above two steps are completed, the output is separate stress paths of having nearly same magnitude and direction. Each of these stress vector groups represent either strut or tie member depending on sign of the stress. In the computer algorithm, all of these screened stress vectors stored

in three dimensional data structure array and these data is used for graphical presentation of strut and tie model and it is called primary extracted strut and tie layout.

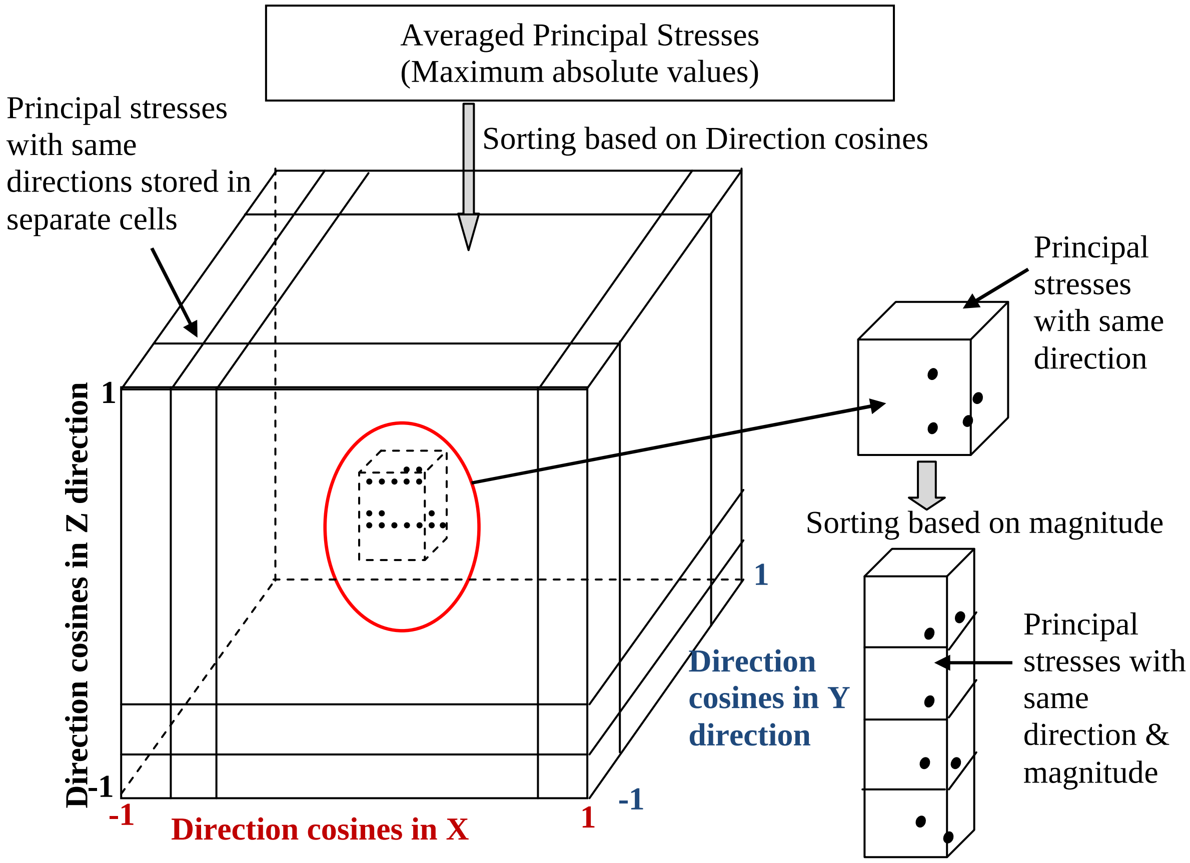


Figure 3.1: *Graphical Representation of Grouping Process*

Definition and Graphical Representation of Primary Strut and Tie Members

Once the grouping of stress vectors completed, all the coordinates of the stress vector points which follows the same strut or tie member is known. The next step involved in STM extraction is identifying of those struts or ties by sizing them. The first step involved in identification of particular strut or tie is the aligning of all the points belongs to that particular stress fields with z (vertical) axis and bring it in to axis origin. Then all the stress vectors are directed in to vertical direction and scattered along the vertical axis. Then plan area of those scattered points is divided in to grid introducing sufficient grid spacing in which each cell in the grid represent the cross section area of the strut or tie. Then, the stress field is divided in to vertical tubes having rectangular cross section based on above grid spacing. The stress points belongs to separate rectangular tubes are identified as strut or tie according to sign of their stress. In here, tubes with no stress vectors can be eliminated and length of strut or tie determined based on lower and upper stress points belongs to particular stress tube. The axis of the tube is considered as axis of the strut or tie and those axes are again rotate back to their original positions in order to locate the strut or tie in their respective positions. This procedure is followed for all stress fields sorted out in order to extract the primary strut and ties layout. In the graphical representation, those strut or ties are displayed by their center lines.

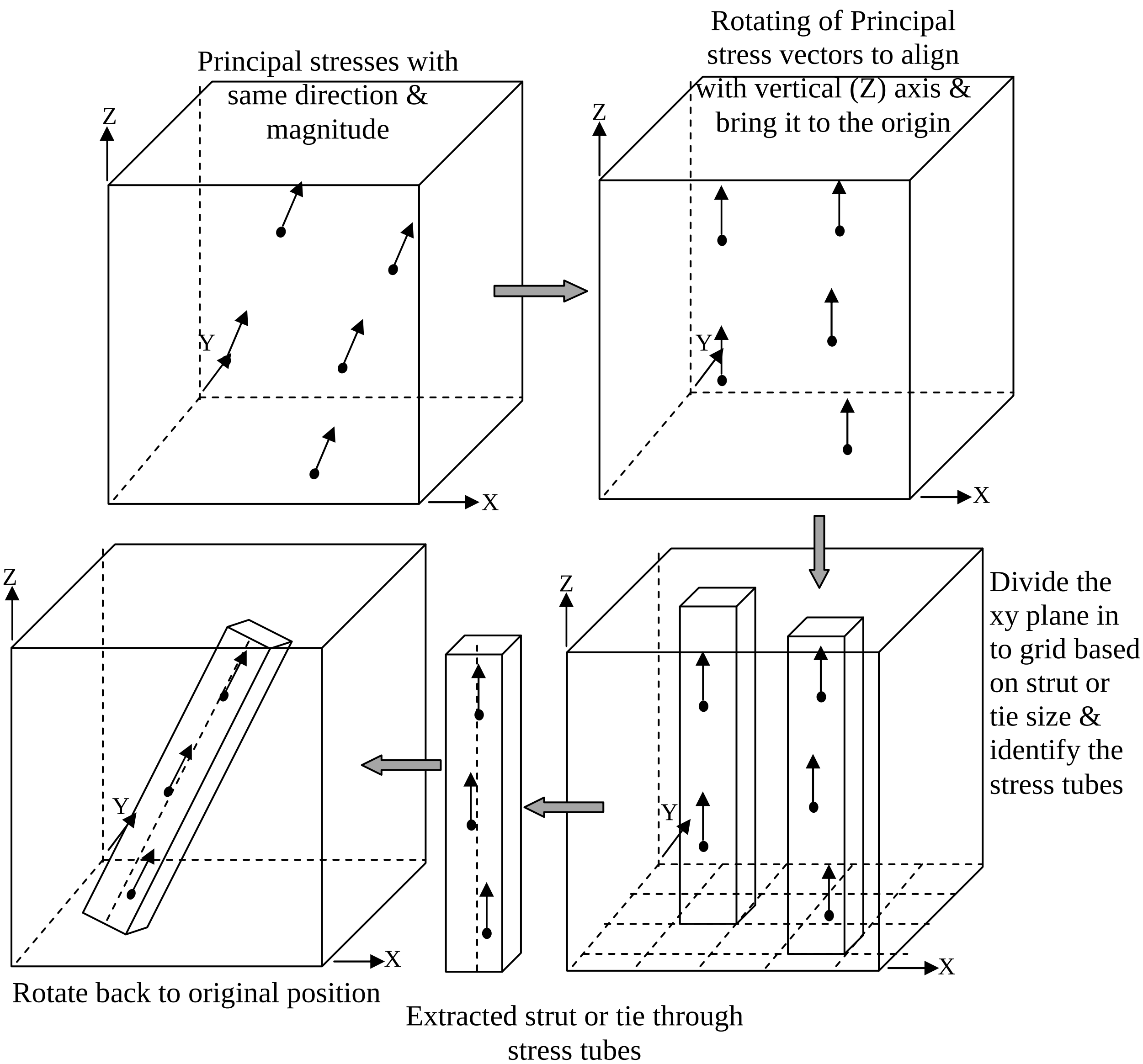


Figure 3.2: *Graphical Representation of Strut and Tie Extraction Process*

Refining of Primary Strut and Tie Layout

Once the primary struts and tie layout is extracted, the refinement of it can be achieved by varying the governing parameters involved in extraction process for better strut and tie layout. There are four parameters introduced in extraction algorithm;

(i) Stress limit tolerance
(ii) Number of divisions in direction cosines
(iii) Number of divisions in stress range
(iv) Strut or tie size

Once this refinement is done, the output is refined strut and tie layout and it is displayed in graphically in program output.

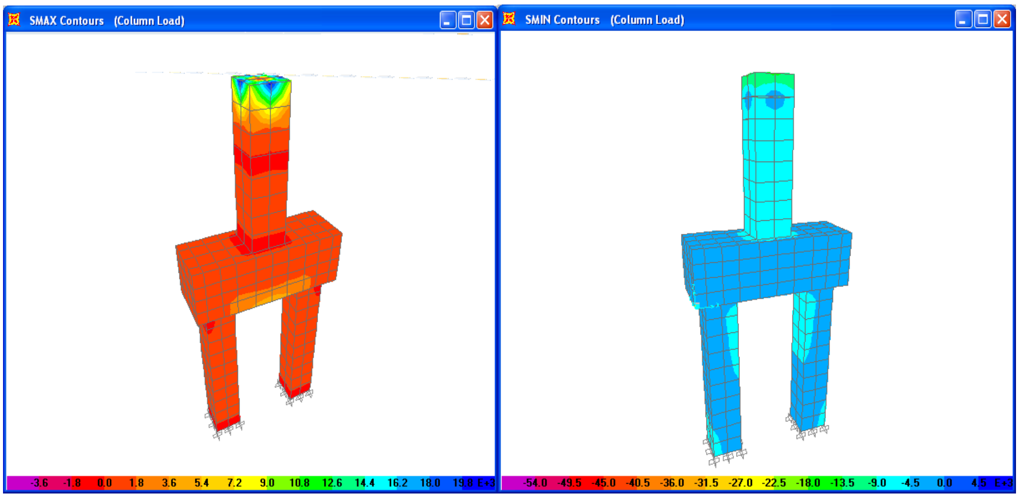
4. Results and Discussion

General

Based on present implementation, the major output coming from the process is appeared in two forms in which graphical interface and text file format. Four graphical views can be viewed through the graphical interface. It includes view of the of stress trajectories of the structural member, view of stresses in directed slots, view of compact directed stress field and view of strut and ties. View of strut and tie is the major output concern in present study. The data text files output coming from the process include details of directed stresses, details of directed stress groups and details of strut and ties. The details of strut and ties include direction cosines and coordinates of the extracted strut and ties together with their stress values and it is used to model strut and ties in SAP 2000. The output results of nine pile cap configurations and three pier configurations modeled and analyzed in SAP 2000 with solid element mesh are used to verify the present implementation and some of the results are adapted in here.

Two Piles-Pile Cap

Two piles-pile cap taken from usual construction practice modeled in SAP 2000 and analyzed with the application of point load at the column center and output data files used in STM extraction process.



(a)Principal Compressive Stress (b)Principal Tensile Stress

Figure 4.1: ***Principal Compressive Stress & Tensile Stress Contours of Two Piles-Pile Cap***

Although bottom tensile zone of the pile cap is cleared from the stress contours, compressive stress flow is not cleared. This is because of usage of solid element for modeling of piles and column. Since column is modeled using solid element, the load coming from column spreads at the top of the pile cap. Normally column and pile caps are modeled using frame elements as it is better to represent axial load transferring members.

The refining configurations of direction cosine divisions=4, Stress range divisions=4, Strut size=0.5 and Stresses > 6% of Maximum stress are used in extracting the above strut and tie layout. Although many strut and tie members still present in the layout, the basic expectation of triangular shaped strut and tie layout is achieved.

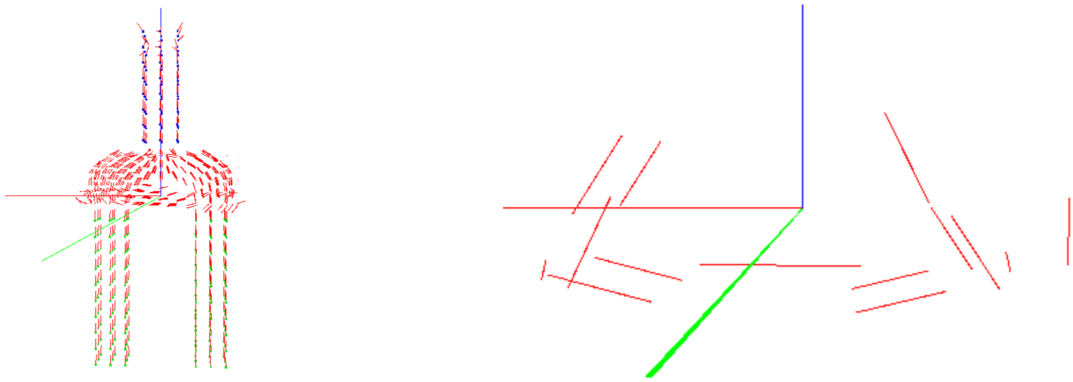
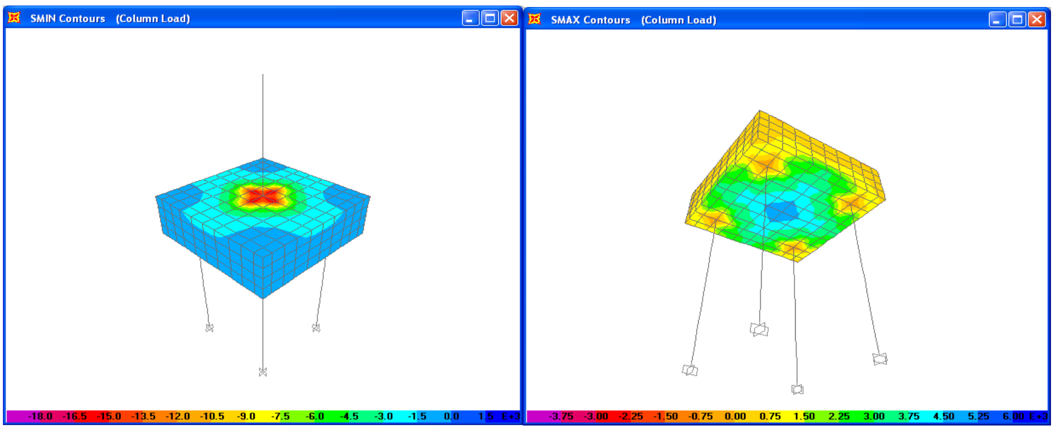


Figure 4.2: ***Stress Trajectories & Refined Strut and Tie Layout from Program Output***

Four Piles-Pile Cap

Four piles –pile cap configurations of having span to depth ratio varying from one to four are considered in the strut and tie extraction. Initially results from all the pile caps modeled from solid element including piles and column are used to test the extraction process but it doesn't show better results. Then, piles and columns in all models are replaced by frame elements to overcome that problem.

The commonly used pile cap configuration of having span/depth is equal to two is considered as first four piles pile cap case.



(a)Principal Tensile Stress (b)Principal Compressive Stress

Figure 4.3: ***Principal Tensile & Compressive Stress Contours of Four Piles-Pile Cap***

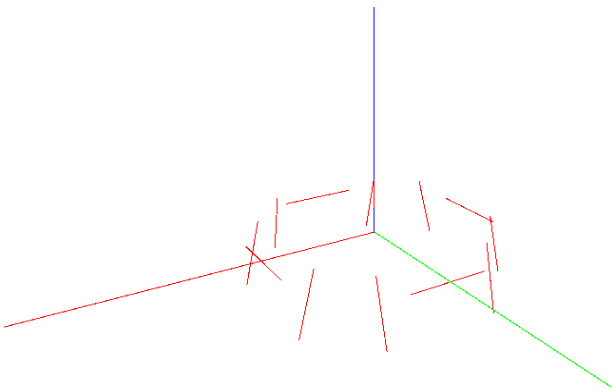


Figure 4.4: ***Refined Strut and Tie Layout for Four Piles Pile Cap (Span/Depth=2)***

The refining configurations of direction cosine divisions=4, Stress range division=2, Strut size=1.2 and Stresses >11% of Maximum stress are used in extracting this strut and tie layout. As it is shown in above figure two inclined struts can be seen in each corner of the pile cap while horizontal tie members lie in between each corner. Although the inclinations of struts are not sufficient to intersect within the body of the pile cap, it can be idealized as a shape of pyramid when each corner struts are represented by a single strut. According to Adebar et al., (1990), the shape of the simple 3D strut and tie model for four piles pile cap is pyramid shaped as shown in Figure 4.8.

Almost similar to pyramid shape four inclined strut layout is clearly shown in program output for the case of four piles pile cap with span/depth is equal to one. This implies that formation of struts possible in rather deep members compared with shallow members.

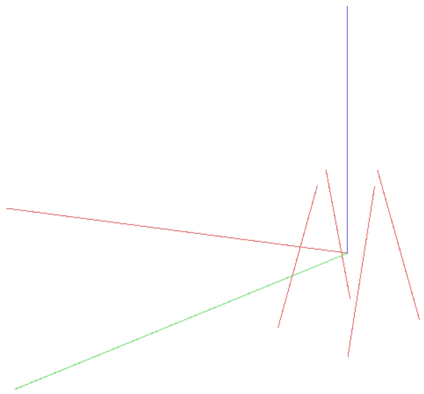


Figure 4.5: ***Refined Strut and Tie Layout for Four Piles Pile Cap (Span/Depth=1)***

The above shown strut configuration is extracted through the use of refining criteria of Direction cosine division=3, Stress range division=2, Strut size=1.5 and considering stresses greater than 15% of maximum stress present in the system.

All other pile cap configurations with four piles considered are rather shallow members compared to the above two cases. So, only tension ties at the bottom of the pile cap are cleared in extracted layout as bending is governing action in such members.

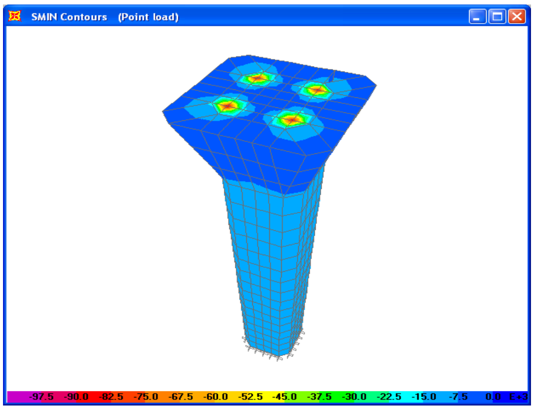
Pier Configurations

The use of strut and tie model for designing of D-region members is the present practice as their stress distribution is complex. Apart from the pile cap configurations, the validity of the results coming out from proposed strut and tie extraction algorithm is checked against such irregular and complex pier configurations used in many of the elevated structures built in the vicinity of the city of Bangkok. Three of such pier configurations are considered in this study. The first one is the common pier configuration used in elevated railway track in Bangkok. The second case is the pier with relatively thick pier head spread over large area. These types of structures are common in most of the interchanges and other locations of elevated highway structures. The third case considered is pier with curved pier head which is commonly used in U-turn bridge structures.

Common Pier Configuration used in Elevated Railway Track in Bangkok

Toller pier with tapered head is modeled by solid elements and fixed support condition is used at the bottom. The load coming from both side spans are applied as point loads acting on pier head as similar to the real situation and results coming from linear static analysis is used as input for the extraction algorithm.

The program output gives the strut and ties layout clearly showing four inclined struts at pier head region and four vertical struts at stem region as shown in following figure. This strut layout is extracted with refining criteria of Direction cosine division=3, Stress range division=1, Strut size=1 and neglecting stresses less than 3% of maximum stresses present in the system. The strut layout modeled in SAP 2000 by removing the small strut and tie element present within the body of the stem is also shown in other figure and it is evident that the program output extract the struts that show the path of the compressive stress flow of the structural member.



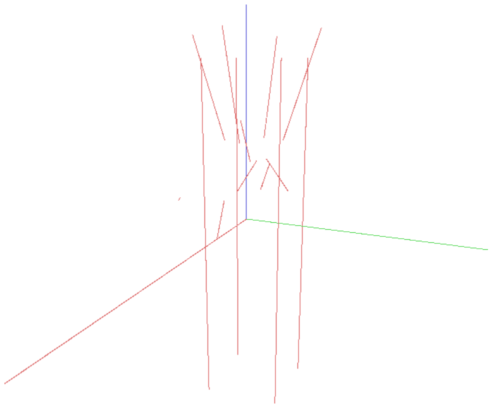
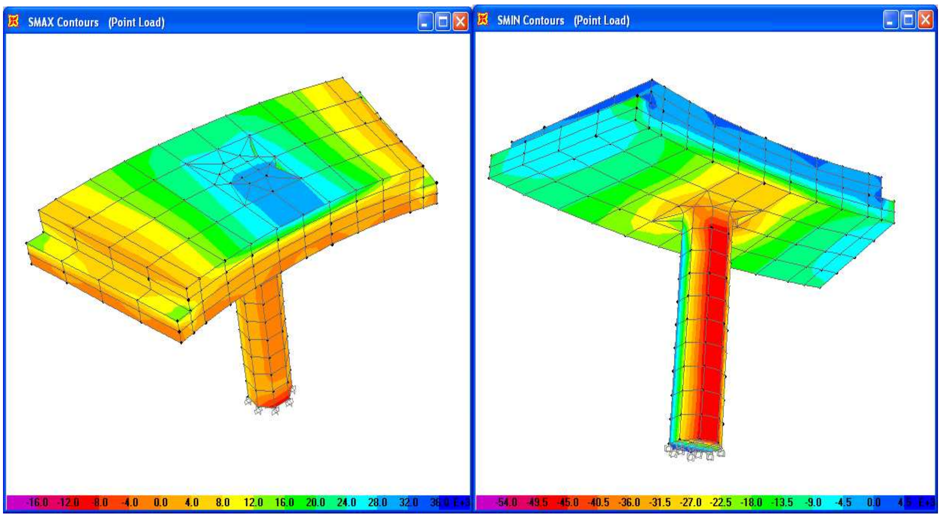


Figure 4.6: ***Principal Compressive Stress Contours of Pier Head under Point Loading & Extracted Strut and Tie Layout***

Pier with Curved Pier Head

Similarly as previous pier configurations, curved head pier is modeled with solid element and bridge girder loads coming from both sides spans of the pier are applied as point loads on relevant locations. A linear static analysis result of the model is used in extraction of possible strut and tie layout for the pier head configuration. Pictures of solid element model, principal stress contours due to girder loads and extracted strut and tie layout are shown in following figures.



(a)Principal Tensile Stress (b)Principal Compressive Stress

Figure 4.7: ***Principal Tensile Stress & Compressive Stress Contours of Curved Pier Head***

Tension ties on top of the pier head and vertical compressive struts at stem of the pier can be seen from the extracted strut and tie layout which is match with principal stress contours of the pier. But compressive struts cannot be clearly seen at the bottom of the pier head. The above strut and tie layout is extracted through the use of refining criteria of Direction cosine division=4, Stress range division=4, Strut size=1 and neglecting stresses less than 5% of maximum stress present in the system.

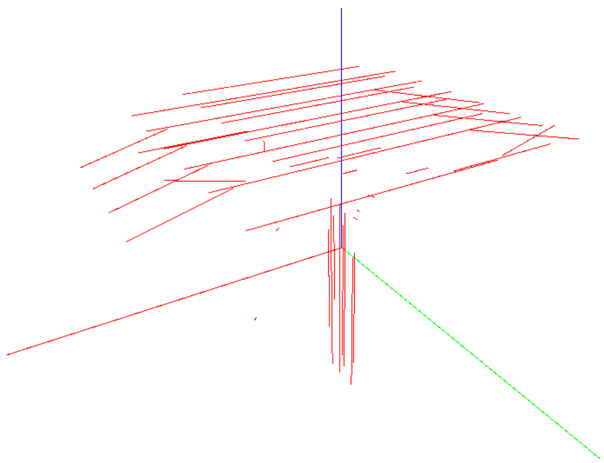


Figure 4.8: ***Strut and Tie Layout for Pier with Curved Pier Head***

5. Conclusions

Based on results and experienced gained through the present study, following conclusions are drawn.

(i) Solid element mesh analysis can display the internal stress flow of three dimensional structural members and initial strut and tie layout can be visualized through it.
(ii) The proposed method can extract the possible strut and tie member layout that match with internal stress flow of three dimensional disturbed region members.
(iii) Further modifications are required to improve the quality of the results.

Acknowledgement

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INFLUENCE OF GRASS COVER ON FLAT REINFORCED CONCRETE SLABS IN A TROPICAL CLIMATE

B.S.S.S. Dareeju[1], J.N. Meegahage[2], R.U. Halwatura[3]

[1] Department of Civil Engineering, University of Moratuwa, Moratuwa, Sri Lanka.
[1]E-mail: sampathdareeju@gmail.com, [1]Telephone: +94773405138

[2]Department of Civil Engineering, University of Moratuwa, Moratuwa, Sri Lanka.
[2]E-mail: mjeewana@gmail.com, [2]Telephone: +94718193531

[3] Department of Civil Engineering, University of Moratuwa, Moratuwa, Sri Lanka.
[3]E-mail: rangika@civil.mrt.ac.lk, rangikauh@gmail.com
[3]Telephone: +94-11-2650567-8(Ext.2212); Fax: + 94-11-2651216

Abstract: Reinforced concrete slab is a better alternative to regain the land due to the buildings because available land is insufficient for usage with respect to the rate of urbanization. However, the reinforced concrete roof slabs do not perform in acceptable level in tropical countries with warm humid climate condition and tend to act as heated bodies which emit long wave radiation to the occupants. As a solution, a green roof is proposed and its thermal performance was determined experimentally using small scale models. Using experimental results, large scale building models were developed to simulate the indoor thermal performance of green roofs and it is shown that the green roof provide satisfactory indoor thermal performance while providing many other benefits such as enhancing aesthetic and regaining lost land for the building.

Key words: Reinforced concrete slab, Green roofs, Indoor thermal performance, Warm humid climate

1. Introduction

The rapid increase in human population over the 20th century has raised concerns about whether Earth is experiencing overpopulation. The scientific consensus is that the current population expansion and accompanying increase in usage of resources is linked to threats to the ecosystem [1]. As a reason of rapid increase in this human population and industrialization, most of facilities such as money, services and wealth were centralized into the cities. Cities are where fortunes are made and where social mobility is possible. According to the UN State of the World Population 2007 report [2], in 2008, the majority of people worldwide migrated to towns or cities, for the first time in history; this is referred to as the arrival of the "Urban Millennium" or the 'tipping point' [3].

Therefore, available land per person is decreasing rapidly in most of developing cities and value of a land is also increasing unbearably. This leads to increase the usage of the available land in an effective manner and people tends mostly to construct high-rise buildings with flat reinforced concrete roofs to gain maximum usage from a minute area. Although this is a more desirable option than having a typical roof with a ceiling, it will create a few additional needs with respect to occupant thermal comfort [4].

However, the concrete roof slabs do not perform satisfactorily in warm humid tropical climate conditions [5] because they act as heated bodies and emit long wave radiation into the occupants in the day time. As a result, indoor thermal comfort of the buildings which having concrete roof slabs becomes fewer and passive features should be included as possible to enhance the indoor thermal comfort in these buildings as using robust roof slab insulation system and enhancing micro climate conditions. The green roof is also a superior alternative for augmenting the indoor thermal performance of a building. It was found that the rooftop temperature can be reduced by about 15^{0}C depending on the vegetation cover, which reduces the transmittance of solar radiation up to 50% [5]. In general, the green roof performs as a capacitive insulation and also short wave radiations are absorbed by the grasses. Hence, the heat flow into the concrete roof slab is insufficient and it enhances the indoor thermal performance in the building in addition to provide other benefits such as enhancing the aesthetic and regaining the land which is lost due to the building.

When introducing an alternative such as green roofs for the buildings as a substitute of reinforced concrete roof slabs, it is better to carry out a comparison between the green roof slabs and the reinforced concrete roof slabs to evaluate indoor thermal performance of each case. It may help to determine the real benefits of the green roofs rather than using artificial ventilations to achieve a better indoor thermal comfort condition in buildings.

2. Objectives and methodology

The main objective of this research is to determine the influence on the indoor thermal performance of a building by introducing a green roof in a warm humid climate condition. The following methodology was used to achieve the above objective:

a) The temperature measurements on actual models with green roof slab by having soil thicknesses of 25mm, 50mm and 75mm were used to find out the heat flow characteristics for computer simulations.

b) Computer simulations were carried out for a typical building with flat concrete roof and flat concrete roofs with green roof slabs.

c) A comparative study was carried out using the above simulation results for reinforced concrete roof slabs and green roof slabs with different soil thicknesses to determine the effect of the sensible heat component on the indoor thermal performance in a typical building.

3. Behavior of heat flow on a green roof

The initial investigation was to determine the indoor thermal comfort conditions in four small scale models. Each model consisted of 125mm thick concrete slab, two 125mm thick cement block walls in each side and the length and the height of walls are 1.25m and 0.5m respectively. A 10mm space was also provided at the top to represent the ventilation condition in a building. Buffalo grass (Bouteloua dactyloides) was used for those models. Buffalo grass is the very native turf grass type for a tropical country because Buffalo grass may reduce the cost for irrigation, maintenance and also it is less vulnerable to the grass diseases.

Four models were utilized to measure the heat flow through a green roof. One model was kept as concrete slab and other models were covered with soil layers of 25mm, 50mm, and 75mm respectively, which were rich with sand to provide a good drainage condition. Figure 1 shows the arrangement for obtaining the temperature measurements.

Figure1- Four models used for the data collection

The slab soffit temperature, slab top temperature, indoor temperature were obtained on a typical sunny day for following cases as without grass cover (case 1) and with a grass cover with soil thickness of 25 mm (Case 2), 50 mm (Case 3) and 75 mm (Case 4) respectively.

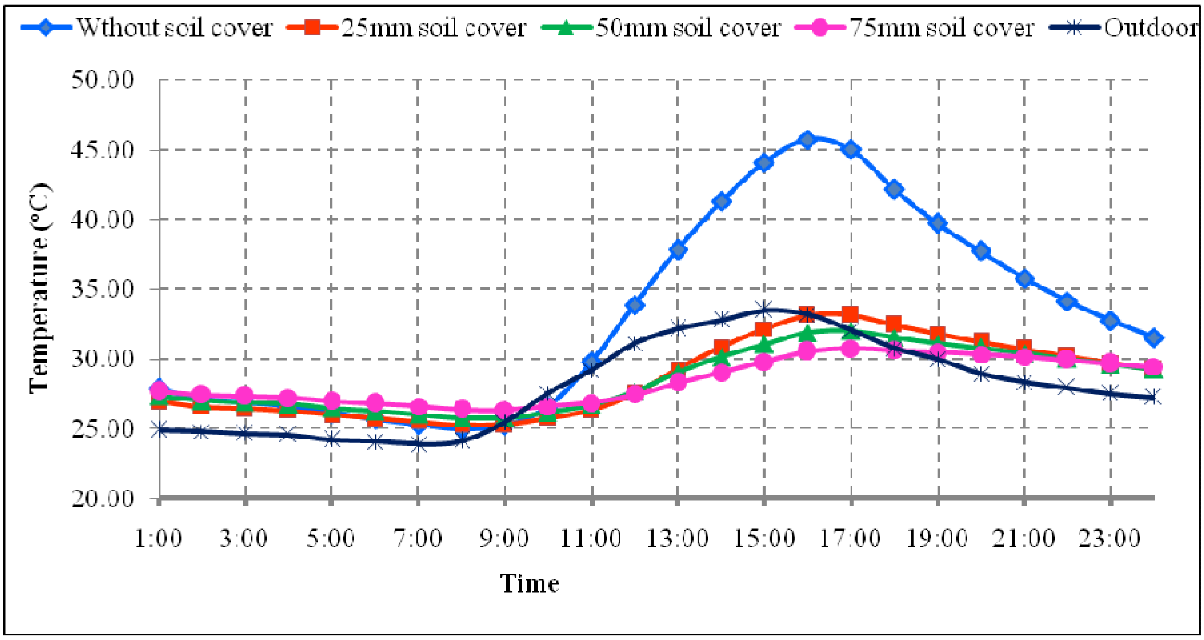

Figure 20- The temperature distribution at the Slab soffit for different soil thicknesses

Figure 2 shows that the slab soffit temperature variation in a typical sunny day for 24 hours. During day time, the maximum slab soffit temperatures of the models without soil cover, with 25mm, 50mm and 75mm soil cover are 45.7°C, 33.2°C, 32.0°C and 30.7°C respectively. According to the slab without soil cover emits considerable amount of long wave radiation and green roofs are not emitting much long wave radiation during day time because they are almost equal to the maximum outdoor temperature of 33.5°C. During night time, soffit temperatures of each model are approximately the same.

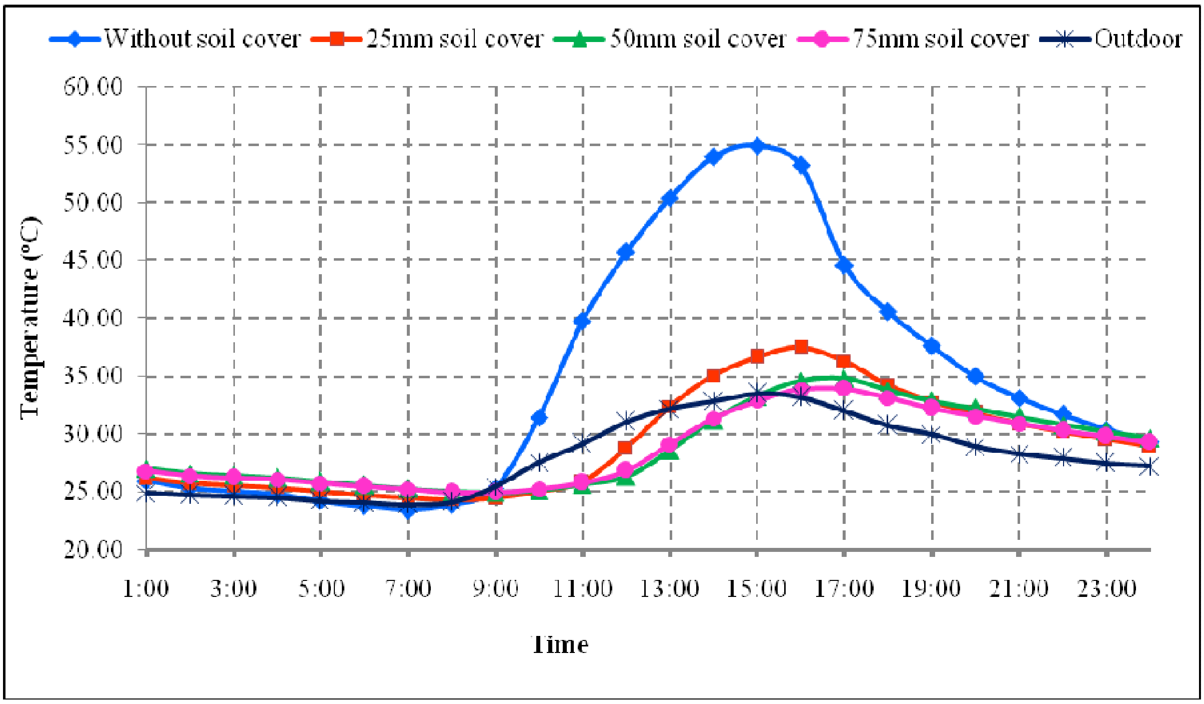

Figure 21-Surface (slab top) temperature distribution for different soil thicknesses

Figure 3 shows the slab surface temperature variations of four models for 24 hours and they also show a similar behavior as the slab soffit temperatures of the models. However they have higher temperature values than the slab soffit temperatures. The maximum slab top temperature reached up to 54.9^{o}C without soil cover slab because it directly exposed to the sun and absorbed a considerable amount of short wave radiation during day time. The maximum slab top temperatures of the models with 25mm, 50mm, and 75mm thick soil covers are 37.5 ^{0}C, 34.8 ^{0}C, and 33.9 ^{0}C respectively. Main reasons for having less slab top temperatures are the thickness of soil cover and the absorption of short wave radiation by grass for the photosynthesis.

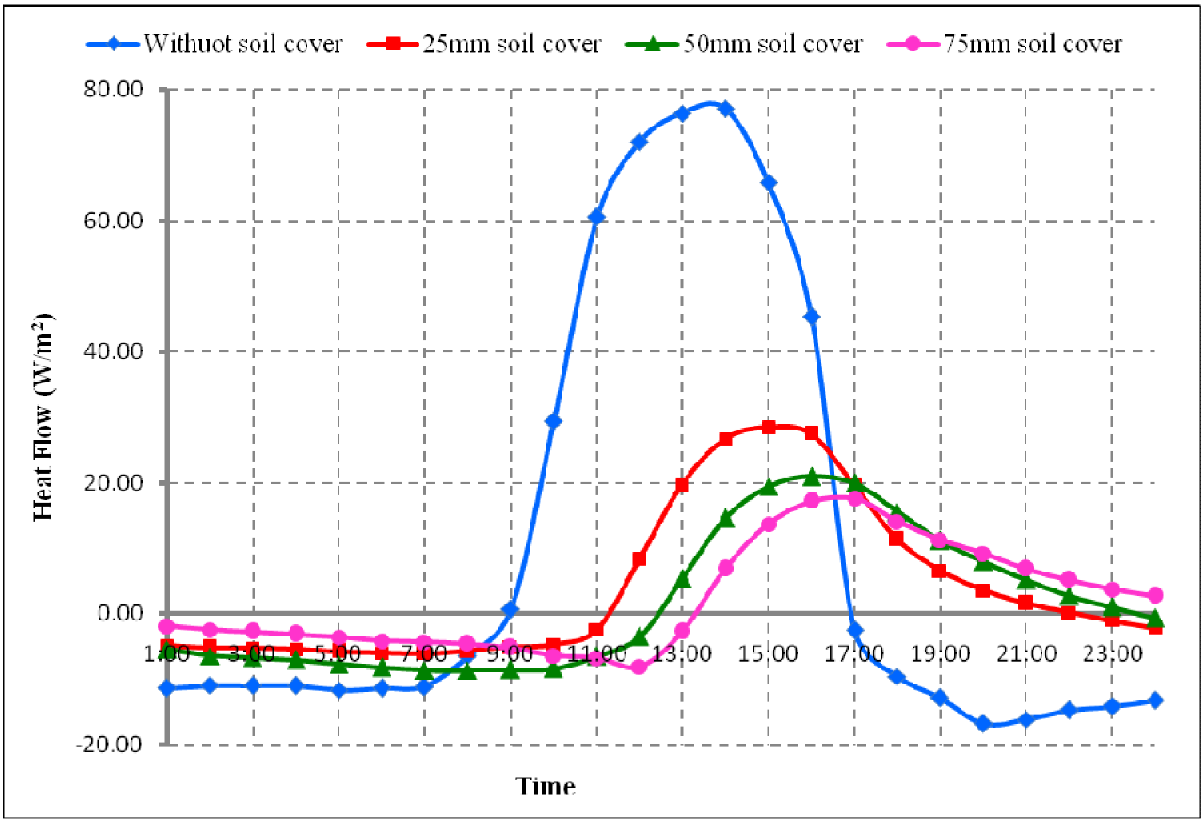

Figure 22-Heat flow values for different soil thicknesses

The heat flow values were calculated using "U" values, called the overall heat transfer coefficient of the materials. The detailed calculation of "U" values is presented in Appendix A. Figure 4 shows the heat flow variations for the models with soil covers and without soil cover. The maximum heat flow of the model without soil cover reached 77.1 W/m^2 and there is a heat flow towards the occupants from 09.00 hours and 17.00 hours. Therefore, in the day time occupants in the buildings having only a flat concrete slab, feel more thermal discomfort due to this kind of heat flow. However, the maximum heat flow obtained was 28.5 W/m^2 for 25mm, 20.9 W/m^2 for 50mm and 17.6 W/m^2 for 75mm thick soil cover models. This is a great amount of heat flow reduction, compared to the buildings having only roof slab. When referring this Figure 4, it can be easily proved that the green roofs performance better than unprotected slabs even in night times. Uncovered slab can reverse the heat flow from 17.00 to 9.00 while in green roofs remains approximately constant.

4. Computer simulation for the models

Computer simulations were used to predict the thermal performance of a building by varying its properties. DEROB-LTH software was utilized and it is a widely used software simulation package in creation of models of actual buildings. DEROB-LTH is capable of calculating thermal energy transmission across the building using of the building properties such as thickness of the layers of the wall and material types. The calibration detail of DEROB-LTH can be found in Halwatura and

Jayasinghe [5]. This has also been validated by many researchers [6, 7, and 8]. In this study computer simulations were used to predict the thermal performance of green roofs.

4.1 Model used

Physical models for computer simulation which described in section 3.0, were modelled using DEROB-LTH and analyzed to obtain the properties of the grass cover. After obtaining the properties of the grass cover, those results were used to analyze the large scale computer models. For large scale models, a two story scale building model was used for simulations because the upper floor of a two storey house is severely affected by the direct solar radiation than a single storey house. The plan area of the house models were 11.5m * 7m. The same house model was simulated with a flat slab and green roof as the roofing system. The aim of research was to investigate the performance of a green roof as a passive technique; therefore, the models were created including all possible passive techniques. The passive features that were included in the models are as follows:

- Windows were faced to Southern and Northern directions, so that the direct solar radiations can be minimized as much as possible
- Shading screens were used to protect the windows from the direct radiation
- 225 mm thick brick walls were selected to reduce the heat transfer through the exposed walls [9]

Such passive features generally have the potential to reduce the indoor air temperature by about 3^0C below the outdoor temperature [10]. The other possible passive features such as courtyards [8] were not considered because they may not be as common as the above mentioned passive features. For the two storey houses with a green roof, a 115mm thick reinforced concrete slab was used at the top floor and a soil layer was introduced over the top slab. This was taken as the case GR. For flat slabs, the top slab of the house models was taken as the roofing system. The thickness of the slab is 115mm and the bare flat slab was considered as the roof. This was taken as the case FS.

4.2 Different cases considered

In this paper two types of roofing systems related with roof slabs were discussed. They were flat slabs and green roofs. For the green roofs three different cases were considered while for the flat slab was considered along. For each case the indoor thermal performance was calculated using DEROB-LTH. The three different cases considered for simulations were as follows.

Case GR1: Green roof slab with 25mm thick soil layer
Case GR2: Green roof slab with 50mm thick soil layer
Case GR3: Green roof slab with 75mm thick soil layer
Case FS: Roof with a flat slab

Figure 5 and figure 6 show the DEROB models used for the simulation.

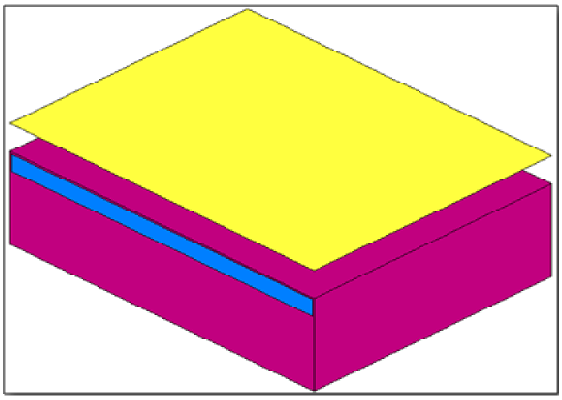

Figure 5- Field model used for the computer simulation

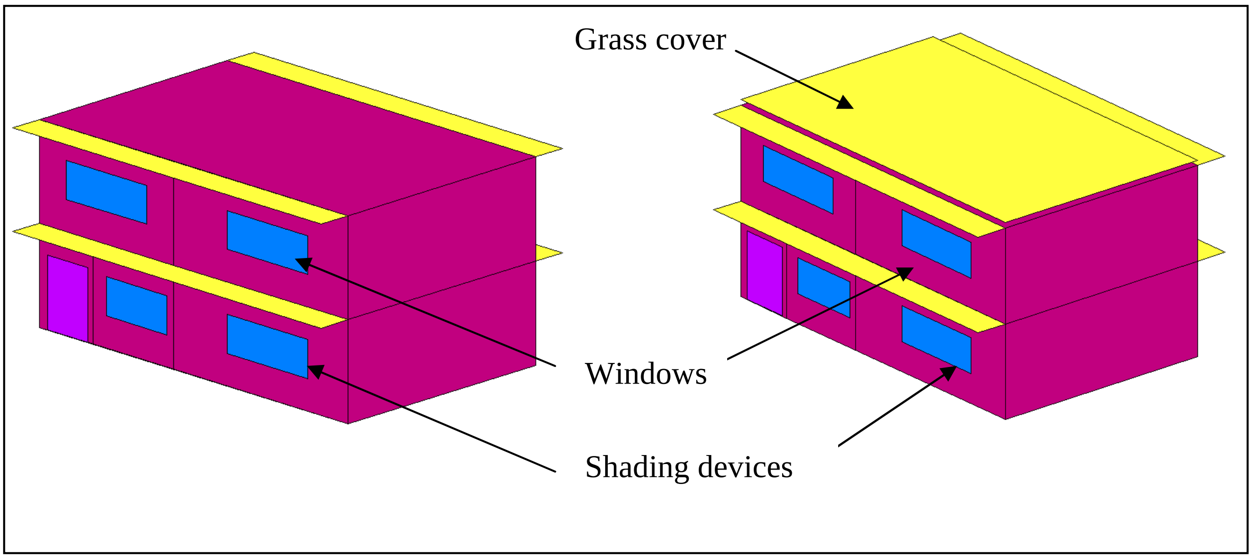

Figure 6- Large scale house models used for the computer simulation

5. Results and Discussion

5.1 Performance of field model

Figure 7 shows the indoor temperatures variation of the field model with a 50mm soil cover. The data was taken in three bright sunny days for a period of 12 hours. In all three days the maximum indoor temperature of the field model was about 32^{0}C. Also the maximum temperature was observed around 14-16 hrs in each day.

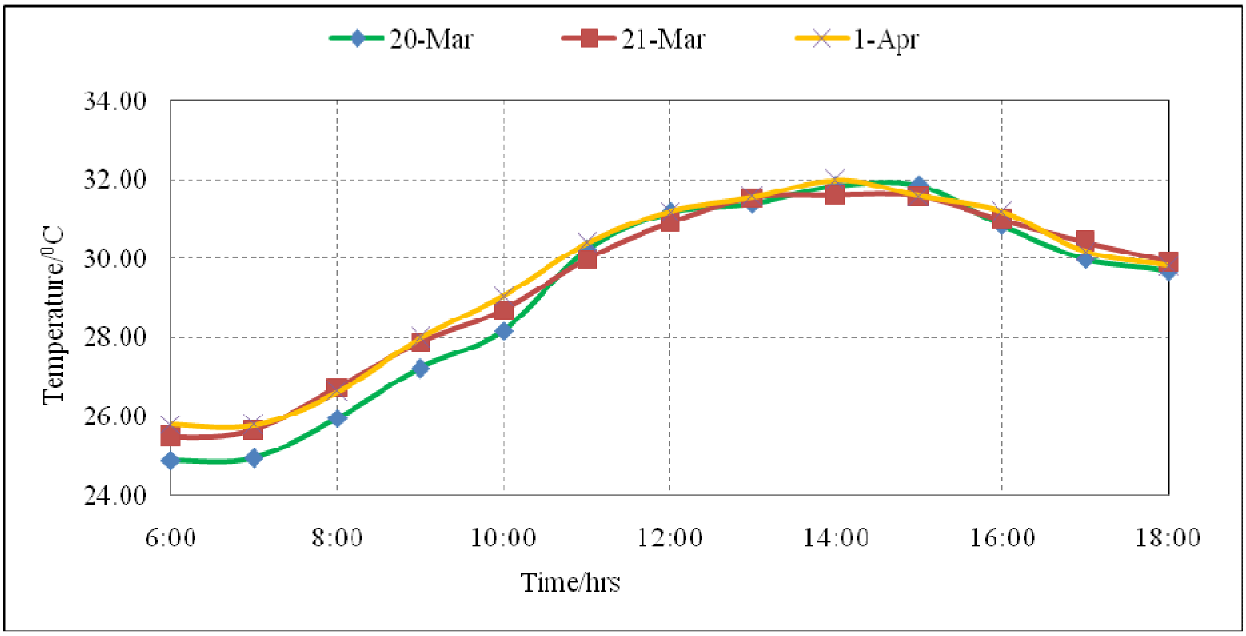

Figure 7- Indoor temperature of the field model for bright sunny days

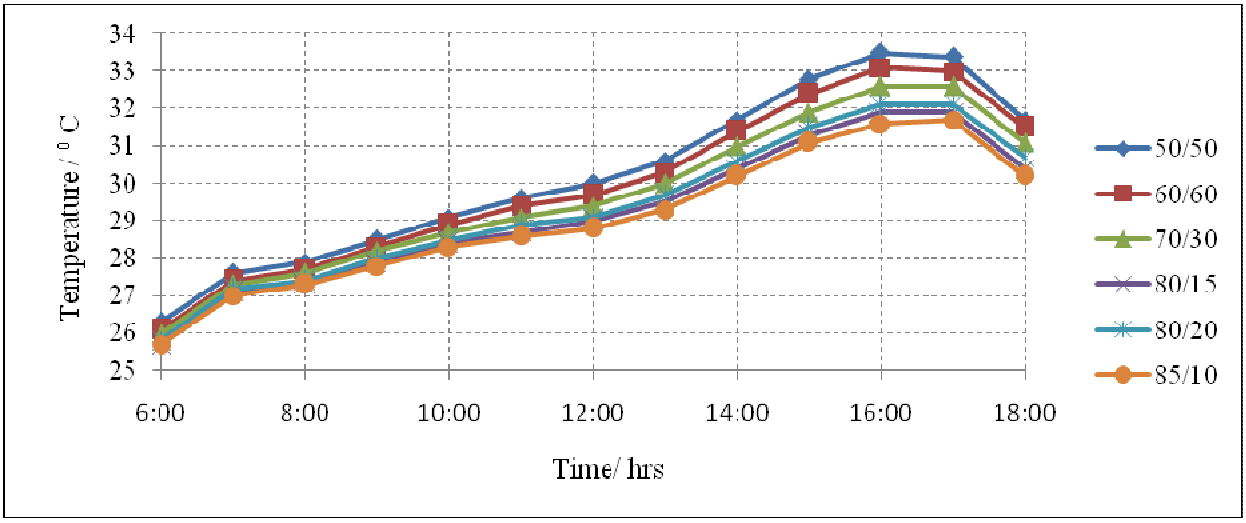

Figure 8- Indoor temperature variation of the DEROB model with different absorptivity/transmisivity

Figure 8 shows the results of the computer simulation that was carried out for the field model.

The indoor temperature variation of the small model with 50mm soil cover was obtained by varying the properties (absorptivity and transmitivity) of grass cover. As shown in Figure 8, the indoor temperature of the computer model also increases with the increase of transmisivity. As mentioned in the previous paragraph, the maximum temperature obtained in the field model was about 32^{0}C. In the computer model, for absorptivity 80% and transmitivity 15%, the same behaviour can be observed as in the field model. For the simulation with large scale models, this data has been used in this paper.

5.2 Performance of the large models

Figure 9 shows the indoor thermal behaviour of a ground floor of the large scale model. The figure shows the indoor temperature of the ground floor is at a low value than the outdoor temperature in the daytime. The variation of the indoor temperature with the variation of the soil cover does not show any significant difference. For all three different soil covers the indoor temperature remains the same. Another important observation in this graph is the variation in indoor temperature is very low. The daily indoor temperature of the ground floor is in the range of 27.6^{0}C and 29.4^{0}C. This will facilitate for more comfort indoor thermal conditions for the occupants.

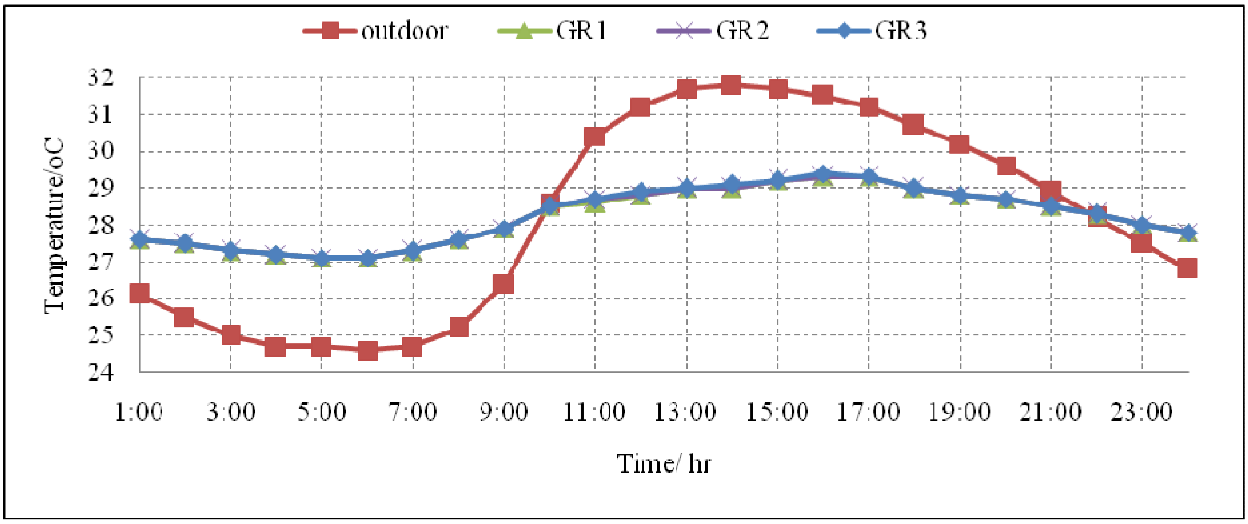

Figure 9- indoor temperature variation in the ground floor for different cases

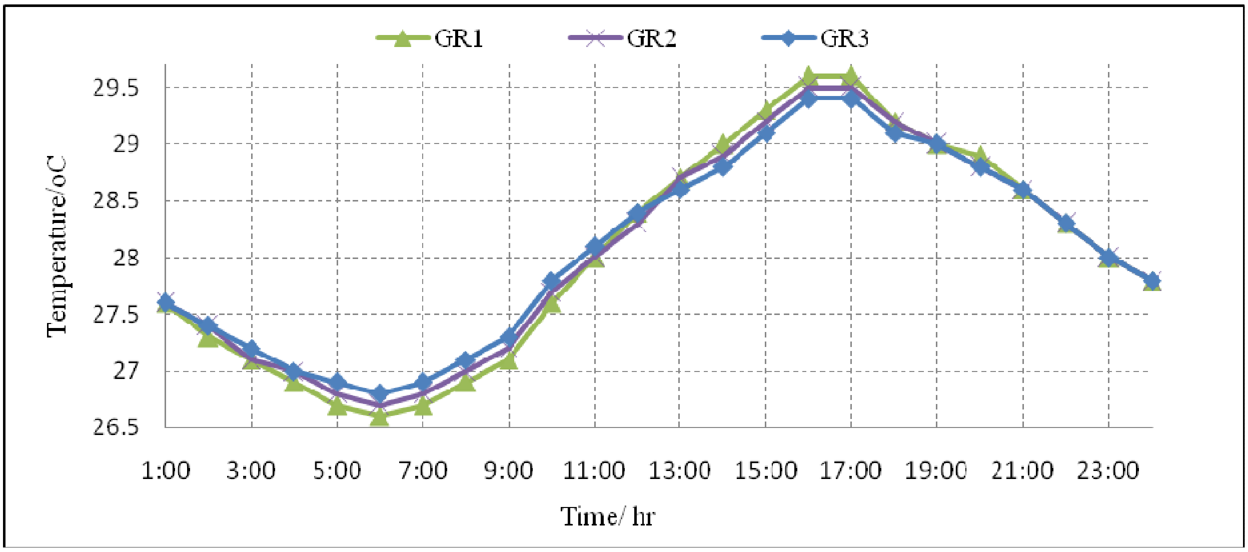

Figure 10- Indoor temperature variation in the upper floor for different cases

Figure 10 shows the indoor thermal behaviour of green roof for the upper floor of the computer model. As Figure 9 indicates, the outdoor temperature reaches a maximum of 32^0C in the daytime. In Figure 10, it shows lower temperatures in indoor than the outdoor. With the increase of the soil thickness, the indoor temperature also decreases. This is because of the capacitive insulation behaviour of the soil cover. Sri Lanka is a country which experiences tropical climatic conditions and in low altitude the neutral temperature can be taken as 26 ^{0}C [9]. The indoor temperature of the upper floor ranges from 27-29.6^{0}C in the above green roofs. This is close to the basic comfort temperature of the Sri Lankan low altitudes.

Case GR1 has the highest indoor temperature among all three cases while GR3 shows the best indoor thermal performance. For GR1, the growing performance of the grass cover should be low due to lower thickness of soil cover. GR3 gives good healthy grass cover while having a high construction costs due to a thicker soil layer.

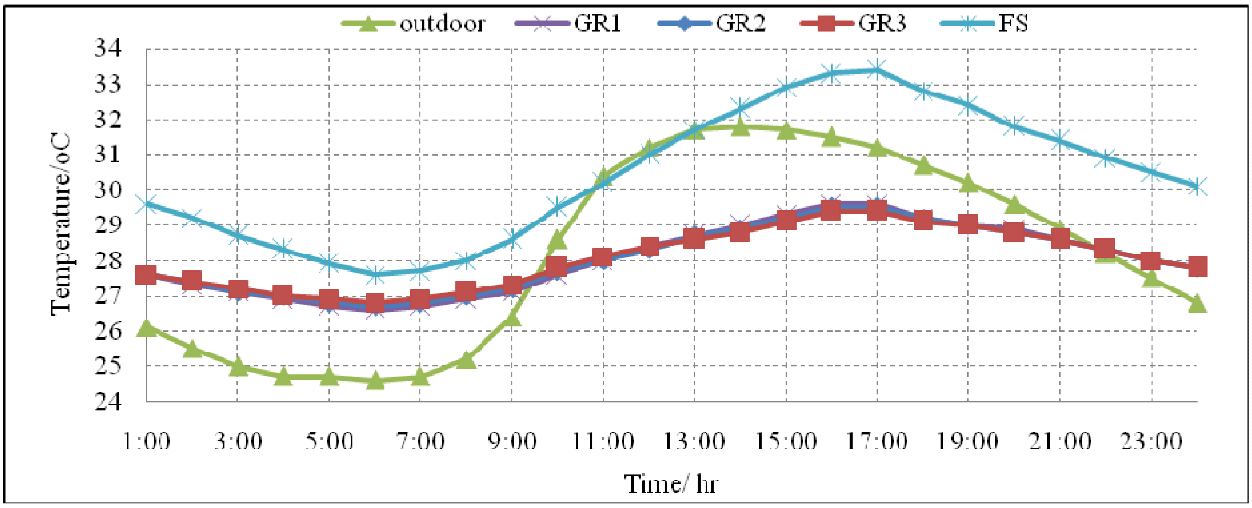

Figure 11- Indoor temperature variation in the upper floor for different cases in green roofs and flat slabs

Figure 11 indicates a comparison of houses with green roofs and flat slabs. It can be seen that the green roofs are much better in thermal performance than bare flat slabs. The indoor temperatures of the flat slab remain higher than in the green roofs almost all the day.

It is important to study the indoor thermal behaviour of flat slabs and green roofs when they get older. In flat slabs, the absorptivity and transmitivity increases with aging. Hence, the indoor temperature of the house with flat reinforced slab will increase significantly. In the case of green roof, the grass cover grows continuously and as a reason, it does not show behaviour like in the flat slab. The worst case in the green roof is the stage that where it has no grass. The analysis of these two situations is also very important in studying the indoor thermal performance of green roofs.

Figure 12 indicates a comparison between green roofs and flat slabs for present and future stages. It clearly shows that the flat slabs increase indoor temperature when they get older. It shows that it can reach up to temperature values close to 35^0C. And also the indoor temperature remains higher than the outdoor, every time for older flat slabs. In the case of green roofs, an increase in the indoor temperature can be visualized. However, it remains lower than the outdoor temperature most of the times in the daytime. The aging of flat slabs were an unavailable phenomenon while the green roofs behave as a fresh green roof always if it is maintained properly.

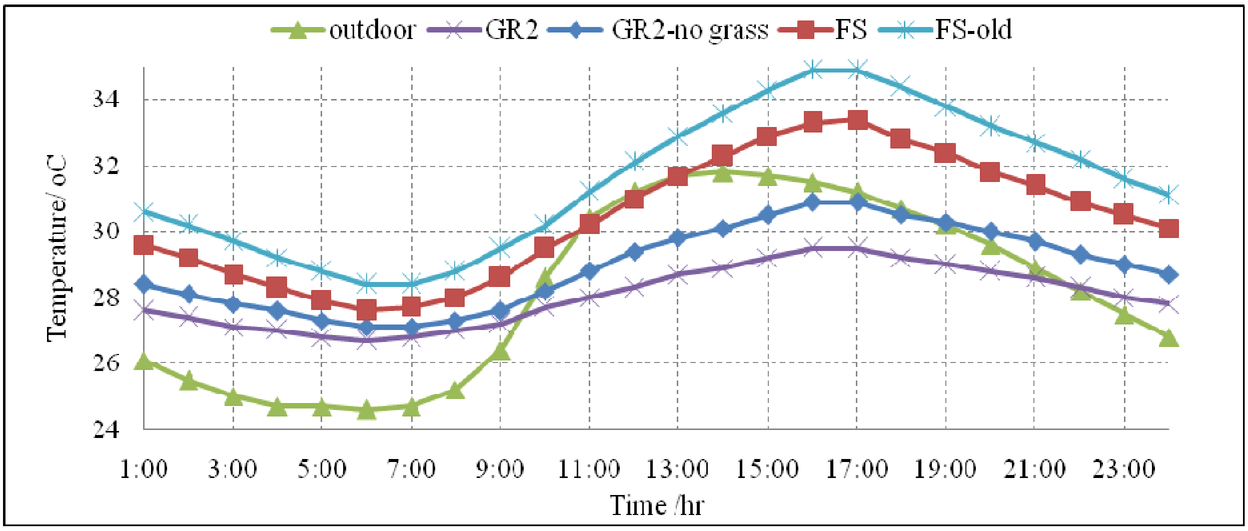

Figure 12- Comparison of indoor temperature for present and future stages in green roofs and flat slabs

5. Conclusions

The turf roof slabs are one of the most attractive trends in house designs in the modern days. It provides number of benefits than a house with conventional roofs. It has the potential to protect occupants from the high temperature conditions prevail in tropical countries by acting in harmony with the natural environment.

Absorptivity and transmitivity are the governing properties of a grass cover when dealing with indoor thermal performance. For buffalo grass, the absorptivity and transmisivity were obtained from the computer simulation as 80 and 15 respectively. The indoor temperature decreases mainly in green roofs due to the low transmissivity of grass.

With the increase of soil thickness, the rate of decreasing in indoor temperature increases. In ground floors of green roof houses, thickness of soil cover does not affect the variation of the indoor temperature. The upper floor of a green roof is much affected by the thickness of the soil cover. The average indoor temperature of an upper floor of a green roof in the day time is close to the basic comfort zone of low altitudes in Sri Lanka

In the case of a good healthy grass cover, low soil thicknesses are not preferred. But in terms of economy and structural stability of the roof slab, thicker soil covers are also not preferred. A moderate soil thickness of 50mm can be considered as the best fit for a green roof.

When considering green roofs and flat slabs, green roofs behave more efficiently on the indoor thermal performance. The buildings with green roofs have considerably low indoor temperatures than flat slabs. With time, the absorptivity of the flat slab increases and the increasing in heat transmission through the slab result in increased indoor temperatures. Well maintained green roofs don not have such effect to the indoor temperature because the properties of the grass cover remains same. The worst condition of a green roof is time where there is no grass on the green roof. However this behaves better than flat slabs.

Green roofs are more effective than flat slabs in terms of indoor thermal performance. In real green roof construction some other materials are also used such as for filtering and drainage purposes. These layers are also helpful to enhance the thermal performance of green roof houses which is not accounted in the simulation. On the hand, a house with a green roof has a regained land on the rooftop which can be use for various purposes. With the upcoming global warming issue, green roofs can be a great alternative to flat slabs.

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About the Authors

R.U. Halwatura, B.Sc. Eng (Moratuwa), Ph.D. (Moratuwa) is a senior lecturer at the Department of Civil Engineering, University of Moratuwa. His research interests are in the areas of energy efficiency and sustainable development

B.S.S.S. Dareeju, B.Sc. Eng (Moratuwa) (undergraduate) is an undergraduate student of the Department of Civil Engineering, University of Moratuwa.

J.N.Meegahage, B.Sc. Eng (Moratuwa) (undergraduate) is an undergraduate student of the Department of Civil Engineering, University of Moratuwa.

Appendix A

Table 1- Conductivity of

Materials	Conductivity (W/m k)
Concrete	1.7

Table 2- Surface Resistances

Ceiling downwards	R_{si}	0.14
Roofs	R_{so}	0.04

Calculation of "U" values.

- For Case1- Without soil cover

125mm thick slab, R_{body} = 0.125/1.7
=0.07 m^2 K/W

Total resistance R_{total} = R_{si} + R_{body} + R_{so}
=0.140 +0.07 +0.040
=0.254 m^2 K/W

U value = 3.94 W/ m^2K

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STRUCTURAL ASSESSMENT OF REINFORCED CONCRETE BRIDGE STRUCTURES EXPOSED TO CHLORIDE ENVIRONMENT

Laksiri Pradeep[1], Sudhira De Silva[2] and Chandana Nawarathna[3]

[1] Engineer, Research and Development Division, Sri Lanka Land Reclamation and Development Cooperation, Colombo, Sri Lanka
[2] Senior Lecturer, Department of Civil and Environmental Engineering, Faculty of Engineering, University of Ruhuna, Hapugala, Galle, Sri Lanka.
[2] E-mail: sudhira@cee.ruh.ac.lk; Tel: +94-(0)91-2245765~7
[3] Engineer, State Engineering Cooperation, Colombo, Sri Lanka

ABSTRACT: This research paper presents a new approach to develop a simple bridge rating system for Sri Lanka. This approach evaluates the current performance of concrete bridges on the basis of simple visual inspection and non-destructive tests. The main reason to conduct this research is to develop a proper bridge management system for Sri Lanka and to develop deterioration prediction curve for bridges. Also some applicable maintenance techniques are introduced according to identify condition state of the bridge based on the durability performance of the each bridge. During field inspection, the major issue to deteriorate concrete bridges was identified as chloride induced corrosion called chloride attack. To understand flexural capacity reduction and area reduction of reinforcing steel due to corrosion, Accelerated Corrosion Testing Method (ACTM) was carried out at the laboratory. By conducting load tests, the flexural capacity reduction of deteriorated concrete beam was compared with control beam. More than fifty percentage of area reduction in steel bars was observed while it reduced flexural capacity reduction more than seventy percent, compared to control beam.

***Keywords**: bridge rating system, visual inspection, acceleration corrosion testing method, non-destructive tests*

INTRODUCTION

Bridges are lifelines of a nation's infrastructure and massive investments are being made in the highway sector year after year. During the last fifty years a number of reinforced concrete (RC) bridges and pre-stressed concrete (PC) bridges were build all over the country. Day by day there is a significant increase of number of bridges in highway sector. Due to high construction cost and replacement cost, maximum utilization of service period is essential for bridges. Currently major infrastructure projects are being made by the government to improve public transportation system. It includes construction of steel and concrete bridges in all classes of roads. To improve transportation system in Sri Lanka, safety is an important parameter to reduce traffic congestion in main cities, where public transportation can be improved with confidence.

Normally bridges are directly exposure to severe environmental conditions and deterioration could occur with time. This deterioration process can lead to eventual failure of the bridge. Therefore periodic bridge inspection systems are required. Also most of these bridges were designed for lower traffic volume, slower speeds, and lower geometric standards than the current utilization. Due to continuous exposure to severe environmental conditions, the performance of bridges can be varying with its life, as well as there is tendency to use existing structures without proper investigation process. For the investigation process, lack of design details of these structures, lacks of expertise to inspect and evaluate cost are the major factors of concern. To obtain better safety from existing structures, it is necessary to assess the current performance of the existing structures. Then replacement or repairing can be determined based on the inspected or predicted results.

Though there are researches that evaluate the current performance of steel bridges in Sri Lanka, reinforced and pre-stressed concrete bridges evaluation attempts are relatively less. With many bridges being older than 50 years, proper bridge management system for Sri Lanka is essential for effective utilization during the remaining service period.

Bridge management system has been recognized as essential in all the developed countries. Though Sri Lanka has thousand of concrete bridges all over the country, there is no proper investigation process to evaluate the current performance these bridges yet. Main objective of this research is to introduce the cost effective, simple bridge evaluation system to identify current condition state of bridges in different exposure conditions. It is also expected to introduce a deterioration prediction curve for bridges, in order to check whether maintenance requirements and offer rehabilitation strategies (maintenance techniques) for bridges.

METHODOLOGY

Visual Inspection and Non-destructive Testing

First it is necessary to identify suitable number of bridges for visual inspections and method of non destructive testing required for particular bridge. Bridges have important structural elements such as; deck, piers, deck layering, drainage, girders, beams, handrail and embankment. Bridges were evaluated through a visual inspection and hence its structural condition and performance could be predicted based on soundness score. The soundness score of a structural element depends on the current condition of the structural element and element weighted value. When bridge evaluation is conducted using this method, a subjective rating was assigned to the bridge components. The presence of cracks, spalling of concrete and corrosion of reinforcement offered important aspects on determining the condition state of each element. Then bridges could be rated according to National Bridge Inventory US (1995) specifications. Also usages of non-destructive tests methods (rebound hammer test and ultra-sonic pulse velocity test) for bridge inspections have gained much reliability on evaluating the structural performance. This is due to its effective ability in evaluating structural conditions of the bridge. Non-destructive testing includes methods of testing on concrete structures which do not reduce the functional capability of the structure.

Two types of non destructive tests were used during field inspections. Rebound hammer test was carried out by pressing rebound tip on the concrete surface. When tip compressed on concrete surface, it rebounded and gives rebound number. After referring to the standard graph, the related compressive strength of concrete can be used to determine the strength of the structural element. In pulse velocity test, at the beginning the trance-meter and receiver was kept at a spacing of 0.3 m and the travel time of the pulse was measured. Then pulse velocity was found by known distance and time. Then referring to standard graphs which show the relationship between compressive strength of concrete and the pulse velocity, the actual strength of the bridge element can be found. Also to increase the accuracy of results, these testing were carried out with the spacing of 0.6 m and 0.9 m. The pulse velocity test can be done by direct or indirect method. But values obtain by direct method are more reliable than indirect method. However, due to access limitations indirect method was applied to the most of the bridges. In addition, pulse velocity test was carried out to identify structural defects such as cracks, voids, etc in the bridge elements. During the inspection, it was identified that bridges along the coastal belt have deteriorated more than the bridges located inland. It is mainly due to corrosion of reinforcements caused that is by chloride induced corrosion cracking called chloride attack along the bridge girders. (see Figure 1)

Experimental Program and Test Parameters

Two reinforced concrete beams were cast in the laboratory using ordinary portland cement with maximum aggregate size of 20 mm. The cross section of the beam is 100 x 150 mm and the length of the beam is 2000 mm. The characteristic design compressive strength of concrete used is 20 N/mm^2. To avoid corrosion of reinforcing steel, the extended length of bars was coated with grease and wax. To simulate the deterioration of beams in short time period, accelerated corrosion testing method (ACTM) was adopted. The specimen is immersed in a sodium chloride (NaCl) solution bath with Cl^- concentration of 5% and that acts as electrolyte. In this method, reinforcing steel of the specimen that have to be corroded, is made as anode and copper bars in the bottom of acrylic tank is used as cathode (see Figure 2). The current supply was connected to one end of steel bar in the specimen and the other end is connected to the copper bars. The constant and continuous current supply of 0.7 Amp was applied to all the specimens until corrosion crack generated. Figure 3 shows the sequence of steps followed during the ACTM in the laboratory.

RESULTS AND DISCUSSION

Visual Inspection and Non-destructive testing

Bridge inspections were carried out along the coastel belt and rural area. Totally, twenty seven bridges were inspected, from those, fifteen bridges located along the costal belt from Hikkaduwa to Weligama

(Bridge Reference Numbers (BRN), 1 to 15) along the main road A2 and twelve bridges, in the countryside (Bridge Reference Numbers (BRN), 16 to 27). Numbering of those inspected bridges is shown according to the inspected sequence from 1-27 (see Table 1). Those inspected bridges were rated according to modified US bridge rating system according to Sri Lankan environmental conditions and type of bridge (Tables 1 and 2).

According to field inspection results, it is revealed that Hikkaduwa Bridge (BRN 5) located very close to main city shows remarkable strength reduction compared to other bridges in the coastal belt. Visual inspection details at Koggala Bridge (BRN 11) revealed that it has large amount of corroded reinforcement but it did not show considerable strength reduction as Hikkaduwa Bridge (see Figure 4). The main reason for that may be the aging of these structures. Also all the bridges located along the coastal belt show average compressive strength of concrete about 40 N/mm^2 while bridges in country region shows average compressive strength of concrete about 30 N/mm^2. Further it was noticed that most of bridges along the coastal belt have been in having a service life of less than 25 years.

Further, analysis was carried out using simple evaluation process to determine required maintenance techniques. The maintenance technique was proposed based on the inspection results and predicted soundness score of the structure. For that, most effective and critical bridge elements (girder, embankment, slab and pier) were identified and their conditions states were used. At the beginning a numerical value is assigned to bridge element considering its importance as shown in Table 3. Each bridge element condition was determined using non-destructive test results (see Figure 4) and assigned the condition state based on the test results (see Table 4). Visual inspection results were important when the access difficulties occurred to carry out the non-destructive tests. Following formula is used to determine soundness score.

Bridge Soundness Score (BSS) = ∑ (Each element condition x element weighted value)

According to the soundness score of the bridge, the required maintenance techniques are listed in Table 5. Table 6 summarizes the method used to determine soundness score values for inspected bridges with necessary recommendations and rehabilitation strategies for bridges.

Acceleration Corrosion Test Results

Acceleration corrosion testing was carried out until corrosion crack appears in the specimen. After 35 days of acceleration corrosion, the test beam showed horizontal corrosion cracks along the beam length. The development and propagation of corrosion cracks along the beam length and underneath of the beam are shown in Figures 5(a) and 5(b), respectively.

After 35 days, it was observed that crack opening increased up to average of 1.8 mm in one-side, average of 2.0 mm on the other side and the bottom crack remained unchanged. Also these cracks were propagated along the whole length of the beam causing it to split horizontally. After 41 days those openings expanded further one side up to 2.6 mm and on other side up to 2 mm (Figure 6).

The flexural capacity of two beams were checked using single point loading test as shown in Figure 7(a). Also it was revealed that there was no proper bond between concrete and reinforcement steel. At the failure load, corroded beam showed only flexural cracks along the mid-span region (Figure 7(c)). But the control beam showed both flexural and flexural-shear cracks at the ultimate failure (Figure 7(b)). After that, the test beam with corrosion cracks was carefully monitored. By inspecting the reinforcing steel, it revealed that the rib had been totally removed from the bar causing it to produce corrosion rust on the surface of the bar (see Figure 7(d)). Due to this poor bond characteristic between steel and concrete, the flexural capacity was reduced by nearly 70%. Figure 8 shows applied load versus mid-span deflection relationship for control and test beams. It observed that the reinforcing steel area reduction was around 55% due to this corrosion. These data were summarised in Table 7.

CONCLUSIONS

This research would be able to improve and modify the contents of the existing inspection sheet that is used by Road Development Authority, Sri Lanka. The proposed bridge management system offers various maintenance plans which can be directly applied to bridges in Sri Lanka. During the inspection, it is identified that most of coastal belt bridges deteriorated than the bridges located inland. It is due to the chloride attack.

It shows that significant amount of bearing capacity reduction between the control beam and test beam (around 70%). Moreover in order to evaluate the condition of existing RC structure, weight loss of the reinforcement steel lost due to corrosion is essential. While this amount cannot be measured directly without removing the reinforcement steel from the structure, it can be estimate indirectly using the width of corrosion induced cracks with the loss of reinforced section area.

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(a) Hikkaduwa town bridge **(b) Ginthota bridge** **(c) Koggala bridge**

Figure 1: Damages in Bridges due to Corrosion

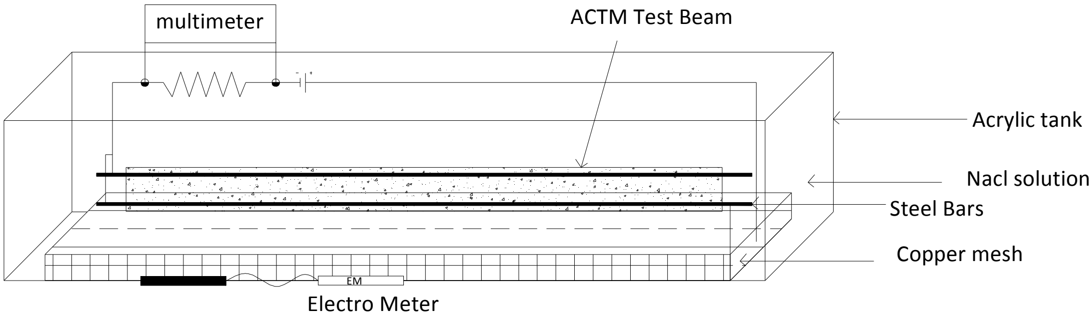


Figure 2: Acceleration Corrosion Test Apparatus

(a) Cast beams **(b) NaCl bath** **(c) Power supply unit**

(d) During ACT process **(e) After 15 days** **(f) Cracks after 35 days**

Figure 3: Corrosion Test Process

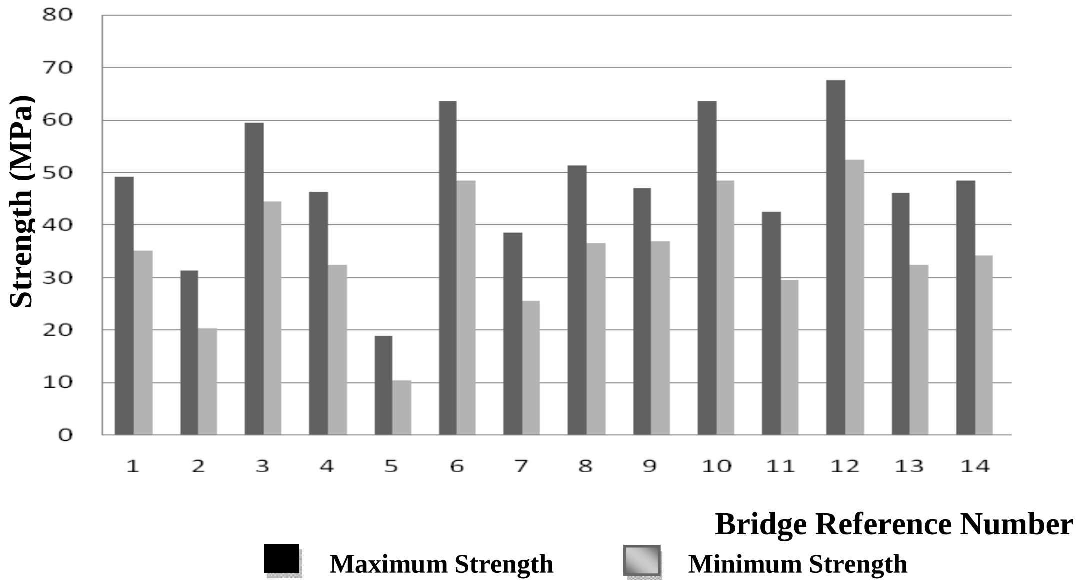


Figure 4: Inspection Summary for Coastal Belt Bridges: Beam/Deck Element

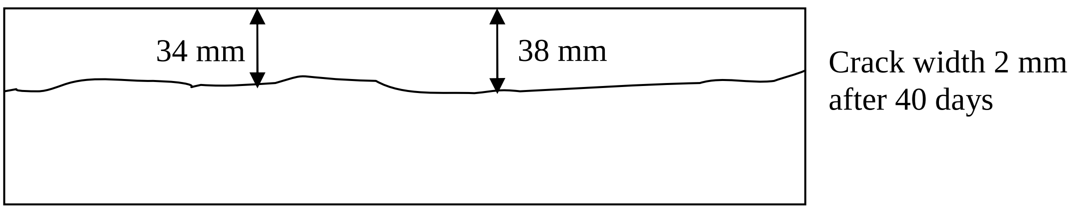


Figure 5(a): Corrosion Cracks on Side

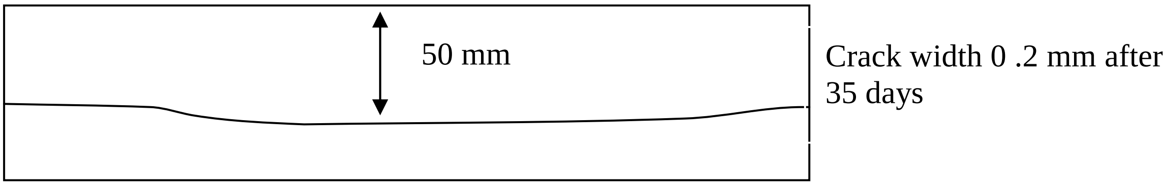


Figure 5(b): Corrosion Crack on Bottom Surface

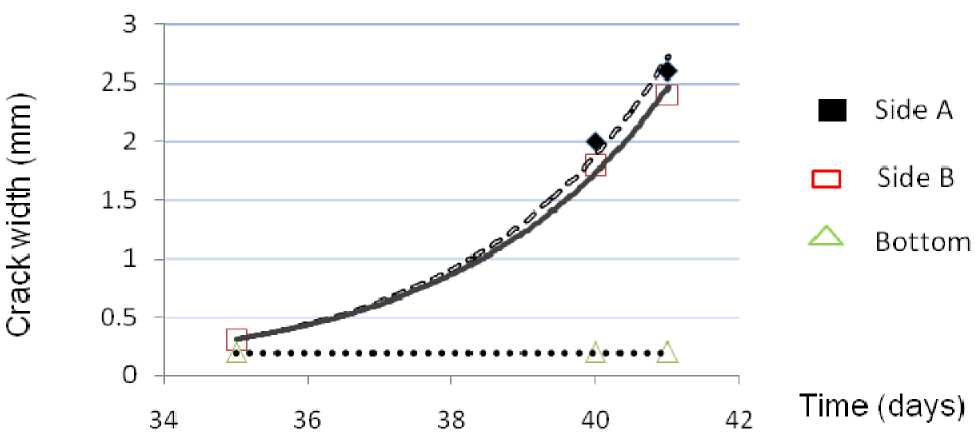


Figure 6: Crack Development with Time

(a) Single point load test (b) Control beam

(c) Corroded beam (d) ACTM results on corroded steel

Figure 7: Loading Test Process

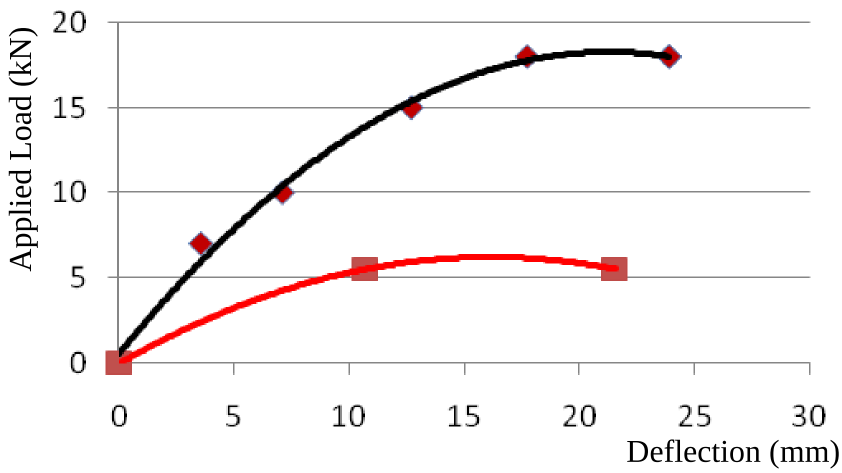


Figure 8: Applied Load vs. Mid-Span Deflection

Table 1: Bridge Reference Number and Condition State

Bridge Number	Bridge Name	Condition State	Condition Number
1	Kapu Ela bridge	Good.	7
2	Railway Across bridge	Satisfactory.	6
3	Ginthota bridge	Very good.	8
4	Dodanduwa bridge	Very good.	8
5	Hikkaduwa bridge	Critical.	2
6	Hikkaduwabridge(Town)	Good.	7
7	Gintota bridge	Very good.	8
8	Mahamodara bridge	Good.	7
9	Dewata bridge	Good.	7
10	Habaraduwa bridge	Good.	7
11	Koggala bridge	Good.	7
12	Ahangama bridge	Excellent.	9

13	Goyyapana bridge	Good.	7
14	Weligamabridge (Town)	Very good.	8
15	Polathumodara bridge	Good.	7
16	Bogahagoda bridge	Satisfactory.	6
17	Agulugaha bridge	Good.	7
18	Imaduwa bridge	Satisfactory.	6
19	Imaduwa bridge	Very Good.	8
20	Kalukadha bridge	Satisfactory.	6
21	Ambalama bridge	Very Good.	8
22	Moraliyadda bridge	Good.	7
23	Polpagoda bridge	Good.	7
24	Makumbura bridge	Good.	7
25	Kottawa bridge	Good.	7
26	Totagoda bridge 01	Satisfactory.	6
27	Totagoda bridge 01	Good.	7

Table 2: Condition State Rating (National Bridge Inventory US, 1995)

Number	**Condition state**	**Physical Description**
9	Excellent.	A new bridge
8	Very good.	No problem noted.
7	Good.	Some minor problem.
6	Satisfactory.	structural members show minor some deterioration
5	Fair.	All primary structural elements are sound but may have minor section loss, deterioration, spalling, or scour.
4	Poor.	Advance section loss, deterioration, spalling, scour.
3	Serious.	Loss of section, etc. has affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.
2	Critical.	Advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed structural support. Unless closely monitored it may necessary to close the bridge until corrective action is taken.
1	Imminent failure.	Major deterioration or loss of section in critical structural component or obvious vertical or horizontal movement affecting structural stability. Bridge is closed to traffic but corrective action may put back in light service.
0	Failed.	Out of service. Beyond corrective action.

Table 3:Element Weighted Values

Element	**Weighted Value**
Piers	1
Deck	2
Abutment	1

Table 4: Element Condition State Value

Minimum Strength (N/mm^2)	Condition Value	Condition State
>=45	9	Excellent.
40-45	8	Very good.
30-40	7	Good.
25-30	6	Satisfactory.
20-25	5	Fair.
15-20	4	Poor.
10 -15	3	Series
10 >	0	Failed.

Table 5: Bridge Soundness Score and Maintenance Techniques Required

Bridge Soundness Score	Treatment Required
36-32	No treatment required
31-24	Simple maintenance techniques required.(Patching Repair)
23-16	Special maintenance techniques required.(Cathodic protection)
15-08	Immediate maintenance techniques required.(Retrofitting techniques)
7 >	Replacement.

Table 6: Bridge Soundness Score and Maintenance Techniques Required For Bridges with Piers

BRN	Bridge Element Condition			Soundness Score	Maintenance Required
	Pear (1)	Abutment (1)	Deck (2)		
1	7	7	7	28	Simple maintenance techniques
4	7	7	7	28	Simple maintenance techniques
8	7	7	7	28	Simple maintenance techniques
9	7	9	9	32	No treatment required
10	9	9	9	36	No treatment required
11	6	6	6	24	Simple maintenance techniques
13	7	8	8	30	Simple maintenance techniques
15	8	8	8	32	No treatment required
17	4	4	4	16	**Special maintenance techniques required**
24	4	4	4	16	**Special maintenance techniques required**
26	4	8	8	24	Simple maintenance techniques

Table 7: Loading Test Results

Beam	Calculated Flexural Capacity (kN)	Failure Load (kN)	Maximum mid-span deflection (mm)
Control beam	17.5	18.0	27.70
Test beam		5.5	21.55

GALVANISED WIRE REINFORCEMENT (GWR) TECHNOLOGY

Earthquake Reinforcement for Non-Engineered Stone and Earth Constructions

Engineer Architect Sjoerd Nienhuys
Project manager and researcher in Yemen Arab Republic, Pakistan, Nepal and Peru
E-mail: Sjoerd@nienhuys.info; or GWR@nienhuys.info
Telephone: +31-35-6232646, Mobile: +31-646-777-544, +44-792-455-2831

Abstract: Earthquakes negatively affect many low-cost houses of low-income people due to poor material choice and construction techniques, especially in non-formal settlements and housing development. In the remote village areas of high mountains such as the Himalayas and the Andes the introduction of cement and steel reinforcement bars for reinforced concrete constructions is very costly and often not affordable for the local villagers. Galvanised Wire Reinforcement (GWR) is an earthquake reinforcement for thick, non-engineered dressed and semi-dressed stone walls, adobe, soil block and rammed earth building constructions. In remote mountain areas masonry with cement mortar is only marginally done due to the high cost of cement, sand and aggregates. The 2.3 mm and cross-welded or knotted GWR provides lengthwise and crosswise reinforcement within the wall thickness and vertically along the borders of all openings such as doors windows and wall endings. The special advantage of the Hot Dip Galvanised Wire is that it does not require a high concentration of cement in the concrete or cement mortar to prevent corrosion of the wires. Villagers can apply the reinforcement throughout the wall construction applying only low-cement mixtures, thus keeping the construction cost low. L-shaped and U-shaped cement blocks provide hollow space for the vertical reinforcement consisting of folded up GWR strips, facilitating construction and providing an aesthetic architectural design. The cement blocks can be cast on site with a simple operated mould and hand compacting, also allowing low cost production techniques. The GWR strips or ladders can equally be used in adobe wall constructions either made with blocks or rammed earth. In addition it is an adequate reinforcement for solid cement block walls. Practically speaking the GWR strips can be applied in every alternating course or stone or block masonry, thus creating adequate stress reinforcement throughout the wall structure and having contact with all stones or blocks.

Key words: mountains, remote, housing, earthquake reinforcement.

Table of Contents

1. Introduction

Earthquakes and Remote Mountain Areas

The western wing of the Karakorum Range of the Pakistan Himalayas, comprising the Northern Areas and Chitral District, is under the influence of plate tectonics that culminate beneath Afghanistan. Earthquake movements are frequently registered in the entire area. A very large earthquake occurred in October 2005, leaving 75,000 people dead and destroying more than 100,000 houses. Building to withstand these tremors is therefore extremely important in saving human lives and minimising economic disaster. Road access to remote villages is by donkey trail. Bringing cement, long concrete reinforcement bars, sand and aggregate is extremely difficult and expensive.

Lack of natural resources, such as timber, have affected the building practices over the last generation. This aspect, combined with rapid population growth, has resulted in a severe deterioration of building quality, especially in areas where no alternatives to traditional stone constructions have been developed. BACIP has introduced the Galvanised Wire Reinforcement (GWR), providing an economically feasible and technically sound method to reinforce traditional dressed stone, semi-dressed stone and soil block constructions in remote mountain villages.

Non-Masoned Semi-Dressed Stone and Rammed Earth Construction

Galvanised barbed wire was first used in 1970 with the reconstruction of low-cost *bahareque* houses (timber frame, bamboo mats and soil plaster construction) in Guatemala by CARE. The use of wire-mesh in stone masonry was used by the author in the earthquake reconstruction programme after the devastating earthquake of 1982 in Dhamar, Yemen Arab Republic.[1] The galvanised wire-mesh technology was particularly suited for the horizontal reinforcement of 18" wide dressed stone walls. In the Dhamar reconstruction project, rolls of pre-manufactured double galvanised cattle fencing were used for rapid application. The openings in this wire-mesh ranged from 6 cm to 20 cm.

The central plateau of Yemen has good quality clays. Two to four-storey houses are traditionally built from rammed earth, having 2-3 ft. thick walls. Before casting and compacting a higher layer of rammed earth between a wooden formwork, a new strip of the cattle wire-mesh is laid from corner to corner. By doing so, a horizontal reinforcement is realized at 1 ft. vertical intervals, providing an even distribution of reinforcement throughout the walls. With the slimming of the walls, the wire-mesh is cut in narrower strips, keeping the length wires along the sides of the wall about one inch from the external face.

Plaster or Cavity Wall

A GWR strip can be made with three length wires. Two wires consist of the main wall reinforcement, while the third external wire is used to create a cavity wall. Cavity walls create some thermal insulation whereby comfort is enhanced and firewood savings realised. Especially in the desert climates and high altitudes, such as on the Yemen plateau, Pakistan, North India, Nepal, Tibet, China and Mongolia, cavity walls are economically and thermally efficient. In wet climates, when the outer cavity wall is made of durable materials, it keeps the inner wall dry.

Insulating Inner Wall

A thin insulating inner wall can be fixed on a third inner parallel wire. Insulating walls are made with cavities, insulation material and reflecting foils. The inside of the inner walls can be finished with gypsum board or plaster on fine wire-mesh.

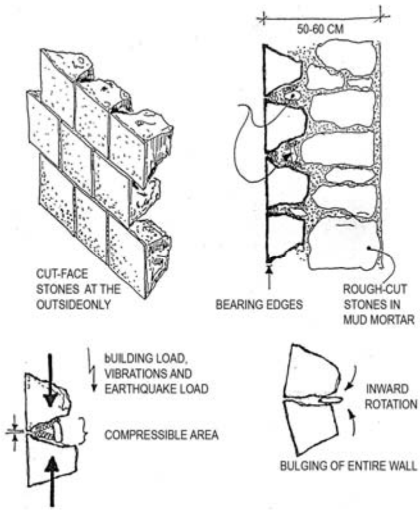
[1] Project financed by the Netherlands Directorate for International Cooperation (DGIS) and DHV Consulting Engineers, The Netherlands. The use of wire-mesh is common in reinforced masonry designs (burned brick).

2. Materials and Method

The need for simple-to-apply earthquake reinforcement for self-help stone constructed houses was highlighted after the December 1982 earthquake in Dhamar, Yemen Arab Republic, where tens of thousands of traditional dressed stone houses collapsed during the earthquake.

Photos of traditional buildings and their damage can be viewed in my paper on site selection see: http://www.nienhuys.info/mediapool/49/493498/data/Dhamar_Site_Selection.pdf

The pictures in the above-mentioned document illustrate the problems related to the technique of making nicely dressed cut-face stones for “dry” masoned stone wall constructions.



The volcanic stones can be finely shaped with enough time and skill (cost), but many masons apply the cut-back technique whereby the face of the stone is tailored with straight sides and the rear side is cut back to minimize the joint in the façade of the building.

The stones are supported with rubble. Vibrations, such as with earthquakes, cause the cut-face stones to bulge outwards and come loose from the rubble masonry of the inside of the walls.

No published information was available on appropriate solutions given the remote mountain situation of the Mahgrib Ans. Therefore, generally available earthquake-resistant construction technology was used, mainly from sources such as the American Concrete Institute (ACI) building codes (ACI 318) and a number of publications from the Indian Institute for Earthquake Engineering (IIEE) on the technical reinforcement of simple houses using reinforced concrete confinement along all wall endings and bringing in tie beams through the windows and at floor level. Roof diagram constructions would be realised on every floor. The details are explained in the following chapters.

The working method was to assess the available building materials in Yemen and the possibility to transport these by light 4WD truck to the building sites, some of them only reachable by donkey and man power. Reduction of weight, transportability and the versatility of the reinforcement were some of the determining factors.

Three other important factors for remote mountain areas, such as in the Himalayas, were:

(1) The possibility to shift part of the manufacturing cost of the reinforcement to the village where the material was to be used.

(2) The use of the steel reinforcement in low cement masonry or even in mud-clay soil masonry. The use of heavy and costly cement was commonly minimised by the house owner and the masons.
(3) The reinforcement should be well distributed throughout the construction because making reinforced concrete columns was costly and the concrete quality generally was substantially below the minimum recommended resistance value due to adverse aggregate, water quality and poor construction practices.

In Pakistan, Northern Areas, the choice fell on 2 mm hot-dip galvanised wire, having ample strength to withstand large tension forces and sufficiently pliable to knot into a mesh structure to enhance adherence in between the stone masonry layers.

The material could be easily transported in rolls of 50 kg and a series of simple hand tools were designed to rapidly knot the wires to make a long roll of mesh having the width of the stone wall – 50 cm, 60 cm or 80 cm.

It is recommended to make factory point welded bands (strips) of reinforcement wire. Post-welding the rolls should be by means of a hot-dip galvanised technology. Rolls of 20 m long are used. GWR can be effectively used in natural stone, adobe and cement block houses.

A small stock of factory welded and post welding hot-dip galvanised wire-mesh in one of the BACIP storage sites in the Northern Areas of Pakistan.
Author with a stack of adobe blocks. When the wire-mesh rolls are slightly smaller than the width of the wall, they should be placed in very soft clay joining paste at every alternating layer.

When the wire-mesh bands are connected at all intersection walls and to vertical corner and door/window side reinforcements, good horizontal and vertical wall reinforcement is obtained. A roof tie beam and roof diaphragm will complete the construction, providing a fairly safe home able to withstand a considerable earthquake without fatal collapse.

A roll of hand-made wire-mesh manufactured for a large house in the mountains. The roll weighs about 100 kg and can be transported in a small 4WD pick-up truck that is able to reach the construction site via the mountain tracks. Smaller rolls can be carried by donkeys.

In remote areas where access is very difficult, rolls of 2 mm wire and the small tools can be hand-carried to the village. After a short training, the wire-mesh can be manufactured on the building site. Women can make the rolls of wire-mesh as well.

3. Theory of the Reinforcement

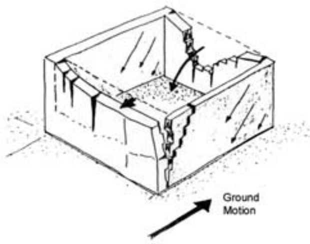
The objective of the GWR is to provide internal stress resistance to walls lacking sufficient natural bonding to function as shear walls or when the bonding is exceeded by an earthquake jolt. Stress resistance is traditionally obtained with wooden tie beams laid in the walls. In modern building technology, iron reinforcement bars are applied by imbedding them in a layer of concrete. This "modern" building technique has a number of serious disadvantages when applied in remote mountain areas because the correct sand and stone aggregates are unavailable, cement is very expensive, and mixture and curing processes are often deficient. Improperly realised concrete constructions become an additional earthquake hazard in themselves due to their massive weight.

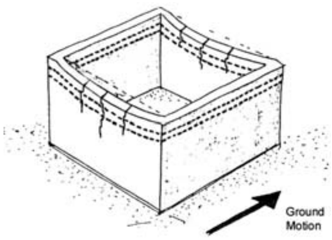
The amount of reinforcement required is determined by the expected earthquake forces; these are directly related to the mass of the construction (weight). The expected horizontal forces caused by an earthquake can be derived from a standard earthquake code.[2] This force varies from 20-30% of the mass of the construction. For public buildings a multiplier of 1.25 or 1.5 is used, depending on its importance. These values are used for low-rise buildings up to four storeys.

Well masoned houses with tie beams, floor diaphragms and consisting of no more than two storeys with proper distribution of doors and windows are usually strong enough to withstand minor earthquakes. In general, these houses are non-engineered, meaning that no special calculations are made. Non-masoned and non-reinforced houses will fail in any major earthquake and cause numerous casualties, as well as great economic loss. For non-masoned (non-bonded masonry) houses to be earthquake resistant, the following is required:

(a) Rectangular cut stones that are fully supported by lower stones.
(b) Minimal one through-stone per square meter of wall.
(c) Floors and roof beams anchored into the walls in two directions, making diaphragms.
(d) Openings that are at least one meter away from the junctions of the walls.
(e) Short freestanding wall lengths (without cross walls) and low unsupported walls.

The diagram right shows the effect of an earthquake on a room made in masonry having no stress capacity.[3]

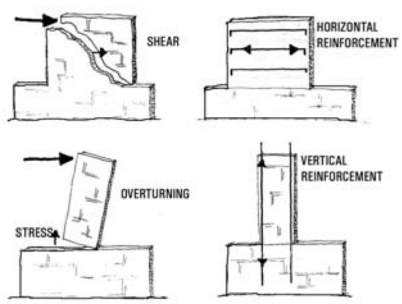




The effect of simple stress reinforcement in the higher parts of the wall is illustrated left. The better the stress reinforcement is embedded in the wall and distributed over the higher part of the wall, the less cracks will appear in the unsupported central part.

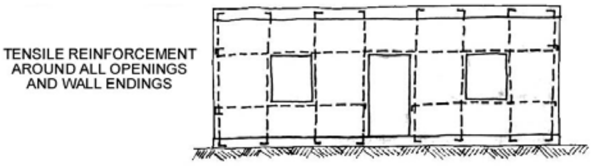
GWR or reinforced concrete tie beams provide such a distributed stress reinforcement in semi-dressed stone or soil block constructions.

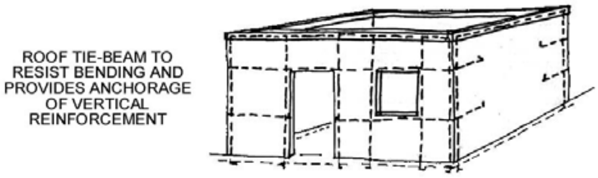
The four diagrams right give an idea of the shear forces in small wall sections, such as those between doors and windows. With the occurrence of an earthquake, diagonal cracks will appear in the walls as indicated in the first diagram. To withstand the horizontal forces, stress reinforcement should be brought into the wall in several layers, crossing the diagonal line of failure (second diagram). The third diagram shows the overturning of a narrow wall section. Here stress reinforcement needs to be placed vertically as indicated in the last diagram. L-shaped and U-shaped cement blocks at wall endings and corners provide room for placing such vertical reinforcement.



The combination of the above two principles of wall reinforcement requires narrow wall sections to be fully framed along their outside borders. With the use of L- and U-shaped blocks, slender stiffener columns can be integrated with the extremities of the walls, connecting the foundation to the upper tie beams. The schematic presentation of such a reinforcement pattern is present in sketches below.

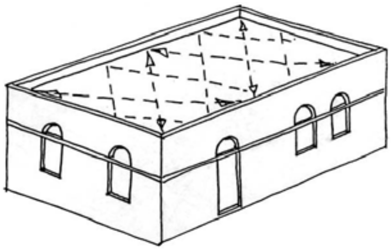
[2] The American ACI 318 and the Indian Earthquake Code are rather similar in their calculation methods.
[3] Sketches on pages 5-7 are copied from Indian Earthquake Research Institute documents, 1976.





When the above reinforcement pattern is combined with a tie beam reinforcement in the top of the wall, it will provide an overall reinforcement pattern as indicated in the right-hand sketch.

The stability of a house not only depends on the reinforcement of individual wall sections, but on the overall coherence of the construction as well. Long walls need to be supported with either reinforced buttresses or anchored cross walls. All floor and roof beams need to be properly anchored into the wall tie beams to create floor/roof diaphragms that function in all horizontal directions. In addition, all inside walls need to be anchored into this floor/roof diaphragm.



The method of reinforcement described can be used for one or two-storey buildings without load bearing reinforced concrete columns. However, the higher the building, the greater the amount of reinforcement required in the lower walls. For buildings with a few storeys, the strength of the shear walls in the lowest part of the building should be more than in the top floors.

The window and door openings should be distributed in the façade in such a way that sufficient wall segments or piers remain to form shear wall sections. For non-engineered constructions, the total section of piers in the lower floor walls should increase with the height of the building. When a two-storey building is planned, but will be built in stages, the amount of piers and internal wall reinforcement should conform to that higher design of the future.

> Earthquake disasters occur when a storey is added on top of a ground floor construction that was not designed for additional floors. This is aggravated when shear walls on the ground floor are eliminated to make room for shops.

The quality control of house construction in villages depends entirely on the knowledge of the house owner. Building advice should provide general rule-of-thumb guidelines to ensure sufficient safety to withstand earthquakes. These guidelines must be understood by both the house owner and the locally available skilled labourers. Some guidelines found in earthquake building codes are:

- No window or door opening should be made within one meter of the corners of the building.
- When the width of a wall section between openings is smaller than its height (piers), the vertical sides of these wall sections need to be reinforced.
- The piers next to a door or window opening should have a minimum width equal to half the height of the opening. For example, if the door opening is two meters high (6 ft.), the pier should be minimum one meter (3 ft.) in width.
- When numerous window openings are required, it is better to make one large opening with a strong shear wall rather than several small openings with many piers. Depending on the design, reinforced columns can be considered instead of several piers.
- For the top floor where there will be no future construction above, the cross section of the shear walls should be a minimum of 40% of the original wall section (without the openings).

- For the floor where there will be only <u>one floor constructed above</u>, the cross section of the shear walls should be a minimum of 50% of the original wall section (without the openings).
- For the floor where there will be <u>two floors constructed above</u>, the cross section of the shear walls should be a minimum of 60% of the original wall section (without the openings).
- When the openings in the lowest floor of a three-storey building consist of more than 30% of the original wall construction, then reinforced column constructions need to be realised.
- For buildings higher than three storeys, engineering calculations should be made.
- The above-indicated percentages can be taken over the <u>entire wall section</u> of the floor <u>only</u> where both the inner and outside walls have a fully integrated network of linked up tie beams, floor and roof diaphragms.
- When horizontal or vertical loads are applied on walls, good bonding from face to face of the wall should avoid internal separation.

In traditionally built houses without the traditional wood framing, the above-indicated conditions seldom exist. <snip drawing>

4. Practice of Dry Stone Construction

Traditional houses in the Northern Areas of Pakistan use four to seven heavy wooden columns in the centre of the room, supporting massive roof beams and having in-filled stone walls on the periphery. These dry stone (non-cemented) exterior walls often have internal wooden posts supporting the heavy roof construction. The roof consists of tree trunks, branches, twigs, grass, birch bark and various layers of clay soil. Adjacent to the central living area, various stores are built having a solid wall construction (no window openings). The only light comes through a central opening in the roof, doubling in function as a smoke outlet.

In the event of a major earthquake, the pillared construction would remain standing, but periphery non-masoned walls would eventually fall out of their framing. If the walls of the adjacent rooms (stores) fail to withstand the earthquake, the pillars would topple sideways causing the whole massive roof to collapse, burying the inhabitants under the heavy roof and rubble. <snip drawing>

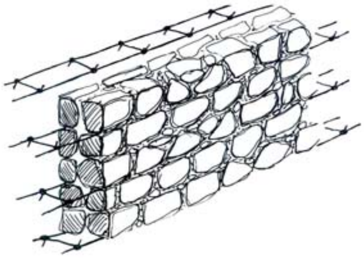
TRADITIONAL HOUSE ~ COLUMNS JUST MANAGE TO KEEP THE HEAVY ROOF UP.
WITHOUT THE REMAINING BRACING WALLS, THE STRUCTURE WILL COLLAPSE FURTHER.

In the past, a wooden tie beam construction was made in the length of the wall consisting of two parallel (fruit tree) wood sections connected to each other with short sleepers. In some cases, these lengthwise wooden strips have been applied in the corners of the walls only.

Population growth has created a high demand for new construction and this has led to an over-exploitation of available forests for building materials and firewood. The result is the non-availability of fruit trees for construction and scarcity of quality wood for the traditional house design with the central columns. The limited hardwood available in the market is unaffordable for wall reinforcement. In the absence of an alternative, villagers are constructing walls without any reinforcement. This makes all such houses highly vulnerable to earthquake jolts and does not allow for the building of two-storey houses.

The GWR replaces the traditional wood reinforcement in dry wall stone masonry.

Sketches: Lateral wall reinforcement ~ traditional method and new GWR solution.

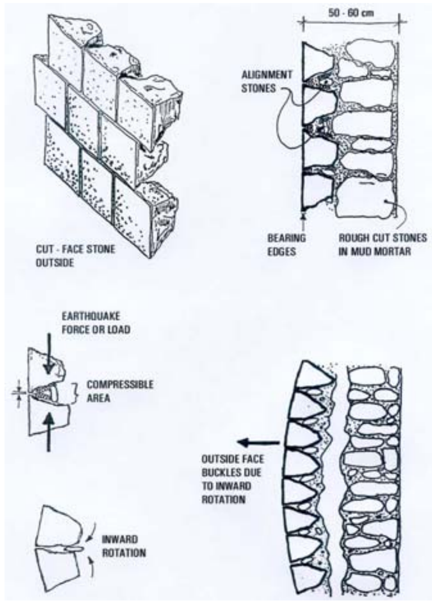


Modern dressed and semi-dressed stone constructions have particular disadvantages in relation to earthquake movements. The worst are nicely cut-face stone works.

a. Earthquake forces are directly related to the mass of the construction. Traditional 18-20" (46-50 cm) dressed stone walls generate tremendous earthquake forces that can only be resisted with either very strong or very stable constructions. Non-masoned stone constructions have neither of those two characteristics.

CROSS-SECTIONS OF TRADITIONAL WALL AND COLLAPSED STRUCTURES SHOWING LACK OF BONDING BETWEEN FACES, BUT WITH THE USE OF A LARGE AMOUNT OF CEMENT MORTAR AND PLASTER. LACK OF ANCHORAGE OF WALLS.

b. Traditional stone walls are composed of two lines of semi-dressed stones (inner and outer faces). Small pointer stones are used throughout the construction (both on the inside and outside faces) to balance the stones vertically in the façade of the wall. When some of these pointer stones fall out due to erosion or vibration, the wall becomes unstable and eventually will bulge and collapse.
c. For a straight uniform finish of the façade, the stones are dressed to even height and size. The stones are cut backwards from the cut-face into a conical shape so they can be easily aligned. The façade stones are supported inside the same wall with roughly cut stones, rubble and clay. Cut-faced stones will resist some vertical vibrations, but the inside of the wall is compressible. The result will be a rotation of the cut-face stones, breaking them loose from the inner wall and fall away in an earthquake.
d. To provide binding between the inner and outer wall faces, through-stones need to be placed providing about one tie per square meter wall. This is insufficient to withstand many tremors over a long period of time.
e. When the 18" two-stone wall is masoned with cement mortar for additional strength and bonding between the two faces, large amounts of mortar are required (30% of wall volume). When steel reinforcement bars are masoned into this cement work, the quality of the cement mortar covering the steel bars is often inadequate and. the steel reinforcement bars will corrode.
f. Steel reinforcement bars embedded in natural stone masonry are always over-dimensioned in relation to their adherence to the stones around them. Many thinner reinforcement wires or GWR provide a better spreading of the stress resistance throughout the construction.



5. Result: Galvanised Wire Reinforcement

The GWR technique provides a simple, cost-effective solution for making houses better resistant to earthquakes. Although binding of stone walls with cement mortar is the best method for making dry stone masonry more earthquake resistant, the disadvantages of using cement mortar in remote mountain villages outweigh the advantages for many inhabitants:

- Cement is costly, heavy and difficult to carry, making transport additionally expensive.
- For loose stone construction, a large quantity of cement mortar (30% of stone volume) is required.
- Reasonable sand quality is required for the cement mortar. Not all riverbeds have the quality and quantity of sand required, and those that do often are located at considerable distance from the villages.

Considering the above problems in remote mountain villages, a reinforcement technique is required that can be applied in:

1. Dry stone masonry using stabilised mortar only.
2. Construction of soil block walls.
3. Adobe and rammed earth constructions.
4. Masoned constructions in which stones, bricks or cement blocks are used.

Steel bars used to reinforce masonry constructions require strong cement-sand mortar (minimum 1:4) or concrete (1:2:3). When the concrete or mortar quality is poor, steel reinforcement bars will eventually corrode and break the surrounding masonry.

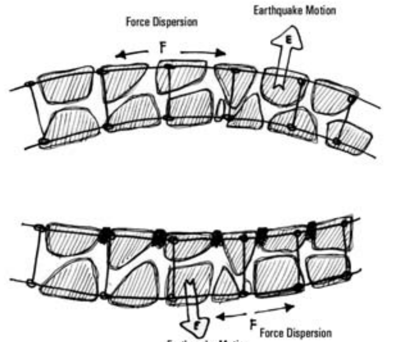
The GWR does not require masonry with strong cement-sand mortar (only 1:10) and can be used in square-cut dry stone construction or adobe walls without the immediate danger of corrosion of the wires. Corrosion protection is adequate with double galvanisation, such as used for barbed wire and fencing wire.

Good adherence between the wires and the wall is necessary. To achieve this adherence, a long ladder-like strip is made from galvanised wire. The many cross wires and knots grip the surrounding stone construction or adobe masonry at regular intervals (1 ft.).

The amount of light sand-cement mortar (mix 1:10) required to fix the cross wires between the stones is considerably less in comparison to other masoned constructions. Less cement mortar results in considerable material and financial savings. Stabilised cement mortar is a large improvement over clay-mud masonry and avoids wind and rain erosion of the joints.

The GWR works best when the reinforcement is placed along the faces of each wall. Thick walls are heavier, but in a thicker wall, the distance between the wires is also increased and the increased moment-arm of the reinforcement will be more resistant to bending. The GWR is difficult to apply in very thin walls; instead, the thin wall sections should be properly framed.

An earthquake motion perpendicular to a wall will bend the wall between the supporting cross walls. With the GWR inside the outer faces of the wall, stress forces will be applied to the reinforcement, resisting the bending force. The alternating forces of an earthquake make the outside placed wires work in alteration.



Plan View of a Bending Stone Wall
showing where the GI Wire takes tension and the stones pointed with cement mortar take the compression.

To improve resistance against compression, the open spaces between the stones of both faces need to be pointed with cement mortar. Cement mortar is needed because the outside face is subject to wind and rain erosion. Thick walls have more benefit from the GWR than thin walls. <snip text>

6. Calculation of the GWR Technique

The maximum horizontal earthquake forces (kgf) in the shear walls of a simple building can reach up to 20% of the weight of the stone mass (kg). A stone wall of 4 meters in length, 2.5 meters high and with a thickness of 50 cm (18 inch plastered) has a volume of 5 m^3 and a mass of nearly 9000 kg. The section of one Ø 2.3 mm GWR wire is 4 mm^2. The stress resistance of the cold-deformed wire is around 1500 N/mm^2 or 150 kgf/mm^2, giving one wire a resistance of well over 600 kgf. Practical stress tests indicated that a double 2.3 mm wire would break at well above 1200 kgf.

Distributing the mass of the above wall over a number of wires indicates that only 8 horizontal wires are needed or five layers of double wires. This is achieved by placing the GWR in every alternating layer. In practice, the forces in a stone masoned wall are partly absorbed by micro-shifting of the stones during a major earthquake and first resisted by the wall itself. The GWR is important in holding the two faces of the wall together; if this is not done, these walls will cause early failure.

The vertical framing reinforcement is created by folding the ends of the wire-mesh up into the hollow of the L- and U-shaped corner blocks. Several double wires will be collected vertically and overlap. Six stress reinforcement wires provide 3600 kgf, being equal to two concrete reinforcement steel bars Ø10 mm, the common reinforcement and adequate for framing.

The GWR as applied by local masons in remote mountain villages is not an exact calculable technology, as much as the actual force of the earthquake cannot be established for the housing site. The purpose off the GWR is to hold the stone construction better together than loose wooden beams (which do not exist) and with that provide better safety for the inhabitants of the house.

The GWR strips can be pre-manufactured, point-welded, double galvanised and supplied in rolls of a few standard sizes. Winding the cross-wire extensions can be done quickly on-site using a small hollow tube tool.

Width of GWR

The width of the GWR depends on the type of building materials, as follows:

- Adobe and Soil Block Walls: For 16-inch walls, 14-inch wide GWR. For 12-inch walls, 10-inch wide GWR.
- Semi-Dressed Stone Walls: For traditional 18-inch walls, 16-inch GWR with light cement mortar in the joints (mortar mix 1:10).
- Cement Block Walls: Hollow 6-inch cement blocks are preferably used in light wall constructions, providing stability and some thermal insulation. For the 6-inch hollow cement block walls, 5-inch wide GWR is recommended.

Factory-Produced Point-Welded

When the wire strips are factory point-welded, pre-manufactured rolls of 100-200 m can be marketed in the same way as rolls of barbed wire. The cross wires should be thinner (2 mm) than the length wires (2.3 mm). The point-welding process needs to be carefully controlled so that the actual section of the length wire will not reduce to less than 2 mm.

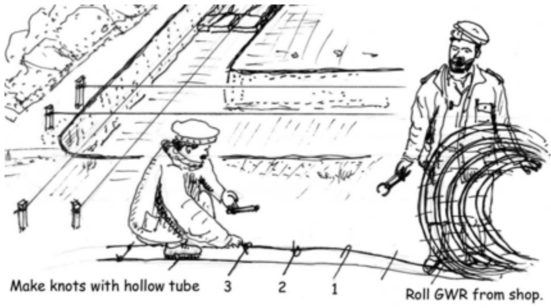
Double Galvanisation after Welding

The cross wires (2 mm) are straight and stick 2 inches out from the length wires. After the welding process, the wire strip needs to be double galvanised conform the treatment for good quality barbed wire. The GWR can be rolled to facilitate transport (200 m = 10 kg).

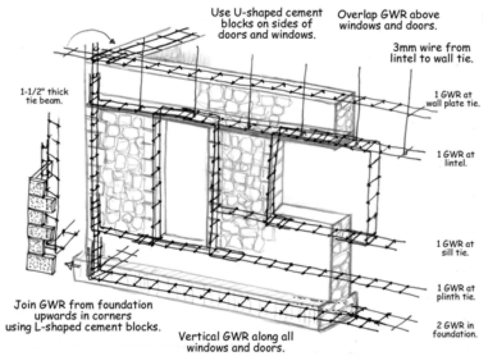
Requirement for a Core House

Minimal 400 m wire (20 kg) is required for a 50 m^2 core house. Each layer in a 50 m^2 core house requires 50 m length of GWR. An earthquake-resistant ground-floor-only house requires six layers of GWR: 2 x foundation, 1 x plinth, 1 x windowsill, 1 x lintel and 1 x roof tie. In addition, vertical reinforcement is recommended along all door and window frames, overlapping above the openings.

On-Site Application



The GWR is unrolled on the building site and cut to length. This length is longer than the wall sections to allow overlap and anchorage to the next section. The mason who applies the GWR in the walls is required to wind the 2-inch extension twice around the main wire or to joining sections of the GWR. This winding will enhance the friction and adherence in adobe constructions and, with a little cement mortar on the winding, it will greatly enhance the linkage with semi-dressed stone constructions. The metal winding tool is about the size of a thick ballpoint pen.



As indicated in Chapter 3, shear wall reinforcement requires vertical reinforcement to be at the ends of the walls and along all window and door openings. The GWR needs to be folded upwards at the corners of the walls and along the doors and windows. The upward folded GWR will meet other GWR sections coming from lower layers. The overlapping strips of vertical GWR form the vertical wall reinforcement.

L- and U-shaped cement blocks have been designed for easy masonry work. These cement blocks are placed in the wall corners and along the sides of door and window openings. The cement blocks have three important advantages:

1. With the placement of the cement blocks, straight vertical edges are first masoned upwards at all corners and allow easy in-fill of the cut-stone masonry. A string is stretched between the raised corners. This saves substantial time in masonry work.
2. The vertical GWR can be pulled upwards in the space between the cement blocks and the stone masonry. The cement blocks will function as formwork allowing easy filling of the space between the blocks and the stone wall with light mortar and small stones.
3. The architecture obtained by the combination of the corner cement blocks and cut-stone in-fill work is aesthetically very appealing.

Various moulds can be used for making the L- and U-shaped cement blocks. One manual type of mould is described below – the rack mould. Large quantities of good quality cement blocks can be rapidly made with this mould.

The rack mould consists of a 2 mm sheet metal mould (open at both the top and bottom) and a rack with a compacting angle iron fitted to it. The mould is placed on a flat, sanded cement floor and filled to the rim with fairly dry aggregate cement mortar (8:1). The rack is then lifted over the mould and set down with force onto the mortar in the mould, compacting the mortar. An additional amount of mortar mix is added to the mould and the compacting with the rack is repeated. Once firmly compacted, the mould is lifted while the rack holds the freshly compacted cement block down. When the mould is free from the freshly cast block, the mould and rack together are further lifted and placed aside to make the next block. The mould may not be tapered inside, otherwise it cannot be lifted.

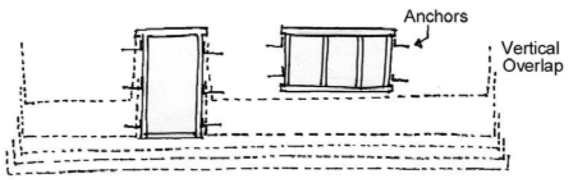
7. Building a House

<u>Foundation</u>

Traditional houses constructed on slopes often have high foundations built in rough stone masonry. The space inside the foundation is frequently used as cattle sheds or storage for fodder and firewood. Although this basement may not be sophisticated enough for living quarters, the walls have to bear the superstructure as it will receive the first horizontal impact by an earthquake jolt.

Depending on the soil type and boulders in the subsoil, reinforcement of the foundation is important to prevent any settlement cracks in the higher walls. The GWR needs to be applied in every layer of the foundation and cover its whole width. For wide foundations, wide GWR strips are used or several strips can be joined lengthwise, overlapping each other.

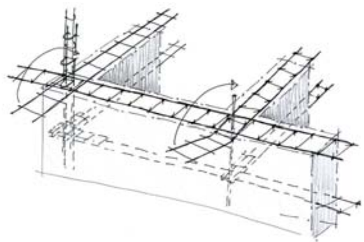
To provide anchorage for the vertical wire reinforcement at the wall ends, several GWR strips are bent upwards at the corners, overlapping for 1-2 ft. to create the corner columns. This vertical reinforcement will run upwards inside the L- and U-shaped cement blocks.



IN FOUNDATION, IN EVERY LAYER

<u>Wall Junctions and Corners</u>

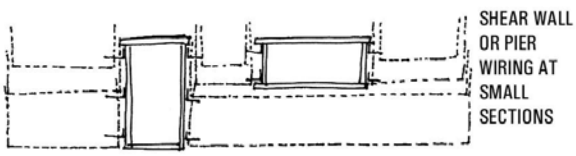
The horizontal GWR is folded over at the corners to make a turn or folded over the cross strip; thus anchoring one wall to the other. This applies to corners and T-junctions.



FOLDING THE EXCESS LENGTH OF THE WIRE-MESH BACK
TO IMPROVE ANCHORAGE WITH VERTICALS

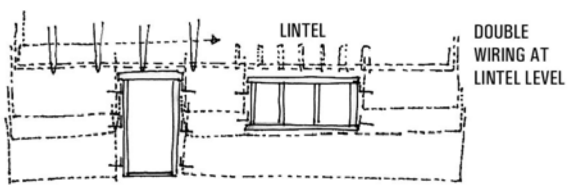
<u>Shear Walls</u>

Between wall ends and openings, shear wall reinforcement needs to be applied. When the wall sections are narrower than the height, pier reinforcement has to be created. Shear wall reinforcement consists of several layers of GWR over the height of the wall section and the bent-up ends of the wall reinforcement into the vertical columns. Pier reinforcement is created by doubling or tripling the vertically folded-up GWR strips along the sides of the doors, windows and wall ends, thus making reinforced columns.



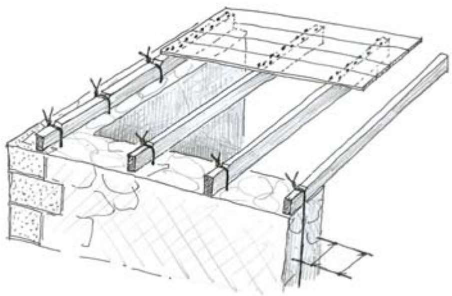
<u>Door and Window Lintel Tie Beams</u>

All door and window lintels need to be connected throughout the building; also over inside walls. For door and window lintels, the GWR is doubled, depending on the span, and loops of 3 mm wire are attached to the GWR at short intervals. The 3 mm wire loops are hooked into the GWR and between the hoops, a solid line of stones or cement blocks is masoned as pressure zone.



STRAPS FOR FLOOR BEAMS

Floor / Roof Beams – Diaphragms



Starting at the door and window lintel level, a 3 mm galvanised wire is hooked into the GWR to provide anchorage for floor and/or roof beams and wall plates. The beams need to be notched over the middle of their supports. The two ends of the 3 mm wire are tied down over the notches to avoid that (in case of a major earthquake) the beams slip out of these anchors. The floors and the roof should form a diaphragm with all walls, providing full bracing at every floor level and the roof.

The 3 mm wire straps are tied together over the notches in the floor beams. To provide a connection between the wall running parallel to the floor beams and the floor beams themselves, a wall beam is tied onto the parallel wall. The flooring or roofing is nailed to this wall beam and also over all the floor beams; thus creating an integral floor diaphragm. <snip text and many drawings>

8. Discussion

When discussing the calculation concept of the GWR with the Pakistan Institute of Engineers (PIE), the staff agreed on the concept, but due to the nature of the vernacular house building practice in the remote mountain areas, precise figures could not be verified. The conceptual thinking about how to reinforce non-engineered constructions was in strong contrast with their detailed knowledge on reinforced concrete design drawings, where steel reinforcement and concrete quality could be precisely defined on paper in order to dimension the construction and its reinforcements.

The professional staff of the PIE agreed that the design value of reinforced concrete quality would seldom be reached in practice due to lack of on-site supervision of the contractor and adverse climate conditions, and similarly the actual construction quality of village mason-built dressed stone houses could not be defined in precise figures.

Nonetheless, the PIE staff agreed that the concept of the GWR was valid. On the other hand, they commented that this building method <u>could not be applied</u> by professional engineers or architects because the methodology was **"not taught at schools, not documented and approved by international testing institutes, and not approved as building practice in Pakistan"**. It was pointed out by the author that by adhering to such criteria, building innovation would not be possible.

In addition, they suggested that the technology could not be technically approved by the PIE when the theoretical and practical field studies and testing had not been realised. The PIE required substantial external finance to realise those tests; funding from the project was unavailable. Moreover, they did not consider the certification of the GWR building method a very feasible option because the application would be totally outside the control mechanisms of the PIE and also outside that of the regular government authorities (Public Works Department). Taking the problems, earthquake risks or interest at heart of the <u>remote rural and mountain population was not their first priority</u>. The pressing needs of upcoming high-rise buildings in the country and the generally poor quality of currently executed concrete works all over the country were of greater concern.

After the Kashmir earthquake of 8 October 2005, the Institute of Architects of Pakistan (IAP) proposed the GWR technology in a number of their design drawings based on cement block construction. The advantage was that the cement block constructions were lighter in mass/weight and the construction could be properly calculated. New houses could be mass produced.

Approximately 75,000 people perished in the Kashmir earthquake, nearly all of them people living in dressed stone, adobe and poor quality reinforced concrete houses. Nearly an entire primary school generation died in the poorly constructed government schools (see page 13 of this paper).

The October 2005 Kashmir earthquake demonstrated the following:

- The apparent lack of understanding of earthquake-resistant design by the Public Works Department (PWD) who built the schools and other buildings, many of which collapsed.
- The need of better reinforcement designs for dressed stone constructions.
- The past total lack of any control on the building design and construction by the PWD.
- The need for intensive training of local masons on earthquake-resistant construction.
- The need for appropriate and low-cost, but durable, solutions for remote mountain people.

9. Conclusions

1. The GWR provides a low cost, but effective, galvanised metal (non-rust) reinforcement for thick **dressed stone wall constructions**. It enhances the internal bonding of the wall, especially between the faces, and provides adequate horizontal stress reinforcement when applied in alternating layers.
2. With the GWR, various widths and lengths of reinforcement can be easily applied in all types of thick-wall dressed stone masonry construction, even in the most remote mountain village because it is easy to transport and can be manufactured locally. It can also be factory pre-welded.
3. Wall constructions should be interconnected with the vertical reinforcement at all junctions and endings. Vertical reinforcement is placed within L- or U-shaped cement blocks. Because the GWR is double galvanised, only light cement mortar (mix 1:10) is used.
4. The use of horizontal and vertical stress reinforcement alone does not guarantee a better earthquake-resistant house; it is the combination of the stress reinforcement placed in the correct way and within good quality stone work. Masoning the stone work with a light cement mortar (mix 1:10) is necessary to achieve a good overall building strength. The GWR is not a magical addition; eliminating other basic precautions, such as light cement mortar masonry and anchoring of all cross walls, will result in a weak construction. Improved housing is a product of proper material use, workmanship and building site technology.
5. The application of the GWR technology in dressed stone construction will allow a first or second storey for traditionally built houses, providing an appealing architecture.
6. With large earthquake impacts, the cross wires will hold the stones in place in the two horizontal directions and absorb part of the impact. The coherence of the construction will be maintained with the vertical framing of all wall sections, the application of tie beams and floor diaphragms, and the application of light cement mortar.
7. The GWR technology should include the following:
    - Instruction manual in its use and application and how to plan a building on a safe site.
    - Manufacturing and delivery of the L- and U-shaped block-making equipment.
    - Availability of the different GWR widths in local shops, together with user guidelines.
    - Availability of 2.3 mm and 3 mm wire and cutting tools for making lintels and roof ties.
    - Assembling instructions for complicated structural designs..
8. Using the GWR in new **cement block houses**, being masoned in light cement mortar, greatly enhances the overall strength while minimising possible rust due to low cement ratios. The GWR can be used vertically or cast-on concrete framing can be used.
9. The GWR technology also greatly enhances the strength of **adobe or rammed earth constructions.** In adobe block constructions, the GWR is placed in every alternating layer; in rammed earth constructions in every layer. These adobe and rammed earth constructions exist in many countries regularly affected by earthquakes, such as Turkey, Afghanistan, Tajikistan, Iraq, Iran, Pakistan, India, Nepal, Tibet, Mongolia and China, as well as in many Andean countries, such as Guatemala, Honduras, Nicaragua, Columbia, Ecuador, Peru, Chile and Bolivia.

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Acknowledgements

The first ideas of the wire-mesh technology were developed in Yemen Arab Republic after the December 1982 earthquake and the following reconstruction programme in the Dhamar district. The technology is based on the behaviour of buildings during earthquakes described in extensive documentation obtained from several earthquake technology institutes worldwide, including India, USA, Japan, Turkey and Chile. In Guatemala in 1975, galvanised cattle fencing wire was already proposed by Care for the reinforcement of rural adobe constructed housing.

During the further development of this technology in 1999-2001, architect Sjoerd Nienhuys (author) was Technical and Programme Director of the Building and Construction Improvement Programme (BACIP), a development programme of the Aga Khan Foundation. He was assisted by Ahmed Saeed Shaikh, Deputy Director, and Qayum Ali Shah, Manager Field Operations, who built his own house using this technology.

BACIP was a project of the Aga Khan Planning and Building Services, Pakistan. The programme was financed by PAK-SID, a collaborative venture between the Canadian International Development Aid (CIDA) and the Aga Khan Development Network. In addition, the cost of the Programme Director (myself) was financed by The Netherlands Directorate General for International Cooperation (DGIS). The technology was fully developed under this programme. Pictures have been added from the Kashmir earthquake of 8 October 2005, showing the deficiency of cut-face stone architecture.

Drawings, sketches and photographs by:
Indian Earthquake Research Institute documents (1976) – sketches on pages 6-7.
Sjoerd Nienhuys – all other drawings, sketches and photographs

In order to comply with the requirements of the ICSBE-2010 , the pictures have been reduced and text sections as well as the chapter 8 on construction technique has been removed.

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Aga Khan Planning and Building Service, Pakistan (AKPBS,P)
310-311, 3rd floor Kassam Court
BC-9 Block-5 Clifton
Karachi 75600, Pakistan akpbsp.k@akpbsp.org website: www.akdn.org

Project Director, Author: Ing. Sjoerd Nienhuys, Architect
Burg. Gülcherlaan 25, 1217 NX, Hilversum, The Netherlands
Email: sjoerd@nienhuys.info website: www.nienhuys.info

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Shear Strength of Precast Prestressed Concrete Hollow Core Slabs

Wijesundara K.K[1], Bolognini D.[2], Nascimbene R.[2]

[1] European School for Advanced Studies in Reduction of Seismic Risk (ROSE School), Via Ferrata 1 - 27100 – Pavia, Italy, email: kushan.wijesundara@eucentre.it, tel: 00393331253734.

[2] European Centre for Training and Research in Earthquake Engineering (EUCENTRE) Via Ferrata 1 - 27100 – Pavia, Italy.

Abstract

Since early eighties, the precast prestressed concrete hollow core slab cross sections with non-circular voids became gradually popular, first in 400 mm thick slabs, then in 500 mm thick slabs. However, it is evidenced that this type of deeper slab sections have subjected to initial web shear cracking when they are provided longer supports and resist for higher line loads acting close to supports. Therefore, the objective of this study is to review the equations specified in American Concrete Institute (ACI), Eurocode 2 (EC2) and Canadian Standards Association (CSA) specifications to evaluate the shear strength of a member having no transverse reinforcement as in the case of hollow core slabs.

For this purpose, the experimental test data on hollow core slabs are collected from past experimental programs and detailed finite element analyses are performed. Based on experimental and numerical results, it could be concluded that the evaluation of shear strength by the equations specified in ACI, EC2 and CSA specifications are conservative for the slab cross sections with circular voids while ACI and EC2 predictions are not conservative for deeper slab sections with flat webs. However, CSA predictions for all types of hollow core slab sections are more conservative than ACI and EC2 predictions.

Keywords: Hollow core slab; Shear strength; Prestressed concrete; Precast members; Circular Voids; Flat weds

1. Introduction

Prestressed hollow-core concrete slabs were developed in the 1950s, when long-line prestressing techniques evolved, and for more than 30 years the type of units produced changed little. These slabs made of high-strength concrete, are prefabricated concrete members with large hollow proportions. In practice, they are interconnected after assembly by joint grouting compound. In comparison with conventional concrete members, this type of concrete slabs has a lot of economical advantages, especially in saving material, energy and in reducing weight of transportation. Outstanding features are quality control, schedule time and costs. Additionally, formworks which are used to produce in-situ concrete are saved in application of these slabs. The first prestressed hollow core slabs were 150, 200 or 265 mm in thickness. They were provided with circular voids. Since the early eighties, slab cross sections with non-circular voids became gradually popular, first in 400 mm thick slabs, then in 500 mm thick slabs. These deeper slab units are increasingly used in industrial buildings, office buildings and also in domestic architecture where it is needed to have large open parking spaces on the ground floor. As a subsequent of that these types of hollow-core slabs were developed to resist the higher loads and to support for longer span. Typical cross-sections of the slabs with circular voids and non circular voids (flat webs) are shown in Figure 1. In a section with non circular voids, the inner webs have a constant thickness over a depth of $h/3$. The outermost webs are only slightly tapered due to the non-verticality of the outer edges.

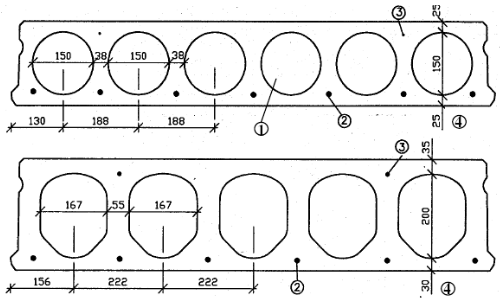


Figure 1. Typical slab cross-sections with circular and non-circular voids

These deeper slab sections provided longer supports or resisted for higher line loads acting close to supports, subjected to the initial web shear cracking. Subsequently, It was realized that resistance of the slabs with flat webs against web shear failure was considerably lower than shear strength evaluated by the equations specified in ACI (2005) and EC2 (2005) specifications. The experimental studies by Pajari (2005) and Hawkins and Ghosh (2006) have alos illustrated that web-shear cracking strengths in end regions can be less than strengths computed by traditional equations specified in ACI (2005) and EC2 (2005) specifications.

Therefore, the first part on the study is going to compare the shear design approaches of 3 different specifications: ACI (2005), EC2 (2005) and CSA (2001). The equations specified in ACI (2005) and EC2 (2005) specifications to evaluate the shear strength are based on predicting diagonal cracking loads by considering stresses at the centroid while the equation in CSA (2001) is based on the Simplified Modified Compression Field Theory (Vecchio et al.(1986), Collins (1997) and Angelakos et al. (2001)) which considers the post-cracking shear strength of a member. This part of study also presents the comparison of observed shear strength from experiments with the code predicted shear strength. The second part of the study presents the results of finite element analyses of typical hollow core slabs with wide range of depths to explore why the deeper slabs subject to web shear cracking at lower loads than the shear strength evaluated by the equations specified in ACI (2005) and EC2 (2005) specifications. For this purpose, 220, 300, 400, and 500mm deep sections are selected and all details of these sections are taken from a manufacturer of prestressed hollow core slabs.

2. Evaluation of Shear Strength

2.1. ACI specification

The use of shear reinforcement is generally not feasible for hollow core slabs and, therefore, the shear strength, particularly of deep slabs, may be limited to the shear strength of the concrete. Section 11.4 of ACI (2005) gives the requirements for evaluating the shear strength of concrete. The provisions of the section 11.4.3 are likely to be used if shear is a controlling factor in the design of hollow core slabs. In Section 11.4.3, the factored shear force V_u is limited to the lesser of φV_{ci} and φV_{cw}, where V_{ci} is the flexure-shear cracking strength and V_{cw} is the web-shear cracking strength. For simply supported hollow core slabs, the shear cracking strength of the web adjacent to the support usually control the design of the unit, unless the design loading includes heavy, non-uniform loads. The φ value for shear calculations is 0.75. The nominal shear strength provided by concrete (web-shear cracking strength) V_{cw} is given in ACI (2005) as:

$$V_{cw} = (0.29\sqrt{f_c^{'}} + 0.3 f_{pc}) b_w d + V_P \tag{1}$$

Where $f_{pc}=P/A$, $d= y_t+e$ but not less than 0.8h, b_w is the width of the section at the centroidal axis and V_P is the vertical component of the prestressing force.

2.2. Eurocode 2 specification

In prestressed single span members without shear reinforcement in regions uncracked in bending (where the flexural tensile stress is smaller than f_{ctk}), the shear resistance should be limited by the tensile strength of the concrete using expression (6.4) in EC2 [2005] as:

$$V_{Rd,c} = \frac{Ib_w}{S}\sqrt{(f_{ctd})^2 + \alpha_1 \sigma_{cp} f_{ctd}} \tag{2}$$

where I is the second moment of area, b_w is the width of the cross-section at the centroidal axis, S *is* the first moment of area above and about the centroidal axis, α_l equal to $l_x/l_{pt2} \leq 1.0$ for pretensioned tendons and otherwise it equals to 1, l_x is the distance between the section considered from the starting point of the transmission length and the section considered at the distance of half of the slab thickness, l_{pt2} is the upper bound value of the transmission length of the prestressing element according to Expression (8.18) in EC2 (2005), σ_{cp} is the concrete compressive stress at the centroidal axis due to axial loading or prestressing ($\sigma_{cp} = N_{Ed}/A_c$ in MPa, $N_{Ed} > 0$ in compression) and f_{ctd} is defined as the design tensile strength.

2.3. CSA specification

The equation for the evaluation of shear strength in CSA (2001) is based on the Simplified Modified Compression Field Theory (SMCFT) which considers the post-cracking shear strength of the member. Factored shear strength V_c shall be determined by clause 11.3.4 in CSA (2001) as:

$$V_c = \phi_c \lambda \beta \sqrt{f_c^{'}} b_w d_v \tag{3}$$

From the clause 11.3.6.4 in CSA (2001), β is defied as:

$$\beta = \frac{0.4}{(1+1500\varepsilon_x)} * \frac{1300}{(1000+S_{Ze})} \tag{4}$$

The longitudinal strain ε_x at mid-depth of the cross-section can be computed from Eq. (5)

$$\varepsilon_x = \frac{M_f/d_v + V_f - A_P f_{po}}{2(E_p A_p + E_c A_{ct})} \tag{5}$$

Where M_f and V_f shall be taken as positive quantities and M_f shall not be taken less than $(V_f - V_p)/d_v$

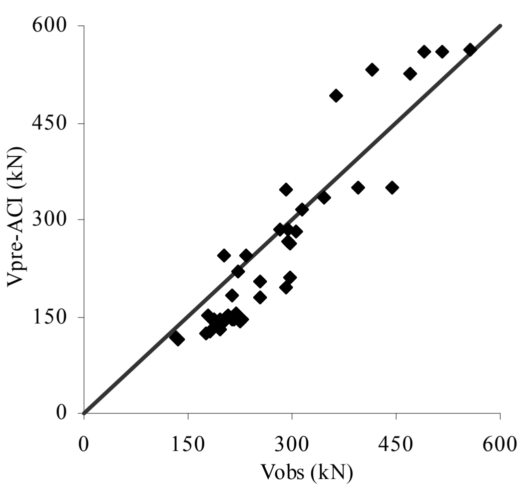
$$S_{Ze} = \frac{35 S_z}{15 + a_g} \tag{6}$$

However, S_{Ze} shall not be taken as less than $0.85S_Z$ and S_Z shall be taken as d_v. a_g is maximum size of coarse aggregate and effective web width shall be taken as the minimum concrete web width within the depth. The prestressing force may be assumed varying linearly from zero to full development in the transfer length which is assumed to be 50 times the diameter of strand as in ACI (2005) Specification. The resistance factor for concrete, φ_c is taken as 0.65 while for low density concrete it is equal to 1.

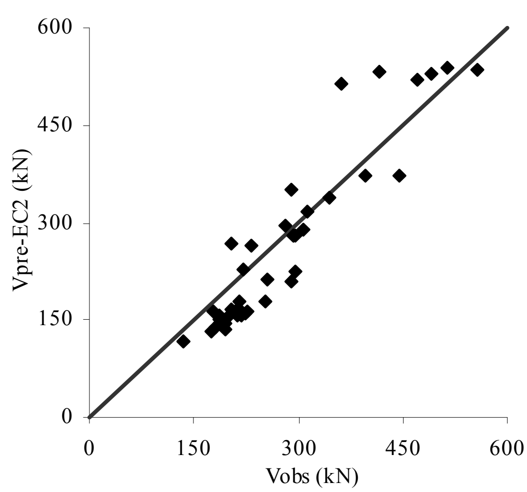
3. Comparison of Code Predictions with Experimental Data

Main objective of this part of the study is to validate the accuracy of evaluation of shear strength of precast prestressed concrete hollow core slabs by the equations specified in ACI (2005), EC2 (2005) and CSA (2001). For this purpose, test data from forty four specimens are selected from the research report by Pajari (2005). It is also important to note that those specimens were simply supported, isolated (not a part of a floor) and loaded with transverse uniformly distributed line loads. The test specimens which have excluded, are only those in which the slabs had grouting at the loaded end, some important data as the measured strength were missing, the shear span (distance from support to the nearest line load) was lesser than 2.4 times the slab thickness and the slippage of strands was greater than that acceptable in the Finnish quality control for type approved slabs. Altogether, 15 different nominal geometries for concrete cross-section were identified in the accepted test specimens.

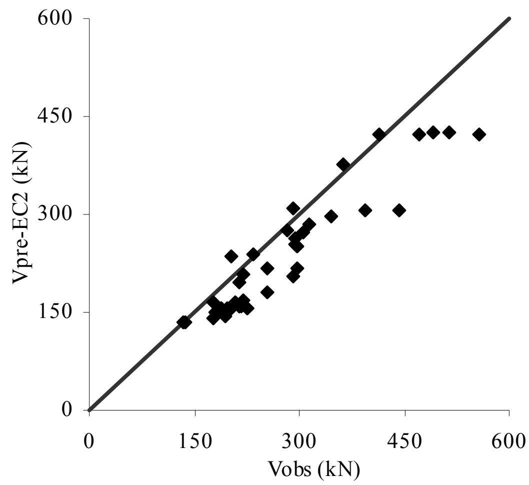
Figure 2 (a), (b) and (c) illustrate the comparison of shear strength values obtained from the tests with the predicted shear strength by ACI (2005), EC2 (2005) and CSA (2001), respectively. In these figures, V_{obs} refers to the shear strength obtained from the test (shear force at support) while V_{pre} refers to the predicted shear strength by the code equation.



(a)



(b)



(c)

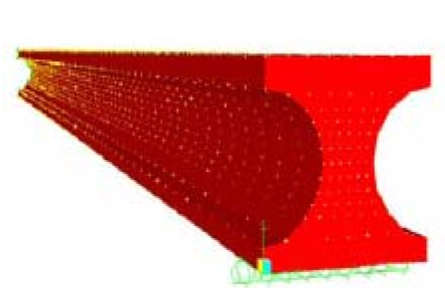
Figure 2. Comparison of observed shear strength with the predicted shear strength by (a) ACI, (b) EC2 and (c) CSA.

Furthermore, it is important to highlight that the predicted shear strength by ACI (2005) and CSA (2001) codes are evaluated using the material safety factors of 1.0 while the predicted shear strength by EC2 (2005) is evaluated using the characteristic tensile strength for this comparison.

It is clear from the comparison that the shear strength values predicted by ACI (2005) and EC2 (2005) for the shallow sections with circular voids are mostly conservative, but they are not conservative for the deeper sections with non circular voids. However, CSA (2001) predicts conservative estimation of shear strength values of all the sections selected for this comparison.

4. Finite Element Modelling

In order to investigate the stress distribution close to the support under a symmetrically loading condition in different slab units and in turn to validate the assumption that the shear stress would reach its maximum value at the neutral axis down to the flexural steel, made in deriving the equations to evaluate the shear strength in ACI (2005) and EC2 (2005) using the Mohr's circle of stresses, the four finite elements models of slab units: 220mm, 300mm, 400mm and 500mm in depth selected from the experimental program are modelled in SAP 2000 program. Figure 3 illustrates the 3D view of a model with circular voids and the loading arrangement.



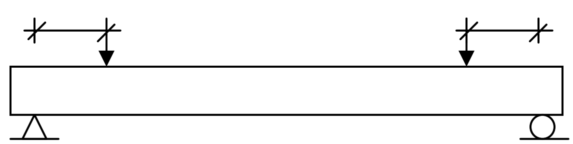


Figure 3. 3D view of a model with circular voids and the loading arrangement

Due to symmetric loading arrangement, half of the beam was modelled. The end details are critical of this type of slab units. Typically, these slabs sit on the small pads only 50mm long right at the ends of the slabs. Hence, all beams were modelled with roller support at one end (50mm distance away from edge of the beam) while all translational degrees of freedom except the vertical translation at other end of the model were restrained.

A vertical line load, corresponding to the experimental failure load, is placed on the top of the model at a distance of five times d_v from the support. The spans of the beams are selected to have same span over effective depth ratio of 25. The prestressing force is transferred in the model as shown in Figure 4. The transfer length is defined as the length required building up the full prestressing force in the concrete. As suggested in the ACI (2005) specification, the transfer length is taken as 50 times the strand diameter.

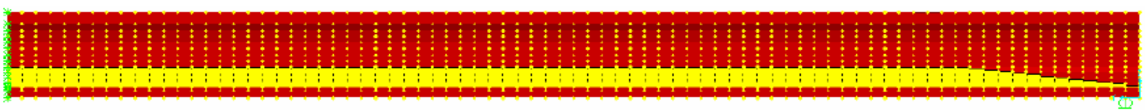


Figure 4. Transferring of prestressing force.

Concrete is modelled as a homogeneous material which results more general behaviour of the beam with the properties reported in the experiments. Modulus of elasticity of the unconfined concrete is calculated using the equation as:

$$E_c = 3320\sqrt{f_c^{'}} + 6900 \text{ MPa} \tag{7}$$

The reinforcements in hollow core slabs are consisted of only longitudinal strand. Each of strands built of seven wires of low relaxation strands are modelled as cable elements using the properties as reported in the experimental program assuming full interaction with concrete.

4.1. Results of finite element models

Figure 5 (a), (b) and (c) exhibit distributions of the direct axial, shear and principle tensile stress components in the model with a section of 300 mm in depth and including the circular voids at the corresponding failure load as reported in experiment program. Figure 6 also shows the distribution of the three stress components of the model with 400mm in depth and consisting of non circular voids.

It is clear from Figure 5 that the maximum shear and tensile stresses are developed at the centroid of the section as it is assumed in deriving the code equations that the shear stress would reach its maximum value at the neutral axis. Since, the maximum tensile stress developed at the web has reached to its peak value, web shear crack could be developed leading in shear failure of the slab rather than having a flexural failure because of rapid propagation of the crack after the initiation of the web tensile crack in the web.

Unlikely previous results, Figure 6 indicates that the maximum shear and tensile stresses are not developed at the centroid of the section. They are more concentrated towards the bottom of the beam because any deformed section close to the support is no longer remained in plane with deeper section consisting of flat webs. Usually, this region is called a disturb region as observed between the support and the loading point and it violates the concept of plane section remained in plane and that it is perpendicular to the longitudinal axis of the slab. Therefore, at lower loads than the evaluated shear strength of such members by the ACI (2005) and EC2 (2005) specifications, web tensile crack could be initiated due to higher concentration of shear stresses.

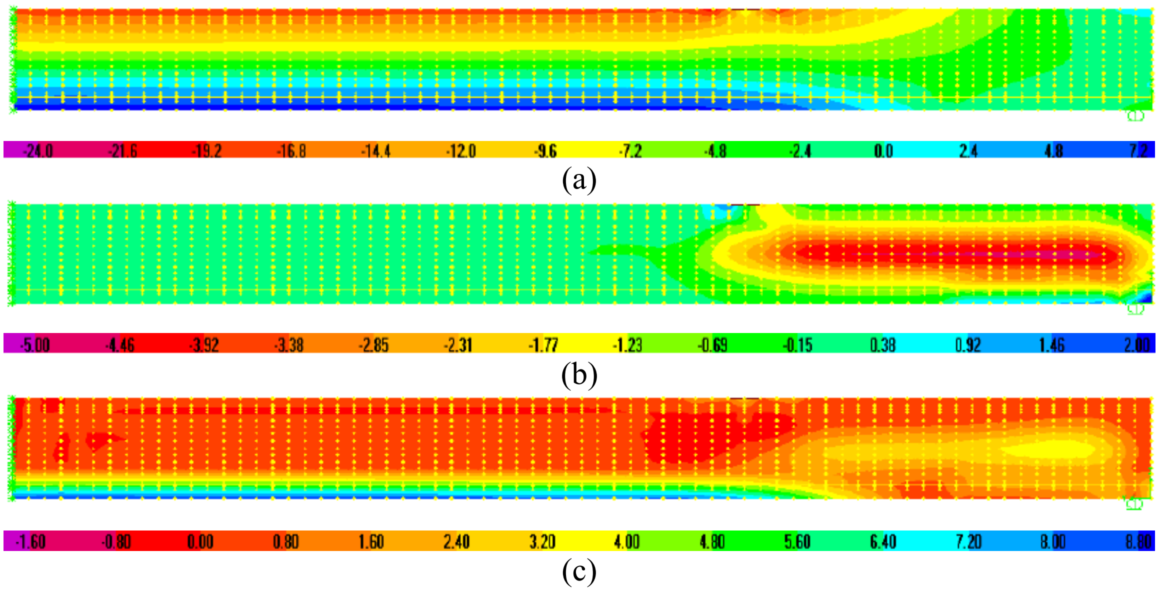


Figure 5. Distribution of (a) direct stress, (b) shear stress and (c) principle tensile stress in MPa of 300mm deep section with circular voids

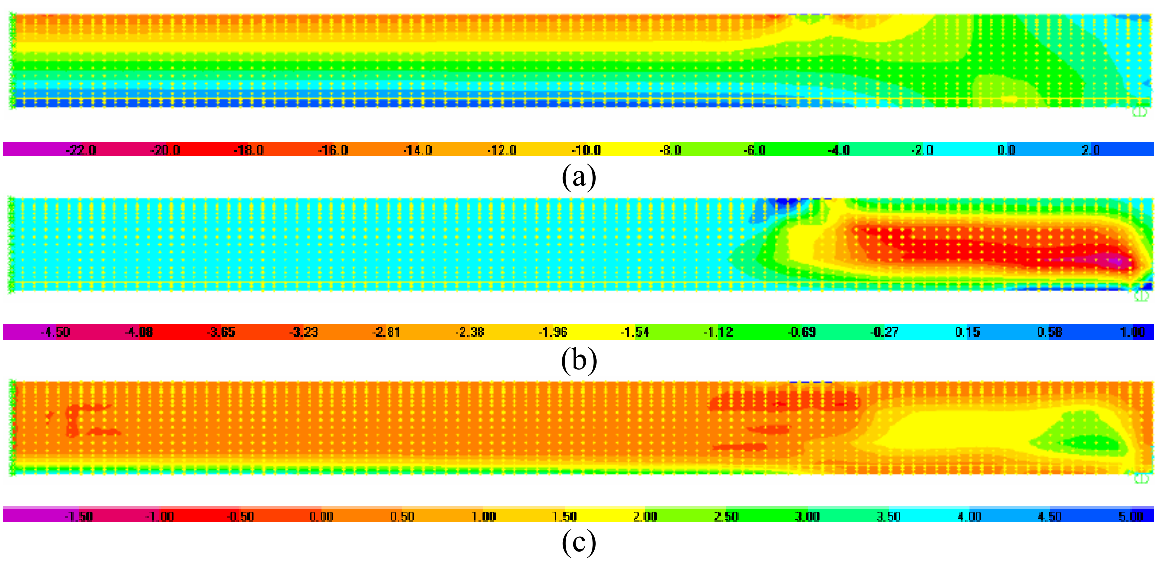


Figure 6. Distribution of (a) direct stress, (b) shear stress and (c) principle tensile stress in MPa of 400 mm deep section with flat webs.

5. Conclusion

ACI (2005), EC2 (2005) and CSA (2001) specifications propose equations to evaluate the shear strength of a member which have no transverse reinforcement. To check the validity of these equations, finite element analyses and 44 experimental tests on precast prestressed hollow core slabs with thickness varying from 220 to 500 mm have been performed. Based on the results the following conclusions can be drawn.

- According to the experimental test data of 265 and 320 mm deep sections with circular voids, ACI (2005), EC2 (2005) and CSA (2001) specifications predictions are conservative. Finite element analyses illustrate that 220,300 mm deep sections with circular voids follow the assumption that plain section remains in plain at the section where the shear forces are high and that the maximum principle tensile stress occurs at the mid depth of the slab. Therefore, the results of finite element analyses give strong support that code predictions are conservative for these types of sections with circular voids.
- As the slab depth becomes larger with flat web, the stress distribution becomes non linear with tensile and shear stresses concentrating towards the bottom of the beam. So, Morch (1902) prediction that the shear stress would reach its maximum value at the neutral axis down to the flexural steel is not going to be valid for deeper precast prestressed concrete hollow core slabs with flat webs. The maximum value of shear stress is much higher the predicted shear stress by code equations and the maximum principal tensile stress is not at centroid of the section .As a result of that, the equations in ACI (2005) and EC2 (2005) specifications, derived from the Mohr's circle of stress and based on the assumption made by Morch (1902), estimate the non conservative strength values for deeper prestressed hollow core slabs with flat webs.
- CSA (2001) prediction on the shear capacity is based on the Simplified Modified Compression Field Theory and it estimate the conservative shear strength values for all kind of sections used in this study. It is because the modified compression field theory, used in the CSA (2001) to calculate the shear stresses at each level, treat concrete as a diagonally cracked material and interface shear stress, often called aggregate interlocking, is estimated by average tensile stress. The interface shear plays an important role in the determination of the shear strength of the members without transverse reinforcement. The stress strain relationship for the concrete indicates that average tensile stresses, after concrete diagonally cracked, are comparatively lower than the tensile stress at the first crack.
- Compressive strength of concrete used in precast prestressed hollow core slabs is comparatively high. High strength concrete member are smoother than in normal strength concrete members with cracks propagating through coarse aggregate particles rather than around them. So the tensile strength at cracking of the members with high strength concrete may be lower than the tensile strength at cracking used in the specifications like ACI (2005) and EC2 (2005). This also should be considered in the design of prestressd hollow core slabs.

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OPTIMUM DESIGN OF BRIDGE SYSTEM SUBJECTED TO DEVASTATING EARTHQUAKE CONSIDERING PERFORMANCE AT ULTIMATE STATE

Kazuhiro Taniwaki[1], Kenta Tanaka[2]

[1]Associate Professor, Department of Civil and Environmental Eng., Fukui University of Technology, Gakuen 3-6-1, Fukui 910-8505, Japan
E-mail: taniwaki@fukui-ut.ac.jp, Telephone: +81-776-292701

[2] Chief Examiner, Chodai Co. Ltd., ootemachi 2-8-4, Naka-ku, Hiroshima 730-0051, Japan
E-mail: tanaka-k@chodai.co.jp, Telephone: +81-82-5456653

Abstract: In this study, a rational and efficient optimal seismic design method for bridge system subjected to devastating earthquakes considering performance at ultimate state is proposed. The bridge system consists of superstructure, rubber bearings, RC piers and cast-in-place concrete pile foundation. In the proposed optimum design method, the optimum solutions for the heights of rubber bearings, cross-sectional dimensions and amount of steel reinforcements for RC piers and the detail of concrete pile foundation are determined for several allowable ductile factors of RC piers considering the constraints on the relative horizontal displacements of rubber bearings to the both bridge and transverse directions, the ductile factor of RC piers, and the constraint on the cast-in-place concrete pile foundation. From the practical design the heights of rubber bearings can take continuous values, but the other variables must be selected from discrete variable sets. Therefore, the construction cost minimization problem can be expressed as a mixed discrete-continuous problem. This problem is transformed into a convex approximation problem with the estimation formulae by using the experimental design, and the dynamic behaviors and those sensitivities are calculated analytically by using the estimation formulae without analyzing the structures. The optimum design problem is solved by a classical branch and bound method with dual algorithm. In the numerical design examples, it is emphasized that the optimum solutions can be obtained efficiently by using the experimental design. It is also demonstrated that the reductions of the heights of rubber bearings and cross-sectional dimensions of RC piers can be observed by increasing the allowable ductility factor.

Keywords: Bridge system, Optimization, Seismic design, Design of experiments, Performance at ultimate state

1. Introduction

After the Hyogoken Nanbu Earthquake in 1995, the seismic design code for highway bridges, JSHB [1], has been revised in order to ensure sufficient ultimate dynamic capacities in the bridge systems for large displacements caused by devastating earthquakes. Recently, the performance-based design method has been introduced for the seismic design at ultimate state in the JSHB. According to the JSHB, the bridge members are not allowed to yield for the frequent earthquakes (Level1), and the bridge members must have the sufficient ultimate dynamic capacities to be able to repair those rapidly after the excitations due to devastating earthquakes (Level2). This task accompanies with tremendous complexity in the process of design of the bridge system. In general, Level2 design is critical for determinations of member sizes for large scale bridge systems. Therefore, the establishment of a rational and efficient optimal seismic design method, which can determine the optimum member sizes considering performance at ultimate state in the Level2 design process, has been awaited expectantly in the practical design.

From this point of view, one of authors proposed an optimal seismic design method using the design of experiments and suboptimization technique [2,3]. In this research works, authors made effort to introduce several relations between construction cost and design variables to make the optimization problem simple. The design variables for bridge members are dealt with as continuous variables and the optimum solutions considering the displacement constraints for bridge direction are determined.

In this study, an rational and efficient optimal performance-based seismic design method for bridge system subjected to devastating earthquakes is proposed. In the design of a bridge system, the dimensions of superstructure are assumed to be given, and the heights of rubber bearings, cross-sectional dimensions and amount of steel reinforcements for RC piers, and numbers of piles and the diameters of piles in the cast-in-place concrete pile foundation are taken into account as design variables. The dynamic nonlinear behaviors of the bridge system are analyzed precisely by using the general purpose nonlinear analysis software (TDAP-III) with the acceleration specified in the JSHB. The relative horizontal displacements between superstructure and piers to the both bridge and transverse directions are dealt with as design constraints for the rubber bearings. The ductile factor, which is given by the ratio of working curvature to the yield curvature, is dealt with as the design constraints for the RC piers so as to ensure the performance specified at the ultimate state. Furthermore, the constraint on the cast-in-place concrete pile foundation is also dealt with to ensure the sufficient ultimate dynamic capacity in the RC pile foundation. However, the constraint on the RC pile foundation is not treated in the optimization process to simplify the optimization algorithm. After determination of optimum solution the constraint on the RC pile foundation is examined, and the RC pile foundation is replaced with the larger one so as to satisfy the constraint.

From the practical design the heights of rubber bearings can take continuous values, but the other variables must be selected from discrete variable sets. Therefore, the construction cost minimization problem can be expressed as a mixed discrete-continuous problem, and it is solved by a classical branch and bound method [4] with dual algorithm and convex approximation [5,6] in this study. The sensitivities of the design constraints need in the optimization process and we encounter the difficulty to obtain those in utilizing the general purpose nonlinear analysis software. To overcome this problem the design of experiments is applied successfully in order to calculate the dynamic behaviors and those sensitivities in the optimization process. In the design of experiments, the estimation formulae for dynamic behaviors are introduced in the expression of quadratic functions of the design variables. The dynamic behaviors and those sensitivities are calculated analytically by using the estimation formulae without analyzing the structures. After the determination of optimum solution the design constraints with the estimation formulae are examined by re-analyzing the bridge system using TDAP-III. In case that the design constraints violate the allowable limits, the estimation formulae for dynamic behaviors are improved and the minimum cost design problem is re-solved. This optimization process is iterated until the relative errors between the estimated design constrains and the exact ones satisfy the allowable limits.

The proposed optimal design method is applied to a five-span continuous steel girder bridge system, and the optimal solutions at various allowable ductility factors of RC pier are compared. In the numerical results, it is demonstrated that the reductions of the heights of rubber bearings and cross-sectional dimensions can be observed by increasing the allowable ductility factor. It is also emphasized that the optimum solutions can be obtained efficiently at a few iterations of improvements of the estimation formulae for dynamic behaviors. The accuracy of the estimation formulae is excellent

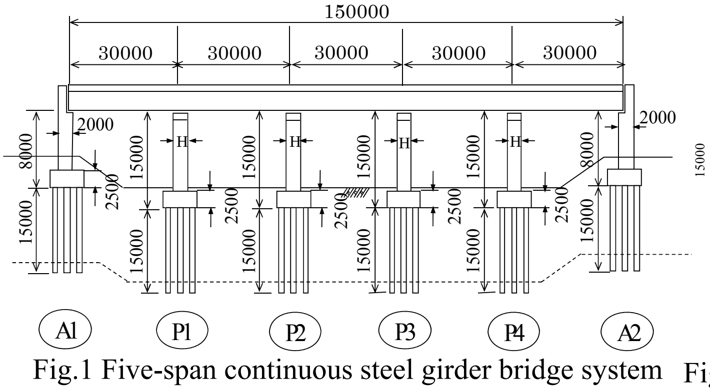

Fig.1 Five-span continuous steel girder bridge system

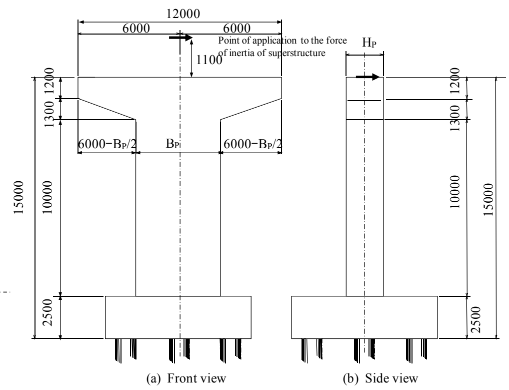

Fig.2 Front and side views of piers and RC pile foundation

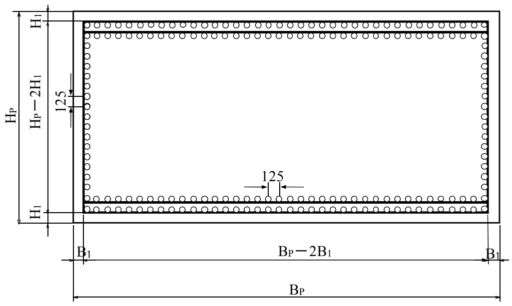

Fig.3 Cross section of a pier

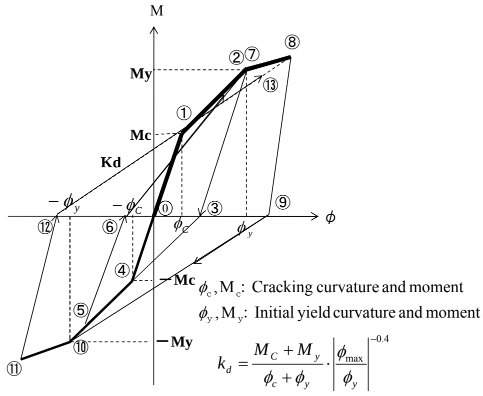

Fig.4 Trilinear rigidity hysteresis model for RC pier (Takeda model)

within 7 percent of relative errors between the exact behaviors and estimated ones.

2. OPTIMUM DESIGN FORMULATION AND OPTIMIZATION ALGORITHM

2.1 Design Model

In this study, the five-span continuous steel girder bridge system shown in Fig.1 is considered in which the superstructure is supported by six rubber bearings, RC piers and the cast-in-place concrete pile foundation. The front and side views of a pier and RC pile foundation are described in Fig.2. The lengths of piles are 15m and five types of soil conditions in stratum are considered to calculate spring constants. The reinforcements in the cross section of piers are arranged in two layers for the bridge direction and one layer for the transverse direction, and the interval of each reinforcement are fixed at 125mm as shown in Fig.3. Following an enlargement of cross section the numbers of reinforcements increase so as to keep the intervals of reinforcements. The stiffness of a RC pier is taken into account as the trilinear rigidity reduction type model (Takeda model) shown in Fig.4. The nonlinear behaviors of the bridge system for the both bridge and transverse directions subjected to devastating earthquakes are analyzed precisely by using TDAP-III in which the Type II standard strong acceleration wave motion model at the Type II soil ground specified in the JSHB is applied. In the time-history response analysis the spring constants of rubber bearings, pile foundations and superstructure are elastic, and both the superstructure and abutment are assumed as rigid body. The piers are divided into 50 segments in order to calculate the nonlinear dynamic behaviors accurately.

2.2 Optimum Design Formulation

In the design of the bridge system, the dimension of superstructure is assumed to be given and widths of rectangular rubber bearings are assumed to be 70cm and 80cm at abutment and piers, respectively. The design variables for rubber bearings are the heights of those at abutment and piers, B_{h1} and B_{h2}. For the cast-in-place concrete pile foundations the numbers of piles and diameters of piles are intensively summarized as the properties of horizontal and rotation spring constants. In this study the horizontal spring constants of RC pile foundation, K_h, which can be commonly used for the time-history response analysis to the both bridge and transverse directions, are considered as the design variables. The widths to the bridge and transverse directions and the amount of steel reinforcements in a cross section, H_P, B_P and A_s, are taken into account as the design variables for RC piers. The bridge system shown in Fig.1 is symmetrical to the centerline and the total number of design variables is six of $B_{h1}, B_{h2}, K_h, H_P, A_S, B_P$.

Engineers have to design the bridge system which have sufficient ultimate dynamic capacities for large displacements caused by devastating earthquakes. Therefore, the relative horizontal displacements between superstructure and piers to the both bridge and transverse directions are dealt with as the design

Table 1 Properties of three types of RC piles

Diameter φ	Number of piles	Width of footing B	Width of footing H	Height of footing	Construction cost(10^3yen)	K_h(kN/m)	$K_{\theta1}$(kNm/rad) (bridge direction)	$K_{\theta2}$(kNm/rad) (transverse direction)	weight(kN)
1.0m	9	7.0m	7.0m	2.5m	13,466	2212657	23604414	23604414	3001.3
1.2m	9	8.4m	8.4m	2.5m	16,544	2762476	38430822	38430822	4321.8
1.0m	12	7.0m	9.5m	2.5m	17,965	2950210	31472551	49511633	4073.1

constraints, $g_{h1}, g_{h2}, g_{t1}, g_{t2}$, for the safety of the rubber bearings. Furthermore, the ductile factors are also dealt with as the design constraints for the RC piers, g_{μ}, so as to ensure the performance specified at the ultimate state. In the design of RC pile foundation, for the case that the horizontal ultimate dynamic bearing capacity for the RC pier is not enough large for the horizontal force calculated using the specified design seismic coefficient, RC pile foundation is not allowed to yield when the equivalent loads corresponding to the horizontal ultimate dynamic bearing capacity for the RC pier are applied to the RC pile foundation. For the case that the horizontal ultimate dynamic bearing capacity for the RC pier is sufficient, RC pile foundation is allowed to yield up to the ductile factor 4.0. This constraint is quite complex to deal with in the optimization process. Furthermore, we need to consider that the design variable for RC pile foundation is dependent on the design variables for RC piers. To simplify the optimum design problem, therefore, it is assumed that the design variable for RC pile foundation is independent, and the constraint on the RC pile foundation is not dealt with in the optimization process. After the determination of optimum solution the constraint on the RC pile foundation is examined.

The total construction cost minimization problem, which is expressed as the summation of bearing construction cost, $COST_B(B_{h1}, B_{h2})$, pier construction cost ,$COST_F(K_h)$, and pier construction cost, $COST_P(H_P, A_S, B_P)$, can be formulated as

find $\quad B_{h1}, B_{h2}, K_h, H_P, A_S, B_P \quad$ which

minimize $COST(B_{h1}, B_{h2}, K_h, H_P, A_S, B_P) = COST_B(B_{h1}, B_{h2}) + COST_F(K_h) + COST_P(H_P, A_S, B_P)$ (1)

subject to

$$g_{h1} = \delta_{h1} - \delta_{a1} \le 0 \quad (2), \qquad g_{h2} = \delta_{h2} - \delta_{a2} \le 0 \quad (3), \qquad g_{t1} = \delta_{t1} - \delta_{a1} \le 0 \quad (4)$$

$$g_{t2} = \delta_{t2} - \delta_{a2} \le 0 \quad (5), \qquad g_{\mu} = \mu - \mu_a \le 0 \quad (6),$$

where δ_{a1} and δ_{a2} are the allowable relative horizontal displacements of bearings at abutment and piers, which are given as the products of the heights of bearings B_{h1}, B_{h2} multiplied by 2.5. μ is the ductile factor of a pier, which is given by the ratio of working curvature to the yield curvature for the bridge direction.

In the optimum design problem B_{h1} and B_{h2} can take continuous values, but the others must be selected from a list of discrete values. In this study, K_h, H_P, A_S and B_P are selected from the following discrete sets in which three types of pile foundations summarized in Table 1 are considered to calculate K_h.

$K_h \in \{2212657(kN/m),\ 2762477,\ 2950210\}$

$H_P \in \{2000(mm),\ 2100,\ 2200,\ 2300,\ 2400,\ 2500,\ 2600,\ 2700,\ 2800, 2900, 3000\}$

$A_S \in \{198.6(mm^2),\ 286.5,\ 387.1,\ 506.7,\ 642.4,\ 794.2,\ 956.6,\ 1140\}$

$B_P \in \{3000(mm),\ 3500,\ 4000,\ 4500,\ 5000,\ 5500,\ 6000,\ 6500\}$

Therefore, the construction cost minimization problem can be expressed as a mixed discrete-continuous problem. Several types of optimization techniques have been developed, and Huang and Arora [4] investigated the efficiency and reliability of those for discrete and mixed discrete-continuous problems.

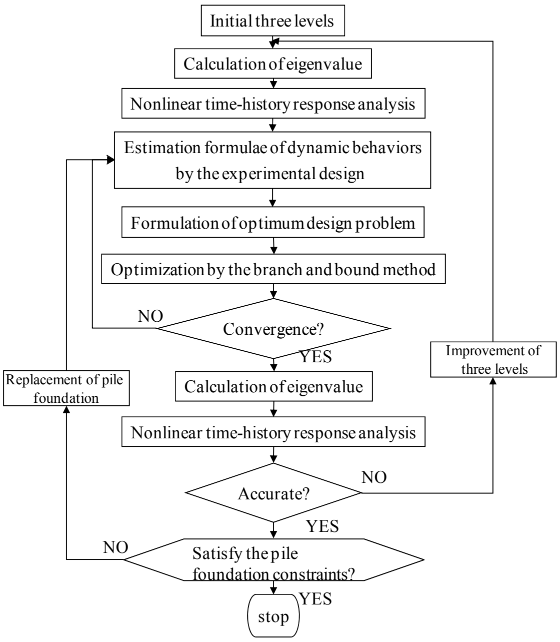

Fig.5 Macro-flow of the proposed optimum design method

Table 2 Orthogonal array table $L_{27}(3^{13})$

No.	Factor 1	2	3	4	5	6	7	8	9	10	11	12	13
No.1	1	1	1	1	1	1	1	1	1	1	1	1	1
No.2	1	1	1	1	2	2	2	2	2	2	2	2	2
No.3	1	1	1	1	3	3	3	3	3	3	3	3	3
No.4	1	2	2	2	1	1	1	2	2	2	3	3	3
No.5	1	2	2	2	2	2	2	3	3	3	1	1	1
No.6	1	2	2	2	3	3	3	1	1	1	2	2	2
No.7	1	3	3	3	1	1	1	3	3	3	2	2	2
No.8	1	3	3	3	2	2	2	1	1	1	3	3	3
No.9	1	3	3	3	3	3	3	2	2	2	1	1	1
No.10	2	1	2	3	1	2	3	1	2	3	1	2	3
No.11	2	1	2	3	2	3	1	2	3	1	2	3	1
No.12	2	1	2	3	3	1	2	3	1	2	3	1	2
No.13	2	2	3	1	1	2	3	2	3	1	3	1	2
No.14	2	2	3	1	2	3	1	3	1	2	1	2	3
No.15	2	2	3	1	3	1	2	1	2	3	2	3	1
No.16	2	3	1	2	1	2	3	3	1	2	2	3	1
No.17	2	3	1	2	2	3	1	1	2	3	3	1	2
No.18	2	3	1	2	3	1	2	2	3	1	1	2	3
No.19	3	1	3	2	1	3	2	1	3	2	1	3	2
No.20	3	1	3	2	2	1	3	2	1	3	2	1	3
No.21	3	1	3	2	3	2	1	3	2	1	3	2	1
No.22	3	2	1	3	1	3	2	2	1	3	3	2	1
No.23	3	2	1	3	2	1	3	3	2	1	1	3	2
No.24	3	2	1	3	3	2	1	1	3	2	2	1	3
No.25	3	3	2	1	1	3	2	3	2	1	2	1	3
No.26	3	3	2	1	2	1	3	1	3	2	3	2	1
No.27	3	3	2	1	3	2	1	2	1	3	1	3	2

Factor1: B_{h1}, Factor2: B_{h2} , Factor3: K_h , Factor4: H_P , Factor5: A_S , Factor6: B_P

In this study the optimization problem is solved by the classical branch and bound method with dual algorithm and convex approximation [5,6] for the reason that the approach is efficient and reliable for a mixed discrete-continuous problem without any parameters.

2.3 Optimization Algorithm

In this optimization process, in general, a number of nonlinear seismic response analyses and sensitivity analyses are necessary to determine the optimal solutions. To avoid these complexity and difficulties and make the optimum design process tremendously efficient, the design of experiments [7] is applied to introduce the estimation formulae for the dynamic behaviors. The dynamic behaviors and those sensitivities are calculated by using the estimation formulae without analyzing the structure. In the design of experiments, according to the orthogonal array table $L_{27}(3^{13})$ [7] given in Table 2, the three levels for all design variables are assumed and the twenty seven runs of nonlinear seismic response analyses are carried out in usage of TDAP-III for the both bridge and transverse directions, respectively. The first six factors among thirty factors in Table 1 are assigned to the design variables B_{h1}, B_{h2}, K_h, H_P, A_S, B_P, respectively. Assuming that the intended variable for the *k*th factor is x_k and the mean value of three levels $(\hat{x}_{ki}, i = 1,\cdots,3)$ for the *k*th factor is $\overline{x}_k$, the general form of estimation formula is introduced in the expression of quadratic functions of the design variables given as eqs.(7)-(10).

$$y = b_0 + \sum_{k=1}^{m} b_{k1} z_k + \sum_{k=1}^{m} b_{k2}(-M_{k2}^2 - M_{k3} z_k + M_{k2} z_k^2) \ , \tag{7}$$

where
$$M_{ki} = \frac{1}{n}(\hat{z}_{k1}^i + \hat{z}_{k2}^i + \cdots + \hat{z}_{kn}^i) \quad (k = 1,\cdots,m) \ , \tag{8}$$

$$\hat{z}_{ki} = \hat{x}_{ki} - \overline{x}_k \quad (i = 1,\cdots,n) \ (k = 1,\cdots,m) \ (9), \qquad z_k = x_k - \overline{x}_k \ , \quad (10)$$

m and *n* are respectively the number of factors, i.e. the number of design variables (= 6), and the number of levels for each factor (= 3). The estimation values of b_0 , b_{k1} and b_{k2} in eq.(7) are given as

$$\hat{b}_0 = \frac{1}{rS_1}\sum_{i=1}^{n} T_{1i} \ , \ \hat{b}_{kl} = \frac{1}{rS_k}\sum_{i=1}^{n} W_{ki} T_{ki} \ \ (l = 1,2) \qquad (11) \qquad \text{where} \qquad S_k = \sum_{i=1}^{n} W_{ki}^2 \ , \qquad (12)$$

r is the number of runs with the level $\hat{x}_{ki}$ (= 9). T_{ki} is the summation of results by the design of experiments with the level of $\hat{x}_{ki}$. W_{ki} is the value of function of coefficient $f_k(z_k)$ in eq.(7) with respect to b_{k1} and b_{k2} where $z_k = \hat{z}_{ki}$, namely, $W_{ki} = \hat{z}_{ki}$ and $W_{ki} = -M_{k2}^2 - M_{k3}\hat{z}_{ki} + M_{k2}\hat{z}_{ki}^2$.

After the determination of optimum solutions the design constraints with the estimation formulae are examined by re-analyzing the bridge system. In case that the design constraints violate the allowable limit, the three levels for all design variables and estimation formulae for dynamic behaviors are improved and the minimum cost design problem is re-solved. This optimization process is iterated until the relative errors between the estimated design constrains and the exact ones satisfy the allowable limit. After the determination of optimum solution the constraint on the RC pile foundation is examined. In the case that the constraint is violated the RC pile foundation is replaced with the larger one and the bridge system is re-optimized. The macro-flow of the proposed optimization algorithm is depicted in Fig.5.

Table 3 Improvements of three levels in the optimization process

Levels		B_{h1} (cm) (spring constant(kN/m))	B_{h2} (cm) (spring constant(kN/m))	spring constant of pile			H_P(mm)	A_s(mm^2)	B_P(mm)
				K_h(kN/m)	$K_{\theta 1}$(kNm/rad)	$K_{\theta 2}$(kNm/rad)			
Iteration 1	1	16.0(15313)	14.0(22857)	2212657	23604414	23604414	2400	794.4	4500
	2	14.0(17500)	12.0(26667)	2762476	38430822	38430822	2600	956.6	5000
	3	12.0(20417)	10.0(32000)	2950210	31472551	49511633	2800	1140	5500
Iteration 2	1	8.0(30625)	8.0(40000)	2212657	23604414	23604414	2400	506.7	3500
	2	10.(24500)	9.0(35556)	2762476	38430822	38430822	2600	642.4	4000
	3	12.0(20417)	10.0(32000)	2950210	31472551	49511633	2800	794.4	4500

Table 4 Optimum solutions for $\mu = 2.0$, 3.0 and 4.0

Allowable ductile factors μ_a	2.0		3.0		4.0	
B_{h1} (spring constant)	14.18cm (17273kN/m)		8.74cm (28035kN/m)		8.37cm (29257kN/m)	
B_{h2} (spring constant)	11.28cm (28366kN/m)		8.0cm (40000kN/m)		8.0cm (40000kN/m)	
K_h (ϕ, n)	2762476 (ϕ=1.2m, n=9)		2762476 (ϕ=1.2m, n=9)		2212657 (ϕ=1.0m, n=9)	
B_P	2700mm		2800mm		2700mm	
As	1140mm^2		387.1mm^2		387.1mm^2	
H_P	4000mm		4500mm		3000mm	
δ_{h1}/δ_{a1}	D.exp.*	1.000	D.exp.*	1.000	D.exp.*	0.992
	Anal**	0.971	Anal**	1.011	Anal**	1.013
δ_{h2}/δ_{a2}	D.exp.*	0.912	D.exp.*	0.604	D.exp.*	0.413
	Anal**	0.895	Anal**	0.602	Anal**	0.390
δ_{t1}/δ_{a1}	D.exp.*	0.903	D.exp.*	0.976	D.exp.*	1.000
	Anal**	0.967	Anal**	1.021	Anal**	0.935
δ_{t2}/δ_{a2}	D.exp.*	0.837	D.exp.*	0.994	D.exp.*	0.897
	Anal**	0.847	Anal**	1.038	Anal**	0.832
μ/μ_a	D.exp.*	1.000	D.exp.*	0.987	D.exp.*	0.971
	Anal**	1.012	Anal**	1.000	Anal**	1.008
yield of pile foundation	Bridge dir: μ_{FR}=1.51 Transverse dir:μ_{FR}=1.62		Bridge dir: not yield Transverse dir:μ_{FR}=1.62		not yield	
Total cost (10^3 yen)	193636		158974		138232	

D.exp.* : Feasibility of design constraints with the estimation formulae by the design of experiments
Anal** : Feasibility of design constraints using exact behaviors by analysis

3. DESIGN EXAMPLES

The proposed optimal design method is applied to the five-span continuous steel girder bridge system shown in Fig.1 and the optimal solutions for several allowable ductile factors μ_a are compared. The unit cost of rubber is as 45yen/cm^3. The construction costs of a pile are assumed as 65200yen/m^3 for the diameter 1.0m and 73800yen/m^3 for the diameter 1.2m. The construction costs of footing and form for pile foundation are assumed as 33500yen/m^3 and 8000yen/m^2. The construction costs of concrete, form and reinforcement for piers are assumed as 18500yen/m^3, 8000yen/m^2 and 120000yen/tf, respectively. Following the flow-chart in Fig.5 the optimization processes for $\mu_a = 2.0$, 3.0 and 4.0 are initiated with the levels of iteration 1 shown in Table 3. In the optimization process, the lower and upper limits for discrete design variables are set at the adjacent discrete values of the minimum and maximum values of the three levels to limit improvements of design variables. The optimum solution for $\mu_a = 2.0$ can be obtained quite efficiently without any improvements of the three levels for all design variables. The optimum solutions for $\mu_a = 3.0$ and 4.0 determined by the lower limits set as the move limits. After then, the three levels are improved to the values of iteration 2 in Table 3 referring to the optimum solutions with the previous three levels. The optimum solutions for $\mu_a = 3.0$ and 4.0 can be obtained efficiently at this stage without additional improvements of the three levels. The optimum solutions for μ_a =2.0, 3.0 and 4.0 are summarized in Table 4.

The horizontal spring constants of pile foundations for all cases are determined by the lower limit which indicates the lowest cost. Then, the RC pile foundations for $\mu_a = 2.0$ and 3.0 are replaced with the larger one so as to satisfy the constraint on the RC pile foundation. In case of $\mu_a = 2.0$ the largest dimensions of cross section, H_P and B_P, and reinforcement in the piers A_S are required in order to satisfy the allowable ductile factor. By increasing the heights of rubber bearings B_{h1} and B_{h2}, namely reducing the values of spring constant, the period of bridge system is made longer and the effect from superstructure is minimized. As the result the total construction cost is minimized. In case of $\mu_a = 3.0$ B_{h1}, B_{h2} and A_S are remarkably reduced compared with those in case of $\mu_a = 2.0$, and B_{h2} and A_S are determined by the lower limits. The total cost is reduced to 82 percent of that in case of $\mu_a = 2.0$. In case of $\mu_a = 4.0$ B_{h2}, K_h, A_S and H_P are determined by the lower limits. The total cost is reduced to 71 percent of that in case of $\mu_a = 2.0$.

As clearly seen from the values of feasibility of design constraints using exact behaviors by analysis in Table4, both the constraints on relative horizontal displacements at abutment to the bridge direction g_{h1} and ductile factors g_μ are active for all cases simultaneously. The displacements at abutment and piers to the transverse direction g_{t2} are also active for $\mu_a = 3.0$. The displacements at piers to the bridge direction g_{h2} are inactive for all cases. The accuracy of the estimation formulae is excellent within 7 percent of relative errors. The exact constraints are enough feasible within 3.8 percent of violation for all cases.

4. CONCLUSIONS

The following conclusions can be drawn from this study:

1) The proposed optimal design method can determine the heights of rubber bearings, cross-sectional dimensions and amount of steel reinforcements for RC piers, and numbers and diameters of piles rigorously and efficiently.

2) By applying the design of experiments, the estimation formulae for the ductile factor in piers and the maximum horizontal displacements to the bridge and transverse directions can be introduced accurately with small number of nonlinear seismic response analyses. The accuracy of the estimation formulae is excellent within 7 percent of relative errors between the exact behaviors and estimated ones.
3) A few iterations of improvements for three levels are required to obtain the optimum solutions in the proposed design method.
4) In the case that the allowable ductile factor is set at a small value, the heights of rubber bearings increase in order to make the period of bridge system longer, and the effect from superstructure is minimized. As increasing the value of allowable ductile factor the heights of rubber bearings are reduced and the dimension of cross section and reinforcement in the piers are also reduced.
5) In the proposed design process, the constraint on the RC pile foundation is not dealt with in the optimization process and, then, the RC pile foundation is replaced with the larger one so as to satisfy the constraint on the RC pile foundation. This design process can simplify the optimization algorithm greatly.
6) The constraints on relative horizontal displacements at abutment to the bridge direction g_{h1} and ductile factors g_{μ} are active at the optimum solutions simultaneously.

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PROFESSIONALS ROLE IN QUALITY ASSURANCE PROCESS

Mr. P. H. Sarath Gamini

Project Director

Greater Kandy Water Supply Project (NWS&DB), Pahala Kondadeniya, Katugasthota, Kandy, Sri Lanka.

Tel: 077-7809661 ; E-mail: nwsdbgkp@sltnet.lk

National Water Supply & Drainage Board (NWS&DB) is a Statutory Board established by National State Assembly of the Republic of Sri Lanka under Law No. 2 of 1974. During the past 35 years, water supply facilities have been improved in urban and rural areas and have objectives of reaching 75% of population to have access to safe drinking water by year 2025.

There are more than 1.2 million domestic piped water connections maintained by NWS&DB. In addition to that, community water supply projects local authority schemes cover more than 800,000 domestic water supply connections. Individual water supply facilities also are available in various parts of the country with in-house water and sanitation facilities, such as, washrooms, toilets etc.

All these water pipeline connections require pipe fittings, such as,

- Stopcocks
- Water meters
- Ferrules
- Clamp saddles, etc.

NWS&DB alone distributes more than 480MCM per year through various reservoirs, storage tanks and flow of water is regulated by various valves, ball float valves and sluice valves etc.

Major water apparatus and measuring devices used in water sector are given below.

- Domestic water meters
- Bulk meters
- Ball float valves for large reservoirs
- Ball float valves for domestic household storage tanks
- Clamp saddles
- Ferrules
- Angle valves
- Bib Taps
- Hot/ cold Water Taps

- Cisterns
- Bidet shower etc.,

A major issue in water sector today is the incidents of high proportions of non revenue water (NRW) by which treated and potable water is wasted. This is not a desirable state as far as the objective of water sector is concerned, and therefore, has to be controlled and curtailed.

It is observed that the non revenue water content is about 35% of the total supply. Main reasons for this situation is –

1. Leakages in the pipes and fittings
2. Illegal tapping
3. Errors in water meters
4. Poor plumbing practices etc.

Major causes of leakages are due to poor quality standards of pipes and fittings. Even though PVC pipes has got certain standards, all other fittings and water meters are purchased following quick procurement processes where quality aspects are not necessarily being considered.

Good quality pipes and fittings cannot be found easily as there are no restrictions on imports of water apparatus, stopcocks, cisterns, ball valves etc.

This paper deals on the weaknesses of such procurement processes and suggests suitable remedial measures at national level as well as individual user levels.

Procurement of Water Apparatus and fittings

NWSDB as the National Organisation procures Rs.5 Billion worth of various water apparatus (including pipes) annually. In the procurement process, quality assuring measures are adopted such as,

1. Quality management system for manufacturing organisations process – ISO 9001 – 2000, ISO – 9001 : 2008
2. Product specific, accreditation certificate (product certificate)
3. Specific test certificates during the delivery of goods
4. Calibration tests for sample of water meters (Domestic)
5. Enduser certificate (applicable only in Greater Kandy Project)
6. Endurance Certification (procurement of water meters in large quantity 50,000; 100,000)

Greater Kandy Water Supply Project inputs to the water sector

Greater Kandy Project has initiated non-revenue water reduction and prevention programmes and various studies have been carried out on existing procurement process related to water apparatus, fittings and found following observations.

1. Except for domestic meters, for all other apparatus only 03 certificates (1, 2, 3) have been insisted during the procurement process.
2. Endurance test certificates were insisted only for 15 mm domestic water meters (it does not apply now)

3. An enduser database has not been properly established in Sri Lanka for water apparatus.
4. Endurance test facilities are not available in Sri Lanka and it is very expensive.
   (Cost of endurance test is done in Singapore and it takes 7 weeks. Initially it is Rs. 6 Million for modifying of test apparatus and after that Rs.4 Million for each three sample tests)
5. Some manufactures produce forged product certificates
6. In some countries, accreditation process (sample testing, random technical auditing, issue of certificate) does not function effectively. Accreditation agencies issue certificates in a positive manner in order to obtain quality accreditation assignments regularly.
7. Most of the local agents or distributors of water and sanitary fittings of Sri Lanka are not aware of applicable standards for water fittings. According to traders, response from the user community on applicable standards is very poor. (more than 50 tender documents were sent to leading manufacturers and other agents and importers for procurement of 40 water / sanitary fitting sets in order to assess the actual situation).
8. Some accreditation agencies issue product certificates with conditions as they are aware of repercussion on quality of fittings.

The studies carried out by Greater Kandy Water Supply Project reveal that traditional procurement process has to be reviewed and new indicators are to be introduced in order to ensure quality water fittings available in the market.

Following actions are recommended.

1. Introduce endurance test certificate for all possible items in order to estimate the service life and performance of the product.

2. An enduser database is to be established based on information from the users by the national utility organizations, such as National Water Supply & Drainage Board for water fittings and in case of electricity fittings by Ceylon Electricity Board.

3. Enduser test apparatus are quite expensive, and therefore, it should be procured or fabricated locally.

4. Awareness campaigns are to be carried out among manufacturers, agents of various water product manufacturers on quality aspects and long term measures that are to be implemented by regulatory authorities.

5. Sri Lanka Standards Institution should take initiative to introduce new standards for water fittings and enduser test apparatus are to be introduced through respective national agencies.

6. Consumer Affairs Authority has legal provisions for maintaining of quality goods and services in order to protect the consumer rights.

7. Respective utility agencies should formulate new strategies to have an integrated approach with Sri Lanka Standards Institution and Consumer Affairs Authority, user groups, manufacturers, their agents and traders to establish quality assurance systems.

Greater Kandy Water Supply Project has developed an endurance test apparatus for stopcocks as a sample to promote quality assurance systems in the water sector. Details are summarised below. For more details, please Refer ***Annex 1.***

<u>ENDUSER TEST APPARATUS DEVELOPED BY GREATER KANDY WATER SUPPLY PROJECT</u>

Performance of this test apparatus is to check the lifetime or performance of stopcocks.

Procurement method	:	Local tendering
		Design and Built

Total cost of the apparatus : Rs.600,000/=

Contractor : M/s. Emac (Pvt) Ltd.

Sample output : Given below

Date	Brand	Country Of Manufacturer	Cause For Failure	Number of Rotations When Failure Occured
23/04/2010	Peglar 709	UK	Leak through gland	3670
1/5/2010	QJ	China	Leak through gland	2652
3/5/2010	SFI	China	Leak through gland	293
3/5/2010	Ningbo	China	Leak through gland	2720
5/5/2010	Melco	China	Leak through gland	3478
7/5/2010	Pegler 774F	UK	Leak through gland	7684

BASIC COMPONENTS OF THE ENDURANCE TEST APPARATUS

CONSUMER'S CHOICE

Product promotion campaigns are extensively carried out using electronic and print media. Therefore, consumers are unable to find the quality of the products easily and most of the consumers are not aware of the methodology on finding quality goods and services. In order to protect the interest of consumers and the service providers, regulatory authorities have been established in Sri Lanka.

Few related regulatory agencies are given below.

1. **CONSUMER AFFAIRS AUTHORITY**

Mission

Safeguard consumer rights and interests through consumer empowerment, regulation of trade and promotion of healthy competition.

Consumer Affairs Authority considers that protection of consumer rights is a means of developing responsible civic society.

Objectives

- *Take necessary action for the protection of the consumer against manufacturers and traders in respect of labeling, price marking, packeting, sale or manufacture of any goods*
- *Issue general directions on labeling, price marking, packeting, manufacture and sale of any goods*
- *For the purpose of protecting the consumer, determined the standards necessary to ensure the quality of goods and services*
- *Determine the "Specific Goods" which is essential for the living and place such goods under price regulation for the wellbeing of consumers*
- *Carryout investigation in regard to the prevalence of any anti competitive practices, which prevents, restricts or distorts competition, with regard to the sale of goods or provision of any services*

2. **SRI LANKA STANDARDS INSTITUTION**

Mission

To undertake, promote and facilitate Standardization, Measurement, Quality Assurance and related activities in all sectors of the national economy in order to

- *Related activities in all sectors of the national economy in order to*
- *Increase productivity and maximize the utilization of resources*
- *Facilitate internal and external trade*
- *Achieve socio-economic development*
- *Enhance international competitiveness of products and services*
- *Safeguard the interest of consumers while improving the quality of life of employees of the Institution*

Strategies

- *To formulate National standards required for the development of the National Economy*
- *To promote the use and application of national standards in all spheres of economic and social activity*
- *To promote quality assurance in all sectors of the economy*
- ***To promote and disseminate valid measurement practices at nation level***
- ***To provide consumer education and consumer protection***
- ***To educate and train industry and service personnel on concepts, practices and techniques of standardization and quality management***
- ***To provide test facilities and develop the national test capability***
- *To provide documentation and information services on standards, technical regulations and related publications*
- *To participate in international and regional standardization activities to safeguard national interest*
- *To constantly develop and upgrade the Institution and its resources*

3. PUBLIC UTILITIES COMMISSION OF SRI LANKA (PUCSL) – ACT NO. 35 OF 2002

Mission

Regulate all the utilities within the purview of the Public Utilities Commission of Sri Lanka, to ensure safe, reliable and reasonably priced infrastructure services for existing as well as future consumers in the most equitable and sustainable manner.

Objectives

- ***Protect interest of all consumers***
- *Promote competition*
- *Promote efficiency in both the operations of and capital investment in, public utilities industries*
- *Promote an efficient allocation of resources in public utilities industries*
- ***Promote safety and service quality in public utilities industries***
- *Benchmark, where feasible, the utilities' services as against international standards*
- *Ensure that price controlled entities acting efficiently, do not find it unduly difficult in financing their public utilities industries*

4. NATIONAL WATER SUPPLY & DRAINAGE BOARD (NWS&DB)

Mission

Serve the nation by providing sustainable water and sanitation solutions ensuring total user satisfaction.

NWS&DB Law, No. 2 of 1974

Extract of Clause 90.2 – Regulatory measures to be implemented by National Water Supply & Drainage Board

a. any matter required by this Law to be prescribed or in respect of which regulations are authorized by this Law to be made ;
b. the licensing of plumbers and the control of plumbing and plumbing fixtures;
c. the preservation and maintenance of the water works of the Board;
d. the control of the use of water supplied from the said waterworks;
e. the prevention of the waste, misuse, undue consumption, or contamination of the water supplied for public or private use;
f. the size, nature, strength and materials, and the mode of arrangement, position, alteration, removal, renewal and repair of the apparatus and receptacles to be used for the purpose of the waterworks of the Board;
g. the control of the public supply of water by stand pipes, and use of such water;
h. the control of the supply of water and the provision of sewerage by private services, and the materials and fittings to be used for the purpose;

CONCLUSION

Considering their mission statements of the said authorities and their strategies related to providing quality goods and services, ample opportunities are available for the society to obtain high quality goods and services. At the moment, even though statutory organizations exist, there are some deviations on implementation of regulatory measures empowered to them.

What are the measures required?
Who should do?

Measures

- Develop standards for products
- Enforce standards by way of measuring quality of goods using test apparatus
- Educate customers / consumers
- Establish enduser database

- Carryout consumer surveys on products, reliability, life cycle, cost etc.
- Identify deficiencies of goods and services
- Provide the feedback to manufacturers, trading agencies who can upgrade the quality of goods and services
- Aware consumers on methodology to select goods and services

Similarly, we can list out so many measures.

<u>Who should do?</u>

All those measures are the typical roles and functions of professionals, such as –

- Civil Engineers
- Mechanical engineers
- Electrical Engineers
- IT Experts
- Scientists
- Legal Draughtsmen
- Sociologists
- Economists
- Administrators, etc.

Considering all above, it is high time to develop an integrated approach to introduce appropriate regulatory measures in the water sector, specifically in order to provide quality goods to the consumers.

- Appropriate quality testing systems (endurance tests, calibration, strength of materials, property of material etc.) are to be developed locally in order to minimize the cost of quality monitoring systems

- Professional associations should take initiative actions to have an integrated approach with all stakeholders to address public issues, such as, availability of poor quality water fittings.

- Professional associations should campaign for high quality services through regulatory authorities and involving other stakeholders.

- Involve Universities, Technical Colleges, University of Vocational Technology to develop and carryout appropriate tests for various products.

- Enduser database is to be established in each utility service agency or major service organisation. A mechanism is to be developed to analyse enduser database and provide the feedback to manufacturers and regulatory agencies, such as, Sri Lanka Standards Institution, Consumer Affairs Authority, Public Utilities Commission of Sri Lanka, Telecom Regulatory Commission etc.

I hope that IESL Kandy Centre may take appropriate action to promote regulatory measures in water sector in order to establish quality goods and services to improve quality of life of people in Sri Lanka.

Annex 1

OBJECTIVES OF ENDURANCE TEST APPARATUS FOR STOPCOCKS

National Water Supply & Drainage Board procures more than 100,000 stopcocks annually to provide water connections to consumers and to replace defective stopcocks. To ensure the quality of same through conducting sample endurance tests is the objective of fabricating this apparatus.

The work scope has been included in the design for the fabrication, supply, installation, commissioning and training the users of an apparatus for performances tests of stopcocks 15mm. diameter.

Greater Kandy Water Supply Project Phase I, Stage II is planning to procure 3,000 stopcocks under its non revenue water programme. This work is planned to implement as a model project with developing quality standards in house water connections and avoiding procuring low quality products to National Water Supply & Drainage Board. Further, we are planning to introduce low cost solutions to water industry for its long term sustainability.

A contract was awarded to design fabricate, supply, installation and training the users of the apparatus for performance test of stopcocks (15mm) procured by National Water Supply & Drainage Board, Sri Lanka. The scope includes all labour, machinery and all materials necessary for the proper manufacture of the apparatus, installing at specified location, testing and commissioning.

Design of test apparatus

The designer / fabricator has been instructed to do the design the apparatus to meet the following specifications.

Technical Capability

The apparatus to be able

- To operate at least one million open close cycles without major repair
- To couple of the mechanism to the hand wheel of the stopcock
- To detect leaks from outside as well as inside (non closure) by water drops
- To prevent dry operation
- To handle power supply failure during testing session
- To withstand water pressure up to 10 bars

Major items to be included

- Test table
- Water tank
- Pressure pump with regulator
- Geared motor or servo motor with fixing and mechanism to open / close valves
- Leak detection sensors inside / outside
- Control panel with remote digital counter to count the number of valve open/close cycles.
- Protection against over torque
- Timers and limit switches to facilitate the test

Mechanical Features of Apparatus

The valve turning mechanism shall be attached to or detached easily from the hand wheel of the valve under test by the operator. It shall not exert any lateral or vertical force to the valve body or stem during the test and shall not vibrate to make undue noise. The coupling of the hand wheel to the drive attachment shall not involve any form of valve dismantling, drilling, cutting or gluing. The attachment shall have provision for hand wheel vertical travel of 3 cm during open / closing.

The test apparatus shall have a resetting facility (0000 to 9999 or higher) counter which counts the number of valve open/close cycles and cumulative counter (000,000 to 999,999 or higher) for total number of valve open/close cycles. This counter is used as a check for the usage for the endurance tester. There shall be two independent timers to set the valve close time and valve open time (0~ 180 s minimum).

The valve close torque shall be adjustable. A separate safety torque switch shall be incorporated with trigger alarm and stop the motor when the open/close toque is beyond the servo motor specification.

The test apparatus once started shall continue to open and close the stop valve until water leak or other alarm is triggered.

Design and Fabrication

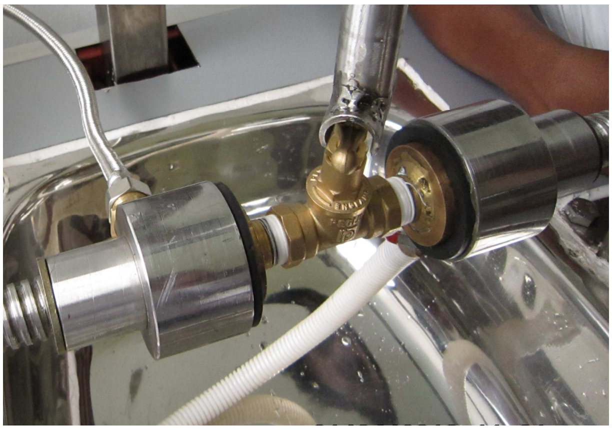
Emac (Pvt) Ltd., Kandy was selected to carryout the design and fabrication work through competitive bidding and awarded the contract.

Their bid price for fabricating this machine including testing and training was Rs.584,500./=.

After submission of design proposal and drawings, the apparatus was fabricated as show below.

4.1 Control & Display Unit

4.2 Valve Clamping Unit



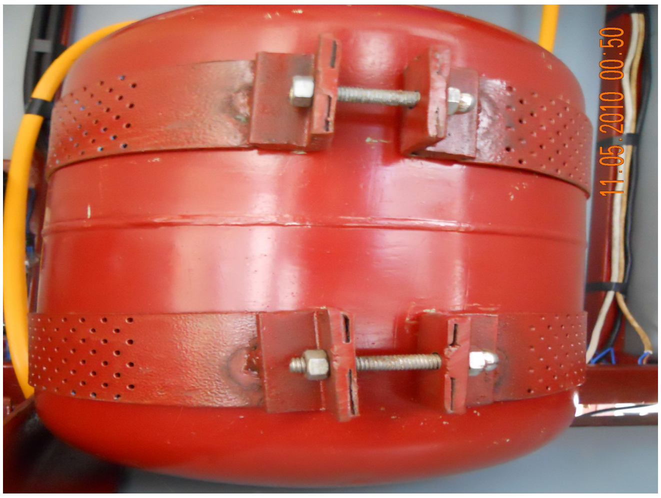
4.3 Pressure Pump

4.4 Battery & Battery Charger

4.5 Servo Motor Unit

4.6 Water Tank

4.7 Pressure Cut-off Valve



4.8 Limit Switches

This machine contains following major components.

Operation principle

The following features are incorporated.

- Operate at least one million open close cycles, without major repair
- Leak detection from outside as well as inside
- Prevent dry operations
- Handle power supply failures during testing session
- System to withstand water pressure up to 10 bars

The equipment include the following facilities.

1. Pressure pump to maintain constant pressure in the system during the testing session
2. Adjustable pressure regulator to maintain required pressure up to 10 Bars
3. 24 hr time is provided to have continuous operation, (For endurance test equipment can be set automatic operation for pre-set time intervals.
4. 0-5 min timer is provided to maintain fully close time up to 5 min and 0-60s time to for fully opened time up to 1 min.
5. Counter (0000 to 9999) is provided for each operation cycle.
6. counter (0000 to 9999) is provided for leakages and cumulative counter for open/close cycles. (if the valve leaks pressure pump will start before the preset times)
7. Geared motor is provided for smooth operation of hand wheel or close/open lever mechanism
8. Universal coupling mechanism to couple the rotary spindle of geared motor to the close/open mechanism of the valve
9. Adjustable limit switches are provided to maintain the optimum seating position of closing and opening.
10. Auxiliary water flow switch.