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Faculty of Engineering, University of Ruhuna, Galle, Sri Lanka
19th March 2012

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Preface

This proceeding is a collection of keynotes and papers presented at the International Symposium on Advances in Civil and Environmental Engineering Practices for Sustainable Development (ACEPS) – 2012 held on 19th March 2012 at the Faculty of Engineering, University of Ruhuna, Galle, Sri Lanka.

Environmental sustainability is a key issue confronting the developing world today. Most of the developing countries in the Asian region have many large scale infrastructure projects at present. Such new developments are likely to cause huge environmental degradation due to lack of proper technical knowledge. Yet if handled with proper technology they can become role models of engineering excellence for sustainable development.

The symposium venue is located in Galle a rapidly developing city which houses the famous colonial fort that has received world heritage status. The Southern region which is well known for its ancient inheritance is going through a rapid development phase introducing many large scale infrastructures. Such development is expected to take place in the light of advanced technical knowledge while giving due considerations to the environment and the heritage. Contribution of experienced professionals from all around the world continues to be vital in this process. Some of the present development work in the Southern region provides such examples for sustainable development driven by proper knowledge. As an example the new Hambantota international air port is designed to be world's first environmental friendly airport.

Technical knowledge transferred from the developing world has immensely contributed to such projects. This symposium is organized to create a venue to discuss such advanced sustainable engineering practices that are adopted in developing the region. The symposium theme mentioned below addresses areas that demands advanced engineering developments should take place while balancing the sustainable considerations.

- Structures and Building Materials
- Infrastructure for changing climate and Sustainable built environment
- Coastal and lagoon environment
- River engineering and Hydroinformatics
- Challenges and Solutions in Geotechnical Engineering
- Environmental Engineering and Management
- Construction Management and Project Planning
- Transportation and Highway Engineering
- Engineering Education and Training

The five key note speeches from four leading academics and an industry professional touched five important aspects of the symposium in depth. The key note address of Professor Norio Tanaka was focused on his work on Overflow Pattern and the Formation of Scoured Region by the Tsunami Propagated in River Channels in Great East Japan Earthquake. Professor M.T.R. Jayasinghe by his key note address presented his experience and vision on designing elevated highways in developing countries. Dr. A. Ranasinghe, representing the industry was able to enlighten the audience with real challenges faced and opportunities available to engineers in developing a country in a sustainable manner. Professor A.W. Jayawardena based on his long career experience delivered a timely speech on Challenges for sustainable water management. Associate Professor Y. Matsumoto presented his experience on structural changes of steel truss bridges.

Despite being the inaugural symposium, ACEPS – 2012 received an overwhelming number of publications. The papers which addressed different themes of the symposium, included studies focused on solving real engineering problems, case studies on sustainable practices, development of engineering tools as well as original studies of academic interest. The manuscripts received in paper format were reviewed by two experts in related area as a part of a double blind review process. Based on the reviewer's comments papers which were of high quality were selected to be presented in the symposium.

The papers included here and presented in the symposium have initiated a dialogue on how the engineering developments should take place within a local and sustainable context. The organisers expect to continue such dialogue by making the ACEPS symposium an annual event.

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March 2012

Message from Dr. H.P. Sooriyarachchi: Symposium Co-chair

Today the world is embraced with the question "how to satisfy the ever broadening needs of human existence". Development has put a lot of strain on the available resources. Making, resources available to current generation, preserved for the future generation, so that their capacity to create wealth is not hindered by our existence is an obligation of the current generation. This has made the sustainable development not just a buzz word but absolute necessity to preserve the cycle of life. It has become a requirement for not just the developed countries but also the developing countries.

With the dawn of the new era of peace, reconciliation and national building, Sri Lanka is presented with an opportunity to embrace development and make sustainable development a practical reality. Being part of this rapidly changing build environment around us, like Hambantota Port, Mattala Airport, New Rail Project and Southern Expressway and possible extension of it to connect Hambantota Port and Mattala Airport, and many more projects on the drawing table, it is important that we assess the social and environmental impact of such development and ensure that the development is sustainable. To this end, there are questions to be raised, lessons to be learned and experiences to be shared. The mission of this symposium on the Advances in Civil and Environmental Engineering for Sustainable Development (ACEPS) is to provide a forum to discuss and share knowledge for healthy practices of development. By creating such a forum the department is fulfilling its social obligation of disseminating the knowledge for sustainable development.

I take this opportunity to thank the main sponsor of the symposium, the Saitama University for their financial assistance and the technical cooperation for not just this project but for other collaborative research and development activities they are engaged with us. The other three faculties of engineering, at the University of Moratuwa, the University of Peradeniya and The Open University of Sri Lanka are also kindly acknowledged for participating and in organising this symposium. Last, but not the least, I take this opportunity to thank the academic staff of the Department of Civil and Environmental Engineering, University of Ruhuna for their untiring effort to make this event a success. My sincere wish is that ACEPS will grow from strength to strength in years to come and continue to address the challenges in Civil and Environmental Engineering.

Dr Harsha Sooriyaarachi has started off his carrier as a structural engineer before moving to academia. Currently he is a Senior lecturer attached to the Department of Civil and Environmental Engineering, University of Ruhuna and have contributed to the development of the faculty from its inception. He is now the head of the department of Civil and Environmental Engineering. He has pioneered in the development of the postgraduate masters programme at the department and was the first coordinator of the programme. His teaching and research interest are mainly on behavior of reinforced concrete and have a large number of publications on the subject.

Message from Professor Norio Tanaka: Symposium Co-chair

The JSPS AA Science Platform Program is designed to create high potential research hubs in selected fields within the Asian and African regions, while fostering the next generation of leading researchers. The three-year program proposed by Saitama University collaborating with three Sri Lankan universities (Univ. of Moratuwa, Univ. of Peradeniya and Univ. of Ruhuna) has been selected as one of the promising distinguished programs by the Japan Society for the Promotion of Science (JSPS). Exchanges will be conducted under the leadership of the core institution (Saitama University) and joint research, seminars and other scientific meetings, and researcher exchanges will be organized and carried out effectively under the program. It is also anticipated that the hubs formed by the core institutions will continue to carry out important research activities after funding for the project has ended.

I am very happy to hold the jointly organized International Symposium on Advances in Civil and Environmental Engineering Practices for Sustainable Development (ACEPS) with University of Ruhuna and Saitama University and in collaboration with the University of Moratuwa, University of Peradeniya and The Open University of Sri Lanka. It is supported by JSPS AA Science Platform Program, JST-JICA SATREPS (Science and Technology Research Partnership for Sustainable Development) that is a Japanese government program that promotes international joint research targeting global issues, and International Collaborative Graduate Program (ICGP) which is the education program to cultivate excellent human resources for international infrastructure development and environmental engineering in association with partner graduate courses.

I hope the seminar will provide an excellent opportunity to establish fruitful international collaboration between the above-mentioned universities.

I would like to thank to Dr H.P. Sooriyaarachchi and Dr. A.M.N. Alagiyawanna, University of Ruhuna, Sri Lanka, Prof. M.T.R. Jayasinghe, University of Moratuwa, Sri Lanka, Prof. K.D.W. Nandalal, University of Peradeniya, Sri Lanka, and Prof. T.M. Pallewatta, the Open University of Sri Lanka, Prof. K. Kawamoto, Saitama University, for their efforts in organizing this symposium.

Norio Tanaka
Coordinator of JSPS AA Science Platform Program on 'Development of bio-engineering by vegetation and for wetlands as a solution of environmental and natural disaster problems for expanding urban fringe zone in Asia'
Project Manager of JST-JICA SATREPS on 'The project for development of pollution control and environmental restoration technologies of waste landfill sites taking into account geographical characteristics in Sri Lanka'
Coordinator of International Collaborative Graduate Program (ICGP)
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Professor at Saitama University, Japan.

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Department of Civil and Environmental Engineering, Saitama University, Japan

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Acknowledgements

ACEPS – 2012 symposium was organised and hosted by Department of Civil and Environmental Engineering, University of Ruhuna in collaboration with Saitama University. It was the first of a series of symposia which the organisers hope to hold annually. Despite being its inaugural symposium, the event created a forum of high quality academic discussion achieving its original objectives. Organising ACEPS – 2012 within a very short period required the generous efforts of many from both Sri Lanka and abroad.

Our sincere thanks are extended to Senior Professor Susirith Mendis – the Vice-Chancellor of University of Ruhuna for his contribution in organizing the event and embracing as the Chief Guest. We would also like to acknowledge the support of Senior Professor Gamini Senanayake – the Deputy Vice-Chancellor of University of Ruhuna.

We are grateful to Dr. A.M.N. Alagiyawanna, for his enormous contribution and guidance in organizing the event and facilitating it within the Faculty of Engineering in his capacity as the Dean of the Faculty of Engineering as well as a member of the organizing committee. His experience and generous advice in all stages of the organizing process facilitated us to do our best.

We have received enormous help from the co-chairs Prof. N. Tanaka and Dr H.P. Soriyaarachchi. Prof. N. Tanaka led all the arrangements on behalf of Saitama University by initiating the discussion, organising sponsorships and encouraging submissions from Saitama University including two keynote speeches. We are grateful for his continuous commitment to the success of the event. We would also like to extend our sincere gratitude to Associate Prof. K. Kawamoto and Assistant Prof. J. Yagisawa for their generous contribution right throughout the process to make this event a reality.

Dr. H.P. Sooriyaarachchi, in his capacity as the symposium co-chair as well as the Head of the Department of Civil and Environmental Engineering worked very hard day and night to make the event a symposium of high quality academic gathering. He led all the organizing work by clearing many obstacles, creating many opportunities and by advising all committees to work to the highest possible quality. We are indebted to him for his brave and high quality leadership.

We are also grateful to the members of the organizing committee, Prof. K. D. W. Nandalal of University of Peradeniya, Senior Prof. M.T.R. Jayasinghe of University of Moratuwa, Prof. T.M. Pallewatta of The Open University of Sri Lanka for their contribution extended in organizing the event.

We are deeply thankful to the contribution of the sponsors, JSPS (Japan Society for the Promotion of Science) AA Science Platform Program and JST-JICA SATREPS (Science and Technology Research Partnership for Sustainable Development). We are also grateful to and Dr K.S. Wanniarachchi and Dr. N.H. Priyankara for their contribution in making optimum use of the available funds.

Five key note speakers, Prof. Norio Tanaka, Prof. M.T.R. Jayasinghe, Dr. Ananda Ranasinghe, Prof. A.W. Jayawardena and Associate Prof. Yasunao Matsumoto and are acknowledged for their stimulating, inspiring addresses initiating valuable discussions among the participants related to the symposium theme.

We are quite grateful to the members of our technical committee consisting of international experts of multiple fields who used their expertise knowledge in the reviewing process. Despite their busy schedules they provided us critical reviews to select the research papers of highest quality on a short notice.

We are grateful to all the session chairs for conducting the sessions smoothly while enabling academic discussions of the session themes. We are also thankful to all the panel members of the evaluation committee who facilitated the selection of the best papers among the high quality papers.

The committee would like to thank all the authors and presenters from all around the world who laid the foundation stones of the symposium by presenting their research outcomes in the symposium.

Our sincere gratitude is also extended to the members of the Logistics Committee, Dr T.M. Rengarsu, Ms N.S. Miguntenna and Ms. T.N. Wickramarachchi who worked with utmost commitment to organise the event to the highest quality. The students of the Department of Civil and Environmental Engineering are thanked for their contribution in terms of designing the symposium web site and logo, making all arrangements before and during the symposium.

We are also grateful to all the academic and non-academic staff of the Department of Civil and Environmental Engineering for their generous assistance in organising the Symposium. The Network administrator Mr Anuradha Gunawardane is acknowledged for his assistance to facilitate smooth functioning paper uploading system and online registering process.

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March 2012

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Section I – Keynote Addresses

Overflow Pattern and the Formation of Scoured Region by the Tsunami Propagated in River Channels in Great East Japan Earthquake

Norio Tanaka (Professor, Hydraulic and Environmental engineering, Saitama University, Japan)

Prof. N. Tanaka is a professor of Hydraulic and Environmental Engineering, Saitama University. He is one of the division head of the Institute for Environmental Science and Technology in Saitama University, a coordinator of Asia-Africa Science Platform Program by JSPS (AACORE), and a project manager of JST-JICA project (SATREPS) in Sri Lanka. He received his doctoral degree in 1991 from the University of Tokyo. Prof. Tanaka is one of the directors of IAHR (International Association for Hydro-environment Engineering and Research) Japan Chapter. He has led numerous research projects on hydraulic and environmental engineering over last 10 years, and has published over many reports and refereed papers.

Abstract: *The tsunami caused by the Great East Japan Earthquake on 11 March 2011, with a magnitude of 9.0, caused catastrophic damage to people and buildings in the Tohoku and Kanto regions of Japan. A field survey was conducted to elucidate the damage to river embankments and their hinterlands (residential area) by tsunami propagation in river channels and overtopping of embankments. Three, three, and four rivers in Iwate Pref., Miyagi Pref., and the Kanto Region, respectively, were selected for the field investigation. In the hinterlands, the tsunami came from coast and river, and the situation, including the evacuation of people, became complex. Tsunami inundation patterns were classified by the river capacity and whether a river or sea embankment was breached or not. This will provide useful information for making new hazard maps and planning new cities.*

Keywords: *tsunami propagation in river channels, overtopping flow, scoured region, erosion of embankment, meandering of river channel, Great East Japan Earthquake*

1. INTRODUCTION

The Great East Japan Earthquake at 14:46 JST on 11 March 2011 had a magnitude of 9.0 and an epicenter 129 km east of Sendai, and it was followed by a large tsunami that broke many of the sea walls (tsunami gates, large embankments)(Takahashi et al., 2011) and coastal forests (Tanaka, 2012), causing catastrophic damage to people and buildings in the Tohoku and Kanto regions of Japan.

Tsunamis can cause catastrophic damage to both human life and socioeconomic property. Extensive experimental (Peregrine, 1967; Madsen & Mei, 1969) and analytical (Benjamin, 1972) studies have shown that tsunamis are also propagated far upstream in a straight channel of uniform depth and width because a solitary wave like a tsunami propagates without changing its shape and speed. In an actual tsunami, river morphology greatly affects the propagation. Although the propagation of solitary waves through curved shallow water channels was investigated by numerical simulations and the deformation of the wave at the outer bank has been described (Shi et al., 1998; Yuhi et al., 2000), the disastrous results of propagation of an actual tsunami in a curved channel were not reported in previous research.

In addition, it is very important to elucidate the role of inland embankments of roads, railways, and channels along the coast in mitigating the tsunami as it inundates the inland, and the relationship between the tsunami propagating from the sea and the flow overtopping the banks of a river.

Therefore, the objectives of this study were: 1) to investigate the interactions between a tsunami propagating from the sea and the flow overtopping from a river, 2) to elucidate the effects of river morphology on the tsunami overtopping the embankment, and 3) to determine the effects of inland embankments on tsunami propagation in the hinterlands of the coast or river.
For that objective, field investigations were conducted of three rivers (Heiigawa, Omotogawa and Sakarigawa Rivers) in Iwate Prefecture, three rivers (Abukumagawa, Old and New Kitakamigawa Rivers) in Miyagi Pref., and four rivers (Kujigawa, Nakagawa, Tonegawa, and Mikawa Rivers) in the Kanto Region in April and May 2011. Figure 1 shows the location of the mouth of each river.

Figure 1 Location of investigation sites (locations of the mouth of each investigated river are shown in this figure)

2. SITE LOCATIONS AND MEASUREMENT METHOD

Table 1 shows the river width at the river mouth, tsunami water depth, and height of the sea/river embankment at each investigation site. The tsunami water depth was obtained from "The 2011 Tohoku Earthquake Tsunami

Joint Survey Group" (http://www.coastal.jp/tsunami2011/). As for the tsunami damage, the width of sea/river embankments that were breached and the width/length of the regions that were scoured by overtopping flow are also shown in Table 1.

The tsunami water depth at each site was determined by the height of scars made by collisions of debris with tree trunks or broken branches, water marks, e.g., collision traits, on the walls of damaged houses, marks on broken roofs, or debris located on roofs. The tsunami directions were analyzed by the directions trees and fences were bent and the location of broken houses and scour regions behind embankments or houses. In addition, estimated tsunami water depths of the river or on the embankment and that in the hinterlands were compared to judge the dominant tsunami direction.

Table 1 Characteristics of rivers investigated

Location	Name of River	River width at mouth (m)	Tsunami water depth* (m)	Embankment height		Breach width		Scoured region due to overtopping flow	
				sea embankment (m)	river embankment (m)	sea embankment (m)	river embankment (m)	Width** (m)	Length** (m)
Iwate Pref.	Omotogawa River	200	6.9	9	4	0	230	70.0 - 100.0	14.0 - 15.0
	Heiigawa River	160	8.9	-	3	-	-	-	-
	Sakarigawa River	150	9.3	4	3	0	0	3.5 - 15.0	2.5 - 6.0
Miyagi Pref	New Kitakamigawa Rive	600	7.4	3	3	680	2100	3.0 - 39.0	2.0 - 5.0
	Old Kitakamigawa Rive	200	7.3	4	3	-	0	15.0	4.0
	Abukumagawa River	900	8.8	5	5	260	200	1.5 - 50.0	1.0 - 19.5
Ibaraki Pref.	Kujigawa River	200	4.2	4	4	0	0	-	-
	Nakagawa River	300	3.3	3	3	0	0	-	-
Chiba Pref.	Tonegawa River	700	3.0	5	5	0	0	-	-
	Mikawa River	25	3.4	2	-	0	-	-	-

*Tsunami water depth data were obtained from "The 2011 Tohoku Earthquake Tsunami Joint Survey Group" (http://www.coastal.jp/tsunami2011/)
**The width and length of regions scoured by overtopping flow are shown in Figure 2

3. RESULTS

3.1. Tsunami propagation and overtopping from river embankment (without tsunami gate at river mouth)

The direction and water depths of the tsunami inundation around the Abukumagawa River are shown in Figure 2. At location A, breaching of the sea embankment was observed. Behind the sea embankment, large areas were scoured by the tsunami overtopping of sea embankment. Near this region, the tsunami water depth was around 5.5 m and the embankment height was 4.8 m. At location B, the overtopping water depth at the top of embankment was estimated to be around 1 m based on the debris attached to the fence on the embankment. Just downstream, the river embankment was also breached by direct attack of the tsunami. The overtopping from the river to the hinterland was severe at location C, but was a little less severe upstream at locations D and E. However, the overtopping became severe again at location F because it was located on the outer-bank side of the river. The extent of overflow was judged from the erosion of the river embankment slope and scoured regions around houses and the broken or washed-out condition of houses. The elevation of the road along the river was higher around location F than in locations D and E, so the difference between the ground level of houses and the road was greater. In location D and E, the road in front of houses had a role to prevent erosion, however location D, downward flow still continued and caused erosion. In that case, the overtopping flow caused scoring and local scour around houses that combined to generate large scour area (Figure 3). Thus, houses around location F were completely washed out, not by the tsunami propagated from seaward, but by the tsunami overtopping the river. Similar overtopping from outer bank of a river was also observed in old Kitakamigawa River.

In case of Heiigawa River (Figure 4), a railway bridge which is 1.5 km upstream from the river mouth was washed out after the damming of flow by debris including a ship (Location G). People who temporally escaped on the railway embankment needed to escape far inland by the destruction of the bridge and railway. Most of the tsunami from seaside was stopped at the railway embankment (Location H), but tsunami also propagated by sewage pipe line, inundated in the inland region of the embankment, and overflown from the upstream river embankment when the railway bridge was broken. This case shows that; 1) bridge has some possibility to be a trigger of the overflow from embankment, and 2) tsunami also propagated from sewage line. This should be considered for the tsunami simulation and hazard mapping.

Figure 2 Damaged situation in Abukumagawa River Basin

Figure 3 Large scoured region by overtopping flow (Right hand side of Abukumagawa River)

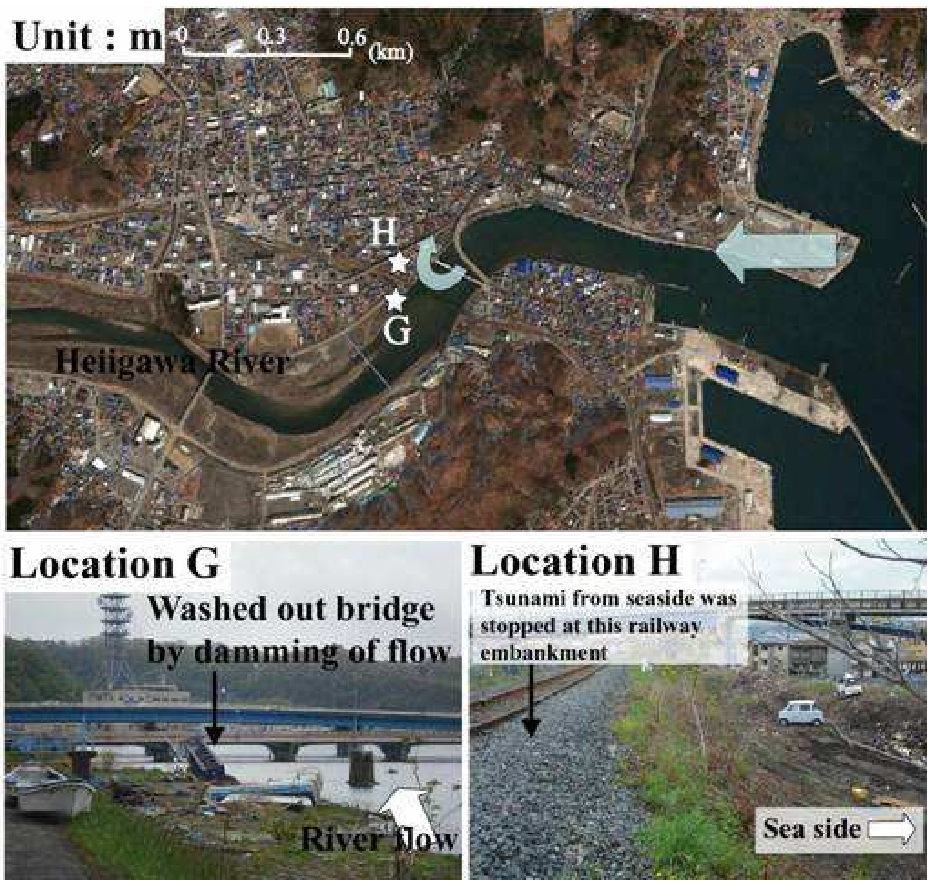

Figure 4 Damaged situation in Heiigawa River Basin

3.2. Change of tsunami propagation pattern with the tsunami gate at river mouth

The Omotogawa River (Figure 5) has a tsunami gate and high sea embankment (around 10 m from the ground on the hinterland side). The overtopping tsunami depth from sea embankment was assumed to around 1.6 m. Many large concrete blocks in front of the sea embankment had been transported by the tsunami, a large scoured region had been generated behind the sea embankment, and the pine trees near the region were overturned (Location I). Most of the houses near the sea and river embankment were washed out by the overtopping flow (Location J). From large scoured regions on floodplain of the river bed, and a scattered region of a broken parapet on the embankment, the breaching was supposed to be occurred from the inland side to the river. Even if a tsunami gate had existed, the tsunami would have been higher than the tsunami gate and the sea embankment; in fact, the tsunami inundation occurred mainly from the high embankment with high potential energy, and washed out or broke the houses. On contrary, in case of a small river in Chiba prefecture, the tsunami was stopped at the gate, because the tsunami height was low in comparison with Iwate Prefecturte.

Figure 5 Damaged situation in Omotogawa River Basin

3.3. Changes of tsunami inundation due presence of road, train embankment, sanitary channel, or mountain

In case of the Sakarigawa River basin (Figure 6), the presence of a road and train embankment changed the direction of most of the tsunami flow intruded from seaward. The tsunami passed through only the culvert of the embankment from seaward to inland, but it continued to overtop the river embankment. Thus, the people who lived upstream of the road and railway embankments received tsunami inundations from two directions. However, in this case, the tsunami overflow from river embankment itself was not large compared with that of the Abukumagawa River. The inland embankment for railway or road is very useful in some cases and needs to be utilized more, considering the tsunami inundation pattern.

In the Kanto Region, the tsunami height was lower than that in the Tohoku area. The Kujigawa, Nakagawa, and Tonegawa Rivers had sufficient capacity to absorb the tsunami. Thus, the areas inundated by the propagated tsunami in the three rivers were restricted. Most of the tsunami intrusions occurred around branches of the river and drainage channels connected to the river. Even when a gate existed between main River and its branches, inundation was also occurred because the earthquake caused a gate trouble by an electricity failure of the system. In the Mikawa River, most of the tsunami was stopped by a sand dune on the coast, but the river itself was open to the sea and a tsunami could easily intrude into and overflow the hinterland from the river. This kind of problem in the gap of an embankment or vegetation barrier was already discussed (Mascarenhas and Jayakumar, 2008; Thuy et al., 2009; Tanaka, 2009, 2011). The river mouth problem is very difficult to mitigate because if a gate is constructed, it may change the tsunami inundation pattern, as in Omotogawa River when the tsunami exceeded the designed gate level.

Figure 6 Damaged situation in Sakarigawa River Basin

4. DISCUSSION

As described in the previous section, overtopping of tsunami from the outer bank side of a river was severe. It is easily assumed from previous studies that the outer bank side is vulnerable to tsunami propagation (Shi et al., 1998; Yuhi et al., 2000). When overtopping occurred here, the scouring of roads in front of houses was not severe, and some houses remained standing although their walls were broken. In contrast, when the difference between the road height and the elevation of the house was large, the scouring became severe and the houses were washed out. The type of utilization of the riverside greatly affects the damage and needs to be studied in more detail in the future.

If a river embankment is not high enough to obstruct a tsunami or a city has rivers or creeks, tsunami inundation occurs not only from the sea but also from the rivers or creeks. In the Old Kitakamigawa River (old channel), or the Sakarigawa River, the tsunami propagated in the river, which is usually faster than a tsunami propagating over land, and the water overflow hit people from two directions.

Many patterns of tsunami propagation in rivers were observed, and they depended on 1) the river capacity (especially embankment height), 2) whether a river or coastal embankment was broken/breached or not, and 3) the existence of an inland embankment or area of high elevation, like a mountain, near the river. For the evacuation from the tsunami, revision of tsunami hazard maps, and new plans for a city design after a tsunami, this complex propagation pattern should be considered and informed to people. The knowledge in this study also needs to be considered in the design of tsunami protection and mitigation systems in a city.

5. SUMMARY

The following conclusions and recommendations were obtained by this study:
1) The flow overtopping embankments occurred mainly on the outer bank side of meandering river sections. Severe erosion occurred on the levee slope, and neighbouring houses were washed out by the scouring due to the overtopping flow.
2) In the hinterlands of coast and river embankment, it is necessary to identify locations where a tsunami can easily overtop for different tsunami conditions, and the information should be utilized for making next hazard map.
3) The tsunami inundation patterns were complex and could be classified based on the river capacity, the existence of gate, and whether a river or sea embankment was breached or not.

6. ACKNOWLEDGMENTS

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Elevated Roads for Sri Lanka

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Abstract: *The development of roads and highways related infrastructure has picked up significantly in the recent times along with a significant increase in the vehicle ownership and also the number of larger and heavier vehicles on the road network. With the proposed expressway system being gradually completed, a new requirement has risen. It is the need to come to the capital of the country, Colombo, with a reasonably short time once a vehicle leaves the expressway. Since the roadside developments are high and the land acquisition for new roads would be difficult, one option is the use of elevated expressways constructed above the existing highways. This paper presents some structural aspects that can be effectively used to create a well planned and robust elevated highways banking on the experience on design and construction of existing elevated structures that have been built in recent times in Sri Lanka.*

1. INTRODUCTION

Sri Lanka has not undertaken the development of expressways as a matter of higher priority until the recent times. As shown in Figure 1, there are only a few expressways that have been planned and the design and construction of them are gradually in progress to improve the mobility within Sri Lanka. The main expressways can be identified as Southern Expressway (Figure 1), Outer Circular Highway (Figure 2), Colombo Katunayake expressway (Figure 1), and the proposed expressway to Kandy/north (Figure 1). With the completion of these expressways, it is expected that higher volumes of traffic would need to gain access to the city of Colombo. However, the present road infrastructure from the proposed expressway boundaries to the city center appear to be already saturated and hence would take a significant time for a vehicle taking exit from an expressway to reach Colombo.

This is not a desirable situation and could negate the benefits that are achieved with time and fuel savings by using the expressways for which a toll also would need to be paid. In this context, it would be a need of the hour to upgrade the connectivity from the expressways to the city center though it would offer significant challenges to find space for increasing the number of lanes that will provide the access. In this context, elevated highways can become the preferred option though the general perception is that elevated highways could be having higher costs of construction. This paper looks at various options available for Sri Lanka with respect to the selection of routes, materials, design methodologies, construction techniques to realize the an effective elevated road network that could bring significant economic benefits.

2. THE PROPOSED EXPRESSWAYS

The proposed expressways of Sri Lanka can be seen on Figure 1. Sri Lanka being an island with a length of about 430 km and a width of about 225 km, it would be sufficient to have a reasonable expressway network for mobility by linking the major cities. This network can then be supplemented by the normal roads that are maintained in a good condition.

This concept can be viewed from the needs of the country as well. Sri Lanka is a country with a GDP of about Rs 6000 Billion (1 US $ = Rs 120/=). The agricultural sector contributes to about 14% and the industrial sector contributes about 26%. The services sector accounts for the rest and stands close to 60%. The exports stand at about Rs 1200 billion.

For the sustaining of the industrial sector, a good road network is necessary with adequate connectivity to Colombo. It will be essential for sustaining the export sector and also supporting the service sectors like tourism. This is the link that can be considered as inadequate with the current road network that provides access to the capital city.

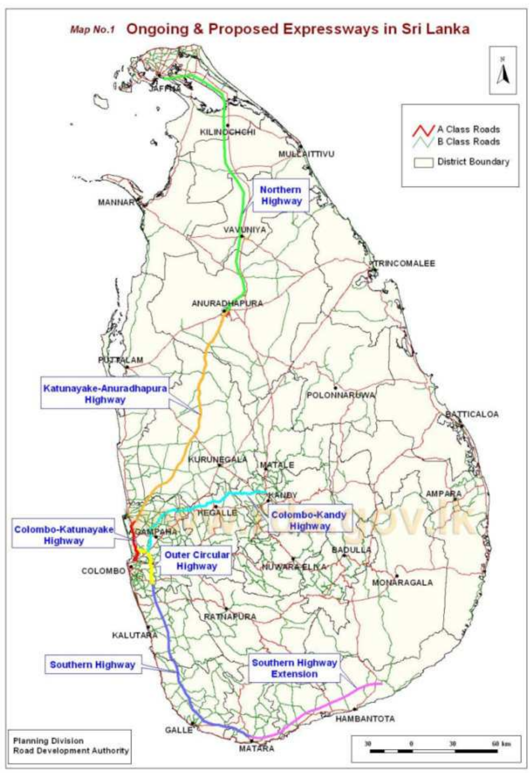

Figure 1 The expressways in Sri Lanka

3. THE POSSIBLE SCENARIOS FOR CONNECTIVITY

The road network to Colombo and within Colombo is generally considered as congested. The implementation of the one way route network in the recent times have reduced the delays within the city of Colombo. One of the reasons for slow moving traffic is the model mix that can be seen that consists of many motor cycles and three wheelers. The separation of faster moving traffic from the motor cycles, three wheelers and slow moving buses is considered as a reasonable solution. Since land acquisition could be a difficult task, elevated highways constructed above the existing four lane main roads could be a viable solution.

Generally, elevated highways are considered as an expensive solution. When, elevated highways are constructed through congested cities, it is possible for those to become a source of noise pollution at a higher elevation thus affecting the medium rise buildings to a greater extent (Mutasem *et al* 2001 and 2002). This could reduce the land value on either side of the road with elevated highway option.

However, the situation in Sri Lanka can be considered as much more favourable. The main highways that would need the construction of elevated expressways above are A1 and A4 for about a distance of 20 km from the city center, up to the boundaries of the proposed expressways as shown in Figure 2. In both these highways, upgrading to 4 lanes from 2 lanes have taken place only in the recent times and hence the large scale roadside developments are yet to happen except within the Colombo city limits and the immediate boundaries. This means that the impact of elevated option could be managed at a reasonable level. This could become an attractive option when the cost of land acquisition and connectivity issues are considered that could arise with alternative solutions.

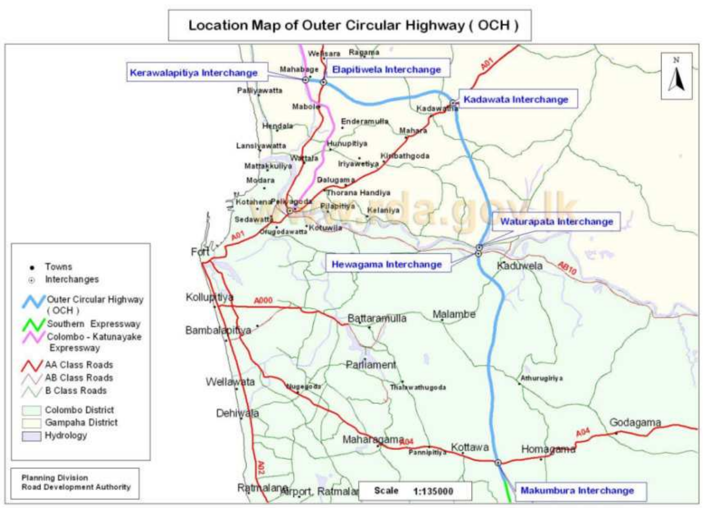

Figure 2 The main roads that would connect the expressways to Colombo

4. THE OPTIONS FOR STRUCTURAL FORMS

It is indicated that elevated expressways with toll charges could be a viable solution for improving the connectivity of the proposed expressways to Colombo. In this context, the selection of an appropriate construction material and methodology will be of primary importance for the successful implementation. One of the greatest challenges will be the construction of an elevated expressway over the existing highway that has to be used on daily basis.

When the future traffic demand is considered, the minimum number of lanes in each direction will be two and it could preferably be three. These details could be finalized with the completion of the feasibility studies that have already been initiated. When the number of traffic lanes are determined, it would be possible to use toll charged as a basis for controlling the number of vehicles so that an adequate level service with appropriate speeds could be maintained. In addition, many other measures will also be necessary to control the number of vehicles such as improvement of rail based passenger services, development of alternative routes, etc. Thus, the elevated expressways will finally become a part of a comprehensive traffic management plan that can sustain the development activities at an appropriate level while ensuring easy access to the city of Colombo from expressways.

An elevated structure can be divided into three main components. They are the foundation, sub-structure and the super-structure. The main material for foundation of this type of large structures will be the reinforced concrete. The sub-structure that consists of pile caps and piers can be out reinforced concrete though piers can be constructed with steel as well. The superstructure will need special attention and the structural forms can be of composite construction of various materials. The use of these are discussed in detail with specific issues addressed.

4.1. The foundations

The soil conditions along the existing highways can be highly variable. One of the solutions that can be successfully adopted in variable ground conditions is large diameter bored piles. Piles can be constructed with diameters up to 2.2 m and the load carrying capacities can be in the range of 15000 kN per pile when the piles rest on bed rock that is at a reachable depth in many locations in Sri Lanka. The advantage of pile foundations is that they can be constructed along edge of the existing highways and the pile caps can be completed below the existing road levels.

4.2. The superstructure

There are two very popular solutions for the superstructure. One is the use of steel beams with insitu cast reinforced concrete acting as a composite material. The other is the use of prestressed concrete beams along with insitu cast deck. In both these, it is possible to have the beams being completed away and then to be placed as the construction progresses. In the Sri Lankan context, prestressed concrete has remained as a preferred material for bridges due to many advantages such as a good durability record with minimum maintenance and the use of locally available or manufactured materials.

There are many issues that need to be addressed with respect to the super-structure. They can be identified as follows:

1. The number of lanes – This will be a critical issue and will depend on many other factors as indicated in BS 5400: Part 2: 1978 or the bridge designers manual of the Road Development Authority of Sri Lanka. However, the width of the existing road is only about 13-14 m and hence the maximum number of lanes will have to be restricted to about 6 with a center median of reasonable width to separate the traffic. An elevated expressway will need a hard shoulder of at least 2.5 m also three lanes of 3.5 m. The other alternative will be to have a wider lane (3.8 m) as the left lane for heavy vehicles and buses and to have two more narrower lanes of 3.2 m for lighter vehicles. This means a width of about 26 m above road of having a width of 14 m. This means the need of cantilevers of about 5.0 m from each pier of about 2.0 m x 2.0 m where the cantilevers will have to act as post tensioned beams.
2. The use of a suitable precast prestressed concrete beam to act as the pier capping beam to support the bridge beams
3. This means that a pier capping beam of suitable shape with two cantilevers on either side that can be post-tensioned to ensure one continuous beam that is connected with post tensioning of the tendons in the ducts
4. A beam of suitable length and shape that can be used to support the insitu cast deck. For this, prestressed concrete I shaped beams of 30-35 m length would be needed since longer spans would need more piles at each pier for which there may not be adequate space
5. An extra-wide pier capping beam that could be used to accommodate changes in slopes and directions while using straight beams of standard section

6. A specially formed precast panel that can be placed on the precast beams so that the insitu cast concrete slab could be constructed only with overnight closure of two lanes of the four lane highway beneath.
7. An expansion joint of suitable type that can ensure smooth ride for the vehicles using this elevated expressway and travel a substantial distance and the special joint that has been applied with a significant level of success is shown in Figure 3 along with the appearance of the completed bridge in Figure 4.
8. Few intersections that will allow vehicles to reach the highway located below via which they will be able to reach their destinations.
9. This indicates a considerable challenge imposed on those involved in the preliminary designs and detailed designs if the elevated highway option has been selected as the preferred option among the alternatives.

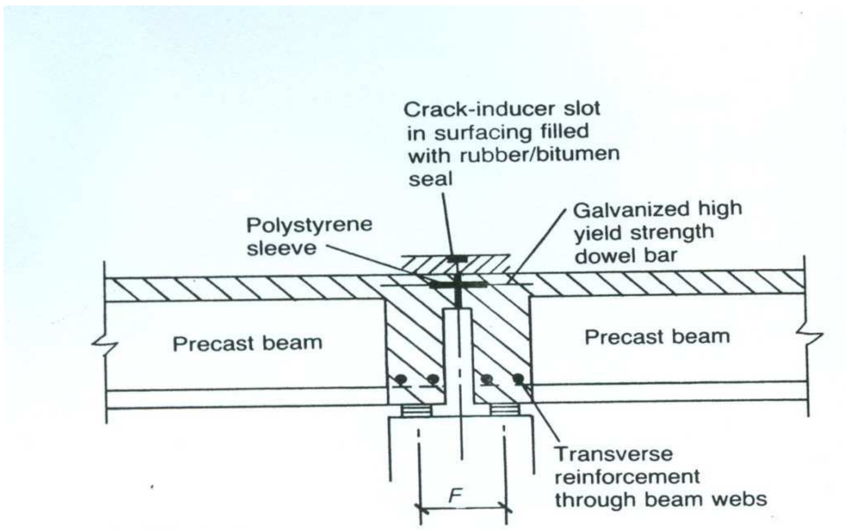

Figure 3 The connection details

Figure 4 The completed expressway without expansion joints

4.3. The sub-structure

The sub-structure will consist of the pile caps and the piers. Since Sri Lanka is a tropical country, it would be ideal to have a good clearance between the existing highway and the proposed elevated expressway such as 7.0 m. In this context, large columns such as 2.0 m x 2.0 m cross sections would be needed for avoiding the slenderness effects and these also would need earthquake resistant details for added safety.

One of the key challenges of the pile caps and pier capping beam will be the durability and the controlling of peak temperatures due to heat of hydration. This will need the use of blended cements that contain pulvarised fuel ash (PFA) along with silica fumes. Due to restricted site conditions that will prevail over the length of the existing highway, high strength self compacting concrete would be needed along with prefabricated reinforcement cages to facilitate the construction at a rapid phase and the post-tensioning operations. The use of latest advances in the concrete technology at its highest level would be needed to ensure the execution of challenging task of this nature.

5. THE EXPERIENCE IN SRI LANKA WITH SIMILAR PROJECTS

A rapid expansion of the highway network of Sri Lanka has occurred in the recent times with many challenging tasks being undertaken successfully. They include the following:

1. Use of 30 -45 m precast prestressed concrete beams for the construction of bridges at a very fast rate as achieved with many concrete bridges and flyovers in Colombo area (Mattakkuliya bridge or Orugodawatta flyover) and the bridges on A4 highway and A15 highway in the eastern province
2. Use of post tensioned pier capping beams with 5.5 m cantilevers at Orugodawatta bridge as shown in Figure 5 and 6
3. Use of self compacting concrete of very low water cement ratio such as 0.25 or less in the toll gates of Southern Expressway
4. Use of blended cements to obtain highly workable concretes that also has high chloride and sulphate resistance as in A32 highway construction (Chindaprasirt *et al* 2005, Neville)
5. Use of water cooling systems in STDP project for preventing the possibility for Delayed Ettringite Formation by controlling the maximum temperatures achieved in the pile caps and other large concrete pores exceeding 700 mm thickness (Bamforth 2007)

Figure 5 A cantilever of 5.5 m with post tensioning ducts in the pier capping beam

Figure 6 The completed bridge with 5.5 m cantilevers supporting the wider roadway

6. CONCLUSIONS

There is a reasonable need to improve the traffic condition of the main approaches to Colombo, especially those serving the gradually updated expressway system. In this context, elevated expressways could offer a viable solution due to various land acquisition problems that are likely to be associated with the new routes. The construction of an elevated highway on a busy road while in use can offer a considerable challenge. The probable need for a six lane highway also could pose a considerable challenge since the width of the existing road is about 14 m at certain locations. The details of all these challenges have been presented in this paper with probable solutions that can be drawn from the past experience in constructing bridges and flyovers in Sri Lanka.

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Development – The Otherside of the Coin

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Keywords: *Environment, engineering, specifications, projects, claims, variations.*

1. INTRODUCTION

Thomas Tredgold's definition of Civil Engineering, may be one of the oldest definitions of Civil Engineering, states that, *"Civil Engineering is the art of directing the great sources of power for the use and convenience of man"*. During the old days, the role of the Engineer was considered extremely important as Engineers reduced the need for human labour and at the same time satisfied some basic physical and social needs in a hostile environment. This attitude eventually developed and the general public believed that the age-old troubles of starvation, disease, floods etc. could be overcome by the proper use of engineering and to some extent it was achieved by the construction of engineering structures such as dams, reservoirs, drainage schemes, communication and road networks.

2. SOCIAL PERSPECTIVE TO ENVIRONMENT

Due to the vast destruction of the environment by our own brothers and sisters, as professional engineers the social perspective towards engineers has changed with time. It has come to a stage where society did not wish to judge finished products solely on engineering criteria. Sometimes, as civil engineers, when we construct a dam and inundate a large area, there will be so many social problems that we have to think of and assess carefully. A Civil Engineer may deal with bricks, concrete and steel whereas an Electronics Engineer might deal with microwaves, satellite communication etc. Whatever we do, we should bear in mind that we, the engineers, are the custodians of the environment. Therefore we have a great responsibility of protecting the environment.

3. SUSTAINABLE DEVELOPMENT

As engineers did not give much consideration to the aforesaid issues, people started looking for professionals who were concerned about the environment, economy etc., who focussed on different perspectives. Therefore, the trend of evaluating the end product or the finished product has completely changed during the past three decades, and the general public sought such professionals to look after their interests while accepting the fact that development should also take place.

Consequently, the professional approach to an engineering problem has now diverged into commercial, social and financial aspects. The commercial aspect simply means that they should provide the client what he desires, and therefore there is a presumption that the client knew what he wanted. Hence, when delivering a product to the client, it is common practice for manufacturers to carry out consumer surveys in order to find out what is required by their clients. The professional engineer also has to play a role to satisfy the requirements of the client, but at the same time he has to also evaluate the other aspects of a product such as its effect on the environment, the society, the long term benefits, the cost etc. Therefore, when considering the engineering aspects of a project, one should bear in mind that it is secondary to the social aspects which I have described above.

Coming back to the engineering aspects, I would like to discuss some of the engineering issues which I have come across as a practising Engineer and Arbitrator.

I was involved as an Arbitrator and Adjudicator in several construction projects during the last 15 years where I have especially concentrated on construction dispute resolution rather than pure engineering. Some of the problems that I have faced in the past, looking at it from different perspectives as a person who sometimes has to defend the Contractor or the Employer depending on who my client is, as described below:

4. INVESTIGATIONS AND SURVEYS OF A PROJECT

In most of the road projects that I had the opportunity to work in, due to lack of preliminary investigation quite often vitiations had to be done. The amount of work in a major road project is so large even one variation could cost a lot of money. For example, if the soil properties are not analysed and therefore stability of embankments are not done the consequence of it would be unstable slopes. Therefore there is a strong possibility of landslides during construction and after construction. As you know when a land slip occurs the amount of additional work to be done becomes an unknown quantity and so there is an effect on the environment. I had an opportunity to sit as an arbitrator in a project at Kandy where the State Authority was constructing power lines and the earthwork quantities had been increased by four times (quadrupled) due to land slides. The consequence of this would be increase in the cost of excavation, time delays, additional costs due to overhead etc. In one instance due to land slips the Contractor's machinery had been buried. Therefore pulling out the machinery and the cost of damages also will contribute towards the increase in prices.

5. INITIAL SITE VISITS AND INVESTIGATIONS TO IDENTIFY PROBLEMS

In one of the projects where I was involved in a Dispute Resolution Process, the Contractor was making a claim for delays due to obstructions that existed in the ground preventing driving of piles. In the said Project the contractor was a piling contractor and while boring for piles he had encountered many obstructions such as foundations of an old building, concrete blocks etc. This type of information should have been revealed during the preliminary investigations by the Employer or the Consultant. The ultimate result is a delay in the Contract and additional expenses to the Contractor, due to idling of the Contractor's machinery.

6. ESTIMATION OF THE COST OF THE PROJECT

Proper Project Estimation at the time of planning of the Project is an extremely important factor. In a building project the Employer had had in his mind to have a Central Air-Conditioning System. However this was not reflected properly in the Contract Documents. It is common practice among the Consultants to issue minimum drawings or details as possible during the “Tendering Stage”. The reason could be either that the Consultant was not ready with the construction drawings or that the Employer/Consultant had given little importance to tender drawings. In the said Project the Contractor had bid for Split Type Air Conditioners whereas the Employer had assumed that he had specified Central Type Air-Conditioners. The ultimate result was that the Contract was terminated as a result of which the Contractor claimed a large sum of money from the Employer for unlawful termination.

I also would like to discuss another example of similar nature. In this Project it is stated in the Bill of Quantities that cabling has to be done from Point ‘A’ to ‘B’ which is about 500 metres in order to obtain 60 Amperes from the Main Grid. In the BOQ it was also stated that Single Core Cable should be used. The Contractor had laid a 4-core Cable which can carry a 3-Phase supply. There is a significant price difference between a Single Core Cable and a 4-Core Cable. The Employer is prepared to pay only the price of a Single Core Cable whereas the Contractor is claiming the cost of a 4-Core Cable. This dispute has gone for resolution and as of now the adjudication process is pending. The amount of the claim is around Rs. 22 Million. This dispute could clearly have been avoided had the Consultant given proper description at the time of tendering. Therefore, documentation is extremely important in contract administration. Hence, it is important to remember that missing an important statement could lead to litigation, the cost of which could be enormous. Apart from this the parties will also lose the good relationship that existed between them. Therefore, the Engineers have to play an important role in the preparation of contract documentation.

7. PROPER CONSTRUCTION DRAWINGS

At the time of tendering it is important that the Employer is ready with appropriate construction drawings. One cannot expect the Contractor to Bid on a certain set of drawings and thereafter proceed with construction on a different set of drawings. In another case in a Road Project, the depth of excavation had been given as 3 metres below the existing surface, which the Consultant may have decided upon the information available to him at that time. After proper investigation that had been commenced after the award of the tender, it was found that the required depth would be much more than 3 metres. The Contractor had already hired machinery which was imported from another country, equipped with a short arm which facilitated excavation only of around 3 to 4 metres in an efficient manner. However, when the excavated depth was increased to 6 to 7 metres, the Contractor had to either modify the existing machinery with a longer arm or hire a piece of equipment with the required arm length locally at different rates. Therefore, due to this lack of investigation at the correct time, the Contractor has lost time and money where all such costs have to be borne by the Employer.

8. RECONSTRUCTION ACTIVITIES

In congested areas, as well as in road construction projects, there are many reconstruction activities that have to be carried out by the Employer or the Contractor. There is however, certain information where it is not reasonable to expect the Contractor to have access to, as at the tender stage the Contractor has very little access to the construction site.

Sometimes relocating a water supply line might take several months. There are extensive telecommunication cables laid by the side of the road which should not be disturbed.

I remember an incident when I was working for the Roads and Traffic Authority in Australia where telecommunication lines had been laid from Sydney to Brisbane. The aforesaid line had been laid by the side of the main highway and this was located only at the time of construction which was a little too late. Relocation of this telecommunication line took about six months as the cable had to be especially fabricated. All these things will add to the cost of the project as the Contractor has to charge for idling resources, overhead costs etc.

Therefore, in construction drawings the location of existing services also should be identified accurately so that the Contractor is made aware in order that he may prepare himself in advance if relocation is necessary.

9. CLEARANCES FROM LOCAL AUTHORITIES

One also has to remember that by merely being a state authority it may not have unlimited powers to carry out construction work in a road project even though the land is acquired for the required lanes and reservations. There may be certain properties still belonging to the other state authorities.

I remember in a road construction project in Sri Lanka, the Employer believed that he could make use of the rock encountered during the excavation of the road. Unfortunately this was not the situation. In accordance with the statutory requirements, any authority should obtain a license or permission from the Geological and Mining Bureau in order to blast and use the rock. There had been a royalty charge also included and this cost had never been estimated in advance by the authority. Therefore the cost and the time had been increased in the project.

When a site is handed over to the Contractor he should be able to proceed with construction without any hindrance. In a road project near Kandy, the Contractor was awarded the contract to rehabilitate the road at a project cost of approximately Rs. 800 Million. When the Contractor was proceeding with the work the Forestry Department interrupted the work stating that the Contractor has to obtain approval from their department as widening will entail encroachment into the forest reserve. The outcome was that the Contractor had to carry out construction in patches as the encroachment had occurred in many places along the length of the road.

10. INCORRECT SPECIFICATIONS

When specifying an engineering project there should be clear and unambiguous specifications for each and every item that has to be performed by the Contractor. If these specifications are not definite the Contractor might have to face two serious problems. Firstly, he may quote the job incorrectly and secondly the Contractor might try to carry out the work as he has priced in his tender and not to the quality or the specifications that is required by the Engineer. This would lead to many problems such as disagreements between the parties, litigation, time delays etc.

There was a specification for the construction of a road where the Contractor was required to lay two layers of metal, and thereafter the gaps were to be filled in the metal with a smaller sized aggregate. When the Contractor commenced compaction in order to achieve the density requested, he was unable to do so as there was no interlocking of materials. The two layers of large size aggregates merely separated without forming a base course. In this instance the Contractor sought advice from the Engineer on how to proceed with the construction and the Engineer came up with many trial and error proposals. The final decision was to use a combination of hand-broken metal and machine crushed metal. Hand-broken metal in large quantities was not available for immediate purchase, and therefore the Contractor had to order hand-broken material and wait for many months until sufficient quantity of hand-broken metal was made available to him. This led to time delays and loss of production to the Contractor. At the same time, the Employer was not happy with the progress of the Contractor and he terminated the contract. Therefore, whatever you do at the planning stage in order to save time will be of no use with your project analysis arrived at with complicated software such as Microsoft Projects or Primavera, since the Planner has failed to look into the most important aspect which is correct specification.

11. DESCRIPTION IN THE BILL OF QUANTITIES

Most of the Engineers have a habit of filling a copied Bill of Quantity when he has to prepare such document. It is extremely important that one has to consider the sequence of events that would take place when an item of job is to be done. If he follows an existing BOQ blindly where the item of work would be different to that specified earlier, the Engineer will be misleading the Contractor. The consequence of misleading the Contractor by providing an improper description for the work item will lead to price variations. A Contractor would never carry out work which is different to the Bill of Quantities which he has quoted for. I remember when I started my career working for a Consultant, and we had to deal with Contractors who were even knowledgeable enough to interpret Engineering and contractual documents. A few decades ago such people were very rare and most of the Contractors were willing to do whatever the Engineer wanted. There are two reasons why the Contractor is obliged to carry out the directions of the Engineer. Firstly, during that time the profit margins of the Contractor were very high; it even ran up to 100%. Therefore the Contractor could make and break things without asking for extra payment as the only loss that he could incur is a little drop in the percentage of profit.

This environment has changed over a period of time. The Contractor's staff is sometimes better qualified than the Consultant's staff. The profit margins of the Contractors are now very low, and therefore if they have to do additional work they cannot simply afford to do so as it would cause a severe loss to him.

Recently, I was involved in dispute resolution where the Engineer was expecting the Contractor to carry out water proofing of a block of toilets before tiling. He has mentioned water-proofing in the BOQ before tiling, but the description does not say the rate of tiling is inclusive of water-proofing. The omission of this sentence has caused the Employer a loss Rs. 8 M as it was a hospital building where toilets covered a large area. Therefore, every work in a contract document has a monetary value, and at the same time every line that you draw in your structural drawing also has a meaning.

I would like to mention another case where, in a lump sum contract, the Contractor had failed to cost for a retaining wall although it is shown in two lines in the drawing. These are costly mistakes and therefore as Engineers I believe, apart from engineering, you ought to be very concerned about the things that you do, and one cannot lose sight at any stage.

Although I was discussing mostly matters that you may consider are not engineering, if you do not concentrate on these issues the industry will deprive you of your status and you will be the third or fourth person to be consulted and the other professionals will be far ahead of you. Therefore, you should never allow this to happen.

As I stated my speech I would like to further add that whatever we do to improve our infrastructure, we should not at the same time disturb the environment. As leaders of development we have a great responsibility to carry out our work in a sustainable manner. People now talk about rainwater harvesting, greenhouses, environmental pollution, recycling etc. Therefore, as custodians of the environment, with the limited natural resources available to us, we as engineers must try to protect the environment as much as possible. Continuously we use fossil fuel which might be depleted within the next 50 years or so. Therefore it is our challenge as engineers to preserve energy and at the same time develop energy from various other sources such as solar, wave, wind etc.

Let me quote from a speech made by Chief Seattle when he was asked to sell to the white man the land which he had been occupying.

His reaction was: *"How can you buy or sell the sky or the warmth of the land? The idea is strange to us. If we do not own the freshness of the air and the sparkle of the water, how can you buy them? Every part of this Earth is sacred to my people. Every shining pine needle, every sandy shore, every mist in the dark woods, every clearing and humming insect is holy in the memory and experience of my people. The sap which courses through the trees carries the memory of the red man."*

He further stated: *"The perfumed flowers are our sisters; the deer, the horse, the great eagle, these are our brothers. The rocky crests, the juices of the meadows, the body heat of the pony and man - all belong to the same family."*

Modern man in his search for pleasure and affluence has exploited nature without any moral restraint to such an extent that nature has been rendered almost incapable of sustaining healthy life. Invaluable gifts of nature, such as air and water, have been polluted with severely disastrous consequences. Man is now searching for ways and means of overcoming the pollution problem as his health too is alarmingly threatened. He also feels that it is irresponsible and morally wrong on his part to commit the future generations to a polluted planet. If man is to act with a sense of responsibility to the natural world, to his fellow human beings and to unborn future generations, he has to find an appropriate environmental ethic today to prevent further aggravation of the present pollution problem. Hence, his search for wisdom and attitudes in a hitherto neglected area of knowledge namely, religion.

12. CONCLUSION

In the modern age man has become alienated from himself and nature. When science started opening new vistas of knowledge revealing the secrets of nature one by one, man gradually lost faith in theistic religions. Consequently, he developed scanty respect for moral and spiritual values as well. With the advent of the Industrial Revolution and the acquisition of wealth by mechanical exploitation of natural resources, man has become more and more materialistic in his attitudes and values. The pursuit of sense pleasures and the acquisition of possessions have become ends in themselves. Man's sense faculties dominate him to an unrelenting degree and man has become a slave to his insatiable passions. (Incidentally the sense faculties are in Pali *indriyas* or lords, because they control man unless he is sufficiently vigilant to become their master.) Thus man has become alienated from himself as he abandoned himself to the influence of sense pleasures and acquisitive instincts.

In his greed for more and more possessions he has adopted a violent and aggressive attitude towards nature. Forgetting that he is a part and parcel of nature, he exploits it with unrestrained greed, thus alienating himself from nature as well. The net result is the deterioration of man's physical and mental health on the one hand, and the rapid depletion of non-replenishable natural resources and environmental pollution on the other. These results remind us of the Buddhist teachings in the suttas discussed above, which maintain that the moral degeneration of man leads to the decrease of his life-span and the depletion of natural resources.

Moral degeneration is a double-edged weapon, it exercises adverse effects on man's psycho-physical well being as well as on nature. Already killer diseases such as heart ailments, cancer, diabetes, AIDS, etc., are claiming victims on an unprecedented scale. In the final analysis these can all be traced to man's moral deterioration. Depletion of vast resources of fossil fuels and forests has given rise to a very severe energy crisis. It cannot be emphasized too strongly that such rapid depletion of non-renewable natural resources within less than two centuries, an infinitesimal fraction of the millions of years taken for them to form, is due to modern man's inordinate greed and acquisitiveness. A number of simple ancient societies had advanced technological skills, as is evident by their vast sophisticated irrigation schemes designed to feed the fundamental needs of several millions. Yet they survived in some countries over 2000 years without such problems as environmental pollution and depletion of natural resources. This was no doubt due to validity of the philosophy which inspired and formed the basis of these civilizations.

"Whatever breathing creatures there may be
No matter whether they are frail or firm,
With none excepted be they long or big
Or middle-sized, or be they short or small
Or whether they are dwelling far or near
Existing or yet seeking to exist
May beings all be of a blissful heart."

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Challenges for Sustainable Water Management

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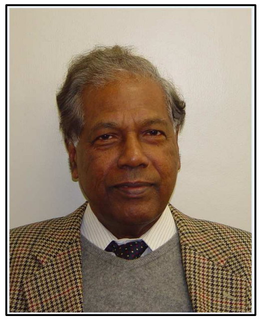

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Abstract: *Water is a precious resource essential for all forms of life. It is abundant in nature but has significant temporal and spatial variability. With increasing population, the per capita share of water on earth is decreasing, and in some regions has reached levels where communities face water stress and water scarcity. Whereas lack of safe drinking water is a major problem for close to a billion inhabitants of the earth, too much water also bring about misery, agony and destruction to many people, places and infrastructure. The former may be attributed to the physical lack of water, pollution or unaffordability whereas the latter is attributed mainly to population growth, urbanization and livelihood issues. Optimal management and use of the available water in the 21st century needs a paradigm shift to a holistic approach since all aspects of water as well as social and economic infrastructures of the world are now more interdependent than ever before. Coping with water problems in the 21st century therefore poses many challenges to water managers. Drinking water security, food security, energy security, climate change, water-related disaster management and maintaining acceptable environmental quality in a sustainable manner are among the major challenges at present and in the foreseeable future.*

Keywords: *Drinking water and food security, climate change, environmental security, disaster management, research, education.*

1. INTRODUCTION

Water is a precious commodity necessary for all forms of life. Its role and importance in societies are exemplified in the following visionary statements in history:

- "Not a drop of water that falls from the heavens shall flow into oceans without being utilized by man" – King Parakramabahu the Great (King of Sri Lanka, 1153-1186)
- "To control China one must first control water" - Guan Zhong, politician and statesman during the Spring and Autumn period of Chinese history (c 720-645 BC)
- "One who solves water problems in the world deserves two Nobel Prizes, one for Peace and one for Science" - Late President of USA, John F Kennedy (1917- 1963).

In the ancient world, civilizations began along the banks of rivers: Yellow and Yangtze in China, Nile in Egypt, Indus in India, among others. The Chinese, during the Wu, Qi and Sui Dynasties, (starting from 486 BC) built the Grand Canal which runs from Hanzhou to Beijing, - a distance of about 1900 km. It is the oldest and the longest water conveyance system in the world - longer than the Panama and Suez canals combined. Hydraulic civilization in Sri Lanka started in the 6^{th} century BC. Romans built the aqueducts in the 7^{th} century BC. In Persia (now Iran), qanats were built some 3000 years ago. Archeological findings point to the existence of aqueducts in India, Mexico, Madeira, among other places, in ancient times. More recently, the Nanzenji aqueduct in Kyoto was built in 1890 during Meiji era. The main purpose of all these engineering feats was to provide water for irrigation and drinking. The kings or the rulers had immense power over the subjects as they controlled the flow of water.

One of the main water problems around the world today is the lack of potable water. After oxygen, water is the most vital ingredient for sustaining life. Food, which requires water for growing comes next. Over one billion people in the world do not have access to safe drinking water and about twice as many do not have access to proper sanitation. The consequences of not having access to potable water are serious particularly in developing countries which suffer from various types of water-borne diseases some of which result in premature death. In terms of quantities, the requirements for domestic and municipal supplies (approximately 8% of total water resources of earth) are far less than those for growing or producing food. The two major users of water are agriculture and industry. The challenge for future is how to guarantee water and food security to all inhabitants of the world.

The second challenge comes as a result of human intervention of the environment over the years. Many of the water resources that were in pristine conditions many years ago are now heavily polluted. They are in a state where they are totally unfit for human use, or that they can only be restored to levels suitable for human consumption at a very high cost. Associated with any development activity, there is always an environmental cost. Waterbodies such as rivers, streams, and lakes also act as waste receiving bodies in many regions. When the release of wastes into such waterbodies exceed their capacities to self purify, they will become 'dead' when no living organism can survive. How the benefits arising from development activities should be balanced against environmental costs is a major issue that needs to be considered holistically in the context of the total water environment rather than as isolated systems.

A third challenge lies in the area of water-related disasters. Water, despite being essential for all forms of life can also at times be destructive. Floods, landslides, and debris flow are all triggered by excess water. Many regions in the world are vulnerable to water related disasters and the damage as well as the resulting casualties are on the increase. It is also important to note that not only the numbers of disasters are increasing but also the number of people affected too because of migration of people into areas with better economic prospects.

2. MAJOR CHALLENGES

2.1. Drinking water security

The health of a nation depends upon the level of cleanliness of the domestic water supply. It is a problem that is often ignored or sidelined by the developed countries as it is only a problem of the poor and the developing countries. Approximately one billion people, or one seventh of the world population, do not have access to clean and safe drinking water. Coupled with the accompanying sanitation problem, which affect approximately twice that number, the situation if allowed to continue can be disastrous. One of the UN Millennium Development Goals (MDG) was to halve these numbers by 2015. Now (2012), 89% of the population of the world have access to improved water supplies, up from 76% in the base year of 1990. (BBC News, March 6, 2012). However, 40% of those still without access to improved drinking water live in sub-Saharan Africa. Worldwide, almost 800 million people still drink dirty water. In the sanitation sector, the MDG has not been reached and in fact it is reported that the situation is worse than in 2000 (BBC News, November 18, 2011). Alarming facts arising from poor sanitation include a child death from diarrhoea every 20 seconds, and more diarrhoea-related child deaths every year than deaths caused by aids, malaria and measles combined (UNDP). Unlike a major flood or an earthquake which affect a small region with high population density, the effect of the lack of drinking water is spread over vast areas with relatively low population densities. From the media point of view, such widespread and prolonged suffering receives much less attention compared with that received for high impact type of disasters such as earthquakes, tsunami and major floods.

Water scarcity can be arising from physical lack of water, poor quality, or due to lack of capacity for developing and maintaining a reliable supply. Arid and desert areas suffer from physical lack of water and such areas also experience pollution problems. Traditional techniques such as rainwater harvesting, and groundwater exploitation would be better appropriate technologies than conventional water supply technology. Since such areas are sparsely populated, achieving individual household self sufficiency is more favoured and should be encouraged than traditional water distribution systems where the conveyance cost can be excessively high. It is also important to introduce low cost water filters which can be used in individual households. It is estimated that $1 invested in improving access to safe water can increase the GDP by $3-14.

2.2. Food security

The global average food intake has increased from 2250 kcal in 1961 to 2800 kcal in 2000, although, in South Asia and sub-Saharan Africa, it still remains at 2450 kcal and 2230 kcal respectively (IWMI, 2006). This increase may be attributed to a number of factors. Land and water productivity has increased with average grain production from 1.4 Tons/ha to 2.7 Ton/ha during the last 4 decades (IWMI, 2006). Global trade in food products also has increased thereby increasing the flow of virtual water. On the negative side are the facts that the population is still increasing, and that the increases are in areas where productivity is low and with inadequate human and economic capacities to upgrade their production.

It is a fact that grains have been the basic form of food for all humans. However, changing life styles have also changed the dietary habits of many societies that have attained some degree of affluence. Meat, milk and fish consumption have increased substantially thereby exerting an additional water cost to food products. For example, the water cost of 1 kg of grain varies between 500-4000 litres whereas the water cost of 1 kg of meat is about 10,000 litres. To produce the grains that are needed for meat and milk production, vast areas of land are irrigated resulting in high environmental costs. On a global scale, agriculture uses about 70% of the world's water resources, followed by industry which uses about 22%. In recent years, many countries have embarked on the production and use of biofuels in place of fossil fuels. This practice also has added a further burden of our water resources because of the additional quantities of water needed for growing the bio species from which the biofuels are extracted.

The challenge in this context is how to produce our food (and energy sources) at the least water cost. Large scale crop production necessarily depends on irrigation to ensure guaranteed successful harvest. Rainfed cultivation on the other hand is weather and climate dependent and therefore does not guarantee a successful harvest every year. The water efficiency in irrigated cultivation is always low. Flooded irrigation results in evaporation from the free water surface as well as from the bare soil. In order to increase the water productivity, it is necessary to reduce crop evapo-transpiration. Better and more efficient techniques such as drip irrigation, low pressure sprinklers are currently being used to increase the water productivity.

Food trade is another area that requires attention. It is not meaningful to attempt to grow food in water-poor regions at high costs and consequent environmental degradation. With global trade expanding, it is quite logical to grow more food in water-rich regions and make them available as food products to those in water-poor regions. However, in the present context, many countries aspire to be self sufficient in food for strategic reasons. Such aspirations will not be needed if fair and reasonable trade agreements and treaties to share trans-boundary water resources are in place. Above all, concern for the well being of other human beings should be the guiding principle for sharing the water resources on earth.

2.3. Climate change

Global warming which is an indicator of climate change is currently a hot topic. The Intergovernmental Panel on Climate Change (IPCC), has concluded that there has been significant temperature rises since the 1970's which they attribute to global green house gas emissions. The issue has also received endorsements and publicity from powerful circles and personalities. There is no doubt that discernible warming is taking place in some parts of the globe as evidenced by melting of ice caps and glaciers, sea level rises, temperature rises, among other changes. At the same time, there is another school of thought, though not as powerful as those advocating the global warming phenomena, who take the view that the issue is blown out of proportion, and that warming exists locally and that it is premature to conclude that the issue is a global phenomenon.

Notwithstanding the arguments for and against global warming, it is s fact that the earth has gone through cycles of warming and cooling in the past. Changes have persisted over decades and sometimes over centuries. Although instrumental measurement of temperature started in 1850, various proxy methods (such as tree rings, ice cores, corals etc.) have been used to understand Paleoclimatology. Examples include the Holocene warm period (circa 1800-4000 BP), the Roman warm period (circa 200-500 AD), the medieval warm period (circa 1000-1100 AD) and the little ice age (circa 1200-1800 AD) (Rundt, 2008). It has also been shown that there is a 1500-year cycle of global warming (Avery, 2008). Some scientists believe that the warming has peaked and that the earth is more resilient than predicted. (Ref: http://news.bbc.co.uk/2/hi/science/nature/7329799.stm). This would mean that the temperature has not risen globally since the 1998 El Nino warm period. There are evidences of cyclical changes of climate in the recent histories of China, UK and Greenland where the warmth has been measured by the ability to grow plant species such as vine, and the ability for animal species to survive. It is also argued that the issue of climate change and global warming in particular has been used as an inhibitor to economic progress in less developed countries. Whether there is global warming or not, earth's resources should not be unnecessarily wasted, but should be shared in an equitable manner. Adaptation, rather than prevention, should be the way forward.

2.4. Environmental pollution

Environmental pollution is a by-product of economic development and goes unabated in many waterbodies as a result of indiscriminate dumping of domestic, agricultural and industrial wastes. Slow accumulation of pollutants over the years in many rivers (e.g. in China and India) has made them aesthetically unpleasant and biologically and chemically toxic. Restoration of such rivers to environmentally acceptable levels is costly and a fair and reasonable approach to recover costs is to follow the polluter pay principle. Many countries have enacted legislation to address this issue but the enforcement becomes difficult as the costs are passed back to the tax payers by the polluters as increased costs of their commercial products. Incidents of pollution caused by accidents such as the one that occurred in Songhwa River in Northeast China are also on the increase as more and more toxic industrial ingredients are conveyed too frequently and over long distances. Introduction of advanced methods of waste water treatment such as membrane technology, recycling, reclamation of waste water etc. help alleviate the pollution problem to some extent. In the long term, an integrated approach of water management in which all aspects of the water sector are considered and optimized within the framework of a single ecosystem appears to be the way forward.

2.5. Disaster reduction

Natural hazards are not preventable. In terms of the damages and the casualties, the main fresh water related hazards are floods, landslides, and debris flow. A hazard becomes a disaster when the region and the community are vulnerable and lack the coping capacity. Therefore any approach for mitigating the consequence of a disaster needs to focus on reducing the vulnerability and enhancing the coping capacity. Although there are many international and regional initiatives aimed at disaster reduction, their implementation is slow and lacks high priority due to political, cultural and economic issues particularly in developing countries. It should also be recognized that capital intensive engineered approaches of disaster reduction practiced in developed countries cannot be applied in developing countries. Rather practices which take into account the local culture, economic status, as well as the political environment would be more effective and implementable. It is only when the community attains a certain degree of affluence that people will begin to think about disasters and invest in disaster mitigation measures. For those living at or below the poverty line, day to day survival by itself is a disaster, and it is very difficult to get them involved proactively on implementing mitigative measures. Although investment in disaster mitigation is considered as a development issue in developed countries, there are other areas of higher priorities where investments need to be channeled in developing countries. More can be achieved by promoting non-engineered approaches of coping with disasters as well as assisting in upgrading the living standards of those in less developed countries.

Water has become a powerful tool that can be used to control the lives and livelihoods of millions and millions of human beings. In the not so distant future, water is likely to replace oil as an economic commodity which may be manipulated by states, enterprises and individuals to their advantage. Water though being a beneficial commodity can at times be destructive too. Floods, caused by excessive rainfalls, snowfalls, etc., can bring immense misery to humankind. This is a serious issue in Asia where about one third of the world population live. The recent floods and other water-related disasters in Sri Lanka, Thailand, Australia, China and Japan amply illustrate this. It is also an unfortunate fact that people return to the disaster-prone areas year after year because of their livelihood issues. More incentives to encourage vulnerable people to relocate to safer areas should be provided by governments to avoid recurrence of similar disasters.

2.6. Research and development

Understanding a problem is a prerequisite to solve it. This applies to the water sector too. Technical issues in the water sector are all contained within the confines of the hydrological cycle in which meteorology plays the upstream role and oceanology the downstream role. In between there are several processes taking place. They can be quantified using the basic concepts of fluid mechanics, hydraulics, hydrology and environmental engineering. Quantification is done using the basic laws of physics such as conservation of mass, conservation of momentum and conservation of energy. Governing equations are obtained using the Lagrangian approach which follows the motion of the same mass of fluid in space and time, or the Eulerian approach which follows the fluid passing through a given fixed position in space. The latter approach uses the control volume concept and leads to the differential form of mathematical representation whereas the former leads to the integral form. The catchment can be considered as the basic unit of domain within which all such processes are considered.

Catchment is a topographically demarcated region which is influenced by the atmosphere from above, the geosphere from below, and the biosphere and the hydrosphere from within. The main challenges of catchment hydrology arise as a result of the interactions of influences from these various components, influences brought about by human activities and the need to ensure that the catchment processes are sustainable. In the past such influences may have existed but to a lesser degree with no conspicuous adverse effects. In the recent times, human influences have accelerated, and the cry for a sustainable future has become louder. A better understanding of the dynamics of the catchment is the key to face such challenges.

Understanding the dynamics of the catchment can be achieved in many ways. For example, collection of better and more comprehensive data could be a starting point. Without data no theory or technique can be validated. Almost all practical tools of hydrology in the past have been empirical. Over the years, techniques of data collection have also changed – from on-site measurements to remotely-sensed. Space based remotely sensed data have become accessible to almost anyone, anywhere, but not without problems. They are grid-based and one data point may represent several hundreds of square kilometres. They then have to be down-scaled to the catchment scale and the methods of downscaling are not perfected yet. Another challenge is that the governing equations of the catchment processes may not be the same at different scales. This is another challenge.

Understanding can also be achieved by modelling the catchment dynamics using conceptual, data driven, and/or physics-based approaches. Nowadays, there is no shortage of models. However, there is no hydrological model that has universal applicability and as a result, more and more models seem to originate at a rate faster than many hydrologists can digest. In fact there are simple models, not-so-simple models, complex models and more complex models with each type having its own pros and cons. There has been a proliferation of models and modeling techniques in the past few decades, and as a result, it is confusing even to an experienced hydrologist. Establishing guidelines on the choice of models under different constraints is another challenge. Simple models have fewer parameters and are relatively easier to calibrate. However, such models do not represent the catchment processes adequately and therefore would in many instances be over-simplified. On the other hand, complex models can potentially describe the catchment processes to any degree of sophistication, but with a price. With increasing complexity of the model, the number of parameters also increases and the principle of parsimony is often violated. The interaction of the multi-parameters give rise to the problem of “equifinality” implying that there is no unique set of parameter values that will give a set of output results but rather a “Pareto” set of feasible parameter space.

There are also emerging techniques of modelling such as for example artificial neural networks that emulate the brain, genetic algorithms and genetic programming that emulate genetic evolution of biological species, phase space re-construction methods that uses the theory of chaos, fuzzy logic that takes into consideration partial truths for dealing with imprecise information, and their various hybrid forms. These methods can be considered as belonging to 'data mining' which attempts to uncover hidden information contained in the data. Such techniques have been mainly developed in the mathematics, statistics control engineering domains and are gradually finding their way into Civil Engineering applications. Embarking on research in these areas would be challenging and rewarding.

2.7. Education

Education plays an important role in the sustainable management of the water environment. Earth's water resources belong to all living things but are controlled by humans only. For its fair and equitable use (and misuse), all human beings should have at least a basic understanding of the water cycle including its relationship to the environment. This is best achieved by introducing related subjects in the school curricula very early in life. Advanced and specialized knowledge should be provided at university level for the professionals who would be managing the resource including their impacts on the environment. A more important aspect is continuing education. Learning is a life-long experience. Knowledge is never at steady state. Many engineering professional bodies promote, and in some cases require, engineers to update their knowledge by attending continuing professional development (CPD) courses to retain their memberships. It is also important to have such CPD courses accredited by reputed and relevant learned societies.

3. CONCLUDING REMARKS

Sustainable management of earths water resources needs the attention, commitment and dedication of all stakeholders. The quantities are dwindling and the qualities are deteriorating. Many issues including societal and cultural matters are interconnected. The problem of water management therefore needs to be addressed in a holistic way. Responding to different challenges sometimes need addressing conflicting interests. If any of the above challenges are taken in isolation, other challenges may have to be overlooked or ignored. An approach in which a balance is sought between development and conservation, and where modern technology and traditional practices go hand in hand would be ideal but the implementation of such an approach requires the will and commitment of all stake holders. A guiding principle would be to share the resources of the planet earth in an equitable manner. Failure to do so will result in a situation whereby the water-rich countries can starve the water-poor countries when conflicts reach critical stages. In the not so distant future, water will take the place of fossil fuel as a political and economic tool that can be used to manipulate communities and governments.

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Relation between changes in the modal properties and structural changes in an existing steel truss bridge

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Abstract: *This paper describes an investigation of the applicability of changes in modal properties of a bridge to structural health monitoring. Vibration measurements were conducted at an existing Warren truss bridge for road traffic in-service for more than 40 years. Experimental modal analysis applied to the measured data identified global vibration mode of a bridge span and local vibration modes dominated by vibrations of diagonal members. There were a partial fracture, cracks and pitting corrosions in diagonal members of the truss bridge detected during recent visual inspections and some of those damages were repaired during the investigation described in this paper. The effects of those structural changes on the modal properties identified were discussed with the results of finite element analysis so as to investigate the applicability of modal properties changes in global and local vibration modes to the identification of structural changes in truss bridges.*

Keywords: *vibration-based structural health monitoring, natural frequency, modal damping, truss bridge, vibration measurement*

1. INTRODUCTION

Recently, structural health monitoring has drawn more attention from bridge engineers, particularly in developed countries where the design life of substantial number of bridges is approaching to its end. The tragic collapse of the I-35W Mississippi River Bridge in Minnesota in the United States in 2007 may have emphasized needs for appropriate structural health monitoring. In Japan, extensive inspections after the collapse of the bridge in Minnesota revealed cracks and fractures in members of steel truss bridges, some of which reached a complete loss of the cross section of a member.

In health monitoring of bridges, visual inspection has been the principal technique, although the reliability of this subjective technique heavily depends on the skills and experiences of the inspector. Objective techniques for structural monitoring should be useful to assist the visual inspection. A possible objective technique that has been investigated worldwide is vibration-based health monitoring (e.g., Doebiling *et al*, 1996; Balageas *et al*, 2006; Boller *et al*, 2009). Most techniques proposed for vibration-based monitoring are based on the identification of changes in the dynamic characteristics of structure from recordings of structural vibration induced by various natural and artificial sources (e.g., Siringoringo & Fujino, 2008). It is assumed that structural damages are associated with changes in the mass, stiffness and damping of the structure that yield changes in the dynamic characteristics, such as natural frequencies, mode shapes, and modal damping.

The objective of the present study was to investigate the relation between changes in the natural frequencies and modal damping and structural changes in an existing steel truss bridge. There were local damages found recently in the bridge used in this study. Sets of vibration measurements were conducted to obtain vibration data for different structural states. The contents of this paper include a part of outcomes from studies reported in Yoshioka *et al* (2008, 2009, 2010a, b) that was summarised in Matsumoto *et al* (2010).

2. FIELD MEASUREMENTS

2.1. Bridge used in this study

A bridge over a river for road traffic in-service from 1965 was used in this study. The bridge consisted of five separated spans, each of which was a Warren truss with a span length of 70.77 m and a width of 6.0 m (Figure 1). The tension diagonal members had a H-section, whereas the compression diagonal members had a box section, as shown in Figures 1 (c, d). There were eight or nine oval holes in the web of each tension diagonal members, except those at the ends of each span, for the reduction of the weight of steel.

During a visual inspection in July 2007, a partial fracture was found near the bottom end of a longest tension diagonal member (D5 in Figure 1), which resulted in loss of the half of its cross section. There were cracks in four other longest tension diagonal members also. In August 2007, those damages were repaired by fixing additional steel plates with the same thickness as that of the member (i.e., 8 mm) by high strength bolts to cover both sides of the flanges and web for a length of about 1.5 m from the end of the member. Additionally, there were pitting corrosions in several shortest diagonal members (D1 in Figure 1) detected in the following inspections in 2009.

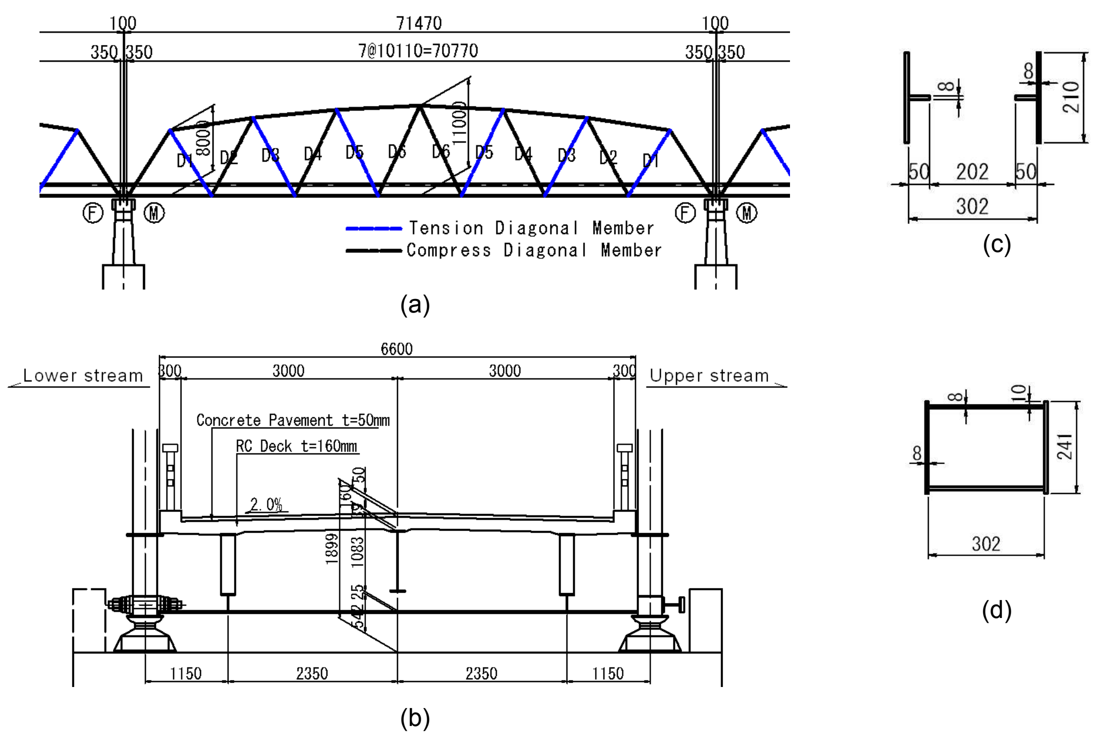


Figure 1 Steel truss bridge investigated: (a) side view of single span, (b) cross section of the bridge, (c) cross section of tension diagonal member, (d) cross section of compression diagonal member. Unit [mm].

2.2. Vibration measurements

Several sets of vibration measurements were made for different objectives. The objectives of those sets of measurements were to identify the effect of the partial fracture, cracks and pitting corrosions on the dynamic characteristics of the local dynamic characteristics of the diagonal members (Measurement 1) and the whole bridge (Measurement 2), and obtain better understanding of the dynamic characteristics of the bridge (Measurement 3), respectively.

Impact testing of the diagonal members was conducted in those with damage and without so as to identify the effect of damage on local vibration characteristics of the tension diagonal members (Measurement 1). Either a three-axis accelerometer unit consisting of three single-axis piezoelectric accelerometers or a set of four single-axis piezoelectric accelerometers was attached to the flange of tension diagonal members at the quarter point from the bottom. Impacts were applied by an impact hammer to the web of the diagonal member at the quarter point from the bottom. Additionally, ambient vibrations of a diagonal member were measured so as to understand the difference between the vibration modes of the diagonal member induced by impact testing and those induced by ambient vibration.

In order to identify the effect of damages on the dynamic characteristics of the bridge (Measurement 2), vibration of the bridge was measured at three locations: the lower chord members on both sides of the bridge and a longest tension diagonal member, D5 (Figure 2). The number of measurement locations was limited because the measurement needed to be completed during a short time period between the detection of the damages and the urgent reinforcement of the diagonal members. The measurement was conducted in the span with the local fracture found and, for comparison, in a span without damages. The acceleration of the lower chord member was measured in the vertical direction at the quarter point of the span. The accelerations in three orthogonal axes were measured in D5 at the quarter point from the bottom. Piezoresistive accelerometers were used in the measurement. Vibration of the bridge was induced by a dump truck with a total mass of about 200 kN running at different speeds between 20 and 40 km/h while the bridge was closed to other traffic.

Figure 3 shows the positions of transducers in Measurement 3 to obtain better understanding of the dynamic characteristics of the bridge. Four servo velocimeters were placed at different positions of a lower chord member and three piezoelectric accelerometers were attached at another lower chord member, as shown in Figure 3. The motion in the vertical direction was measured at all those locations. Additionally, the motions of five tension diagonal members were measured with piezoelectric accelerometers. At the quarter point from the bottom, two accelerometers were attached to the web to measure in-plane motion of the diagonal member and another accelerometer was attached to the flange to measure out-of-plane motion. A particular interest in this measurement was to understand more about the dynamic coupling between the diagonal members and the whole structure.

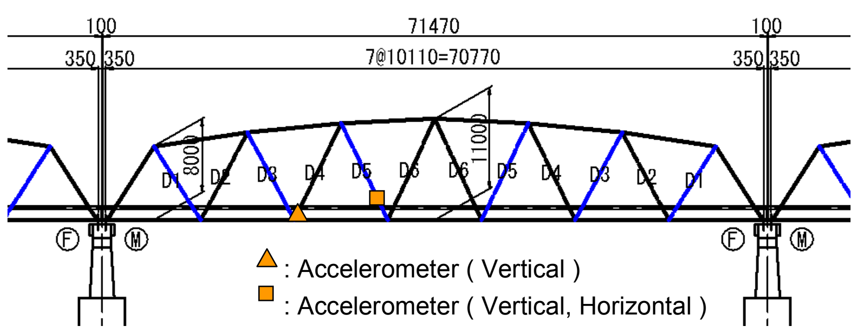


Figure 2 Positions of accelerometers in Measurement 2

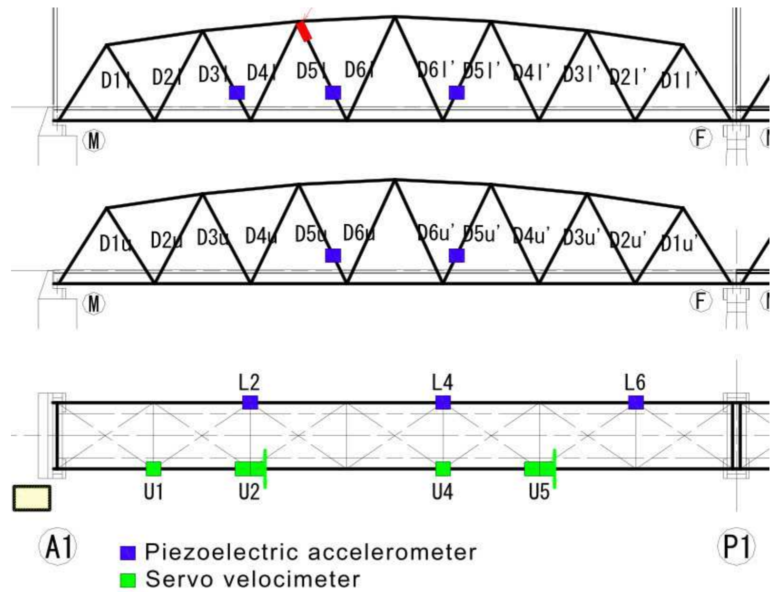


Figure 3 Positions of accelerometers in Measurement 3

3. NATURAL FREQUENCY CHANGE IN LOCAL MODES

Figure 4 shows an example of the results of impact testing that compares the Fourier spectra of the response to impact of a diagonal member D1 with pitting corrosions (referred to as corroded in the figure) and that of a D1 without corrosion (referred to as healthy). In the figure, the stabilization diagram obtained from the Eigensystem Realization Algorithm, ERA (Juang & Pappa, 1985) for the diagonal member without corrosion is compared with the corresponding Fourier spectrum. The Modal Amplitude Coherence, MAC, was used to identify reliable natural frequencies in the ERA. The comparison between the Fourier spectrum and the stabilization diagram from the ERA implies that the natural frequency identified from the measurement records in the impact testing were reliable.

It was observed that the natural frequencies observed at frequencies above 100 Hz were different between healthy and corroded diagonal members, while there were minor differences in the frequency range below 100 Hz (Figure 4). The natural frequencies of the diagonal member with pitting corrosions appeared to be lower than those of the diagonal member without corrosions. Similar trend were found with the comparison between this diagonal member with pitting corrosions and other healthy diagonal members with nominally the same dimensions. The decreases in the natural frequencies in the frequency range above 100 Hz for the corroded diagonal member may be associated with decreases in the modal stiffness of higher order local vibration modes that are attributed to the pitting corrosions.

Figure 5 compares the natural frequencies of the healthy diagonal member identified by impact testing and those identified from twenty different records of ambient vibration in the frequency range between 100 and 200 Hz where the effect of the pitting corrosions was observed. In the analysis of the ambient vibration records, the ERA was applied to free vibrations observed after vehicle pass-bys. The natural frequencies identified from ambient vibration records were a subset of the natural frequencies identified by impact testing. However, what natural frequencies were observed varied depending on the ambient records used in the analysis as observed in Figure 5. In practical applications, a continuous ambient vibration measurement combined with some statistical analysis can be used to identify all natural frequencies within a frequency range of interest.

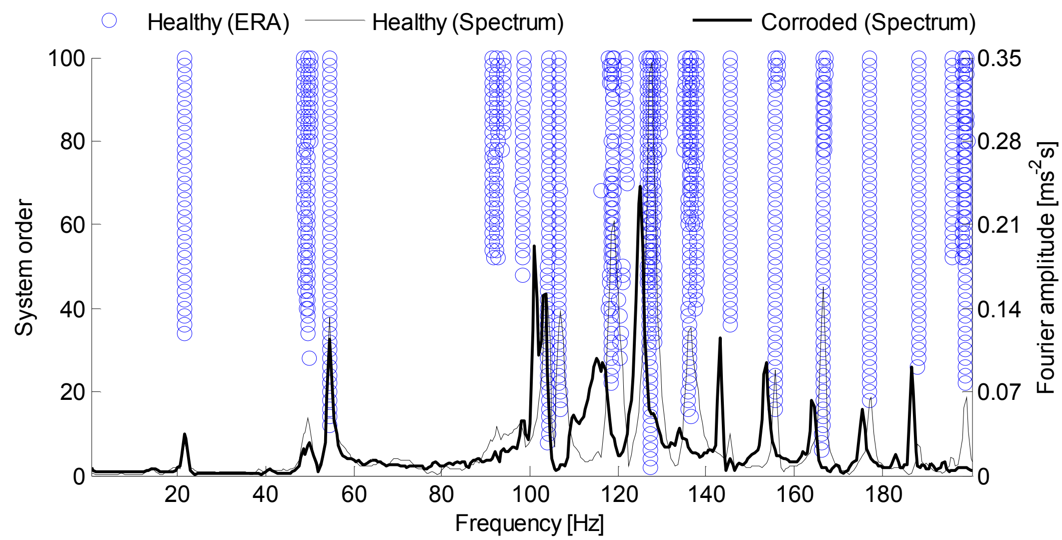


Figure 4 Fourier spectra of response of healthy and corroded diagonal members to impact. Stabilization diagram from ERA obtained for health diagonal member are also shown.

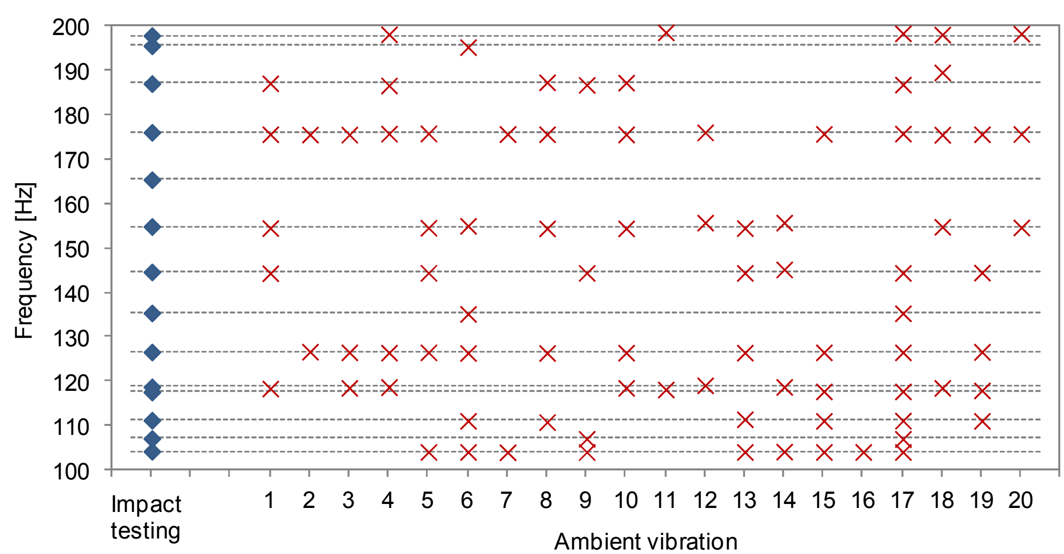


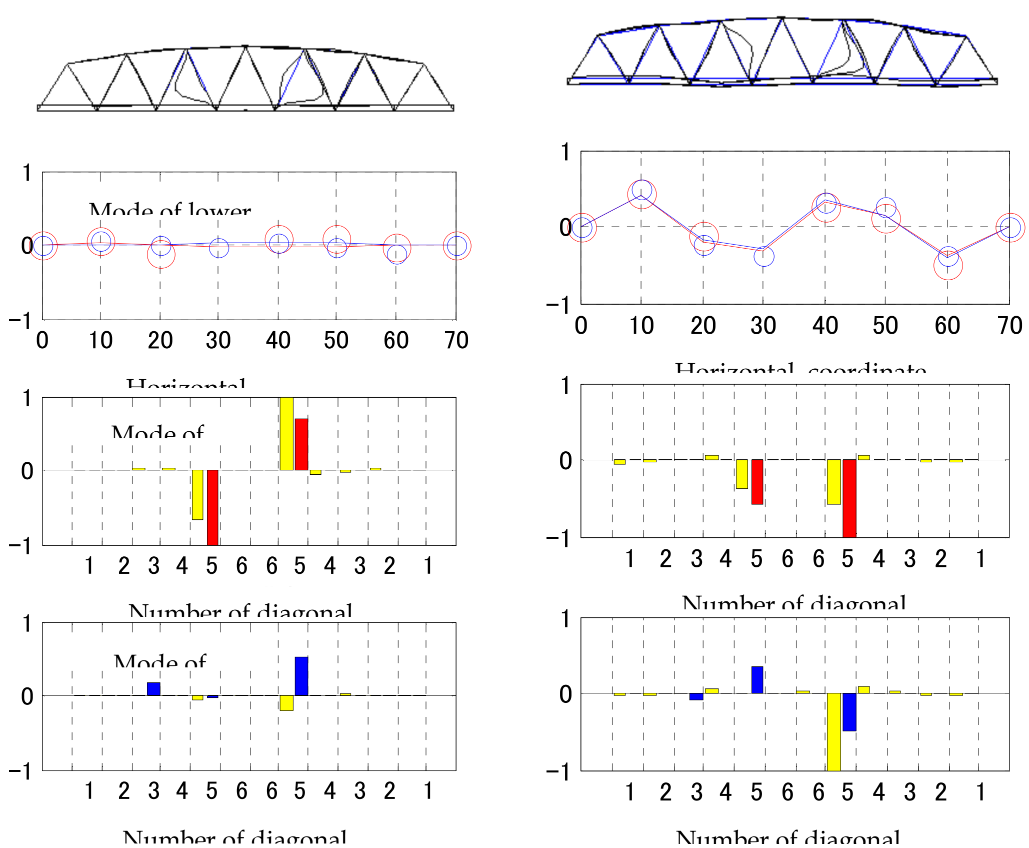
Figure 5 Natural frequencies identified by impact testing and 20 ambient vibration records

4. DAMPING CHANGE IN COUPLED MODE

It was understood that there were closely spaced vibration modes at frequencies around the natural frequencies of the diagonal members. This was partly because there were four diagonal members in a span that had nominally the same dimensions. Additionally, there were coupled vibration modes between the diagonal members and the whole structure. Figure 6 shows the mode shapes involving significant motion of the longest tension diagonals (D5). In the figure, the mode shapes identified from the field records measured in Measurement 3 by the ERA are compared with those obtained from theoretical modal analysis by finite element analysis. The figure shows a vibration mode dominated by the motion of the diagonal members (referred to as a local mode in this paper) and a mode involving the motions of the diagonal members and lower chord members (referred to as a coupled mode).
Figure 7 shows the changes in the modal frequencies and damping ratios before and after the

reinforcement of the diagonal members identified in Measurement 2. The modal properties shown in the figure were obtained by the ERA. The figure shows that, in the local vibration mode dominated by the motion of the diagonal member damaged and repaired, the modal frequency increased from 7.1 Hz to 9.8 Hz, approximately, and the modal damping ratio decreased from 0.0055 to 0.0039 after the reinforcement. In the global vibration modes involving the motion of the whole structure, there appeared to be changes in the modal damping ratio with minor changes in the modal frequency. It was noted that there was more variability in the identification of the modal damping ratio from the measurement records in the lowest order vibration modes, such as the mode at about 2.6 Hz in the figure, although the data are not presented in this paper. This variation in the damping was considered to be caused by the friction damping at the bearing supports that was dependent heavily on the displacement amplitude of vibration. In the global mode at about 7.3 Hz, however, there was less variability in the identification of modal damping ratio and the change in the modal damping shown in Figure 7 was more reliable than the changes in the lowest order vibration modes.

Figure 8 shows the relation between the changes in modal damping ratio found in the vibration mode at 7.26 Hz, as shown in Figure 7, and the dynamic coupling between the diagonal and lower chord members. The dynamic coupling between the diagonal and lower chord members are represented by the ratio of the modal amplitude of the diagonal member to the modal amplitude of the lower chord member in the figure. The modal amplitude of the diagonal member was obtained by subtracting the modal amplitude at the end of the diagonal member from that identified for the measurement location in the diagonal member.



(a) (b)

Figure 6 Modal amplitudes corresponding to (a) local mode at 9.264 Hz and (b) coupled mode at 9.325 Hz. The vi-bration modes identified experimentally are compared with those obtained theoretically.

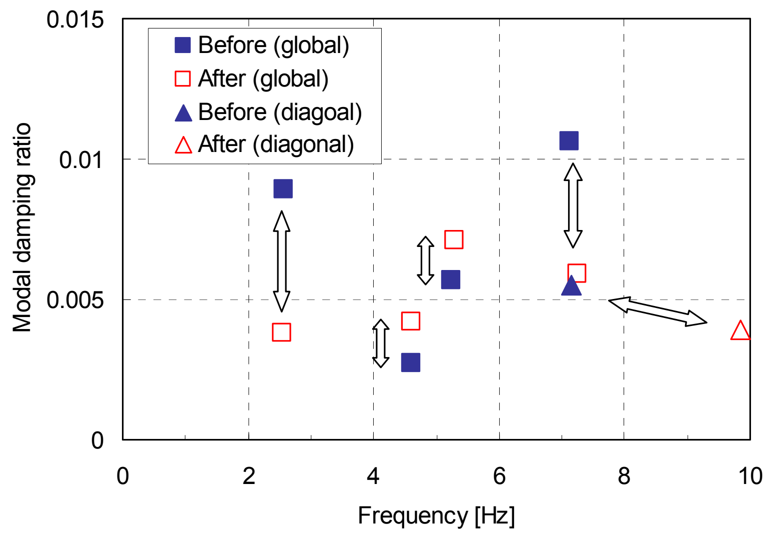
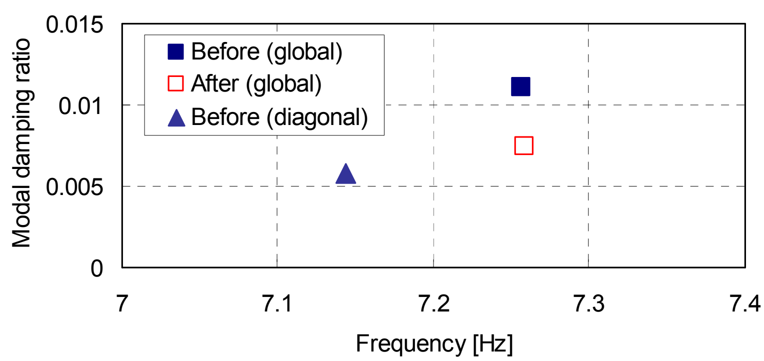
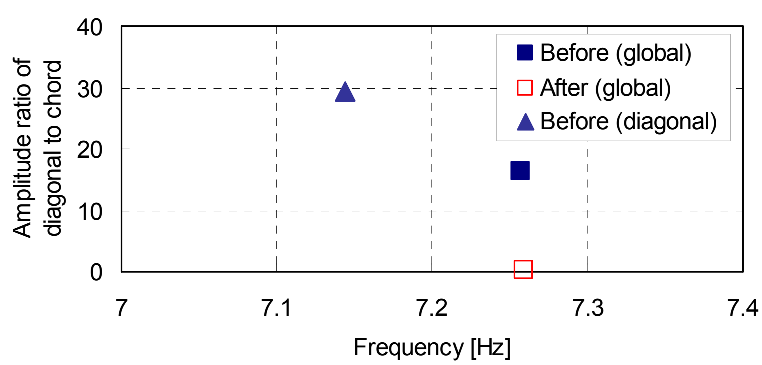


Figure 7 Examples of changes in modal frequencies and damping ratios before and after reinforcement of diagonal members.



(a)



(b)

Figure 8 Relation between changes in modal damping ratio and dynamic coupling between diagonal and lower chord members. (a) modal damping ratios and (b) amplitude ratios of diagonal member to lower chord member in local and global vibration modes before and after reinforcement.

The vibration mode at 7.14 Hz shown in Figure 8 was the local mode dominated by the motion of the damaged tension diagonal member D5 as indicated by a high amplitude ratio of the diagonal member to the lower chord member (i.e., about 30). As shown in Figure 7, the modal frequency of the local mode increased from 7.14 Hz to 9.80 Hz after the reinforcement of the diagonal member: the modal properties of the mode after the reinforcement are not shown in Figure 8.

The decrease in the amplitude ratio of the diagonal member D5 to the lower chord member from about 16 to 0 in the global mode at 7.26 Hz, as shown in Figure 8, implies that there was no dynamic coupling between the diagonal member D5 and the lower chord member after the reinforcement due to the increase in the modal frequency of the local mode of D5. The decrease in the modal damping ratio of the global mode at 7.26 Hz may be associated with the loss of coupling between the diagonal and lower chord members that was caused by the change in the mechanical property of the diagonal member. This finding suggests that the modal damping ratios of global modes may be used as an indicator of local damages in diagonal members in steel truss bridges.

5. CONCLUSIONS

The results presented in this paper shows a possibility of the identification of local damages in steel truss bridges, such as damages in diagonal members, from changes in the modal properties of the structure obtained from vibration measurements. The natural frequencies of higher order local vibration modes of diagonal members can be a direct indicator of damages, although a practical implementation of the identification of natural frequencies of a number of diagonal members may need further development. A possible solution may be applying impact testing only on diagonal members that are identified as critical members in redundancy analysis. Ambient vibration may be able to be used instead of impact testing to identify natural frequencies of diagonal members. The identification of changes in the damping of global mode may be more practical in terms of the feasibility of measurement, although there is a need to improve the reliability of the identification of modal damping. The development of quantitative relation between changes in modal properties and damages requires further investigations.

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Section II- Solutions for Challenges in Infrastructure Planning and Development

Application of Remote Sensing and GIS techniques for exploring Construction Material from inaccessible terrains; case study in Eastern Province of Sri Lanka

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Abstract: *Demand for construction materials in the North and Eastern provinces of Sri Lanka has risen significantly due to accelerated construction work in progress to restore the destroyed infrastructures during the civil war. Ground surveying methods to locate quarries within this area are not applicable due to security reasons and the larger area to be covered. Thus a Remote Sensing method is suitable for this task. This study introduces a methodology to locate suitable quarries and sand mines for constructions in the Eastern province. LANDSAT ETM+ images were used for classification. Feasible rocks and sand deposits for quarry operations were located using Multicriteria Analysis (AHP).The highly suitable rock quarries and sand deposits were considered in closest facility analysis. The results showed that the integration of Remote Sensing (RS) and Geographic Information System (GIS) can serve as an effective tool in demarcating suitable sites for construction material and this methodology can also be applied for projects of similar nature.*

Keywords: *Closest facility analysis, MCDA, Sand deposits, rock quarries.*

1. INTRODUCTION

The twenty years long civil war has resulted in impoverishment and under-development, especially in the areas of Northern and Eastern provinces of Sri Lanka. Most of the infrastructures were severely disrupted by bombs, barbed wire, blockades and land mines. It is estimated that the 290,615 houses were destroyed during the civil war in Northern and Eastern provinces of Sri Lanka. About 58% of the total housing stock in the North and East and around 90% of the houses belonging to the displaced population were destroyed as the result of the war (World Bank 2008).

Presently, it has become a challenge to rebuild those areas. The demand for construction materials have also gone higher as a result of the major development and construction projects.There are plenty of resources in those areas to cater to the demands, but most of them are unknown. Hence exploration programs needed to be carried out to find suitable places to obtain construction materials. There are several methods available for this task. Many of them are related to direct field surveys and observations. Since the field access and observations are obstructed by land-mines lying beneath those areas and the removal process will be extended over many years, Remote Sensing (RS) and Geographic Information System (GIS) techniques can be used for this task (Sabins, 1998).

Therefore, the main objective of this project is to use satellite images for identification of the available rock and sand deposits for construction work and also use of GIS analysis for demarcation of economically viable construction material occurrences in the area considering all the factors such as construction sites, road networks and environmental factors.

This method could be developed as a technology to be applied in similar situations that will save our time, money and tireless efforts towards the old conventional methods of locating quarries.

2. MATERIALS AND METHODS

2.1. Methodology

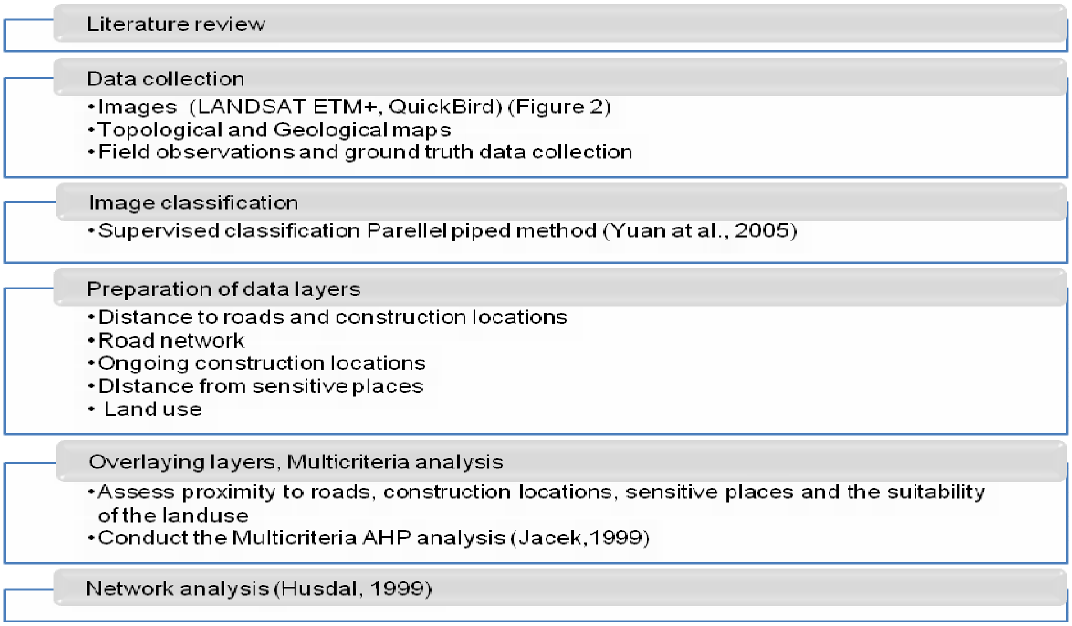

Figure 1 Methodology

2.2. Images Used

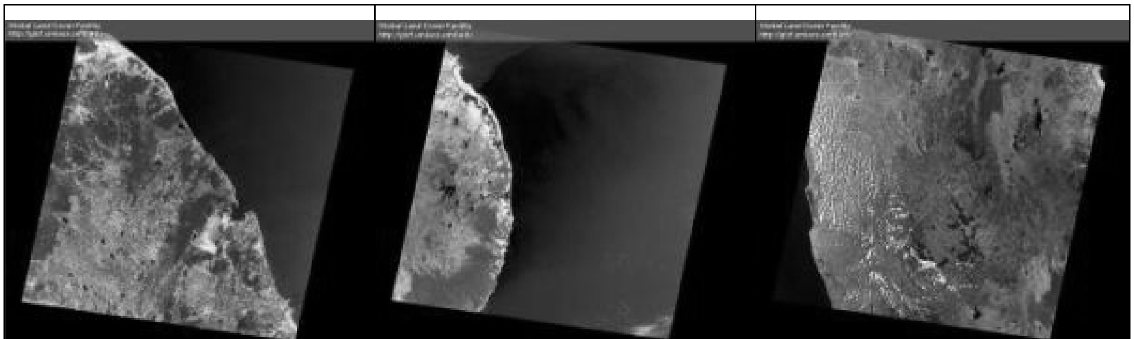

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Figure 2 Landsat ETM+ images (Global Land Cover Facility)

3. RESULTS AND DISCUSSION

In this study, highly exposed rocks and sand areas were accurately classified, but the classification accuracy (1) of rocks is greater than the sand (Table 1).Sand deposits along the flood plain of the Mahaweli River, sandy soil, beach and lagoons were classified as sand abandoned areas. Examples of comparison of the rock and sand classification results with Google earth were given in Figure 3 and 4.

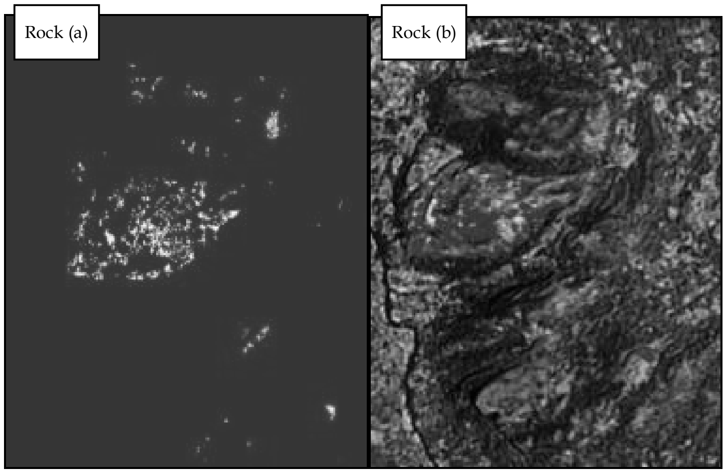

Figure 3 Classification for rocks using Landsat ETM+ (a) and Classification overlay on Google Earth (b); yellow, green & red rock classification overlay on Google Earth in red color

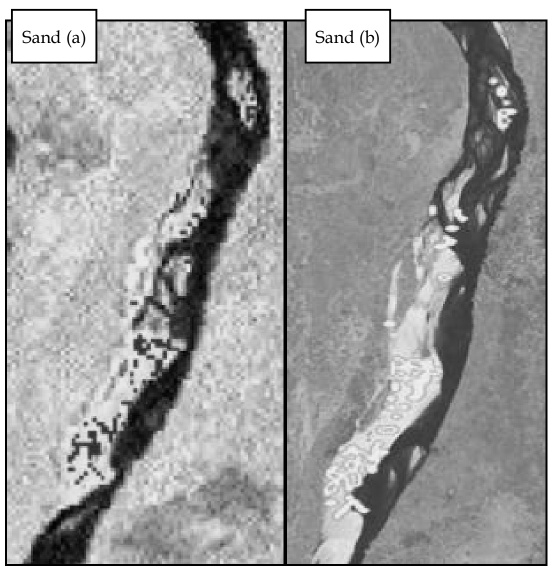

Figure 4 Classification for sand using Landsat ETM+ (a) and Classification overlay on Google Earth (b); red sand classification overlay on Google Earth in yellow color

$$\textit{Accuracy} = \frac{\textit{No of classified rocks/sand}}{\textit{Total No of rocks/sand}} \times 100\% \qquad (1)$$

Table 1 Accuracy of classification

Construction Material	Accuracy of Classification
Rocks	91.47%
Sand	85.21%

The classification gave a few erroneous results; mostly the roofs thatched using “Asbestos” and some flat bare lands were wrongly classified as rocks and sand respectively. The collected ground truth data (Table 2) showed that rocks were not classified where the vegetation cover and overburden is thick and rock slopes are steep. Also, where vegetation is less but scattered all over the rock top minimizing the visible area for one pixel has excluded that area from the classification. In the classification of sand, the major obstacle encountered was the classification of sandy soils which are not suitable for sand mining. Locating only the possible sites for rocks and sand is not sufficient as they may not be economically viable to put up a quarry.

Table 2 Ground truth data collection – Rocks and Sand

<table>
<tr><th>Location</th><th>classify</th><th colspan="3">Description</th></tr>
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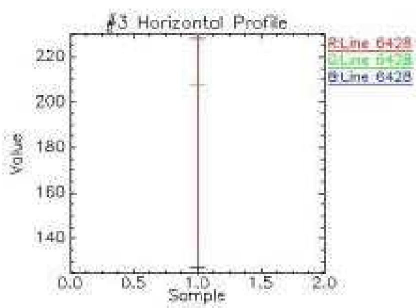
Rock</td><td rowspan="2">yes</td><td colspan="2">

Pixel values (7,5,4 in Red, Green and Blue channels)</td><td></td></tr>
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Area: 100 x 50± 25m

GPS Points:
7.33792 N
81.019567 E</td><td>Mineral Present:
Mafic 50% (black)
Felsic (50%)
charnockite gneiss
Vegetation cover: <10%</td><td>Remarks:
Accesible
Mining Feasible
No quarry in operation (people have shown interest though)
State owned
No apparent problems</td></tr>
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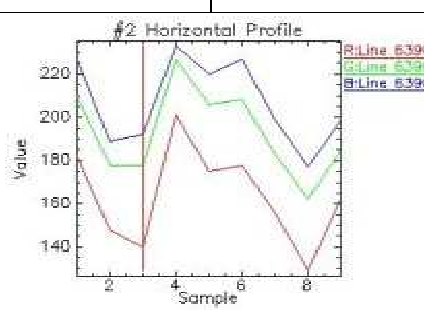
Rock</td><td rowspan="2">no</td><td colspan="2">

Pixel values (7,5,4 in Red, Green and Blue channels)</td><td></td></tr>
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Area: 50 x 400 m²

GPS Points: 7.3382 N
81.01957 E</td><td>Mineral Present:
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Mafic <20%

Vegetation cover:
40 – 50%</td><td>Remarks:
Quarry in operation</td></tr>
</table>

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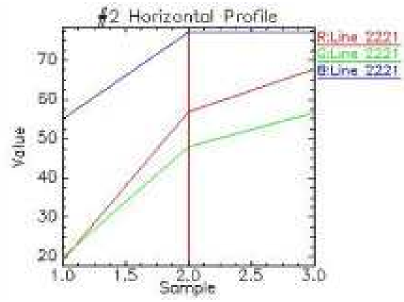
Sand</td><td rowspan="2">yes</td><td colspan="2">
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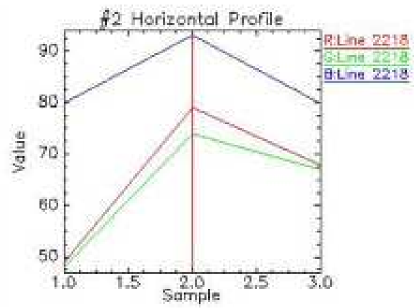
Sand</td><td rowspan="2">yes</td><td colspan="2">
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Sand mining in Operation</td></tr>
</table>

Calculated weights for Individual criterion using Multicriteria AHP techniques were shown in Table 3, the following criteria were ranked;

- The rocky land should be easily accessed,
- Must be located close to the construction sites,
- Should be easy to acquire for quarry operation and
- The quarry should be located considerable distance away from sensitive locations.

Table 3 Calculated weights for Individual criterion

Criteria	Weight
Proximity to roads	0.4929
Proximity to construction locations	0.3082
Land use	0.1056
Proximity to sensitive locations	0.0936

The overall land suitability (2) should address all the above needs (Figure 5). Economically viable Quarries and sand mines should be categorized based on that land suitability.

*Land suitability = [DR] * 0.4929 + [DC] * 0.3082 + [LU] * 0.1056 + [DI] * 0.0936* (2)

where [DR] is the Proximity to roads, [DC] is Proximity to construction locations, [LU] is the Land use and [DI] is Proximity to sensitive locations

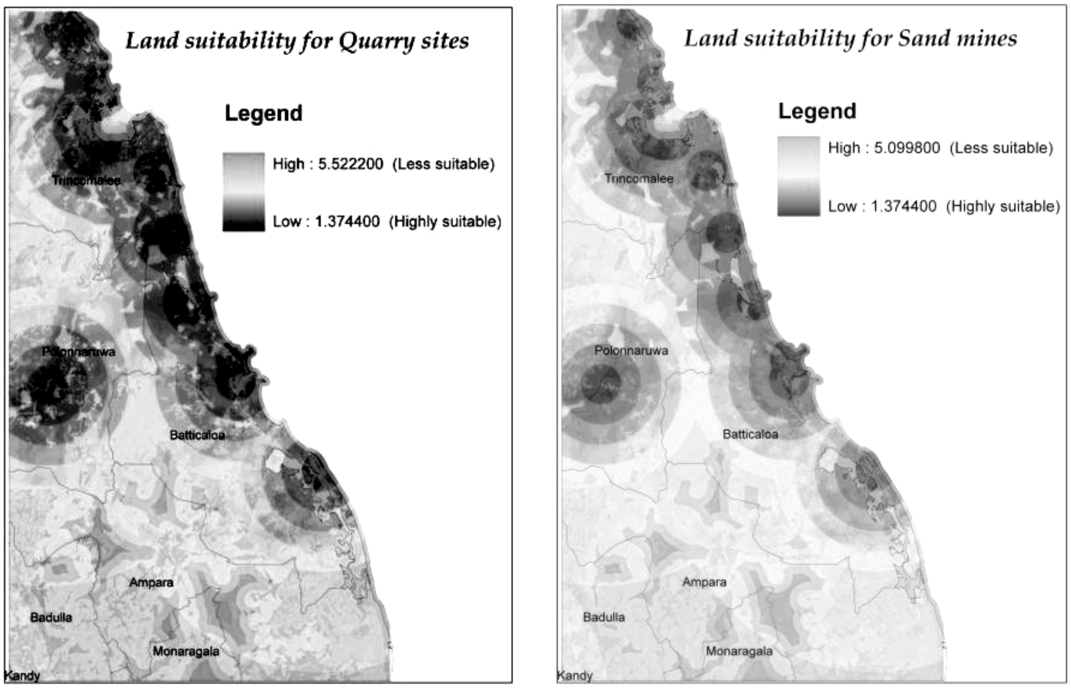

Figure 5 Land suitability for Quarry sites and Sand mines

Under the prevailing categorization of the suitability of rocks and sand deposits for quarry operations, highly suitable category contains the most preferable places to locate quarries and sand mines (Figure 6). The criteria and ranking in the decision making process can vary depending on the type of data input and the objectives of the stakeholders.

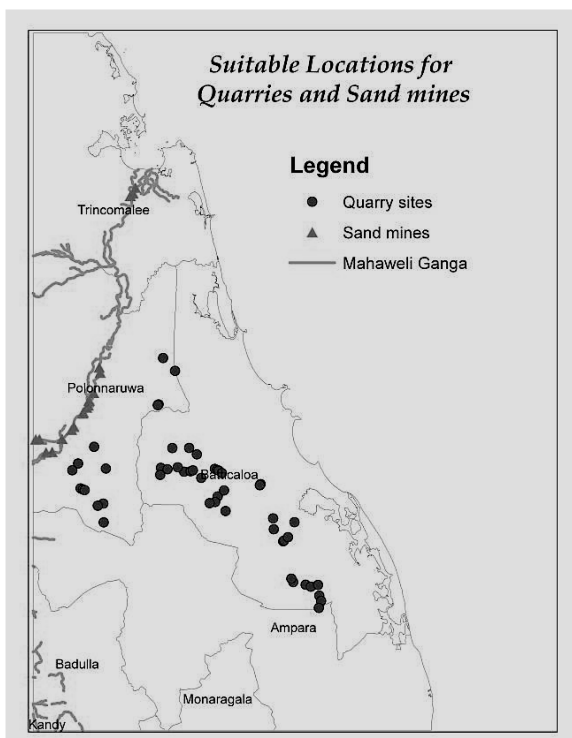

Figure 6 Suitable Locations for Quarries and Sand mines

Finally, the network analysis was used to obtain the best quarry sites and sand mines with least cost (distance, time) manner to reach construction sites. That is the best path in which a particular quarry site can transport its construction material to a construction site with minimum cost.

The accessibility to roads and construction locations were assessed based on the distance from them to the quarry sites. The cost of taking the relevant path way was not taken in to account. Any network analysis requires impedances (speed) associated with each road in the network, by which priorities are given to the different roads for analysis. The vehicle speeds were assigned to individual road types that are main roads and jeep or car tracks (Vinod and Sukumar, 2003). The time taken for the turns was also not considered.

The shortest path between quarries and the customers will facilitate the truck drivers to deliver construction materials to maximum number of customers in an efficient manner. This helps a quarry to have an idea about how many customers are within a particular distance and the closest cities from the quarry to carry out selling of construction materials.

Shortest paths between quarry sites and construction sites are shown in Figure7.

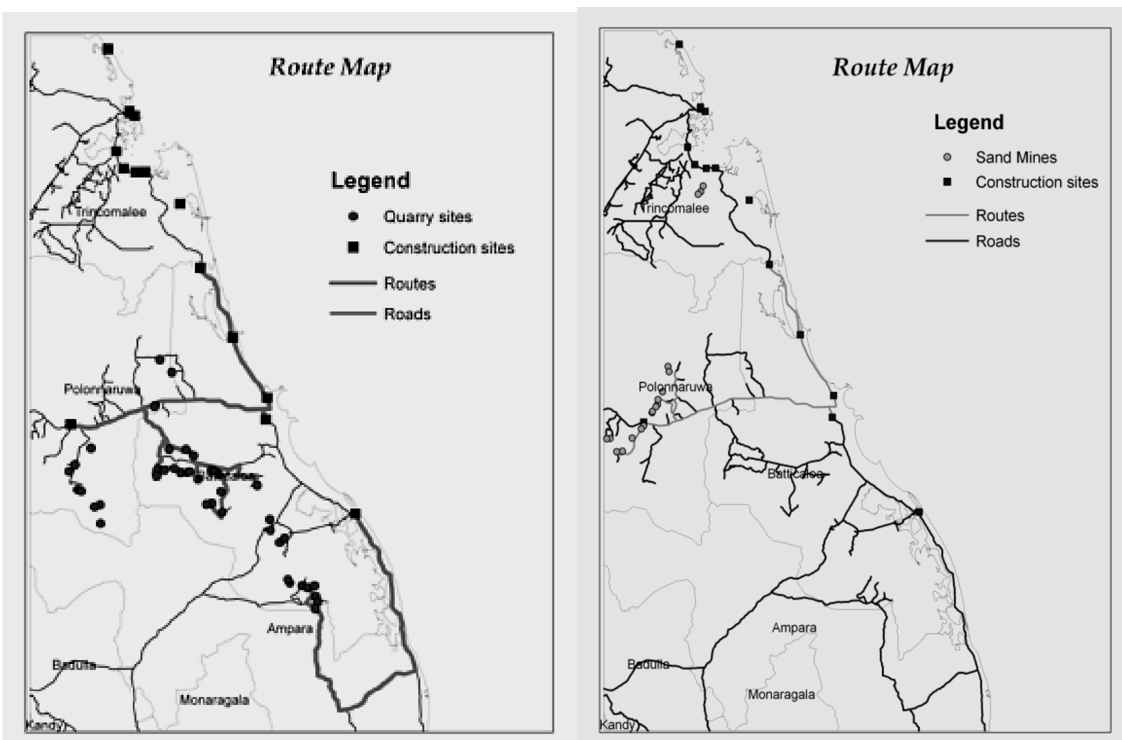

Figure 7 Route maps for Quarry sites and Sand mines

4. CONCLUSIONS

The accuracy of the rocks and sand classification is satisfactory enough to proceed with the remaining analysis. Therefore, 15m upgraded resolution LANDSAT ETM+ images are suitable for rocks and sand identification. Rocks tend to classify accurately if the exposure is high with a less overburden and low vegetation cover.

The highly suitable quarries indicate that area is within the favorable limit of the criteria defined according to the study objectives.

Final results after network analysis gives feasible quarries based on the route suitability selected to reach the construction sites in a minimum time.

The extent of the archeological sites cannot be identified by satellite images and need to be integrated to the GIS database to optimize the final result. For a more comprehensive result mineralogy and the properties of rocks can be incorporated to the multicriteria analysis.

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Gas and Heat Transport in Variably-Compacted Landfill Cover at Variably-Saturated Condition

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Abstract: *Understandings of gas and heat transport in the landfill covers are essential for enhancing the landfill site stabilization and reducing the greenhouse and toxic gas emissions. Gas diffusion and thermal conduction are main mechanisms for gas and heat transport in soils. Gas diffusion coefficient and thermal conductivity govern gas diffusion and thermal conduction, respectively. In this study, we developed a unified predictive model for gas diffusion coefficient and thermal conductivity considering soil compaction level. Numerical simulations of gas (methane, carbon dioxide, and oxygen) and heat transport in a landfill cover were performed using the developed predictive model. Increase of compaction level enhanced not only heat transport in the landfill cover but also methane gas emissions due to reduced methane gas oxidation nearby soil surface.*

Keywords: *Landfill cover soil, gas diffusivity, thermal conductivity, numerical simulation.*

1. INTRODUCTION

Landfill sites are a significant source of methane (CH_4) which has a high global warming potential, estimated to be more than 20 times that of carbon dioxide. The estimated CH_4 emissions from landfills are 550-635 Mt CO_2-eq. year^{-1} corresponding to about 9% of global anthropogenic methane emissions (Rogner et al., 2007; Bogner et al., 2008). In addition, the emissions of toxic gases such as a hydrogen sulfide and volatile organic chemicals from landfill sites affect surrounding local environments. The exothermal reactions also occur due to the microbiological processes in the waste layer. These gases and heat produced in the waste layer move through landfill covers and emit to the atmosphere. Therefore, the understandings of gas and heat transport in the landfill covers are essential for enhancing the landfill site stabilization and reducing the greenhouse and toxic gas emissions. Gas diffusion and thermal conduction are main mechanisms for gas and heat transport in soils. Gas diffusion coefficient (D_p) and thermal conductivity (λ) govern gas diffusion and thermal conduction, respectively.
Gas diffusion coefficient is controlled by air-filled networks, while thermal conductivity is affected by both solid phase configuration and water-filled pore networks. Since the bulk soil-pore structure is composed of the three phase (air, water, and solid) geometries, gas and heat transport characteristics at different moisture conditions are expected to be interrelated. Such a relation enables to develop a unified predictive model for gas and transport parameters which are promising for simulating simultaneous gas and heat transport in the landfill covers.
In this study, we developed a unified predictive model for gas diffusion coefficient and thermal conductivity considering soil compaction level (i.e., dry bulk density). Numerical simulations of gas (methane, carbon dioxide, and oxygen) and heat transport in a landfill cover were performed using the developed predictive model. The effects of soil compaction level and thickness of the landfill cover on gas and heat transport were investigated.

2. MATERIAL AND METHODS

2.1. Soil Samples and Gas Diffusivity Measurements

A waste landfill site in Saitama Prefecture, Japan, was selected as a sampling location. The final cover soil (size fraction less than 2-mm) was used in this study. The soil texture was a sandy loam. Compaction tests were performed for soil samples at different water content. In the compaction tests, the soil samples were repacked into large soil cores (i.d. 15-cm, length 12-cm) at two different compaction levels (high: 2700 kJ m^{-3} and low: 600 kJ m^{-3}).
After compaction tests, 100-cm^3 core samples were taken inside each repacked large core. The core samples were classified into two different ρ_b ranges (1.80-1.90 g cm^{-3}, labelled as extreme compaction (EC), and 1.70-1.80 g cm^{-3}, labelled as high compaction (HC)). After the core samples were water-saturated, they were drained at different matric suctions and the gas diffusion coefficient (D_p) was measured. For comparison, disturbed soil samples at different water contents were repacked into 100-cm^3 cores at dry density of 1.55 g cm^{-3}, representing normal compacted soils (labelled as normal compaction, NC), and the D_p was measured on the repacked soil samples at different soil-air contents.
The D_p was measured on the repacked 100-cm^3 soil cores with a diffusion chamber method (Rolston and Moldrup, 2002). Oxygen was used as tracer gas and measured as a function of time in the diffusion chamber. The D_p was calculated according to Osozawa (1998). In this study, the gas diffusion coefficient of oxygen in free air (D_0) at 20 °C was taken as 0.20 x 10^{-4} (m^2 s^{-1}).

3. DEVELOPMETNS OF UNIFIED PREDICTIVE MODEL FOR GAS DIFFUSIVITY AND THERMAL CONDUCTIVITY CONSIDERING COMPACTION LEVEL

3.1. Gas Diffusivity (Dp) Measurements and Development of Compaction-Dependent Predictive Dp Model

Figure 1a shows the measured gas diffusivity. At the same soil-air content (ε, m^3 m^{-3}), higher D_p for soils at higher ρ_b were observed. It is because the soil samples at higher ρ_b have lower volumetric water content as compared to those at lower ρ_b. This effect is more significant for soils under wet conditions (i.e., lower ε). The following power-law function was fitted against the measured data.

$$\frac{D_p}{D_0} = \alpha_p \varepsilon^{X_p} \qquad (1)$$

where α_p, X_p are fitting parameters to represent air-filled pore connectivity. Figure 1b shows the fitted α_p and X_p values as a function of ρ_b. Both α_p and X_p linearly decreased with increasing ρ_b, suggesting that larger pore-networks for loosely-compacted soils more dramatica

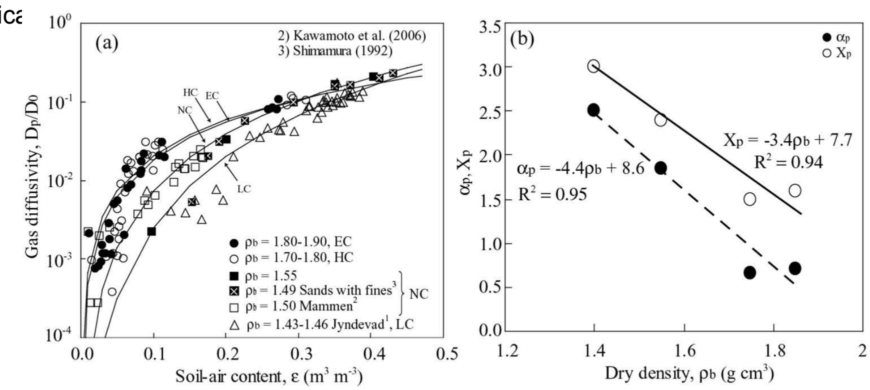

Figure 1 (a) Gas diffusivity as a function of soil-air content, (b) fitted □$_p$, X_p as a function of dry bulk density. Redrawn from Hamamoto et al. (2011).

3.2. Unified Predictive Model for Gas Diffusivity and Thermal Conductivity

In addition to the measured D_p data for landfill cover soils, D_p and thermal conductivity data (λ, W m^{-1} K^{-1}) were collected from literature (e.g., Lu et al., 2007). Figure 2 shows transport parameter (P: D_p or λ) normalized by P at fluid saturation (D_p: air saturation, λ: water saturation), P_{sat}, as a function of fluid saturation (ϕ/ϕ_{sat}). ϕ is the fluid content (m^3 m^{-3}) and ϕ_{sat} is the total porosity (m^3 m^{-3}). The ϕ for D_p and λ represents soil-air content (ε) and volumetric water content (θ), respectively. As shown in Fig. 2, the D_p rapidly increased at higher fluid saturation since at dry condition (higher ϕ/ϕ_{sat}), gas diffusion is enhanced due to well-connected larger pore-networks. On the other hand, more marked increase in the λ was observed under lower fluid saturation. Since thermal conduction process is mainly governed by thermal conduction through solid phase, at dry condition, water bridges begin to form between soil particles, and the λ starts to increase rapidly because of the improved thermal contact between particles.

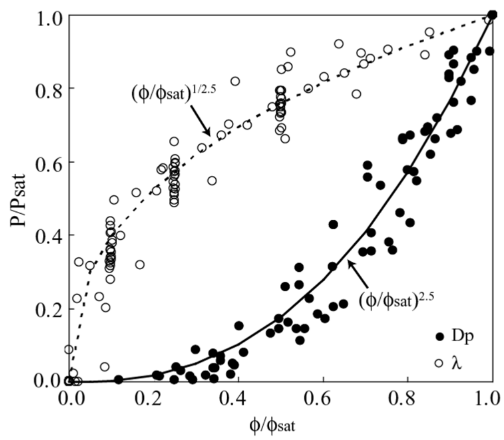

Figure 2 Normalized transport parameter as a function of fluid saturation. Redrawn from Hamamoto et al. (2010).

As shown in solid and dotted lines in Fig. 2, the normalized D_p and λ values could be well expressed by $(\phi/\phi_{sat})^{2.5}$ and $(\phi/\phi_{sat})^{1/2.5}$, respectively. Thus, the clear mirror image was obtained for D_p and λ behaviours as a function of fluid content. By combining the obtained relation between D_p and λ, and predictive D_p model as a function of ρ_b (Fig. 2), the following unified predictive model for D_p and λ considering compaction level (ρ_b) can be derived.

$$\frac{P}{P_{sat}} = \left(\frac{\phi}{\phi_{sat}}\right)^{X_p} \tag{2}$$

where the X_p in Eq. (2) can be expressed as a function of ρ_b shown in Fig. 1b, and for λ, inverse of X_p ($1/X_p$) instead of X_p. In this study, P_{sat} values for D_p and λ were estimated by Eq. (1) and predictive model from literature (Lu et al., 2007), respectively.

4. NUMERICAL SIMULATIONS ON GAS AND HEAT TRANSPORT IN LANDFILL COVER

4.1. Governing Equations for Gas and Heat Transport

In this study, CH_4, CO_2, and O_2 movements in the landfill cover were simulated. By assuming Fickian diffusion process and methane oxidation as main gas transport mechanisms, the governing equation for gas transport for each gas species can be expressed as,

$$\varepsilon\frac{\partial C_i}{t} = D_p\frac{\partial^2 C_i}{\partial z^2} - \chi_i R_{CH4} \tag{3}$$

where C_i is the gas concentration (mol m^{-3}) of species i (i.e,. i = CH_4, CO_2, or O_2), R_{CH4} is the methane oxidation rate (mol m^{-3} s^{-1}), and x_i is the stoichiometric factors (χ_{CH4} = 1.0, χ_{02} = 1.5, χ_{CO2} = -0.5), t is the time (s), z is the length (m). Following Michaelis-Menten equation, the R_{CH4} can be expressed as,

$$R_{CH4} = \rho_b V_{\max} / \left\{ \left(1 + \frac{K_{m,CH4}}{C_{CH4}}\right)\left(1 + \frac{K_{m,O2}}{C_{O2}}\right) \right\} \qquad (4)$$

where V_{max} is the maximum methane oxidation rate (mol g^{-1} s^{-1}), $K_{m,CH4}$ and $K_{m,O2}$ are the half-saturation constants of CH_4 and O_2 (mol m^{-3}), respectively. In this study, V_{max} = 750x10^{-12}、$K_{m,CH4}$ = 0.29、$K_{m,O2}$ = 0.49 were used based on de Visscher and van Cleepmut (2003).
When thermal conduction process is considered as a heat transport mechanism, the governing equation for heat transport can be expressed as,

$$C_v \frac{\partial T}{\partial t} = \lambda \frac{\partial^2 T}{\partial z^2} \qquad (5)$$

where C_v is the volumetric heat capacity (J m^{-3}, K^{-1}) and T is the soil temperature (K). The C_v value can be estimated from specific heat and volumetric ratio of each solid, liquid, and air phase. The temperature dependency on D_p is considered as $D_{0,T(K)}/D_{0,293(K)} = (T/293)^{1.67}$ ($D_{0,T(K)}$, m^2 s^{-1}: gas diffusion coefficient in free air at T (K), $D_{0,293(K)}$, m^2 s^{-1}: gas diffusion coefficient in free air at 293 (K)).

4.2. Model Domain and Parameters for Numerical Simulations

Figure 3 shows the model domain for the numerical simulations. The thickness of the landfill final cover was set as 1 m. For CH_4 movement, a constant flux boundary of 1.2x10^{-5} (mol m^{-2} s^{-1}) was applied at the bottom boundary based on the field measurements of methane emission flux at the landfill site where the soil samples were taken in this study. In addition, to express the exothermal reactions in the waste layer, the heat rate, P(t) (W m^{-2}), was applied as P(t) = 200exp(-t/10^6) based on Klein et al. (2003).

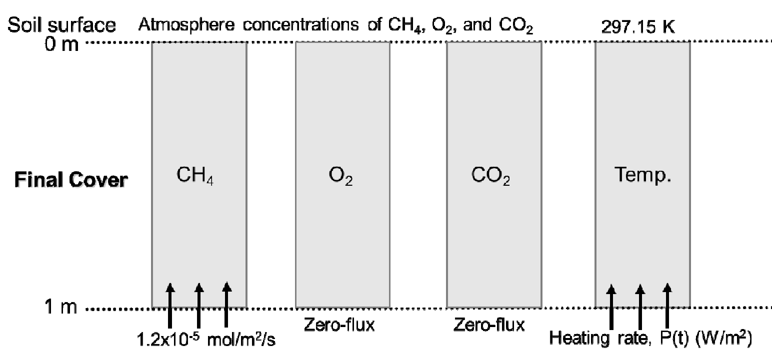

Figure 3 Model domain and boundary conditions for numerical simulations.

Numerical simulations were performed against two model cases: landfill cover soils at extreme compaction (ρ_b = 1.85) and loose compaction (ρ_b = 1.44) levels. The moisture condition for each model case was assumed as a field moisture condition represented by θ at soil-water matric potential of -100 cm H_2O. The total porosity, soil-air content, volumetric water content at field moisture condition for each model case were obtained by separate measurements using repacked soil cores in a laboratory. In addition, based on the obtained soil physical properties, volumetric heat capacity (C_v) and D_p and λ were estimated by Eqs. (1) and (2), respectively. Table 1 shows the parameter values used in the numerical simulations. The COMSOL Multiphysics Ver. 3.5a was used for solving gas and heat transport in the landfill cover soils.

Table 1 Parameter values used in the numerical simulations.

Bulk density	Total porosity	Soil-air content	Soil-water content	Soil heat capacity	Gas diffusivity	Thermal conductivity
ρ_b	ϕ_{sat}	ε	θ	C_v	D_p/D_0	λ
g/cm^3	m^3/m^3	m^3/m^3	m^3/m^3	$J/m^3/K$		$W\ m^{-1}\ K^{-1}$
1.44	0.46	0.363	0.10	2.27E+06	1.35E-02	1.46
1.85	0.30	0.041	0.26	3.51E+06	5.91E-04	3.24

4.3. Numerical Simulation Results

Figure 4a shows changes of soil temperature with time in the landfill cover soils at ρ_b = 1.85 and 1.44. For the extremely-compacted cover soil, soil temperature increased up to 60 ^oC after 5 days at bottom boundary (at the interface between waste layer and cover soil), while for loosely-compacted cover soil, increased up to 90 ^oC. Furthermore, soil temperature reach equilibrium with atmospheric temperature after 60 days for the extremely-compacted cover soil but longer time period was needed for the loosely-compacted cover soil. The finding suggests that the extremely-compacted cover soil has higher heat exchange ability since the extreme soil compaction increases contact number of soil particles, giving higher λ value (Table 1). Figure 4b shows changes of methane concentration profile with time in the landfill cover soils at ρ_b = 1.85 and 1.44. After 120 days, extremely-compacted cover soil exhibited around 90% of CH_4 at the bottom boundary, while CH_4 concentration rapidly decreased nearby soil surface due to a methane oxidation effect. The CH_4 concentration for the loosely-compacted cover soil increased up to only 30% and more marked effect of methane oxidation was observed. Since at field moisture condition, the extremely-compacted cover soil has higher water retention, soil-air content is lower as compared to loosely-compacted cover soil, giving lower D_p (Table 1). Hence, lower gas diffusion characteristics for extremely-compacted soil caused higher CH_4 concentration and lower methane oxidation effect as compared to the loosely-compacted soil.

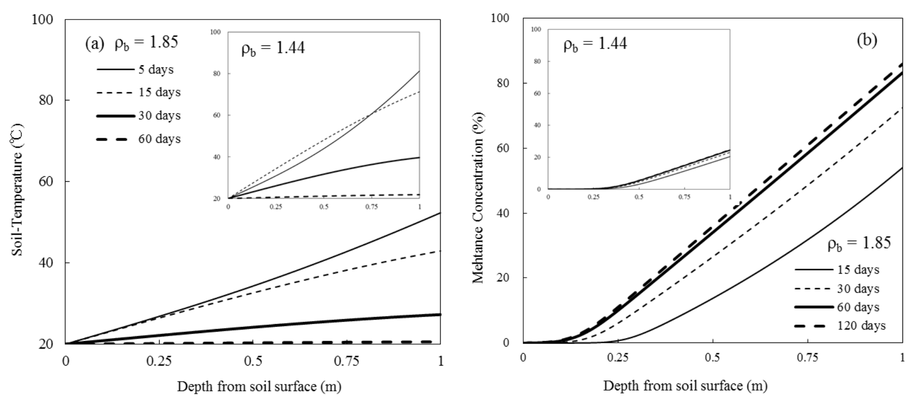

Figure 4 (a) Soil temperature profile and (b) methane concentration in the landfill cover soils at two different compaction levels.

CH_4 and CO_2 emission fluxes to the atmosphere were calculated for cover soils with different thickness and compaction levels shown in Fig. 5. The CO_2 emission flux was higher as compared to the CH_4 emission flux due to methane oxidation near soil surface. In addition, with increasing thickness of the cover soil, CO_2 and CH_4 emission fluxes increased and decreased, respectively. Higher methane oxidation ability for the loosely-compacted soil (Fig. 4b) caused higher CO_2 and lower CH_4 emission fluxes as compared to those for extremely-compacted cover soil. The numerical simulation results suggest that landfill cover soils with extreme compaction may contribute to global warming due to its high CH_4 emission ability since CH_4 has 20 times higher global warming potential than CO_2. In addition, when the thickness of the landfill cover is less than 50 cm, the CH_4 emission flux rapidly increased, indicating methane oxidation is not effective for cover soils with very thin thickness, enhancing the CH_4 emission.

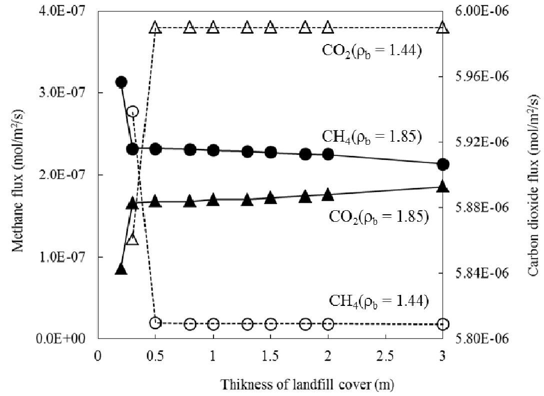

Figure 5 Methane and carbon dioxide flux to the atmosphere from the landfill cover soils at two different compaction levels and different thickness.

5. CONCLUSIONS

Using a developed unified predictive model for gas diffusivity and thermal conductivity, gas and heat transport in landfill cover soils at different compaction levels were simulated. Higher compaction enhances heat exchange through landfill cover soil, possibly contributing rapid site stabilization in the landfill site. On the other hand, lower gas diffusion characteristic in the highly-compacted cover soils decreased gas diffusivity, hereunder causing enhancement of CH_4 emission flux to the atmosphere due to ineffective methane oxidation ability. The methane oxidation ability is also highly affected by the thickness of the landfill cover. In perspective, more accurate simulations to represent more complex gas and heat transport such as heat-induced density-driven gas flow and further model developments for biological kinetic parameters to evaluate methane oxidation rate are needed to reduce and control greenhouse and toxic gas emissions from the landfill site and more rapid site stabilization.

6. ACKNOWLEDGMENTS

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Suitability of Expansive Soil to Use as Clay Liners in Arid Zone of Sri Lanka

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Abstract: *Solid waste is a growing problem in urban areas of Sri Lanka and management of waste, both liquid and solid have become a critical environmental concern due to the absence of engineered methods of disposing waste. Development of a simple engineered landfill facility utilizing a locally available material to suit landfill liner requirements is the most economical and the timely solution to this problem. In the present study, it was investigated the suitability of expansive soil which is commonly available in the south arid zone of Sri Lanka to use as clay liners in landfill facilities. The soil was improved by the addition of commercially available Bentonite to build a low hydraulic conductivity barrier. Further, the long term effect of soil -leachate interaction on hydraulic conductivity of the suggested liner was studied. Results showed that the engineering properties of expansive soil can be improved by the addition of bentonite to meet the landfill liner requirements. However, the original engineering properties of soil-bentonite mixtures were significantly affected by the leachate interaction over a period of time.*

Keywords: *expansive soil, hydraulic conductivity, landfill liner, leachate.*

1. INTRODUCTION

Solid waste, especially Municipal Solid Waste (MSW), is a growing problem in urban areas of Sri Lanka and management of waste, both liquid and solid have become a critical environmental concern. The absence of engineered methods of disposing waste and the open dump approach adapted has created this major environmental and social problem of waste within most of the cities. Under open dumping, which is the main trend among local authorities at present, solid waste are disposed haphazardly and they are subsequently subjected to open burning.

Currently, the attention given to the solid waste management in dry zone especially in arid zone of the country is very low due to the fact that all most all the major cities in Sri Lanka are situated in wet zone. However, solid waste management in dry zone is very important as the people depend very much on ground water for their drinking purposes and therefore, the contamination of ground water especially by the leachate generated in waste disposal sites should be kept at a minimum by following engineered waste disposal methodologies.

Bagchi (2004) and Daniel (1993) had identified that engineered land filling is one of the best options to overcome the problems associated with contamination of ground water with leachate. The liner system in an engineered landfill acts as a barrier for leachate and prevents the transportation of contaminants to the surrounding pollution prone environment. Hence liner system in a landfill becomes one of the critical design considerations. Bagchi (2004), Daniel (1993) and Jayasekera (2007) had studied the hydraulic conductivity of different landfill liners and according to the results a landfill liner is intended to be a low permeable barrier which is generally involves the application of clay or synthetic material layer. Since, synthetic materials are very expensive, compacted clay liners (CCL) are the most common liner system in developing countries. (Ameta et al, 2008)

Expansive soil is a locally available material in Hambantota area which can be used as a liner material. Gourley et al (1993) had investigated the engineering properties of expansive soil and had defined expansive soil as fine grained clay which occurs naturally and subjects to swelling and shrinkage, varying in proportion to the amount of moisture present in the soil.

Only a limited number of researches are reported with respect to investigation of suitability of expansive soil to use as a CCL material in landfill sites. Therefore, a compacted clay liner was developed using expansive soil in this research study. Further, effect of soil-leachate interaction on engineering properties of suggested clay liner was investigated.

2. METHODOLOGY

2.1. Engineering Properties of Expansive Soil

In order to investigate the suitability of expansive soil to use as a clay liner, basic engineering properties of original soil collected from Hambantota were determined in the laboratory and presented in Table 1.

Table 1 Engineering properties of soil-bentonite mixtures

Physical Property	Bentonite Percentage			
	0%	5%	10%	15%
Liquid Limit (LL) (%)	41	41	43	49
Plastic Limit (PL) (%)	24	28	30	22
Plasticity Index (PI) (%)	17	13	13	27
Linear Shrinkage (LS) (%)	16	17	19	31
Maximum Dry Unit Weight (kN/m^3)	17.00	16.94	16.56	16.25
Optimum Moisture Content (%)	19.0	19.5	22.0	28.0

X-ray diffraction test is the most accurate methodology to determine the mineral content of soil. However, due to the limited facilities available and the high cost involved, the mineral content could not be found through x – ray diffraction method in this research study. According to the findings of Savage (2007) it was realized that Illite is the most dominating clay mineral in this particular expansive soil using Atterberg Limits and clay content.

Swelling potential of expansive soil can be determined in accordance with both British Standards and Australian Standards. According to the British Standard, swelling potential is defined in terms of swelling pressure whereas according to the Australian Standards, swelling potential is defined in terms of Shrink – Swell Index. In this research study shrink-swell index was evaluated according to the AS 1289.7.1.1(2003) in order to present the swelling potential of the expansive soil and shrink-swell index was recorded as 1.48%.

According to AS 1289.7.1 (2003) Shrink-swell index can be found by shrink swell test which consists of two separate laboratory tests, a swell test and a simplified core shrinkage test. These tests should be carried out on undisturbed soil samples from their initial field moisture contents.

As fine content of the soil is high, falling head method was used to determine the hydraulic conductivity of expansive soil. The major difficulty encountered during the experiment was saturation of the soil sample. As soil sample was compacted at its optimum moisture content to represent the clay liner in an engineered landfill site, it takes very long time for the saturation process. In order to overcome this difficulty, a vacuum was applied from top of the sample while sample was submerged in water.

2.2. Improvement of Engineering Properties of Expansive Soil

Different percentages of bentonite varying from 0-15% in steps of 5% on dry weight were mixed with expansive soil in order to improve the engineering properties of soil and depicted in Table 1.
Stewart et al (2003) had defined bentonite as an important naturally occurring clay mineral of great commercial importance possessing inherent bleaching properties. It falls mainly under montmorillonite group and presents strong colloidal properties. Further variations of hydraulic characteristics of soil-bentonite mixtures were studied.

2.3. Long Term Effect of Soil-Leachate Interaction

The long term effect of leachate contact on hydraulic conductivity and volume change properties of liner material was evaluated by allowing the compacted soil-bentonite mixtures to interact with leachate for a period of four months.

3. RESULTS AND DISCUSSION

3.1. Engineering Properties of Soil-Bentonite Mixtures

Variation of engineering properties with the addition of bentonite is presented in Table 1. According to the laboratory test results with the addition of bentonite, Atterberg limits tend to increase. Similarly maximum dry unit weight decreases whereas the optimum moisture content increases. The large increase in liquid limit, linear shrinkage and optimum moisture content when the bentonite percentage is 15% may be due to the high water absorption of bentonite.

3.1.1. Compaction Characteristics

The variation of maximum dry density and the optimum moisture content with the addition of bentonite is clearly illustrated in Figure 1.

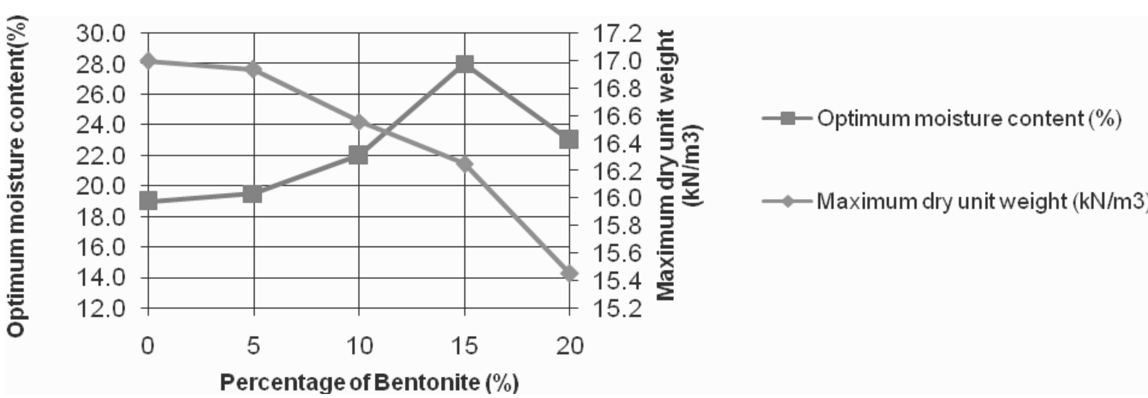

Figure 1 Variation of maximum dry unit weight and optimum moisture content with bentonite percentage

It can be observed that as the bentonite percentage increases, the maximum dry unit weight decreases whereas the optimum moisture content increases except the case of 20% bentonite addition. The decrease in maximum dry unit weight with increase in bentonite content may be attributed to high swelling characteristics of bentonite that forms a gel called as diffused double layer around soil particles. When this diffused double layer forms around the soil particles, the effective size of soil particles increases which causes increase in void volumes and thus decreased dry unit weights.

3.1.2. Hydraulic Conductivity

The variation of hydraulic conductivity of soil-bentonite mixtures is illustrated in Figure 2. It can be noted that the hydraulic conductivity of soil-bentonite mixtures decrease with the increase of bentonite.

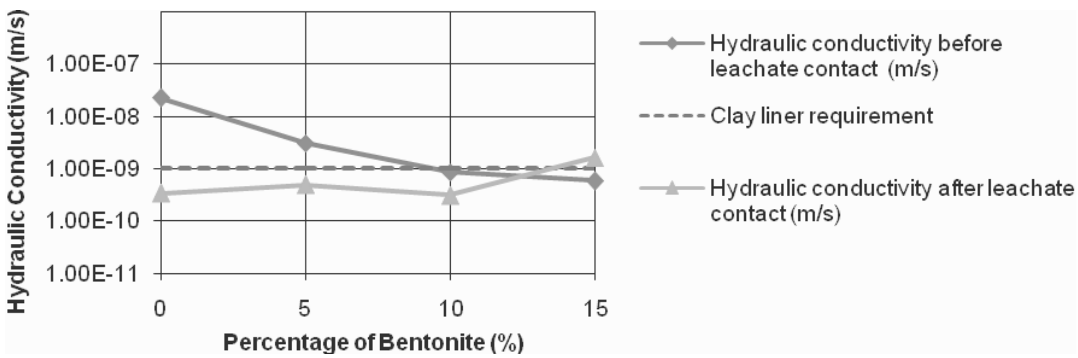

Figure 2 Variation of hydraulic conductivity with bentonite percentage

With the increase of bentonite, which mainly consists of montmorillonite mineral, the diffused double layers surrounding the clay particles are getting thicker. (Figure 3) As a result, the flow paths between the double layers become pinched off and the hydraulic conductivity decreases. Further, according to the Gouy-Chapman theory, the hydraulic conductivity is inversely proportional to the double layer thickness. It can be noted that, a significant reduction in hydraulic conductivity with the addition of bentonite to the original soil. The clay liner requirement with respect to hydraulic conductivity, i.e. 1×10^{-9} m/s, can be achieved with the addition of 10% of bentonite to the original soil. [Bagchi (2004), Daniel (1993) and Iqbal et al (2003).

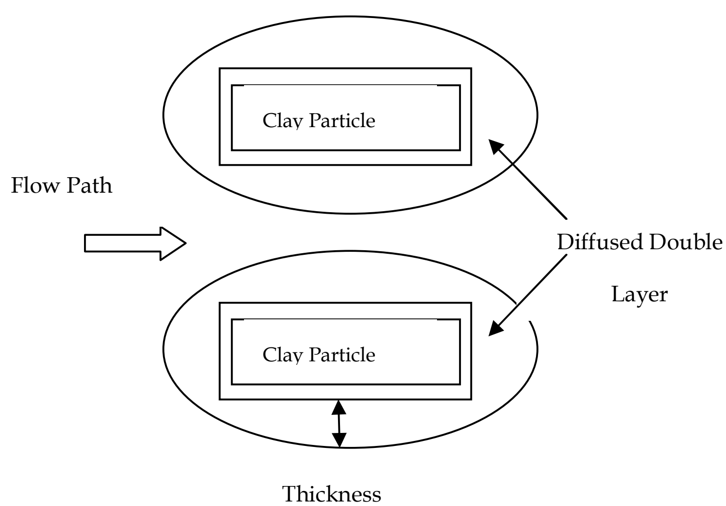

Figure 3 Reduction of hydraulic conductivity due to increase of double layer thickness

On the other hand, due to the formation of diffused double layer creates repulsive forces along the sides of the clay particles making it difficult for individual clay particles stay closer to each other. Under these repulsive forces, these clay particles align themselves in a more parallel orientation forming a dispersed structure; hence increase the void ration over the increase of bentonite percentage. (Figure 4)

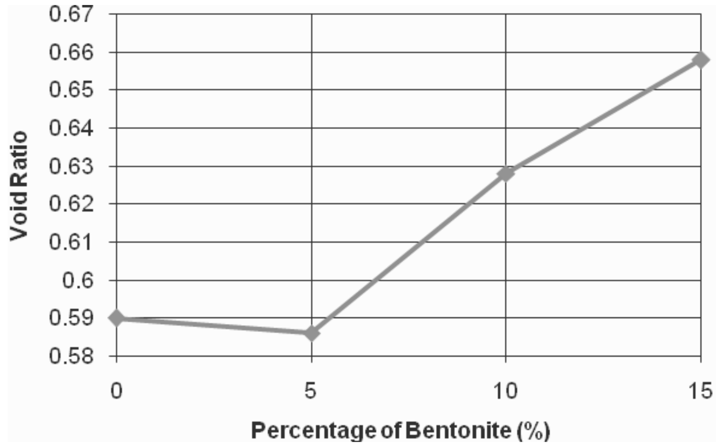

Figure 4 Variation of void ratio with the addition of bentonite

3.1.3. Plasticity Index

Plasticity index is another parameter which should be checked against the landfill liner requirements. Results of the laboratory Atterberg tests are presented graphically in Figure 5.

It can be observed that liquid limit increases with the addition of bentonite whereas the plastic limit has no such relationship. Plastic limit increases with the addition of bentonite up to 10% of bentonite. However, the rate of increase in both the liquid limit and plastic limit are almost similar.

Plasticity index doesn't show any clear relationship with respect to the percentage of bentonite added. Initially plasticity index decreases and with the addition of 15% of bentonite it shows a huge increase. But for 5% and 10% plasticity index is almost similar and this may be due to the same rate of increase in liquid and plastic limits. However, the requirement of a landfill liner material which is plasticity index should be more than 7-10% is satisfied for all soil samples. [Bagchi (2004), Daniel (1993) and Iqbal et al (2003).

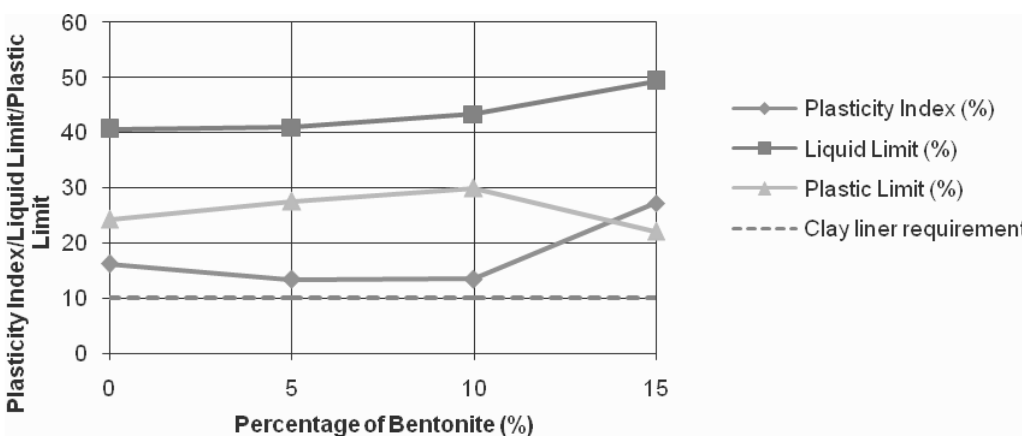

Figure 5 Variation of Liquid Limit, Plastic Limit and Plasticity Index with bentonite percentage

3.1.4. Particle Size Distribution

Particle size distribution plays a crucial role in evaluating the suitability of a soil to use as the liner material in a landfill site. The fine fraction should be high with low gravel content to ensure a low hydraulic conductivity through the soil. According to the results of the sieve analysis and hydrometer analysis the fine content of the natural expansive soil is 66%. This means that natural soil itself contains a higher fraction of fines.

The desired value of fine content of a landfill liner material is equal or greater than 20-30%. [Bagchi (2004), Daniel (1993) and Iqbal et al (2003)] Therefore, the natural soil also satisfies this requirement. Since bentonite contains a higher fraction of clay minerals this fraction of fines in natural soil is expected to be increased with the addition of bentonite. Results of hydrometer analysis for different percentages of bentonite addition are shown in Figure 6.

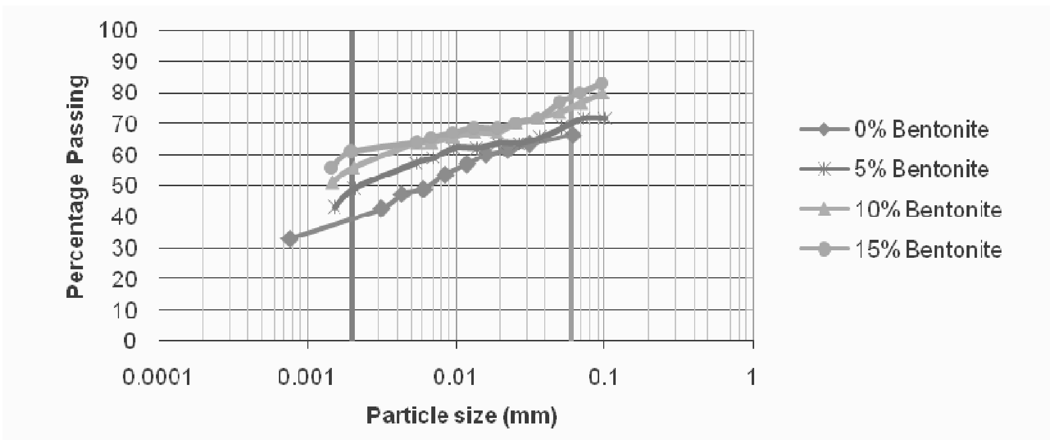

Figure 6 Particle size distribution curves of different soil-bentonite mixtures

According to the above results hydraulic conductivity is the governing factor to select the best percentage of bentonite that should be mixed with expansive soil as all the other clay liner requirements are satisfied with the natural soil itself. Therefore, as suggested in section 3.1.2 mixing of 10% of bentonite by weight with expansive soil yields the best mixture for a clay liner and the comparison of clay liner requirements with that mixture properties are given in Table 2.

Table 2 Clay liner requirements

Engineering property	Clay liner requirement	Expansive soil + 10% bentonite
Hydraulic conductivity	$\leq 1\times10^{-9}$ ms^{-1}	8.56×10^{-10} ms^{-1}
Plasticity index	≥ 7-10 %	13 %
Percentage fines	≥ 20-30 %	75 %
Percentage gravel	≤ 30%	< 2 %
Maximum particle size	≤ 25-50 mm	2 mm

3.2. Engineering Properties of Soil-Bentonite Mixtures after Long Term Leachate Contact

Long term effect of soil-leachate interaction on hydraulic conductivity is the major factor which determines the satisfactory performance of a landfill liner. The variation of hydraulic conductivity of compacted soil-bentonite mixtures after contact with leachate is also illustrated in Figure 2. It can be seen that hydraulic conductivity of original compacted expansive soil has been decreased significantly after the interaction with the leachate. However, with the increase of bentonite percentage the hydraulic conductivity has been slightly increased and when it comes to a bentonite percentage of about 14%, hydraulic conductivity has been increased comparing to the before leachate contact state. Consequently, clay liner requirement gets dissatisfied.

This reduction of hydraulic conductivity in original expansive soil after contact with the leachate is mainly associated with the clogging of soil particle tops due to precipitation of the suspended particles existing in the leachate and form a less permeable thin layer at the top. The slight increase of hydraulic conductivity over the soil-bentonite mixture after contact with the lachate is mainly due to reduction of diffuse double layer thickness, which causes increase of flow paths between the diffused double layers. In other words, soil-bentonite mixture has become less reactive (decrease swelling potential) after contact with leachate for a certain period. This is mainly due to the physic-chemical reactions between leachate and soil-bentonite mixture.

Similar results can be observed with respect to void ratio as shown in Figure 3 in soil-bentonite mixture after contact with leachate. The void ratio has been increased with the addition of bentonite, due to the effect of diffused double layer, where repulsive forces of clay particles increased the void spaces. The reduction of void ratio in the original expansive soil after interact with the leachate is mainly due to the precipitation of the suspended particles existing in the leachate, in the void spaces of soil, which leads to increase the volume of solid state in the soil; thus void ratio has been reduced. There is no any significant effect on the other clay liner requirements from the long term leachate interaction. Therefore, hydraulic conductivity becomes the governing factor which determines the optimum percentage of bentonite.

According to these results compacted natural expansive soil will also act as a hydraulic barrier after the attenuation by landfill leachate for a certain period of time. On the other hand there is an optimum percentage of bentonite that should be added to expansive soil to build a clay liner in landfill facilities since the soil structure will be altered after the leachate interaction which affects the performance of the clay liner.

4. CONCLUSIONS

Engineering properties of expansive soil can be well improved by mixing it with different percentages of bentonite. However, the rate of improvement of those properties gets reduced with the increasing bentonite percentage. Therefore, excessive addition of bentonite to expansive soil will not form a suitable mixture to suit clay liner requirements.

All the clay liner requirements other than the hydraulic conductivity get satisfied by the natural expansive soil itself and therefore hydraulic conductivity is the governing factor which determines the most efficient percentage of bentonite. According to the laboratory experiments it can be concluded that addition of 10% of bentonite by weight will yield the most economical soil-bentonite mixture to build clay liners in arid zone.

The original engineering properties of soil-bentonite mixtures can be significantly affected by leachate interaction over a period of time. After interact with leachate, the hydraulic conductivity has been significantly decreased in the original expansive soil whereas it has been slightly increased with the increase of bentonite percentage. Therefore, it can be concluded that, the satisfactory performance of the compacted clay liner is highly depends on the alteration of soil structure due to the soil-leachate interaction over a long period. These consequences will affect the satisfactory performance of the clay liner over time.

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Instrumentation and Monitoring of Mahawewa Landslide off Walapane in Central Hills of Sri Lanka

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Abstract: *As instability of the slope area at Mahawewa, Walapane has been identified as a major threat to the people since 1986, National Building Research Organization has started a project of monitoring and stabilization of the slope, cooperating with the Japan International Cooperation Agency in 2010. Main involvement of this study was to do the instrumentation and monitoring, mainly based on analysis of un-stabilization parameters of the area which measured using five instruments. That is horizontal movement at sub surface using Inclinometer, fluctuation of ground water level using piezometers, characteristics of slip surfaces using strain gauges, movement of the surface using extensometers and rainfall using rain gauges. In addition to above drill log data of the site, visual information of cracks generated and springs available were also used. Finally, according to the results of analysis the unfavourable ground water level in slope area has been identified as the most critical factor of the landslide. According to that conclusion a suitable mitigation options for Mahawewa landslide were proposed.*

Keywords: *Landslides, Slip surfaces, Ground water level, Slope stabilization, Slope instrumentation*

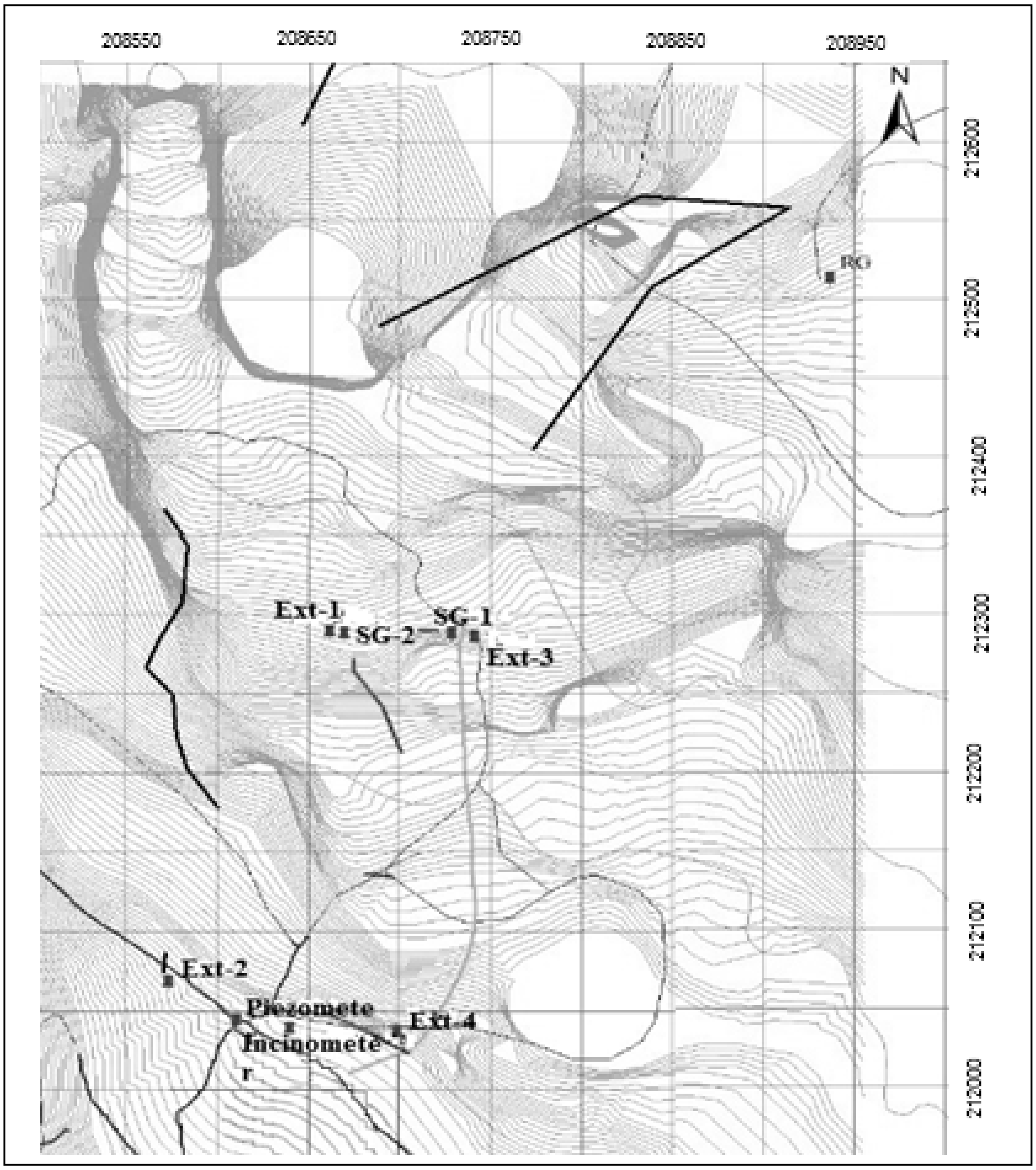
1. INTRODUCTION

Landslide is a natural disaster which is caused significant impact to the economy of the country and creates number of problems such as, loss of human life, damage to the property and also damage to the natural environment. Landside disaster has been reported with increasing frequency in recently at central hilly areas of Sri Lanka.

The instability of the rock slope at Mahawewa in Walapane was identified as a major risk to the safety of the people and property. That slope is located behind the Padiyapallalla town in Nuwara Eliya district in Central Hills of Sri Lanka. It belongs to Kumbalgamuwa GN division. The landscape in that area is very undulating with steep slopes. The initial signal of landslide was occurred in 1986, but in that time it was not highly concerned. Then again in 11th and 12th January 2007, 3 landslides were occurred in the area. Among them, one was activated by losing 18 lives and fully damaging 68 houses (http://www.wsws.org /articles / 2007 / jan2007/sril-j31.shtml).

Mahawewa area off Walapane has some importance in economical, agricultural and sociological aspects. If the landslide will be re-activated again, it is highly affecting to the Walapane Hanguranketha main road and also it can destroy the Keerthi Bandara School situated on the extreme down slope (http://www.nbro.gov.lk/web, NBRO home page, Mitigation of landslides). Therefore, after the 2007 tragedy, National Building Research Organization (NBRO) has started monitoring this landslide using instruments with some mitigating programme. This is the first time of doing such a total instrumentation procedure for monitoring the landslide, Sri Lanka. There are 5 types of instruments have been installed such as, Inclinometer, Piezometer, Extensometers, Strain Gauges, and Rain Gauges (Fig 1).

Main involvement of ours on behalf of this study was to do the monitoring part by analysing the relationships of slope stabilizing parameters which were measured using those instruments. We used the relationships between following factors; rain fall of the area, fluctuation of the water table, movement of the surface ground (direction and extent of movement) and movement of the sub - surface ground (no of slip surfaces, possibility of their activation, depth from the surface to slip).



Ext – Extensometer

SG – Strain Gauge

Figure 1 Instrumentation Location Map of Mahawewa Landslide

2. METHODOLOGY

2.1 Data Collection and Field Survey

As the first step of our research, instrument installation processes and their particular locations were studied. Then one year data of all instruments were collected within suitable intervals. Data collection was done by manually in Inclinometer once a month while data has been automatically recorded in other instruments at each half an hour or one hour.

In addition to that, a Global Positioning System survey was conducted for the affected area to demarcate the boundary of the both past and current landslide, around the lake, along the cracks appeared and also at the location of instruments. Then using data of the map from NBRO, prepared a map of the area using Arc map 9.3 and 3D model of the slope using Auto Cad 2010.

2.2 Data Analysing for Surface movement

Generally, the first step of landslide monitoring consists of exploring the behaviour of surface material such as their direction of movement, extend of movement, rate of movement etc. In addition to that these initial evidences are directly affected on the installation of other instrument such as strain gauge and inclinometer. Therefore at the beginning, data extracted from the extensometer were studied for each day.
As it provides the relative movement between two points on the sliding body the readings were plotted against the time to realize the continuous movement during a considered time period. Then it was compared with the rainfall as it is the main driven force of all most all the landslides in Sri Lanka. Then regarding those graphs the particular sections which indicate rapid movements were further analysed. The errors identified were removed to clear visualization of the critical movement (Fig 2).

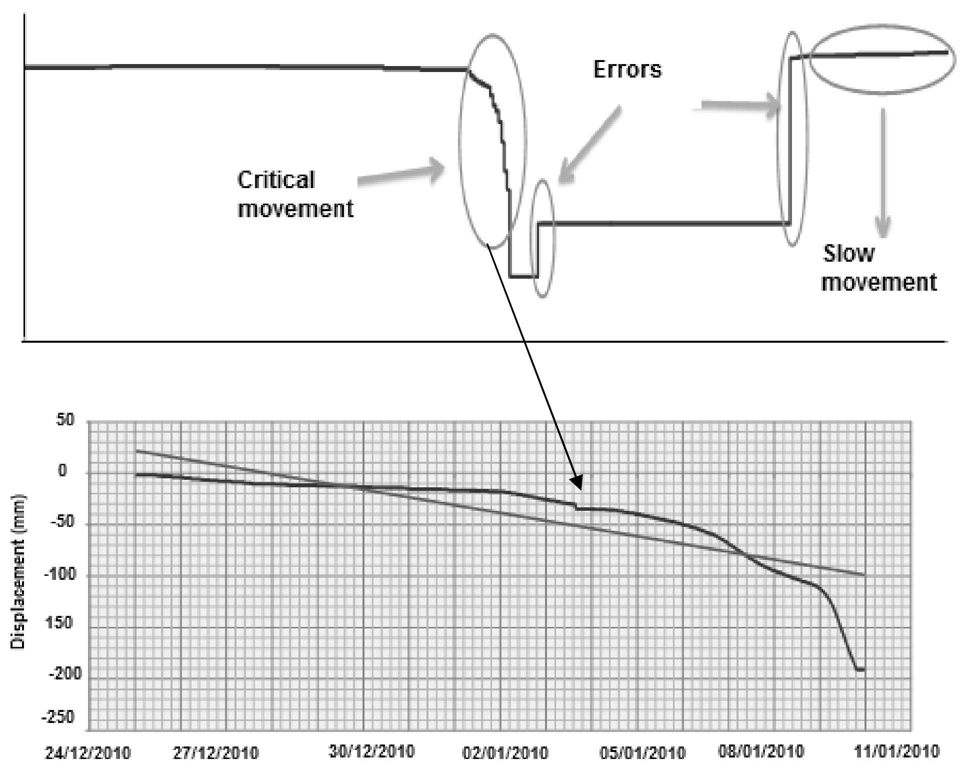


Figure 2 Analysing of surface displacement

In here with the help of a trend line, an equation was obtained in the format of "***y =Mx + C***". It gives a relationship between movement and time. Then the value received for the "M" indicates the rate of movement (mm per Day) while the sign mark (+,-) indicates the direction of movement. If a mines sign is received it indicates that the lower part is moving fast and vice versa.

2.3 Analysing of water level fluctuation

Water always tends to reduce the stability of the earth slopes. As an example when Cracks are filled with water, they create tension on slope. This tension force is a function of the height of water column in the crack. So ground water level decides the height of the water column in the tension crack.

Piezometer provides two values called **water level** and **groundwater level**. The fluctuation of groundwater level with the rainfall was plotted as follows (Fig 3).

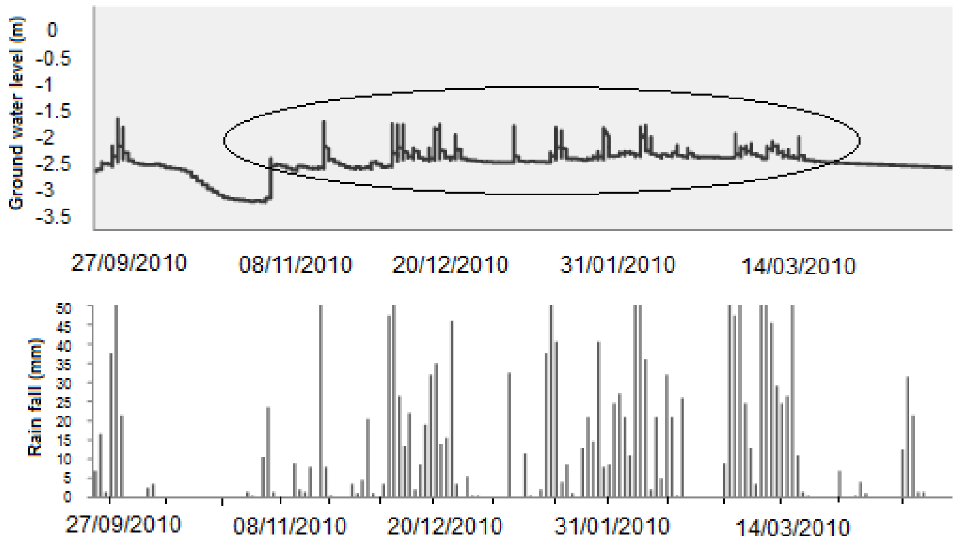


Figure 3 Groundwater level with the rain

As this diagram indicates groundwater level has been rapidly changed during the rainy season. It means that water get accumulated within the slope area which cause to increase the hydraulic pressure and reduce the factor of safety. To have a much accurate result upon ground water the data obtained from water gauge was also referred.

Then some crucial points which indicate rapid deviations of ground water level from the normal conditions were recognized and tried to find some reliable reasons which may cause those disturbances. By mean of closely analysing, the ground water level at the normal condition was determined. Then evaluate the amount of deviation from the normal value at a particular rainy condition. Then the water column height at a particular condition can be compared with the occurrence of the major slip surfaces.

2.4. Data Analysing for Sub Surface movement

2.4.1 Inclinometer data

Data has been taken inserting a sensing unit (electronic probe) consists with wheels that track within the grooves of the inclinometer casing. These grooves are aligning to the direction of the slope sliding and it is named as “X” direction. The values visualized in readout unit also named as “X” values. Simultaneously it also gives the values regarding with perpendicular direction to the “X” and it is called the reading of “Y” direction. That data taken provide the information about inclination of the casing at various depths which implies the horizontal displacement of the ground with the depth. Normally two sets of readings were taken in each depth by running the instrument in the casing once and then repeating the procedure with the instrument turned 180^0.

The instrument provides the angle of movement of the sliding slope, while after the data has been processed, the depth with respect to critical movement was recognized. On behalf of this, three graphs were plotted in the same X-Y plane for adjacent three months. Then check the comparison movements of the subsurface for each month.

2.4.2 Strain Gauge data

At the Mahawewa slope there are two strain gauges are installed and it provides the valuable result of depth to the potential slip surfaces. At first Strain Gauge data were plotted and identified some rapid changes (Fig 4). For having such a huge strain changing, there should be critical failures at the corresponding depth.

At the same time, the behaviour of these recognized failure surfaces has been daily monitored throughout the year to see whether the depth of failure surfaces has been changed. Because it changed with various kinds of external influences such as water percolation, slope consolidation, change in geological structures etc. And also for mitigations like soil nailing, these analysed data are very much crucial to decide the length of supported nails. The coordinates of major crack appeared at the top of the landslide were also used to correctly locate the identified Slip surfaces

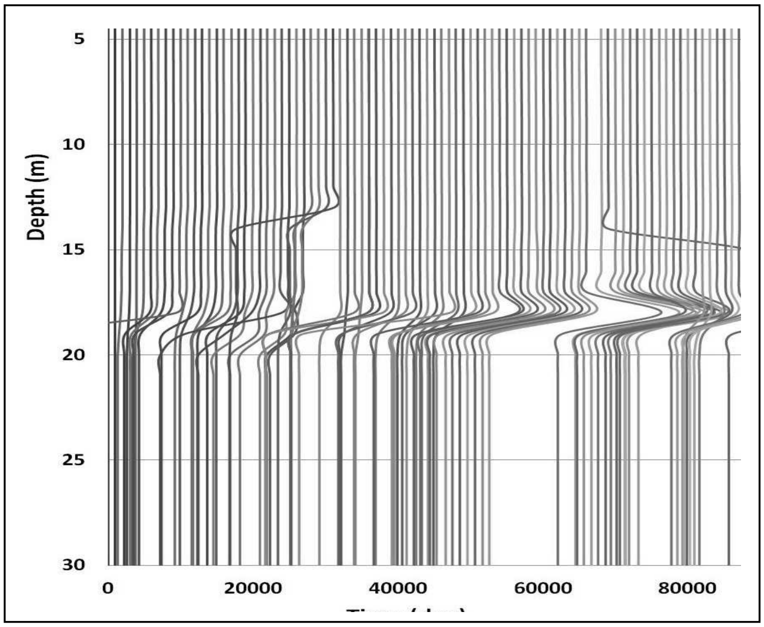


Figure 4 Identification of slip surfaces – Strain Gauge data

3. RESULTS AND DISCUSSION

3.1. Geomorphology of the area

The landscape of Mahawewa is very much undulating with steep slopes of 15^0 to 35^0. There are two escarpments at the top area of the slope and near to that the whole area is filled with a colluvium deposit consisting of large size boulder (1 m to 5 m in diameter). There is a lake bellow the escarpment. It is now abandoned for some extent but still water can be appeared little below to the ground level even in dry season. Villagers in that area take the water through pipe lines from that abandoned lake. Newly appeared water spills could be identified at the middle area of the slope.

Newly formed cracks which are largely extended in the ground were identified at the whole area of the landslide while the major crack has been visualized at the top of the slope. And also walls and floor of the most houses are severely cracked.

3.2. Results of Extensometer (Ext)

Although there had been five extensometers installed before the very recent landslide occurred, by now it has been reduced to four. By analysing data received from these four instruments following valuable information was exposed.

- **Ext 01:** Critical movement - 25/12/2010 to 10/01/2010. Lower portion from the Crack has relatively moved downward. Rate of movement is 7.56 mm per day.
- **Ext 02:** Critical movement- 08/12/2010 to 09/01/2011. Rate of movement is 3.629 mm/day. Lower portion from the major boundary has moved downward.
- **Ext 03: C**ritical movement- 25/11/2010 to 09/12/2010. Rate of movement is 17 mm/day. Lower portion from the crack has relatively moved downward.
- **Ext 04: C**ritical movement- 27/11/2010 to 27/12/2010. Rate of movement is 1.3 mm/day. Low downward movement compare with the other movements.

3.3. Results of Strain Gauge (SG)

Following are the characteristics of identified slip surfaces using Strain Gauge data.
First 6 months – Mainly two slip surfaces were identified at 13 m and 18 m depth bellow the location of SG2 (Fig 8). The slip surface at 13 m depth also can be detected at the 11 m depth bellow the SG1. This has a slow movement with the rain. The effect is varying up to 1m above the slip surface and varies with the rain fall.
Slip surface at 18 m depth bellow the SG2 has a rapid movement in whole time period which concerned. It has affection area of 1 m above the slip surface and 2 m bellow the slip surface. That affection area range is changed with the rain fall and continued the same in dry season.
Second 6 months – New two slip surfaces are formed in shallow area with the depth of 4m and 10m bellow the SG2

3.4. Results of Inclinometer

As strain gauge data is not sufficient for correctly defining the slip surfaces, the Inclinometer data also has been used. That represents the horizontal movement in sub surface bellow the location of Inclinometer. Those movements are always measured with respect to the end point of the inclinometer bore hole. Those movements are shown in below (Fig 5) and it gives an idea of occurring of a slip surface at 26m depth below the particular location.

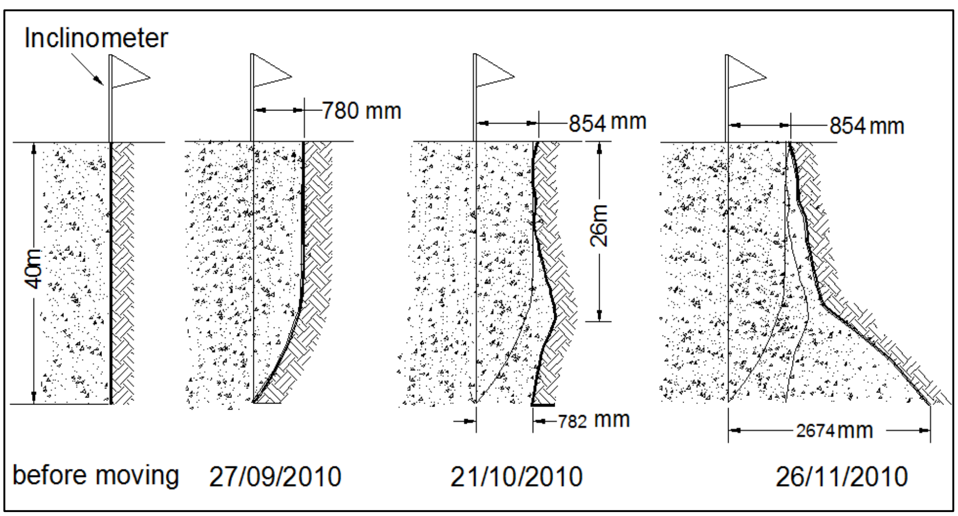


Figure 5 Results of Inclinometer

In here the end point of the Inclinometer is also have a considerable horizontal movement. It can be happen only if there are slips surfaces occurring bellow the end point of the Inclinometer. Then gives an idea about availability of slip surfaces even bellow the 40m depth from the location of Inclinometer. All the analysed data about potential slip surfaces in first 6 months which were taken from two strain gauges at the middle area and from 1 Inclinometer at top of the slope is summarised as follows (Fig 6, Fig 7).

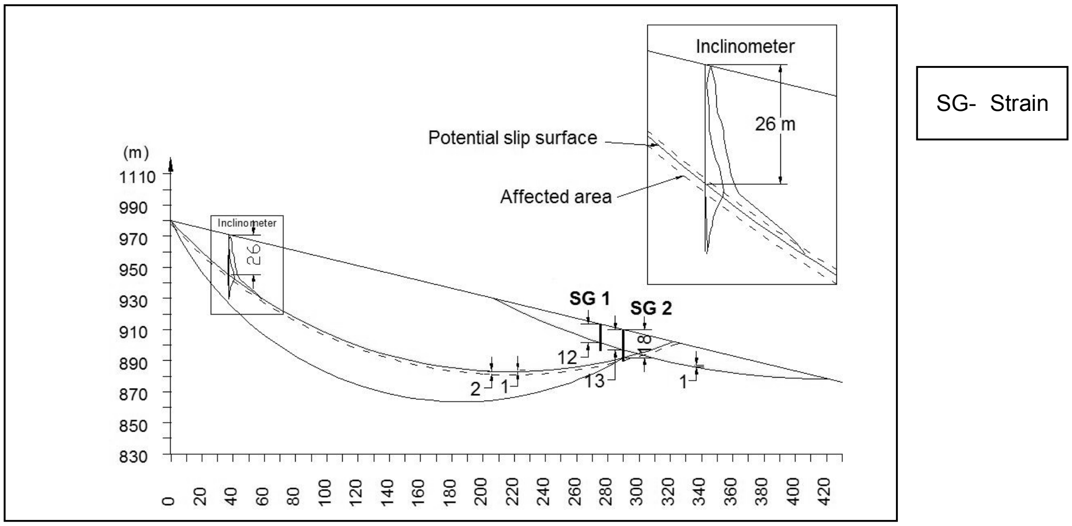


Figure 6 Potential Slip Surfaces at the Mahawewa slope (24/07/2010 to 10/01/2011)

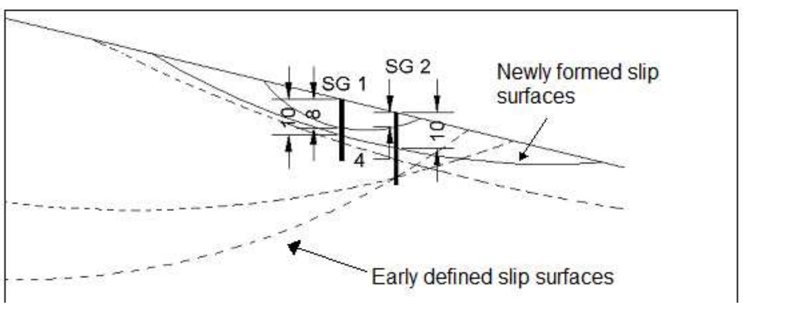


Figure 7 Newly formed Slip Surfaces (10/01/2011 to 14/06/2011)

3.5 Ground Water Level

In dry season (Fig 8)

- Top of the slope – at 2.5 m depth
- Middle area of the slope – 9.82 m depth

In Rainy season (Fig 8)

- Top of the slope – at 1.75 m depth
- Middle area of the slope – 6.64 m depth

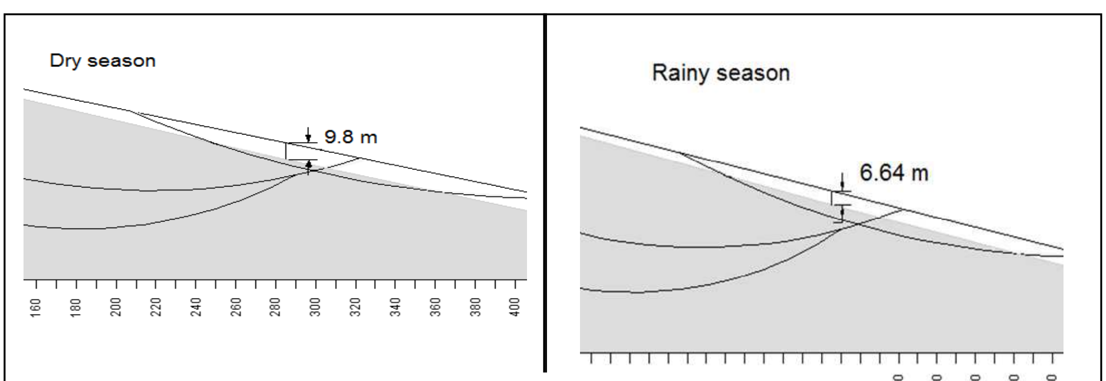


Figure 8 Ground water level in dry and rainy seasons

4. CONCLUSIONS

According to the monitoring data, the type of this Mahawewa landslide is a debris rotational slide, and also rock falling can be seen only at the top of the landslide. Mainly four slip surfaces could be identified and there is a possibility to have another deep slip surface bellow the 40 m depth of Inclinometer station.

However the most critical factor of this landslide is the unfavourable ground water level in slope area. It has been clearly identified that in dry season Piezometer indicates the average ground water level of 2.5 m at the top of the slope and Strain Gauges indicate average ground water level of 9.82 m at the middle area of slope moving axis. In rainy season at the top of the landslide ground water level has been reduced up to 1.5 m level while middle axis of the slope has 6.64 m ground water level. Then it can be concluded that the ground water level is always above the critical slip surfaces even in dry season. So it is better to reduce the water column well below the deepest critical slip surface.

But it is not feasible as well as practicable to establish such a process to reduce the water level completely below o the main critical failure surfaces. Then it is suitable to minimize the effect of the water column by reducing the water level down to some optimum extent. Then the weight component of this unfavourable water column is minimized and hence the safety factor can be increased.

Temporally mitigation options also can be used to minimizing the rate of ground displacement of the slip surfaces while introducing proper water proof sealing techniques for the prevention of water entering to the surface cracks. Introducing of well maintaining ditches system is essential. Then can minimize the amount of water accumulated in the abandon reservoir area up to some extent. It also can be performed by designing a surface drainage system of diversion cannel system to the top of the landslide to divert the direct water entries from the escarpment which cause to reduce the erosion of the slope.

Not only surface drainage system but also subsurface drainage system can be introduced to remove the accumulated excess water in abounded lake and sliding area. Inserting horizontal or vertical drains are preferred. Furthermore, to increase the feasibility of the project, extracted water can be utilized for the benefit of the people living around Kumbalgamuwa area. For instant surface water can be used for the agricultural activities while using the subsurface water for drinking purposes.

5. ACKNOWLEDGEMENTS

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Characterization of geotechnical properties as affected by sediment environment in Kanto lowland clays in Japan

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Abstract: *The purpose of this study is to investigate effects of sedimentary environment (freshwater and marine deposits) on geotechnical properties for Kanto lowland clays. Freshwater and marine sediments were used to perform standard consolidation tests, undrained triaxial compression tests, and unconfined compression test. Pore-water compositions were also measured for the sediments. As results, higher compression index, higher sensitivity, and lower shear strength ratio were observed for the marine sediments as compared to those for the freshwater sediments. In addition, the measurements of pore-water compositions revealed that the sensitivity values were well related to the proportion of monovalent cations, especially for the sediments with plastic index of less than 30.*

Keywords: *geotechnical properties, sedimentary environment, pore-water composition*

1. INTRODUCTION

It is well known that differences in structure of clays (i.e., fabric and bonding) affect the geotechnical properties such as a compressibility or sensitivity. The structure of natural clays highly depends on the sedimentary environments since the depositional geochemistry and deposition rate contribute to the fabric (arrangement of particles) and interparticle bonding.

In this study, geotechnical properties including Atterberg limits, compression index, undrained shear strength, and sensitivity for alluvial clays in Kanto lowland with different sedimentary environment were measured. In addition, ion compositions in pore water were also measured to characterize the sedimentary environment and the relations between ion compositions and geotechnical properties were also investigated.

2. MATERIALS AND METHODS

2.1. Study Site

The samples used in this study were taken at Kasukabe (Bigohigashi, Kasukabe-city, Saitama) and Kameido (Koto-ku, Tokyo) in Japan. Kasukabe and Kameido sites are located in Nakagawa lowland and Tokyo lowland, respectively. The sample cores of the latest Pleistocene to Holocene sediments up to 50 m depth were drilled at both sites.

2.2. Measurements of Physical, Chemical, and Mechanical Properties

The core samples were used for the measurements of physical, chemical, and mechanical properties. The particle size distribution was measured by a sieving and hydrometer method. The soil pH and EC (EC, mS m^{-1}: electrical conductivity) were measured in a 1:2.5 and 1:5 (by weight) mixture of soil and distilled water, respectively.

The compressibility (i.e., compression index) of the sediments was investigated by a standard oedometer test. The undisturbed and remoulded shear strengths were determined by the unconfined compression test using the specimen with 5 cm of diameter and 10 cm of height. The sensitivity value of some samples, where remoulded shear strengths could not be measured since the specimen did not stand, were defined as infinity in this study. Undrained triaxial compression test ($\overline{CU}$) was also performed according to JGS (Japanese Geotechnical Society) 0523 for undisturbed soils using the cylinder specimen (i.d. 5 cm, height 10 cm).

The pore water was extracted by a direct extraction method where the samples of around 300 g were squeezed by centrifugation at different rotation speeds (8000 rpm). The extracts were used for measuring ion concentration. Cations of Na^+, K^+, Ca^{2+}, and Mg^{2+} were measured by an atomic absorption spectroscopy and anions of Cl^- and SO_4^{2-} were measured by an ion chromatograph, respectively, following JIS K-0102.

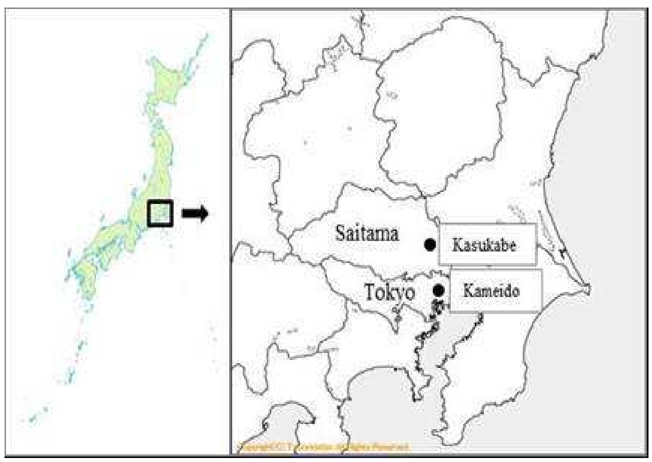

Figure 1 Sampling site

2.3. Monovalent / Divalent Ion Ratio

Gapon's equation was used to investigate the effects of proportion of cations monovalent and divalent in the pore water on mechanical properties. The Gapon's equation can be expressed as below.

$$\frac{M_e}{D_e} = k\frac{M_0}{\sqrt{D_0}} \qquad (1)$$

where M_0 and D_0 are molar concentration of the monovalent, divalent cations in the free pore water, M_e and D_e are the exchangeable monovalent, divalent cation in meq/100g of air dry soil, k is Gapon's constant (depends on soil type).

Eq. (1) suggests that a plot of $M_o / \sqrt{D_o}$ with depth would indicate a variation of the monovalent / divalent ion ratio in the exchange complex of the pore water provided the soil type is constant with depth (i.e., constant k value with depth). In this study, the monovalent / divalent ion ratio in the soil has been calculated according to Eq. (2).

$$\frac{M_0}{\sqrt{D_0}} = \frac{Na^+ + K^+}{\sqrt{Ca^{2+} + Mg^{2+}}} \qquad (2)$$

where Na^+ , K^+ ,Ca^{2+}, and Mg^{2+} were measured monovalent and divalent concentrations.

3. RESULTS AND DISCUSSION

3.1. Physical and Chemical Properties

The sedimentary facies, variation of physical and chemical properties with depth at Kasukabe and Kameido sites are shown in Figures 2 and 3, respectively. For both sites, the sediments were deposited under either freshwater or seawater respectively. Based on the previous studies (Miyachi et al., 2004; Nakanishi et al., 2011) sedimentary facies, the sediments for Kasukabe site at the depth ranging from 7 to 27 m and Kameido site at the depth below 9m were classified as marine sediments.

The marine sediments in Kasukabe showed higher natural water content and void ratio as compared to those for freshwater sediments. The natural water content and void ratio in Kameido increased with depth. The marine sediments at both sites generally showed higher natural water contents than liquid limits, resulting in higher liquidity index more than 1.0. The silt and clay are dominant for the sediments at Kasukabe site, while the shallow samples (less than 18 m) at Kameido site contained more than 15% of sands. The pH values for Kasukabe and Kameido samples varied from 6.5 to 9.4 and 8.2-9.2, respectively. Especially, the marine sediments for both site showed higher pH value. In addition, relatively higher EC values more than 50 (mS m^{-1}) were observed for marine sediments at Kasukabe site as compared to fresh-water sediments below 27 m.

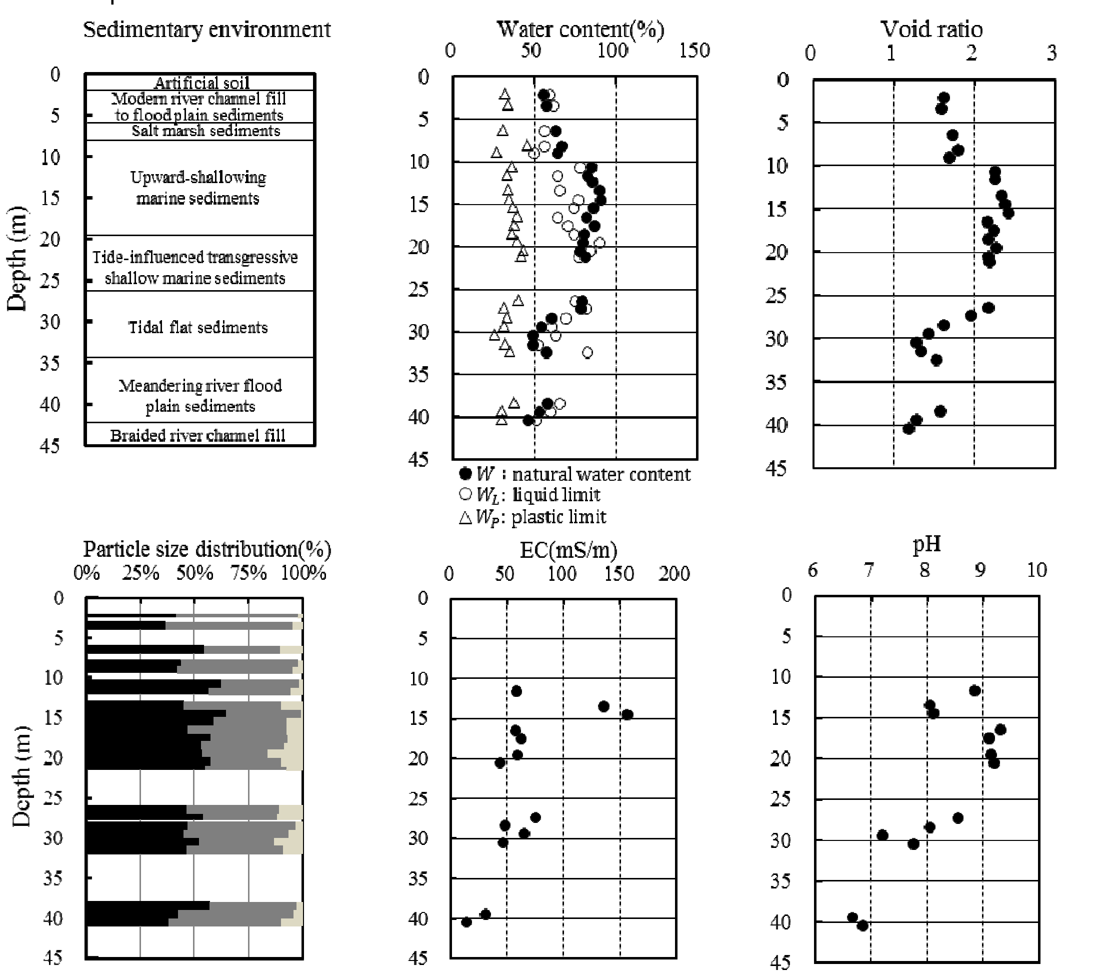

Figure 2 Sedimentary facies, variation of physical and chemical properties with depth at Kasukabe.

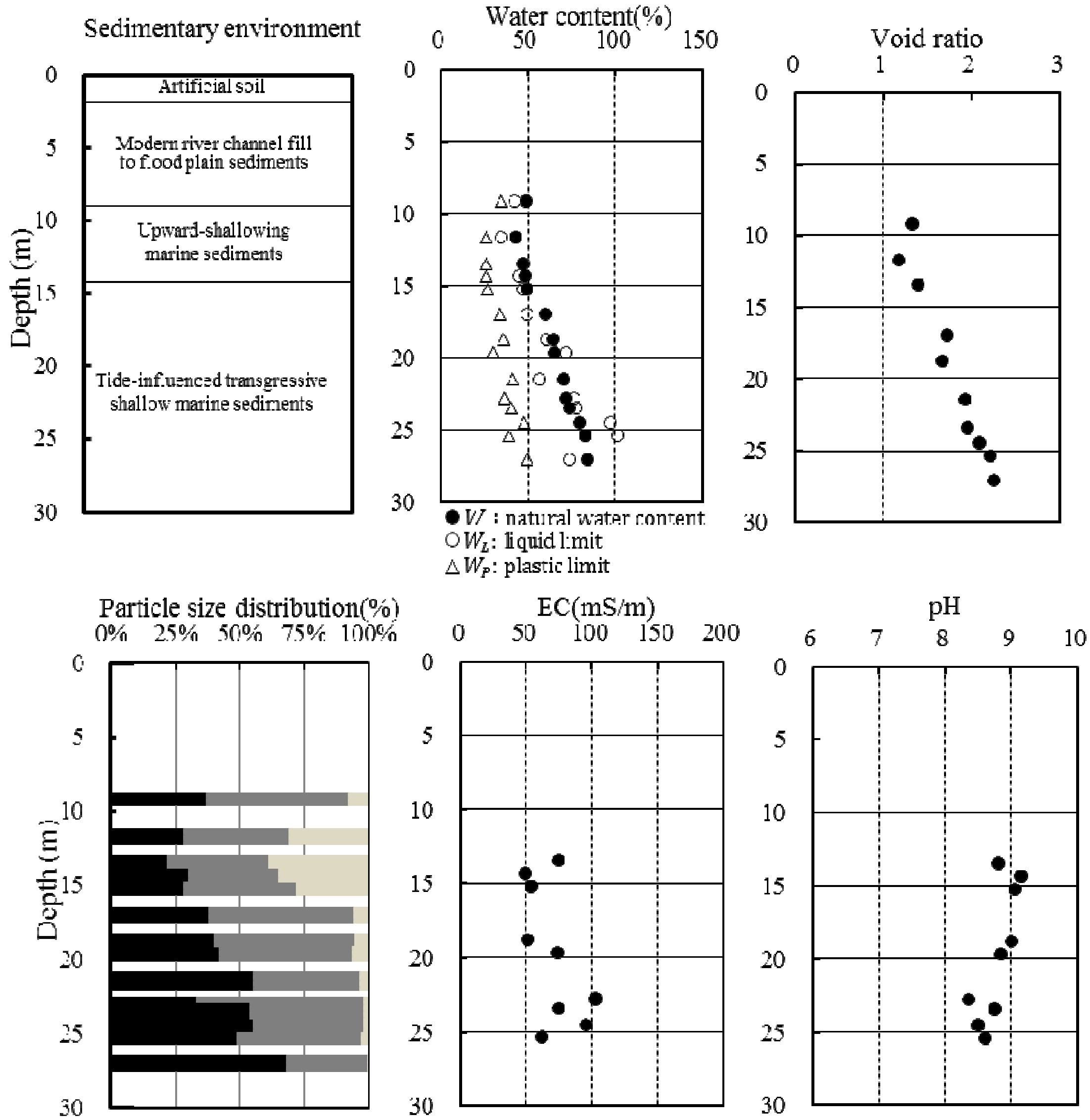

Figure 3 Sedimentary facies, variation of physical and chemical properties with depth at Kameido.

Figure 4 shows pore-water chemistry in the Kasukabe and Kameido sediments. For cation concentration profiles at both sites, Na^+ showed highest percentage in the pore water, followed by Ca^{2+}, K^+, and Mg^{2+}. The ion concentrations, especially Na^+ concentration, in marine sediments were higher than those for freshwater sediments, supporting higher EC values for the marine sediments. For anion concentration profiles, higher SO_4^{2-} values than Cl^- were observed for the sediments at both sites, suggesting that the oxidation of the pyrites (FeS_2) which are generally contained in the marine sediments caused the formation of sulphuric acid. To support this, a clear relation between SO_4^{2-} and pH was observed for the marine sediments. In addition, the average Cl^- concentration (0.01 N) at both sites were around 1/50 of the one for the sea water, indicating the marine sediments were affected by the leaching of the pore water.

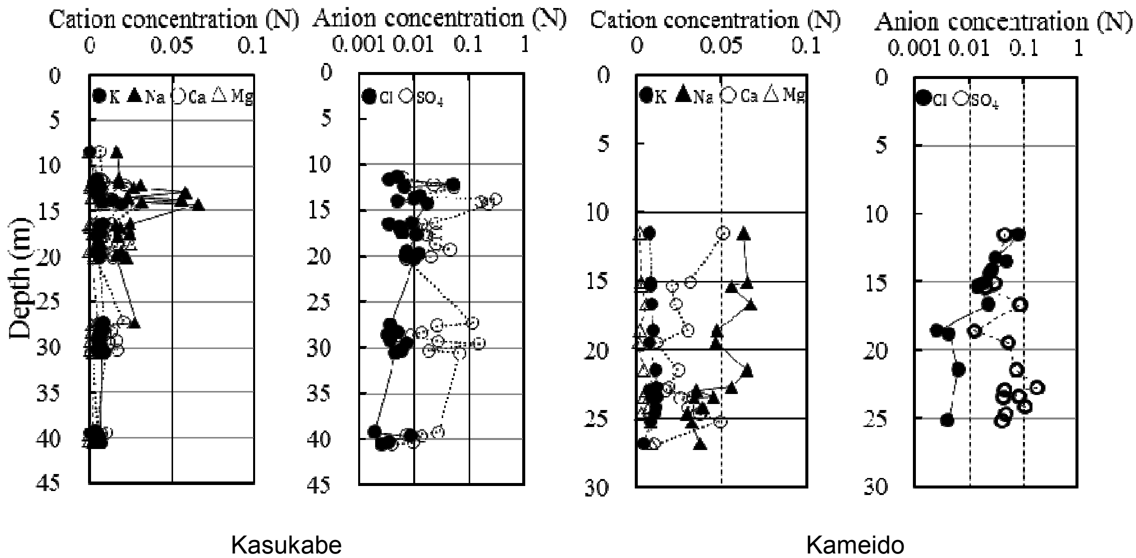

Figure 4 Pore-water chemistry in the Kasukabe and Kameido sediments.

3.2. Consolidation Properties

Figure 5 shows measured consolidation index (C_c) with depths for the sediments at both sites. As a literature data (Yamamoto, 2000), the C_c data at Oshima (a site around 2 km far from Kameido) was also added into Figure 5. The C_c at freshwater sediments was around 0.5, while the marine sediments exhibited significant higher C_c values of around 1.0. The results indicate that well-developed fabric likely represented by edge-to-face orientation of clay particles is formed for marine sediments due to the sedimentation under high salinity.

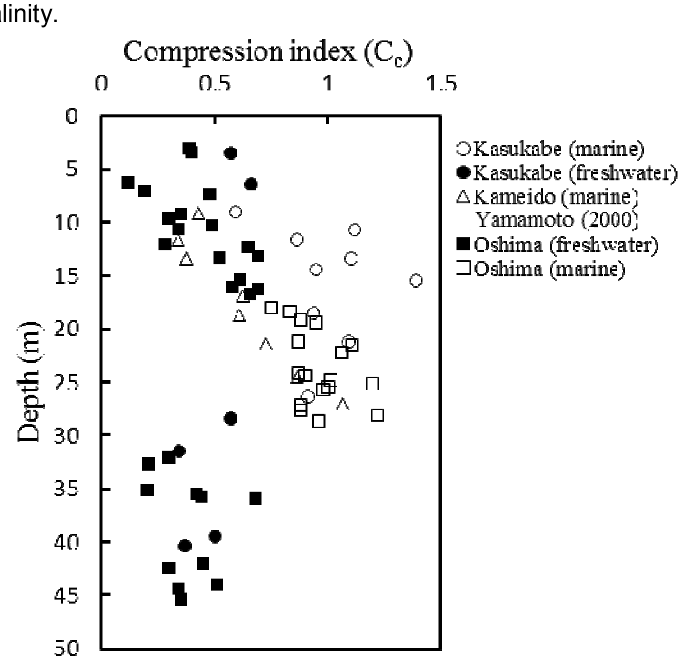

Figure 5 Consolidation index (C_c) with depth.

3.3. Shear Strength Properties

3.3.1. Undrained triaxial compression test ($\overline{CU}$)

Figure 6(a) shows the deviator stress and excess pore-water pressure against an axial strain for the marine (20 m depth) and freshwater (28 m depth) sediments at Kasukabe site. As examples shown in Figure 6(a), the excess pore-water pressure relative to the maximum deviator stress more significantly increased for the marine sediments as compared to one for the freshwater sediments. To support this, Skempton's pore water pressure coefficient (A_f) was generally higher for the marine sediments, showing A_f values more than 0.70. On the other hand, the A_f values for the freshwater sediments were less than 0.65. Figure 6(b) shows relation between undrained shear strength and consolidation pressure for the sediments at Kasukabe site. Undrained shear strength was increased with increasing consolidation pressure. The shear strength ratio was higher for the freshwater sediment than marine sediment, exhibiting the ratio of 0.56 and 0.36 for the freshwater and marine sediments, respectively. Again, this difference in shear strength characteristics in freshwater and marine sediments is probably attributed to the difference in the soil structure for each sediment where the marine sediments have well-developed more porous soil structure.

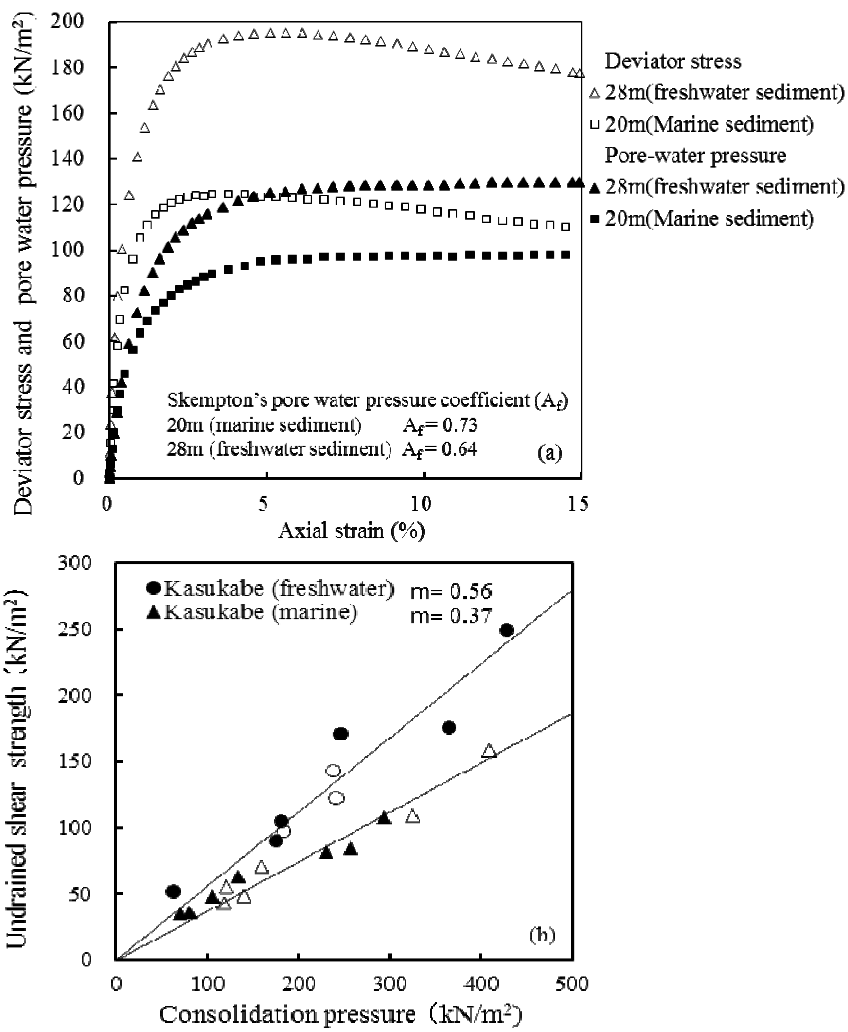

Figure 6 (a) Deviator stress and excess pore-water pressure against an axial strain and (b) Shear strength ratio for Kasukabe sediments. Open plots in Fig. 6(b) represent the samples which were consolidated at 1.5 to 3.0 times of in-situ effective stress.

3.3.2. Unconfined compression test

Measured unconfined compression strength for the undisturbed sediments and calculated sensitivity, which is the ratio of undisturbed to remolded strength at the same water content, for the sediments at both sites with different depths are shown in Figures 7(a) and 7(b), respectively. Unconfined compression strength increased with depth. Hence, the effects of sedimentary environment on the unconfined compression strength was not clearly observed. However, higher sensitivity values were observed for the marine sediments. Especially, the marine sediments at Kasukabe site with higher silt and clay contents than those at Kameido site showed significantly higher sensitivity values.

Figure 8 shows the relation between sensitivity and proportion of cations in the pore water. The sensitivity of the sediments with plastic index of less than 30 at both Kasukabe and Kameido sites increased with increasing the proportion of monovalent cations. Higher sensitivity for the sediments with higher proportion of monovalent cations suggests that degree of repulsion between clay particles, highly affected by pore-water composition, contributed to the compression strength for the remoulded specimens.

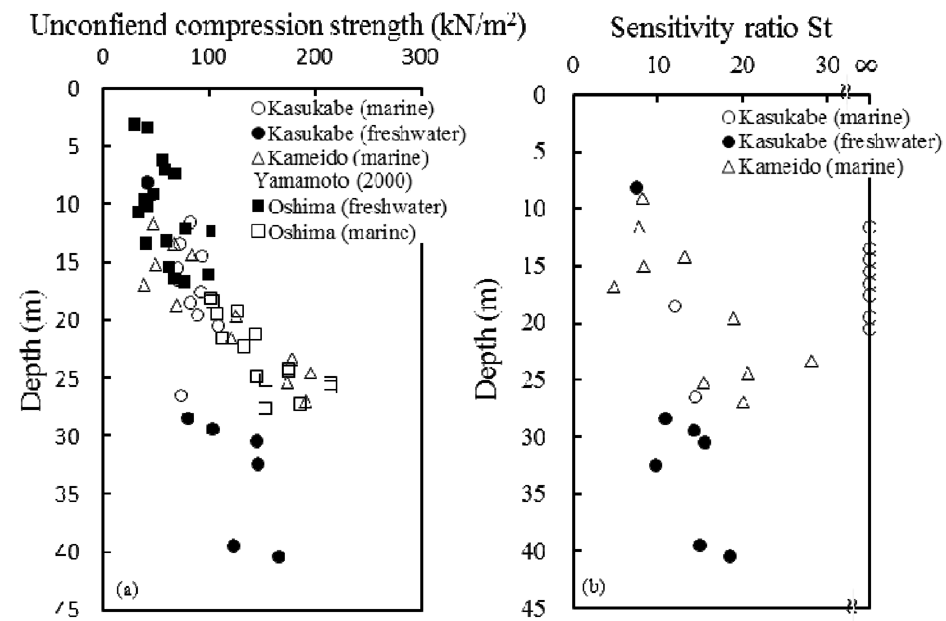

Figure 7 (a) Unconfined compression strength and (b) sensitivity with depth.

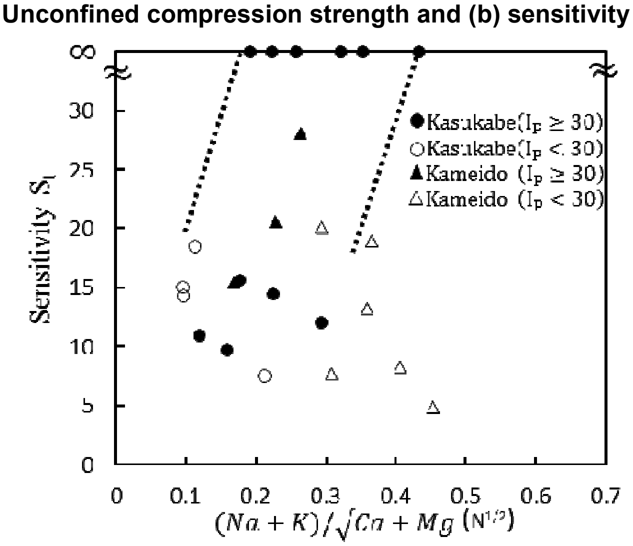

Figure 8 Sensitivity as a function of $M_0/\sqrt{D_0}$

4. CONCLUSIONS

In this paper, geotechnical properties of alluvial clays affected by sedimentary environment are characterized with geochemical investigations of pore-water compositions. The findings are as follows;

(i) Higher C_c values were observed for the marine sediments than those for fresh-water sediments at Kasukabe and Kameido sites.

(ii) Shear strength ratio was higher for the freshwater sediment. The difference in the fabric of the marine and freshwater sediments might affect the compressibility and shear characteristics.

(iii) Unconfined compression strength for the undisturbed samples increased with depth and the effects of sedimentary environment on the compression strength were not clearly observed.

(iv) Sensitivity for the marine sediments was higher than freshwater sediments. In addition, the sediments with higher proportion of monovalent cations and plastic index of less than 30 showed higher sensitivity values for both Kasukabe and Kameido sites.

5. ACKNOWLEDGMENTS

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Heat and Gas Transport Characteristics in Differently-Decomposed Peaty Soils at Variably-Saturated Conditions

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Abstract: *Knowledge of heat and gas transport properties of the peat soils is important to simulate greenhouse gas behaviour and changes of soil-temperature in the wetlands. In this study, the heat and gas transport properties for differently-decomposed and variably saturated peat soils were measured in order to investigate the general analogies and differences between transport properties of the peat soils. As results, changes in liquid-phase tortuosity under different moisture conditions and volume shrinkage under dry conditions did not significantly affect the thermal conductivity, showing linear increase of thermal conductivity with moisture content. On the other hand, marked effects of soil-water blockage and volume shrinkage on the soil-gas diffusion coefficient under wet and dry conditions, respectively, were observed.*

Keywords: *Wetland, Peat soil, green house gas, gas transport, heat transport.*

1. INTRODUCTION

Humus-rich peaty soils in wetlands are known as one of the major sources of global carbon which emits the greenhouse gases to the atmosphere (e.g., Alm et al., 1999; Pilegaard et al., 2003). Knowledge of gas transport characteristics in differently-decomposed and variably textured peaty soils is important for simulating the emissions of the greenhouse gases, especially methane, from the wetlands (e.g., Alm et al., 1999; Pilegaard et al., 2003). In addition, since soil temperature in the wetlands is a key factor to control the microbiological and chemical processes, better understandings of heat transport characteristics in the peaty soils are required for accurate simulations of greenhouse gas production and emissions. Unique physical characteristics of peaty soils such as high organic matter content, high total porosity and volume shrinkage may influence various transport properties of peaty soils. Objectives of this study are to measure the heat and gas transport properties for peat soils at variably saturated conditions and discuss the possible analogies and differences between various transport properties of peat soils.

2. MATERIALS AND METHODOLOGY

Soil samples for this study was obtained from the Hokkaido Bibai marsh, Japan (43°19’ N, 141°48’ E). Undisturbed peat samples were taken from three different sites in Bibai marsh at different depths using 100cm^3 cylindrical cores (i.d. 5.01cm, length 5.11cm). Peat 1 samples were sampled inside the marsh area, while Peat 2 samples were sampled from the area nearby a drainage ditch surrounding the marsh. Peat 3 samples were obtained from forested area located outside the wetland. Basic soil physical and chemical properties are tabulated in Table 1.

Table 1 Physical and chemical properties for soil samples.

	Depth	Layer	Water Content	Particle density	Dry bulk density		Porosity				
					Saturated	Air dried		LOI	SOC	C/N	pH†
	cm		%	g cm^{-3}	g cm^{-3}	g cm^{-3}	cm^3 cm^{-3}	%	%		
Peat 1	0	H1	942	1.42	0.07	0.09	0.96	97.00	44.60	48.00	5.8
	5	H2	1233	1.44	0.05	0.12	0.94	91.10	65.70	67.00	5.6
	10	H2	1211	1.42	0.07	0.08	0.93	89.34	60.60	28.00	3.1
	15	H3	854	1.65	0.10	0.16	0.92	74.50	29.60	17.00	4.6
	20	H3	573	1.49	0.16	0.26	0.86	36.17	33.30	19.00	5.1
	30	H3	591	1.37	0.16	0.23	0.92	47.54	36.50	26.00	4.8
Peat 2	0	AH	124	2.05	0.31	0.39	0.85	55.94	66.09	33.17	4.3
	10	AH	282	2.63	0.17	0.24	0.94	46.55	89.70	41.75	4.5
	20	H1	699	1.86	0.09	0.17	0.95	95.25	72.89	54.46	4.6
	30	H2	959	1.70	0.09	0.12	0.95	95.70	86.70	84.74	3.5
	40	H2	922	1.44	0.08	0.10	0.91	94.37	86.63	81.28	4.6
	50	H2	954	1.80	0.09	0.16	0.94	94.61	72.82	85.02	4.3
	60	H2	762	1.51	0.10	0.16	0.91	97.72	71.28	90.77	4.8
Peat 3	0	H1	81	1.93	0.24	0.36	0.92	48.00	31.27	28.59	4.3
	10	H2	315	1.58	0.14	0.34	0.92	32.80	52.62	27.00	4.1
	20	H2	488	1.49	0.10	0.19	0.90	94.70	50.87	25.67	4.1
	30	H2	720	1.54	0.09	0.19	0.92	91.23	64.78	28.79	4.6
	40	H2	833	1.52	0.09	0.20	0.94	93.33	81.24	23.83	5
	60	H3	953	1.48	0.08	0.15	0.93	84.53	81.64	28.55	4.8
	90	C1	419	2.08	0.27	0.38	0.90	52.10	43.37	24.91	4.6
	100	C1	277	2.04	0.45	0.62	0.91	24.80	27.26	25.21	4.5
	120	C1	196	2.32	0.19	0.28	0.95	27.00	28.33	24.40	4.6
	150	C2	71	2.64	0.75	1.62	0.71	8.40	3.23	16.26	5.1

† pH was measured directly by using extracted soil solution.

The peat samples were initially saturated and subsequently drained using two different methods corresponding to the matric suction ranges. A hanging water suction method was used for low matric suctions up to pF 2 (- 100 cm H_2O) and a pressure plate apparatus for medium suctions (pF 2 to pF 4, i.e., - 100 cm H_2O to -10000 cm H_2O). The sample cores were then oven-dried at 30^{0}C for three days and kept in a climate controlled room at 25^{0}C and a relative humidity of 60% for three days to reach air-dry condition (Resurreccion et al., 2008). The thermal conductivity (λ) and gas diffusion coefficient (D_p) were measured at different soil moisture suction levels. The thermal conductivity of the samples was measured by using Decagon KD2-Pro probe. The gas diffusion coefficient was measured using a chamber method where oxygen was used as a tracer gas.

3. RESULTS AND DISCUSSION

3.1. Water retention characteristics and volume shrinkage during drying

Figure 1 shows the changes in the total porosity (Φ), volumetric water content (θ), and volumetric solids content (σ) for Peat 1-3 samples as a function of pF at different depths. All soils except the surface layers (i.e., to 10 cm depth), exhibited high water retention characteristics up to pF 1.8 where around 60-80% of water saturation was still retained. This characteristic clearly indicates a formation of a well-developed organic matrix with micro-pore structure that increased with increasing degree of decomposition. Most peat soil samples showed a decrease of Φ and increase of σ with increasing pF after around pF 1.8 due to volume shrinkage of the samples during drying. Most of the samples showed peaks in θ at around pF 2 indicating a reduction in total soil volume (and thus increase in σ) caused a relative increase in θ, although soil-water for each sample drained with increasing pF at the range of pF 1.5-2.0.

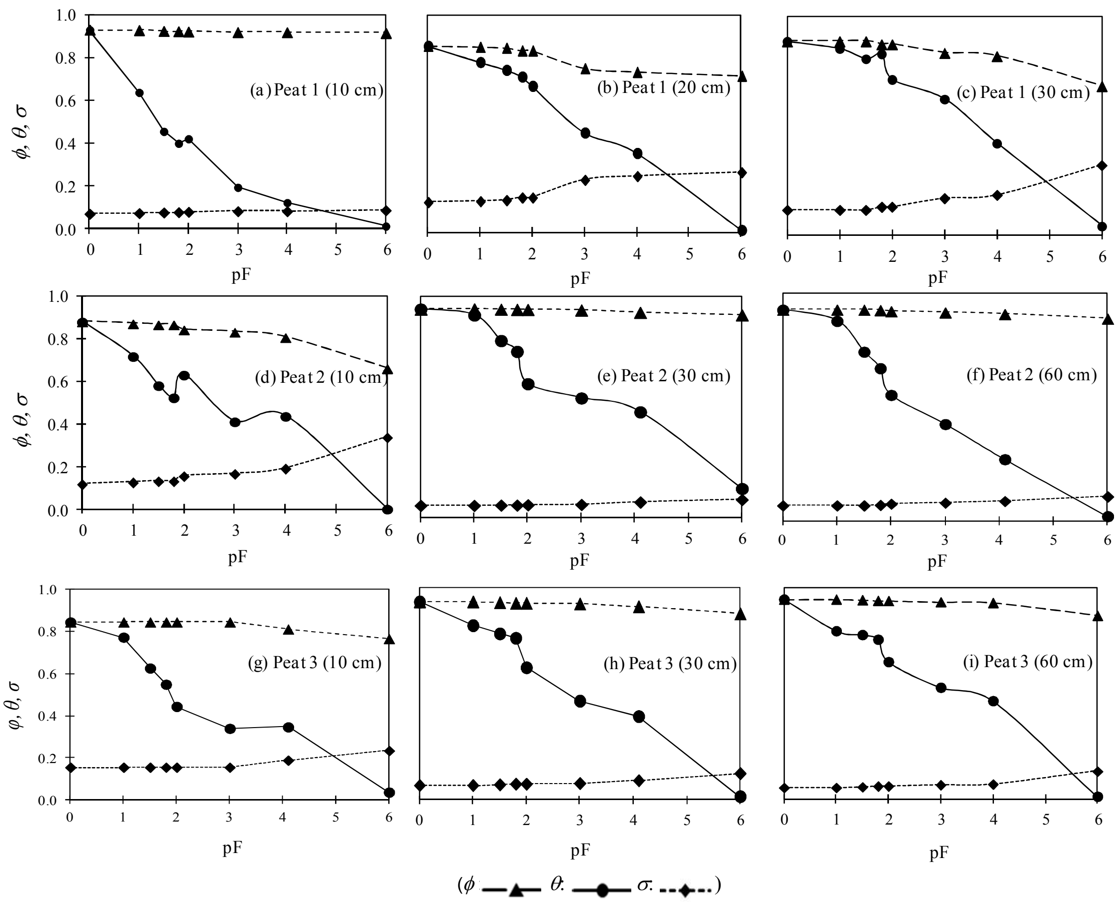

Figure 1 Total porosity (ϕ), volumetric water content (θ), and volumetric solid content (σ) for Peat1, Peat 2, and Peat 3 samples as a function of pF (= log $|\psi|$, where ψ is in cm H_2O).

3.2. Measured thermal conductivity and gas diffusion coefficient of peat soil under variable-Saturated conditions

Figure 2 shows the measured thermal conductivities (λ) and gas diffusion coefficients (D_p) for three different peat soil samples as a function of volumetric water content (θ) and soil air content (ε), respectively. Both transport properties, λ and D_p showed increase with increasing fluid content (i.e., soil water content or soil air content, $m^3\ m^{-3}$).

Thermal conductivity (λ) of the mineral soils is rapidly increasing with increasing θ especially at dry conditions due to the improvement of thermal contacts between soil particles by soil water (Becker et al., 1992; Hamamoto et al., 2010). A linear increase of λ with increasing θ was observed in all three peat soils. This suggests that water content and liquid-phase tortuosity are the most controlling factors for the λ, as supported by the fact that the thermal conductivity of organic matter (0.25 $Wm^{-1}K^{-1}$) is much lower than that of water (0.57 $Wm^{-1}K^{-1}$) (de Vries, 1963; Brovka & Rovdan, 1999). The Thermal conductivity for Peat 3 at deeper layers showed higher values than those for other peat types. Difference in mineral composition in the deeper layers might affect this characteristic behaviour as partially expected by lower loss on ignition (*LOI*) values and higher particle density (ρ_s) values of the soil samples.

The D_p rapidly increased at high ε, indicating gas transport in soils is highly enhanced through well-connected, large-pore networks under dry conditions (Hamamoto *et al*., 2010). In addition, the D_p data clearly suggests the existence of percolation thresholds which is inactive air-filled pore space due to soil-water blockage.

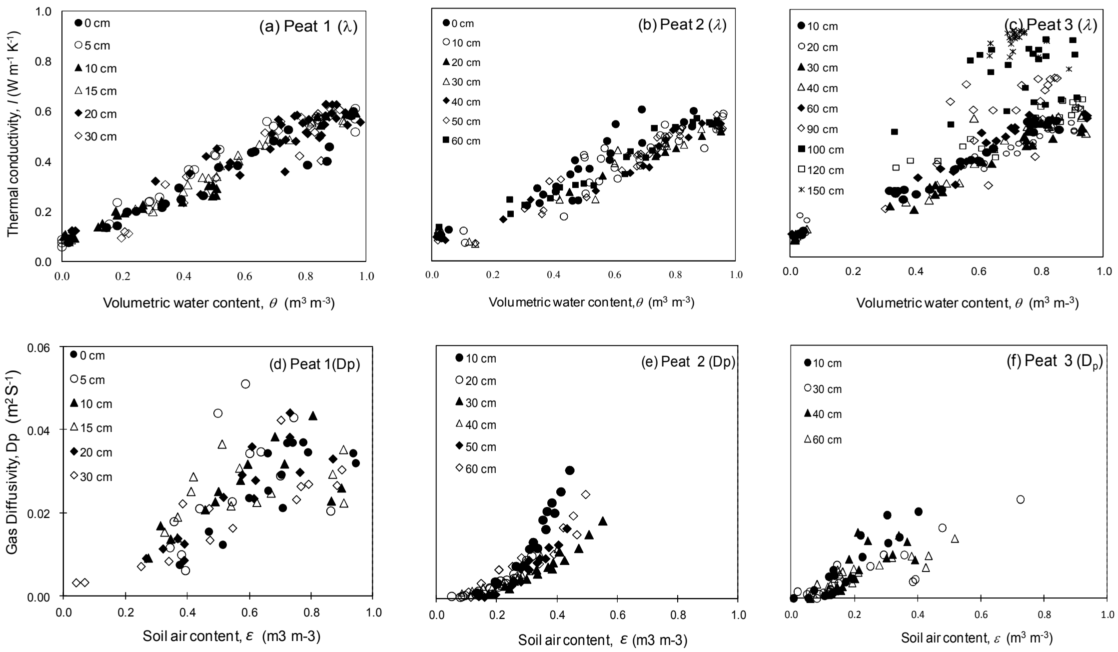

Figure 2 Thermal conductivities (λ) and gas diffusivities (D_p) as functions of volumetric water content (θ) and soil air content (ε) for Peat 1, Peat 2 and Peat 3 samples.

3.3. Application of Archie's Second Law with Reference Point to Thermal Conductivity and Gas Diffusivity

Under fluid-unsaturated conditions, a generalized Archie's second law (Archie, 1942) for transport parameters with reference point can be written as,

$$\frac{P}{P_{ref}} = \left(\frac{\phi}{\phi_{ref}}\right)^n \tag{1}$$

where P is the bulk parameter value under fluid-unsaturated conditions (i.e., λ and D_p), P_{ref} is the bulk parameter value at the reference point, ϕ is the fluid content (i.e., soil water content for λ and soil air content for D_p, $m^3\ m^{-3}$), ϕ_{ref} is the fluid content at the particular reference point and n is the saturation exponent representing a fluid-pore tortuosity–connectivity in porous media.

In this study, the pF 2 was selected as a reference point since the peat soils at less than pF 2 (wet condition) do not exhibit significant volume shrinkage as shown in Figure 1. Therefore, dry (region 1) and wet (region 2) regions for λ, and wet (region 1) and dry (region 2) regions for D_p were defined for matric potential regions ≤ pF 2 and > pF 2, respectively. In addition, λ at dry condition (λ_{dry}) was considered in both P and P_{ref}, giving P/P_{ref} of $(\lambda-\lambda_{dry})/(\lambda_{pF2}-\lambda_{dry})$ where λ_{pF2} is the λ value at pF 2. The n values for each region 1 and 2 were defined as n_1 and n_2, respectively.

Figure 3 shows the normalized parameter values, P/P_{ref}, for each parameter as a function of normalized fluid content, ϕ/ϕ_{ref}, (θ/θ_{pF2} for λ and $\varepsilon/\varepsilon_{pF2}$ for D_p, where θ_{pF2} and ε_{pF2} are θ and ε at pF 2, respectively). Table 2 shows the best fit n_1 and n_2 values for thermal conductivity and gas diffusivity of each peat soils at different depth.

The normalized λ values generally showed a linear trend as a function of fluid content, giving n_1 is ranging from 0.8 to 2.0 and n_2 is ranging 0.7 and 1.8. Again, this supports that λ is mainly governed by a degree of water saturation and soil-moisture dependency on liquid-phase tortuosity for thermal conductivity is small. In addition, almost similar n_1 and n_2 values (average value of 1.0) for each peat soil at different depths (Table 2) suggest less effects of volume shrinkage on the λ.

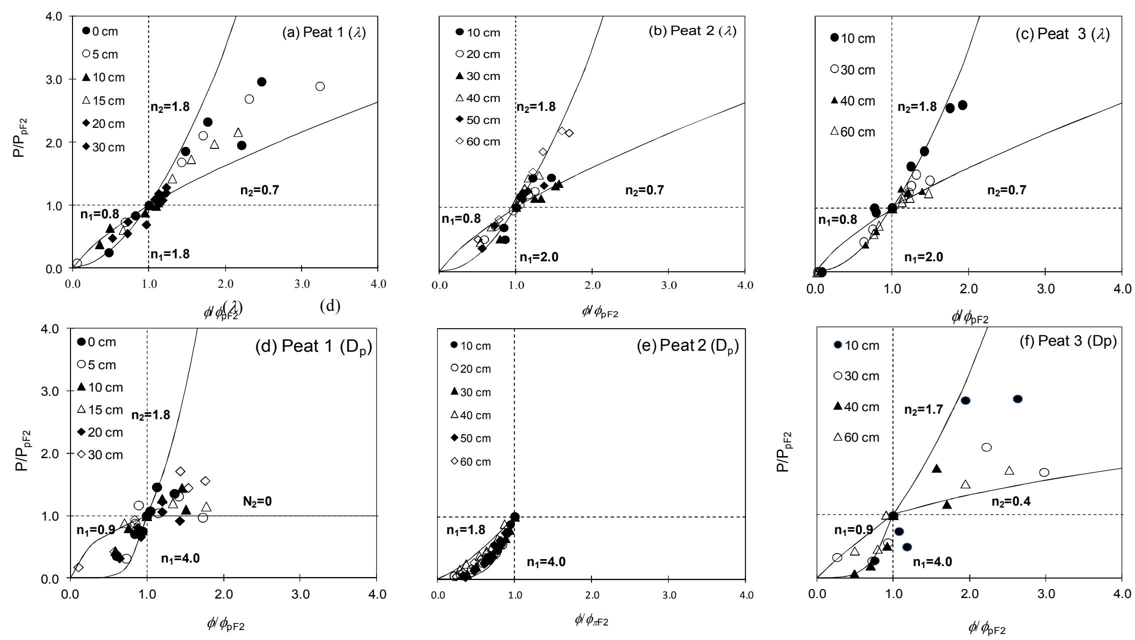

Figure 3 Normalized parameter value, P/P_{pF2}, as a function of normalized fluid content, ϕ/ϕ_{pF2}, for for Peat 1, Peat 2 and Peat 3 samples.

For D_p, the normalized D_p values showed a two-region behaviour against fluid content. Higher n_1 values ranging 0.9 to 4.0 as compared to those for the λ indicate the significant effects of soil-water blockage on gas diffusion process under dry conditions. The n_2 values for the D_p were generally lower than those for λ and more wider range of n_2 values were obtained (Fig. 3 and Table 2). The findings suggest that increase in air-phase tortuosity dramatically reduced incremental increase in the D_p under dry conditions.

Table 2 Best fit n_1 and n_2 values for thermal conductivity and gas diffusivity of each peat soils at different depth

soil		n_1		n_2	
		λ	D_p	λ	D_p
Peat 1	0 cm	1.5	3.0	1.2	1.5
	5 cm	0.8	2.0	1.1	0.3
	10 cm	0.8	1.5	1.0	0.5
	15 cm	1.5	-	1.1	0.3
	20 cm	1.3	3.0	0.9	0.0
	30 cm	2.1	1.5	1.0	1.0
Peat 2	10 cm	2.0	2.9	1.2	-
	20 cm	1.2	2.6	1.1	-
	30cm	-	2.6	0.7	-
	40 cm	1.1	1.5	2.2	-
	50 cm	1.4	2.4	1.2	-
	60 cm	0.9	2.2	1.7	-
Peat 3	10 cm	-	-	1.6	1.3
	30 cm	1.5	4.5	1.0	0.9
	40 cm	2.0	5.0	1.0	0.7
	60 cm	1.9	2.0	0.7	0.6

4. CONCLUSIONS

In this study, thermal conductivity (λ) and diffusion coefficient (D_p) for differently-composed peat soils at variably-saturated conditions were measured, and heat and gas transport characteristics were characterized by applying Archie's second law with a reference point of *pF 2* to the measured data.

Linear increase in normalized λ values as a function of fluid content for all peaty soils except for soils with high mineral contents suggested that changes in liquid-phase tortuosity under different moisture conditions and volume shrinkage under dry conditions do not significantly affect the λ behavior. On the other hand, normalized D_p values suggested marked effects of soil-water blockage and volume shrinkage on the D_p under wet and dry conditions, respectively.

The fitted *n* values by the Archie's law for the λ were ranged from 0.8 to 2.0 (on average 1.0) for dry and wet regions. In general, as compared to λ, higher and lower *n* values for the D_p under wet and dry regions were obtained, respectively. In perspective, the obtained differences in the *n* values of the λ and D_p will be associated with physical and chemical properties such as organic matter content or clay content and unified predictive models of λ and D_p considering key factors to control λ and D_p behaviors will be proposed.

5. ACKNOWLEDGEMENT

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Study of Effectiveness of Road Fund as a Solution for Maintenance Problem in Sri Lanka

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Abstract: *Transportation makes a key contribution to the economic and social development in every country. Among other transportation means roads play a considerable role with the lack of other means like rails, ships and airlines. Well-conditioned roads have many tangible and intangible benefits to all road users. Well-planned maintenance programme is an essential in this regard. There are many causes for not having effective sustainable road maintenance practices. Those are related to management, technology and finance. Among them, inadequate financing is a critical problem. Many countries depend on country's general budget even though it is not adequate. Time to time individual countries and international lending agencies have introduced off-budget financing methods. Road Funds concept was introduced by the World Bank as an off-budget method. In this thesis main purpose is to analyze effectiveness of Second Generation Road Fund and to propose whether it is desirable solution for Sri Lanka road maintenance problem. The Road Funds in developed countries, middle-income countries and developing countries were evaluated separately and final conclusion was drawn according to evaluation results*

Keywords: *Road maintenance, Road Financing, First & Second Generation Road Funds, Effectiveness*

1. INTRODUCTION

Well-conditioned road network in a country brings many tangible and intangible advantages to all road uses. To keep the roads in well-conditioned, a well-planned maintenance programme is an essential. If maintenance of road is not implemented at the right place and right time, it may cause many problems. Many countries do not have well-established maintenance plan. As a leading cause for this problem, it can be high-lighted the institutional related problems. Those are institutional framework, human resource constraints, lack of clearly defined responsibilities, inefficient management structures, weak management systems, and inadequate financing arrangement (Heggie, Ian G. and Piers Vickers,1998).
With the identified problems, for an effective road network, the solution are strengthening the management and financing of road sector. The concept is commercialization: or putting road on fee-for-service basis and managing like a business (Kenneth M. Gwilliam & Zmarak M.Shalizi). To achieve this, reforms should be done to focus on; Establishing responsibility for managing roads by clearly assigning roles, Creating ownership of roads, Strengthening management of roads, Stabilizing road financing. With this background the main purpose of this research is to study existing Road Funds (RFs) and analyse its effectiveness.

2. METHODOLOGY

Literature review was conducted to identify problems with road maintenance financing and for possible solutions. During the literature survey it was understood that Road Fund (RF) is practicing in many developing countries but no effectiveness analysis has been done to date. Then evaluation methods (both qualitative and quantitative) for RF were defined to analyze the effectiveness of implementation of

Road Funds (RFs). The indicators used to measure the quality of (RFs) were categories into three sections namely, structure, process and performance of the RF(World Highways, Vol. 9, No. 7, October 2000 "Second Generation Road Funds: The Way Ahead).
Many countries around the world responded to the growing shortage of finance by attempting to earmark selected road related taxes and charges and depositing them into a special off-budget account, or road fund, to support spending on roads (Mustapa benmaamar). The first introduced mechanism is called as the First generation Road Fund. The mechanism is, levy, duties etc are collected by the relevant ministry and deposited to the Road Fund. Those funds are supposed only to use for road maintenance work. With the deficit of the First generation Road Fund, the Second Generation Road Fund was introduced. Here funds are collected by an independent body and deposited to the Road Fund

Structure
1. Does the structure of the Road Board (RB) introduction of professional management run according to sound business practices? (**S1)**
2. Does the RB have adequate representation of road users and civil societies to encourage better management demand for efficiency and control of monopoly power? **(S2)**
3. Does the RB have a firm legal basis? **(S3)**

Process
4. Adequacy of road financing **(P1)**
5. Share of work outsourced **(P2)**
6. Stability of road financing **(P3)**
7. Performance monitoring **(P4)**

Performance
8. Impact of RF on the quality of road maintenance **(F1)**
9. RF contribution to improved operational efficiency **(F2)**
10. RF impact to improve the capacity of executing agencies.**(F3)**
11. Has the RF improved the allocation?**(F4)**

Quantitative analysis was done using road maintenance expenditure compare to Gross Domestic Product (GDP).

3. RESULTS, DISCUSSION AND CONCLUSION

3.1 Results

Second Generation Road Funds in various countries were qualitatively evaluated under defined indicators and results are shown in **Table 1**.
Using the qualitative method, effectiveness of RF in each country can be evaluated. It cannot be used as a comparison method. Therefore to compare countries, road statistics and Gross Domestic Product (GDP) data are used. It is often helpful to look at the size of the financing plan relative to GDP. Cross country comparison of this indicator can be helpful although they cannot be definitive due to unreliability of data. Evaluation results of quantitative methods are shown in **Table 2** and **Table 3.**

Table 1 Achievement of Object Indicators

Country	S1	S2	S3	P1	P2	P3	P4	F1	F2	F3	F4
Ethiopia	+	-	-	+	-	+	+	+	+	-	-
Benin	+	+	+	+	+	+	+	+	-	-	n.a
Ghana	+	+	+	+	-	+	+	+	+	+	n.a
Zambia	+	+	-	+	+	-	-	+	+	-	-
Kenya		+									
Honduras	+	+	+	n.a	+	+	n.a	n.a	n.a	n.a	n.a
Guatemala	+	+	+	n.a	+	+	n.a	+.	n.a	n.a	n.a

+: there is an improvement after Road Fund establishment, -: there is no improvement even after Road Fund establishment, n.a: no information available

Table 2 Developed countries: Road expenditure on roads as proportion of GDP

		Average of Total Road Expenditure/GDP % (1970-2003)	Average of Maintenance Expenditure/GDP % (1970-2003)	Average of Maintenance Expenditure/GDP % (1990-2003)
Developed Countries with Road Fund	Switzerland	2.20	0.67	0.1
	USA	1.63	0.42	0.25
	New Zealand	1.30	0.65	0.74
	Japan	2.69	0.39	0.36
		Average= 1.96	**0.53**	**0.36**
Developed Countries without Road Fund	Finland	1.23	0.88	0.78
	Germany	1.58	0.16	no data
	Ireland	1.40	0.51	0.21
	Italy	1.55	0.59	0.48
	Denmark	1.47	0.56	0.39
	Spain	0.69	0.18	0.22
	Sweden	1.06	0.61	0.55
	Canada	2.06	0.72	0.27
	Norway	2.25	1.71	0.52
	Korea, Rep	1.70	0.19	0.2
	Australia	1.98	0.54	no data
		Average= 1.54	**0.60**	**0.40**

Source: Road expenditure data are from World Road Statistics, IRF and GDP data from World Bank Statistics

Table 3 Developing and middle income countries: maintenance expenditure on roads as proportion of GDP

		Average of Maintenance Expenditure/GDP % (1970-2003)	Average of Maintenance Expenditure/GDP % (1990-2003)
Developing Countries with Road Fund	Hungary	1.35	0.17
	Latvia	1.44	1.44
	Romania	0.20	0.20
	Ethiopia	0.24	0.14
	Benin	0.25	0.16
	Yemen	1.62	1.86
	Ecuador	0.22	0.22
	Ukraine	0.44	0.44
	Cameroon	0.35	no data
	Madagascar	0.30	no data
	Mali	0.27	no data
		Average= 0.61	**0.58**
Developing Countries without Road Fund	Sri Lanka	0.0569	0.0031
	Pakistan	0.26	0.2254
	Albania	0.0542	0.0542
	Croatia	0.1741	0.1626
	Bolivia	0.5172	0.4833
	Mongolia	0.1344	0.1344
		Average= 0.1995	**0.18**
Middle-income Countries without Road Fund	Thailand	0.21	0.21
	Philippines	0.29	0.14
	Poland	0.52	0.51
	Turkey	0.13	0.13
	Chile	0.20	0.42
	Brazil	0.10	0.014
	Hong Kong	0.49	0.52
	Malaysia	0.36	no data
	Argentina	0.13	no data
	Mexico	0.15	0.13
		Average= 0.26	**0.26**

Source: Road expenditure data are from World Road Statistics, IRF and GDP data from World Bank Statistics

3.2 Conclusions and Recommendations

Even though First Generation RF was established to ensure stable and adequate financing, still it is under the Ministry of Finance. So Ministry of Finance can use it as its general budget. Agency model RB is not a financing mechanism. Post-war RFs are similar to Second Generation RF and performing well. There is a trend in developing countries to practise Second Generation RF.
RF to be more effective following proposals can be given.

(i) Fuel levy should be set up to meet road expenditure.
(ii) Direct transfer of revenue from petroleum enterprises to RF is the most desirable way.
(iii) Resource allocation should not be based on ‘standard formula’ but according to necessity.

(iv) Surplus of RF can lead to many criticisms by other sectors. This should be considered when setting fuel levy.

Considering quantitative analysis, road spending is increasing with increasing GDP for developed countries and middle income countries. It does not show significance of road fund on maintenance expenditures. For developing countries there is no clear relationship between maintenance expenditures and GDP. But average of spending on maintenance as a proportion of GDP is high in developing countries with RF than that of without RF. According to the analysis, the countries having good road networks are spending 0.36% of countries GDP value for countries with Road Fund and 0.40% of countries GDP value for countries without Road Funds. Considering these results and road conditions, RF should be able to spend at least about 0.4% of country's GDP value.

Finally it can be concluded that establishment of a RF is effective in every countries but RF alone cannot take all responsibilities to improve the road sector at the initial stage. With parallel to stabilizing financial arrangement, other problems related to management of road sector also should be addressed with establishing responsibility for managing roads.

It is very clear that inadequate financing for maintenance is a critical problem faced by Sri Lanka. Spending on road maintenance is about 0.0031% as proportion of country's GDP in Sri Lanka. With the deficit budgetary arrangement, it is hard to achieve money from the general budget. Therefore it is essential to look for an off-budget financing method.

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Estimation of Pedestrian walking speeds at controlled cross walks in Sri Lanka - a pilot study

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Abstract: *This study aims to develop and test a methodology to extract pedestrian speed data from video footage and to find the pedestrian walking speed Sri Lankans. This study was done in the premises of the Faculty of Engineering, University of Ruhuna. Pedestrians were walking across a measured length was videotaped and the resulting footage was analysed after breaking the footage into frames. For each pedestrian walking event sex of the subject and the starting and ending clock times were recorded. Results showed that sex of the pedestrian did not have a significant effect on the walking speed of the pedestrians. Further, results showed that test subjects walked with a 15th percentile speed of 1.20 m/s. This walking speed is as par with the USA walking speed and faster than the Singaporeans' walking speed when compared with the speeds reported in the previous studies. However it is noted that this walking speed may go down if we include more test subjects from elderly age category.*

Keywords: *Pedestrians Walking speed, Pedestrian crossing, Speed data from*

1. INTRODUCTION

Pedestrian walking speed at a cross walk is an important and essential parameter in intersection traffic design. Pedestrian walking speed determines the pedestrian crossing time. 15th percentile of the pedestrian walking speed distribution is used in the signal designs as the design pedestrian walking speed. For a safe crossing, the "Walk" phase in a signal cycle must be at least equal to the pedestrian crossing time based on the design pedestrian walking speed. In related literature design pedestrian walking speeds are suggested.

Manual on Uniform Traffic Control Devices (MUTCD) for Streets and Highways (Federal Highway Administration 2009) published by the Federal Highway Administration USA, suggest a speed of 1.2 m/s (4 ft/s) as the appropriate pedestrian speed at cross walks. This speed of 1.2 m/s (4 ft/s) was first introduced in the 1961 version of MUTCD for streets and highways. According to LaPlante and Kaeser (LaPlante & Kaeser 2007) the speed of 1.2 m/s is based on an unpublished study by Exnicios in 1952 (Exnicios 1952). However, many studies have questioned this suitability of this speed and different pedestrian walking speeds have been found. A 1996 study by Knoblauch (Knoblauch et al. 1996) found that a 1.19 m/s speed is appropriate for younger pedestrians while 0.91 m/s was suggested for older pedestrians. A 1999 study by Young has found pedestrian walking speed to have a mean of 263.26 m/min with a standard deviation 52.49 m/min. A 2001 study by Tarawneh (Tarawneh 2001) showed that the 15th percentile pedestrian walking speed for general public in Jordan is 1.11 m/s and 0.97 m/s for old age people. A 2005 study by Gates et al. (Gates et al. 2006) using 1,947 pedestrian crossing events measured at eleven intersections in Madison and Milwaukee, Wisconsin, recommended pedestrian walking speed values base on percentage of old age pedestrians. According to Gates et al. when the proportion of pedestrians over the age of 65 exceeds 20, 30, 40, and 50 percent of the total pedestrians at a location, walking speeds of 3.6, 3.5, 3.4, and 3.3 ft/s, respectively, are recommended as design pedestrian walking speed. A 2006 study by Fitzpatrick et.al (Fitzpatrick et al. 2006) showed that for the general public; a pedestrian walking speed of 3.5 ft/s and for older pedestrians walking speed of 3.0 ft/s is suitable. In 2011, the Public Rights-of-Way Access Advisory Committee of USA (Public Rights-of-Way Access Advisory Committee 2011) has released draft guidelines for public comment. For pedestrian signal phase timing, maximum pedestrian walking speed of 1.1 m/s (3.5 ft/s) was recommended.

In Sri Lanka, there is no clear national guideline on pedestrian walking speeds. Instead, signal designs in Sri Lanka depend on foreign guidelines originated from USA or UK. Being a developing country in tropics, Sri Lankan road environment is very different from USA or UK. Further, Sri Lankan pedestrians' ergonomics and reason for crossing might be different from those of USA and UK. Therefore, suitability of guidelines originated in USA or UK to Sir Lankan conditions is questionable. For example a study by Tanaboriboon et al (1986) has found the Singaporeans walk slower than the Americans or UK citizens.

Need for a national guideline on pedestrian walking speed was not felt in the past because of very limited installations of traffic signals in Sri Lanka. At present, with the economic growth, the number of traffic signal installation in Sri Lanka is in the rise. Therefore, the need for a national guideline on pedestrian walking speed is also increases. This intern derives the need to find the pedestrian walking speeds for Sri Lankan pedestrians under prevailing conditions. Further we need to study other related pedestrian behaviors in the Sri Lankan context. Therefore we need a comprehensive study on pedestrian behavior. This paper presents a part of such a study. There are two objectives for this part of the study they are: (1) to determine pedestrian crossing speed in Sri Lanka; and (2) to find the effect of gender on pedestrian walking speed. The following text is organized into three chapters; chapter 2 gives the methodology followed by results and conclusions.

2. METHODOLOGY

A 5 meter level stretch of a road inside the faculty of engineering was selected for the study as the test stretch. 5 meter represents the typical width of the roads in Sri Lanka. Start and the end points of the test stretch were marked with temporary markers. It was made sure that the marks are clearly visible in the final video. Student of the Faculty of Engineering leaving the lecture rooms were chosen as the test subjects. Test subjects were not informed of the test so that they will not behave differently in front of the camera. For the same reason above, camera was placed so that it is not prominently visible to the test subjects.

Students were videotaped using a digital camcorder with PAL video system. PAL video system will yield a video with 25 frames per second. This will give a 1/25 s accuracy in the time measurement. Video was kept at an angle to the test stretch. Keeping at an angle will induce some parallax error; it is believed that parallax error can be eliminated using legs of the test subjects as the reference point. Audio Video Interleaved format produced by Microsoft Inc was used as the video format. Multiple pedestrian crossing events were videotaped at single stretch. Maximum time for a single stretch was limited to 10min so that the resulting video file will be easy to handle.

Table 1 Sample data collected for this Study

Initial Time	Final Time	Time Duration (s)	Speed (m/s)	Gender
00.01.06	00.04.14	3.13	1.597	F
00.10.08	00.15.05	4.95	1.010	M
00.11.21	00.15.13	3.87	1.292	F
00.15.16	00.20.10	4.9	1.020	M
00.17.04	00.21.06	4.03	1.241	M
00.24.19	00.28.25	4.1	1.220	F
00.34.11	00.38.05	3.9	1.282	F
00.38.00	00.42.16	4.27	1.171	M
00.38.00	00.41.28	3.47	1.441	M

The resulting video was analyzed using Ulead© video editing software. This software breaks the video in to 1/100 s, thereby increasing the accuracy of the time measurements. For each pedestrian crossing event time of start, time of end will be collected in addition sex of the test subject was also recorded. A Sample data collected for this study is shown in Table 1. Here the third column speed is calculated as the distance (5m) divided by the time duration (s).

2.1. Data Analysis

Data was analyzed using a non parametric approach. A non parametric model was developed for the data. SPSS package called Clementine was used to develop the model. All statistical significance will be evaluated at a P value of 0.05. Non parametric model called HTBRM was used to model the speed sex relationship. At this stage data for 50 pedestrian crossings were collected. 'Sex' was the only independent variable. There were 34 male and 16 female pedestrian crossing events. Basic statistic of the data used for the analysis is shown in Table 2.

Table 2 Basic statistic of the data used for the analysis

	Speed (m/s)				Number
	Average	Std.Dev	Max	Min	
Female	1.31658	0.179166	1.59744	1.07991	16
Male	1.32960	0.187716	1.67785	0.95785	34
All	1.32543	0.183291	1.67785	0.95785	50

3. RESULTS

Results of this study showed that pedestrians in Sri Lanka walked with a mean speed of 1.3m m/s having standard deviation of 0.183 m/s. From this the 15th percentile data was calculated to be 1.20 m/s.

Several HTBRM models such as CART, CHAID were tried on the models. However the HTBRM models did not show any significant relationship between the sex and speed.

4. DISCUSSION AND CONCLUSIONS

According to the T-test (p=0.40) and HTBRM model results, test data showed that males and females walked at a similar speed. However it must be pointed that test subjects in this study were university students with in the age of 22-26, this may have some effect on the walking speed. To get the real situation we have to include all other age groups as well. In future study it is intended to include more explanatory variables such as dress type, shoe type, weather, presence of police officer, walking with a peer, age, sex, carrying stuff etc. it is believed all of this data can be obtained from the videotape itself without disturbing the pedestrians.

Study showed that 15h percentile speed of Sri Lankan test subjects to be 1.20 m/s. This value is equal to the speeds recommended by MUTCD and a 1999 study by Milazzo et al (1999) recommended a same value. Mean walking speed of Sri Lankans is greater than that of reported in a 1986 study by Tanaboriboon and & Chor.

Using the digital video camcorder and Ulead© software we were able to obtain the speed of the pedestrians to a 1/100s high accuracy. No major difficulties were encountered in the process. Extracting speed data manually from video is time consuming however it can be done with hi precision required for this study. It is expected that the presence of zebra crossings will ease the distance measurement. With the experience gained from this pilot study it is believed that we can use the same procedure to get real life data from the field.

This study has produced walking speeds of pedestrians in Sri Lanka. However, it is very much premature to recommend any guideline from this study; main reason is being the limited number of test subjects used in this study. It is believed that inclusion of a larger data set will improve the reliability of the results obtained from this study.

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User comfort on urban roads

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Abstract: *Urbanization was one of the movements taken by man to impose his super powers on the natural world to make his life more comfortable, ignoring that the trees plays pivotal role in the natural world and also in his life. As a result, it has been responding to the man by changing the climate, creating, daunting challenges of global warming, air pollution, etc. Due to the urbanization of most of the cities in the world, trees were replaced by the infrastructure and accordingly, the users of urban areas were suffered from the discomfort due to high temperature, especially, when they are on the roads. Therefore, man was forced to rethink about replanting, trees on the urban roads. From this research, it was identified the logical relationship between the actual thermal performance of the urban roads and how it feels to the users of the roads, on different hours of the day. Further, this research, emphasized, that having properly planted trees on the roads, can provide the comfort to the users without any disturbance to their activities on the roads, and also contributes to minimize the environmental issues that the world is facing today.*

Key words: *Urban roads, Air temperature, Humidity, User comfort, Tree shading*

1. INTRODUCTION

Climate change has become the most threatening problem that the world is experiencing today. According to Houghton (1997) and Spencer (2010), variations in day-to-day whether are occurring all the time; they are very much part of our life. As a result, the phrase "Global Warming" has become familiar to many people as one of the environmental issues of our day. Further, several studies carried out by different scholars have indicated that the world climate has got hotter during last two decades. Similarly, there is a very significant periodic seasonal variation in the air temperature of the city of Colombo. Figure 1, represents the air temperature distribution of Colombo city from 1901 to 2001. It clearly indicates the deviations of the temperature from the average temperature. The average air temperature of the city for 100 years from 1901 to 2001 is 27.25°C (Manawadu & Liyanage, 2008). The early 1910 to 1960's the temperature has a negative deviation from the average temperature and afterwards the temperature increases year by year.

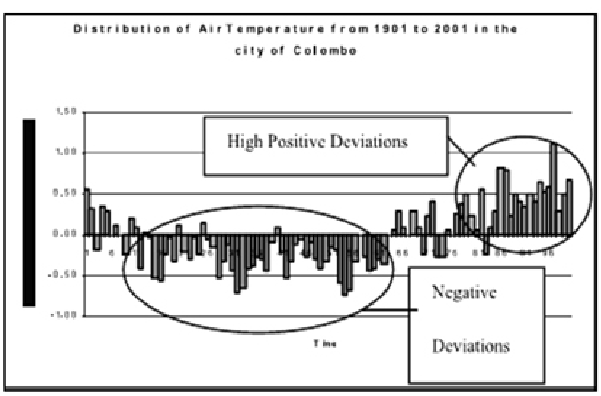

Figure 1 Air temperature distribution Source ; Manawadu & Liyanage, 2008

Figure 2, shows the trend of increasing temperature of the city in recent past. The increase in the temperature is quite rapid and the trend line shows that the temperature increase within the 60 years from 1940 to 2000 is 1° C which is far higher than the global temperature increase. Further, parallel to the temperature increase in city of Colombo and since Sri Lanka is located close to the equator with latitude between 5^0 to 9^0 north's, so there can be direct solar radiation on roads for about five months of the years, at the different times.

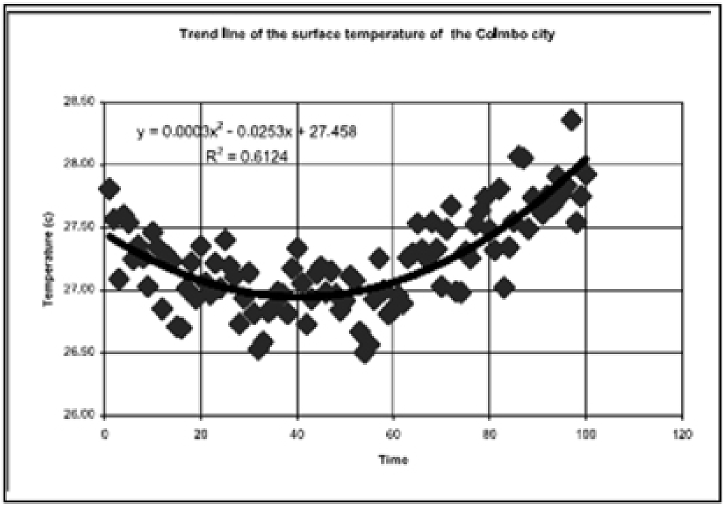

Figure 2 Trend line of the temperature of city of Colombo Source ; Manawadu & Liyanage, 2008

Every day, the road surface will be subjected to direct solar radiation for about half a day. Therefore, screening of sunlight is also important owing to its reduction of direct radiation from the sun during clear days (Manawadu & Liyanage, 2008).

Further, urban heat island effect is a more immediate concern in cities related with impervious surfaces. In the peak of summer in warm climate areas, temperatures of asphalt pavements can reach 77^{0}C (Karen et.al, 2007). In Sri Lanka most of asphalt paved roads are exposed to sun light directly. Asphalt surfaces generally absorb solar radiation in day time and release it overnight. Asphalt absorbs most of the visible light that falls on it. (Asphalt pavements similarly absorb the invisible infrared solar radiation.) Thus the energy in the absorbed sunlight becomes heat in the pavement. Generally the absorption capacity of bitumen surfaces is much higher as it is a black body. Because of that it keeps surrounding warmer than if the bitumen road surface covers with tree shadings. If there are trees overlap streets which shading large proportions pavement it will intercept radiation before it reaches sealed surfaces. So the amount of solar radiation is less which reaches the asphalt surface due to reflection by trees and also some parts of solar radiation are absorbed by trees. Because of that tree shadings reduce the surface temperature of asphalt surfaces and surrounding (Waltz & Hwang, 2011).
According to, Planet Ark Environmental Foundation (2011), basic positive impact of trees is that is help to reduce surface temperature. Most tropical countries are using trees for reducing surface temperature. The urban heat island is in part caused by heat production from cars, heating and air conditioning of buildings, industrial processes, and urban structures that may slow winds and hence prevent heat exchange. However, another main component of the extra heat in cities comes from dry sealed surfaces. Sun-exposed pavement absorbs radiation throughout the day, charging up on heat like a battery, and then releasing it overnight. This release of sensible heat [heat that causes a change in temperature in an object is called sensible heat, (Mcketta, 1992)] keeps the neighborhoods warmer than if the surface was vegetated or shaded. Likewise, one benefit of urban trees, there is the relationship between tree shade and pavement performance (Watson & Labs, 1983).
The aesthetic value of old, large trees has been shown to increase the attractiveness of town streets and the large trees in road side the roads may positively affect the psychology of residents, pedestrians and drivers, including reducing solar radiation which hit the road surface directly, limiting runoff of rain water which can cause to erode the road surface, absorption of urban noise which comes from vehicles, improving human health and reduced psychological stresses (Heisler, 1997). Also urban tree shadings in road side reduce regional air pollutants (Ozone, PM10, NO2, SO2, CO) by 1% to 3% of anthropogenic sources and the major advantage of roadside trees is roadside trees capture more large-size particulate matter than trees not near the road. (Scott, et.al, 1999).These effects have implications for air quality standards. Also vegetative canopies provide a cooling effect on microclimate directly by shading the ground surface and indirectly through transpiration. Therefore many cities have started tree planting programs in order to improve environmental conditions and alleviate the urban heat island effect. (Carol, et.al, 2011) and (Maco, et.al, 2002)

Conversely, one of the problems related to roadside trees is visual disturbance to drivers and pedestrians. This problem is mainly occurred at the junctions and curves. Use of a “clear zones” or lateral roadside space free of rigid obstacles is a major policy related to roadside trees. Yet street trees are often perceived as liabilities due to litter drop, root damage to side walkways, and visibility and security problems created by blocking signs and lighting. Also root of trees can be caused to damage for drainage system. Localized pavement failures are often caused by poor drainage systems. When side drains and culverts silt up, water ponds against the road embankment eventually weakening the lower pavement layers (Gregory, 2005).

2. OBJECTIVES AND THE METHODOLOGY

The objectives of the research;

- To find out, the actual performance of an urban road with and without trees, with respect to temperature and humidity during the daytime.
- To find out, the user satisfaction on urban roads, with and without trees, in different times of the day.
- To understand the relationship between the performance and the level of user comfort on urban roads with and without trees during the daytime.

The following methodology was developed to achieve the objectives of the research:

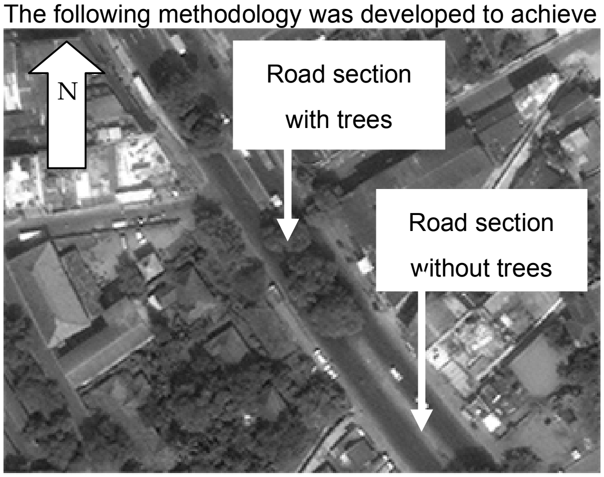

Figure 3 Selected road sections

As in Figure 3, most suitable urban road section, with morning and evening heavy traffic and also having shaded and un-shaded areas in close vicinity was selected, in between Katubadda and Rathmalana, concentrating on same location ,close to each other, to get more accurate field measurements. Further, the selected road was oriented to north-west and south-east directions. Required measurements were taken, throughout the day starting from 9.00 am to 4.00 p.m. in every one hour interval, with respect to temperature and humidity of the road and these measurements were taken, above 1m from the road surface and about 100 m from each road sections on the same day.

A questionnaire survey was conducted to come across, how users of urban roads feel, on having trees on the roads. The analyzed sample size was randomly selected 261 and that represented both, pedestrians and drivers. The questionnaire enclosed 6 sections; the first section was to identify the user and to understand the first impression on trees on urban roads, second section was created to comment on the trees on the road in more optimistic manner, third section was highlighted with the problems of having trees on the road, fourth section was to understand the level of comfort that experiencing by the user, when driving or walking under a tree canopy, fifth section was to comment on the visible distance affected by trees and the final section was to mark the hours of the day, that user feel more discomfort on a urban road, spanning from 7.00 a.m to 7.00 p.m.

Finally, the readings that were taken with respect to temperature and the humidity on a urban road with and without shade was compared with the findings, with respect to the personal feelings of the user, that were surveyed from the questionnaire.

3. ANALYSIS AND FINDINGS

3.1 Actual thermal performance of the road sections

Temperature and humidity measurements were taken in both road sections with and without trees, from 1m above the road surface and 100m away from each road sections.

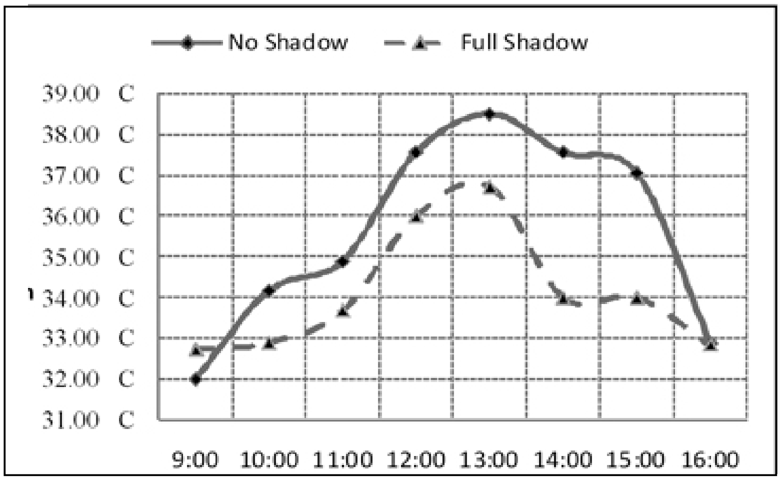

Figure 4 Air temperature variation

According to Figure 4, the peak temperature is obtained during 13:00 hour and the peak value is 38.5°C in the no shade area and 36.75°C in the full shade area and air temperature difference is 1.75^{0}C. Further, when compared the two graphs it was clear that, the temperature drop down between 14.00 hour to 15.00 hour has drastic difference when compared to the other hours of the day and it was around 3.5^{0}C.

However, comparison of these temperature readings clearly indicates, the impact of the trees and the shade, on the urban roads and how they reduce the temperature

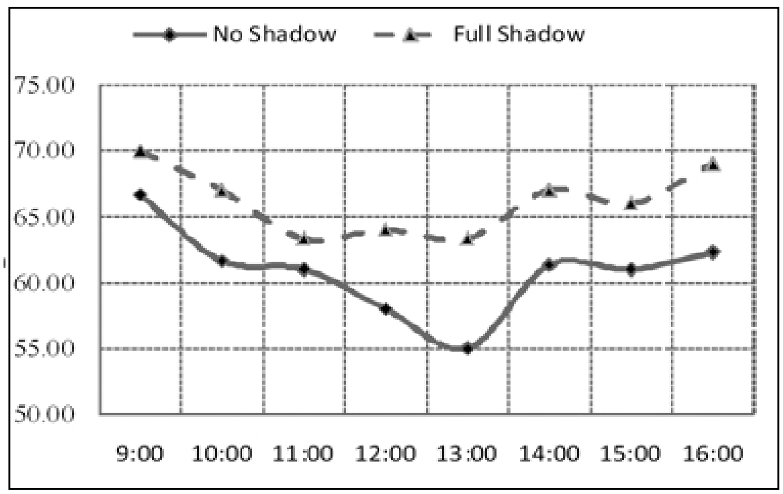

Figure 5 Humidity variation

According to Figure 5, urban road section with full of trees and shades were high in humidity with compared to urban road section with no trees and shades. However, during morning and evening the humidity in both road sections were high and the lowest humidity reached at 13.00 hours. When considered the road section with full shade, the humidity varies from 67% to 55% at 9.00 hours to 13.00 hours and again it increased up to 63% by 16.00 hours.

Likewise, in no shade area the humidity was high as 70% at 9.00 hours and it was decreased up to 64% during the day time and again it was increased up to 69% at 16 hours. However, when compared both graphs, it was clear that at 13 hours, the full shaded road section was high in humidity of 8 % than the road section no shade.

3.2 Survey on user satisfaction on urban roads

According to the questionnaire, stated in Appendix A[1], the 6 sections were separately analyzed to understand the macro picture in more broader and accurate manner. The section 1 was to understand the user and whether the users prefer to have the trees on urban roads or not. When, analyzed the data, according to Figure 6, 7 and 8, it was identified that from the surveyed sample, majority represented a 55% of male population and most popular category was the pedestrians and it was 66 %. From, remain of 34% of drives, 56% have driving experience more than 5 years.

[1] Appendix A is available with the corresponding author to be obtained upon request.

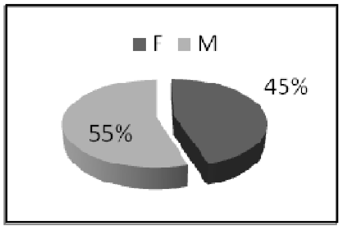

Figure 6
Sample composition on gender

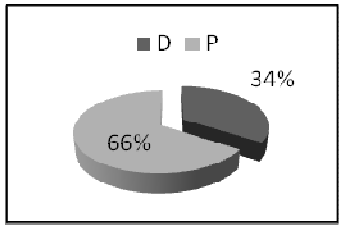

Figure 7
Sample composition user category

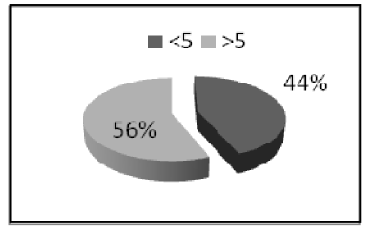

Figure 8
Sample composition driving experience

Section 2 of the questionnaire, was designed to understand optimistic thinking of users regarding trees, on the road and on how they feel when they experiencing such urban roads with full of trees and shades.

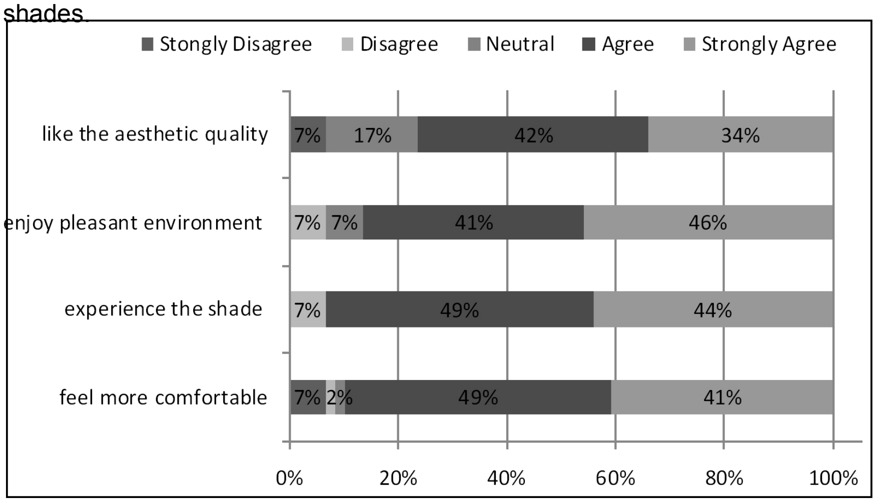

Figure 9 Preference to have trees on urban roads

According to Figure 9, significant majority user population was agree on the comfort, shade, pleasantness and aesthetic quality on the urban roads with trees. Further, equal 7% of users who experience discomfort on the shady roads and who doesn't experience any aesthetic quality of urban roads with trees.

Section 3 of the questionnaire, reasoned out the problems of having trees on urban road. According to Figure 10, there were some users, who were strongly disagreed, that the amount of daylight fallen on the roads get reduced due trees on the roads and at the same time no one is strongly agreed on that as well. Further, some uses don't think that road signs get hidden by the trees. However, when compared all four types of problems that can create from trees, majority of 48% agreed on the idea, that most road blocks are creating during rainy season due to fallen trees and branches.

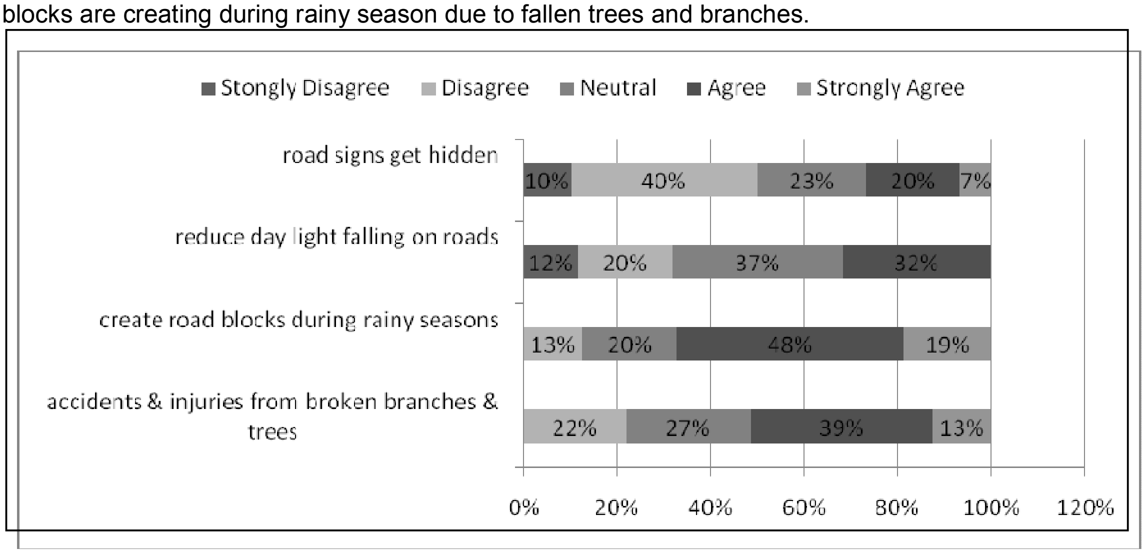

Figure 10 Problems of having trees on urban roads

The level of comfort that users can experience on a shady urban road was analyzed under section 4. When analyzed the Figure 11, majority of 47% strongly agreed that they feel more relaxed on an urban roads under a tree canopy and also 20% on air pollution reduced by the tree on urban roads. However, very less population thought that, they feel forbidden, tired and not safe on shadowy urban roads. At the same time, 64% of majority population disagreed, the idea of tiredness when walking or driving under a tree canopy.

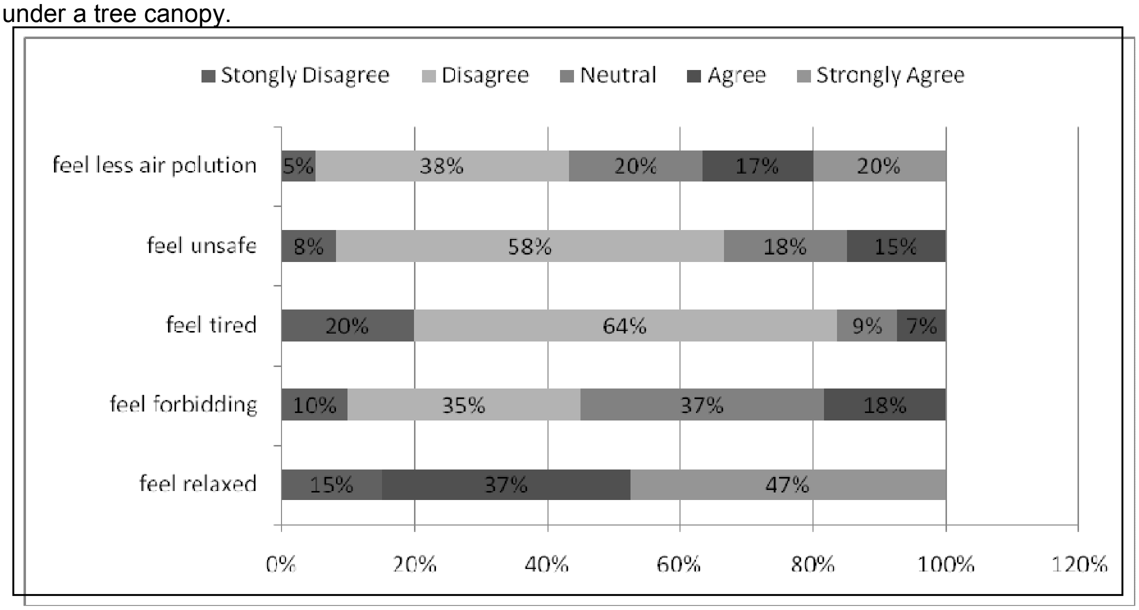

Figure 11 Level of comfort under a tree canopy

Section 5 of the questionnaire was concentrated on the visible clarity under tree canopy. According to Figure 12, 42% of majority strongly agreed that, trees on bends and curves can reduce the visible clarity of urban roads and 40% have neutral experience on that and no one strongly feel that when shadows of the trees falls on to the windscreen, create difficultness to drive. Further, majority of 52% disagreed, the idea that, lower branches of trees block the visible distance.

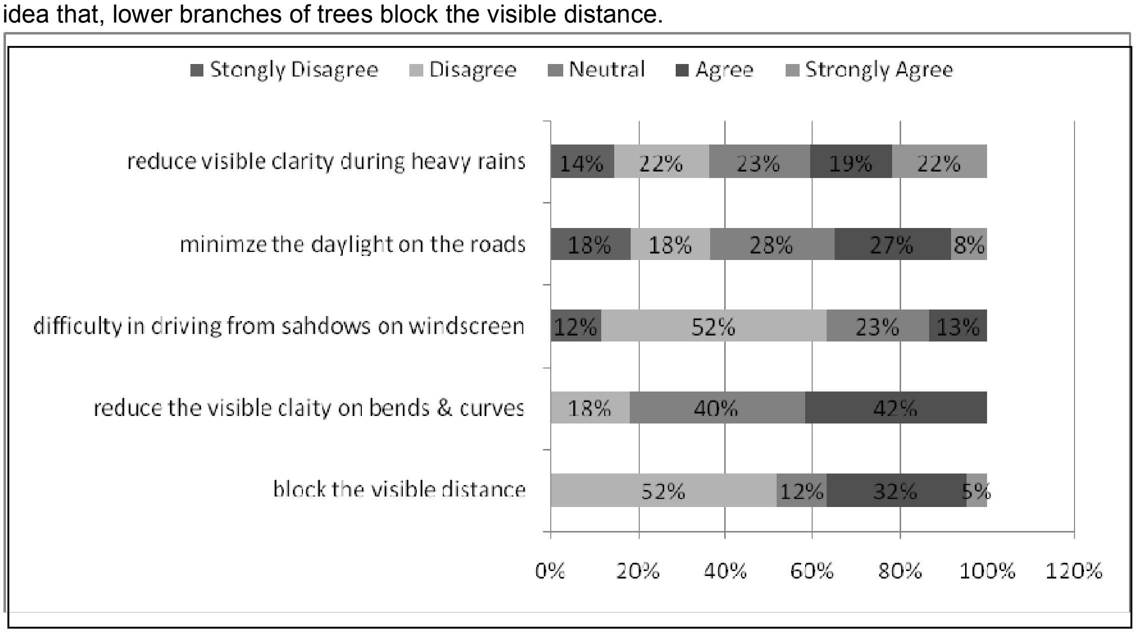

Figure 12 Visible distance under a tree canopy

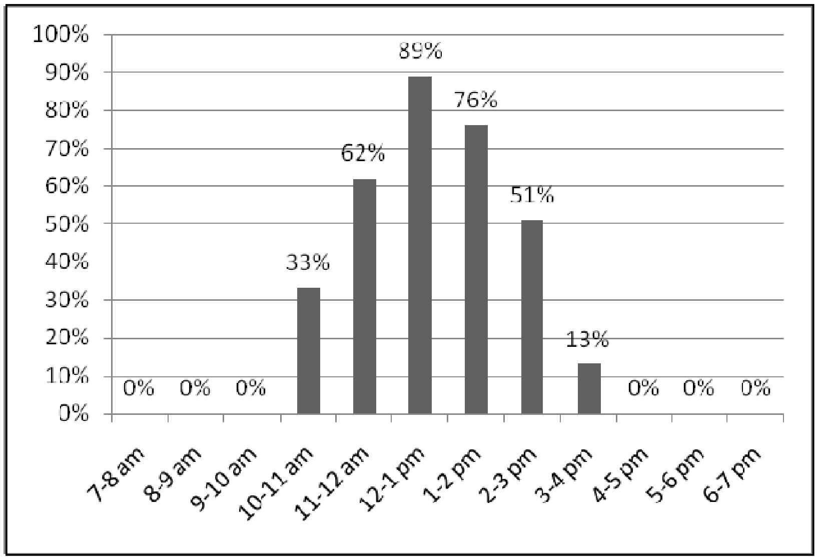

Figure 13 Discomfort hours of the day with respect to user

The section 6 was entirely based on to identify the most discomfit hours of the day for users on urban roads. From the Figure 13, it was spotted that, the discomfort start from 10.00 am to 4.00 p.m and majority of the sample population commented that 12.00 to 1.00 pm is the most uncomfortable hour of the day.

4. DISCUSSION

Figure 14: Satellite image of Colombo

When carefully analyzed the satellite image of Colombo, it can be seen that majority area was covered by infrastructure with very less greenery and also with the contribution of high number of vehicles; the tendency of heating up the environment was highest in Colombo as an urban area. Therefore, necessity of having shades on urban roads especially in Colombo, are very important and critical aspect, that have to be considered by the users in the area.

During the research, the actual thermal performance of the urban roads with and without trees were analyzed from the readings that were taken on the effecting variables for thermal performance; temperature and humidity and findings were compared with user's responses with respect to their personal feelings and experiences.

From the Figure 4 and 5, it was cleared that, the temperature increases and humidity decreases during the day, starting from 10.00 a.m to 4.00 p.m. Though, there was a variation between the road section with shade and without shade, the thermal performance of both road sections indicated the same. When this was compared with the Figure 13, the user experienced the discomfort of the day starting from 10.00 a.m to 4.00 p.m, and that proves the thermal performances of the urban road with the actual discomfort that user experience on the urban road has a strong association.
However, from this research, it was recommended to carry out further analysis to find out, whether, the actual thermal performance of an urban road with fully shaded, continuous tree canopy and entirely

deserted road with no trees at all, falls within the comfort zone and how the user responds on the same roads with respect to level of comfort.

5. CONCLUSION

From the entire analysis, carried out during the research, it was proved that, users prefer to have shades on urban roads rather than experiencing deserted roads without any shades. Though, the trees provide higher humidity level, they also reduce temperature and that makes the users who travels on the roads, covered by trees, more comfort than travelling in a road without trees. On the other hand, according to point of views of some users, even though the trees provide maximum comfort, there are some problems of having trees on urban roads such as; road blocks during rainy seasons, reduction of visible clarity, accidents and injuries due to lower and broken branches and trees, etc. However, these can be reduced by giving adequate attention and implementing appropriate maintenance procedures by relevant authorities. Therefore, by reducing the negative impacts of trees on urban roads and encouraging properly maintained tree canopies on them, other than the comfort, that will also contributes to control the critical environmental issues such as global warming and air pollution; most daunting challenges of the world today.

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Technology Transfer to Local Construction Industry through Foreign Contractors: Barriers and Enablers

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Abstract: *Sri Lankan construction industry has developed considerably during past few years due to ongoing infrastructure development projects and post war reconstruction and rehabilitation projects. These will further trigger a construction sector boom over the next few years. Construction industry boom has invited more foreign construction firms into the local construction industry. Technology Transfer to local construction industry is one key benefit from the foreign construction firms' involvement. Technology Transfer is a crucial and dynamic factor in social and economic development and has been transferred intentionally or unintentionally from one party to another when right conditions are provided (Li Hua and Greenwood, 2001). The development of a construction industry of a certain country can be enhanced through the involvement of foreign construction firms. However, authors' initial investigations raised question is, whether the Sri Lankan construction industry is ready to acquire the possible construction technology through foreign firms. This paper identified enablers and barriers of Technology Transfer and suggest local construction industry to create a suitable conditions to acquire construction technology more dynamically through foreign firms.*

Key words*: Technology Transfer, Learning Environment, Enablers, Barriers*

1. BACKGROUND

Sri Lankan construction industry contributes about 8 per cent to the country's GDP and ranks seventh among the thirteen major sectors contributing to the country's GDP (ICRA industry report on Sri Lanka, 2011). The end of destructive conflict in 2009 has revived the economic activity and resulted in a strong focus on infrastructure development. With the construction boom, the presence of foreign construction firms has increased considerably in Sri Lanka. It is evident that the involvement of foreign construction firms within a construction market of a certain country, improves capacity of construction industry of that host country. When foreign contractors are invited, host country expects technology transfer to the local construction industry through their involvement which encourages the flow of new technology to the local industry to upgrade the capabilities of local contractors (Ofori and Lean, 2001). According to Ofori et al, (2002) foreign contractors have generally shown their ability to deliver higher quality of work with timely completion. The process of technology transfer requires a recipient industry should possess a suitable mode of gaining knowledge (Ganesan and Kelsey, 2006). Thus, the local construction industry should have an appropriate environment to grab and absorb suitable technology and knowledge from foreign construction firms. But the question remains, is whether the present situation of Sri Lankan construction industry is prepared to acquire advance technologies and knowledge. It is essential to understand the barriers and enablers to create such cohesive environment.

1.1. Technology Transfer

Technology Transfer (TT) is a crucial and dynamic factor in social and economic development. Abbot (1985) defined TT as “the movement of the science from one group to another, such movement involving its use”. Simkoko (1989) defined the TT for construction industry as “the planned conveyance and acquisition of technical knowledge and techniques of construction firms”. This implies that there is no true TT until the technical knowledge received from the donor has been put into effective use. Technology Transfer involves a two way process which can succeed only when both the donor and the recipient work together in deciding what needs to be transferred and implement (Sridharan, 1994). Moavenzadeh and Hagopian (1984) suggested foreign contractors’ involvement as a key requirement for the development of local construction industry and shown that local contractors progressively enhance their capability by working with foreign contractors, until eventually they become able to export their services. It is evident with Singapore construction industry, that the local contractors’ capacity and ability have been enhanced as a result of their involvement with foreign firms (Lam, 1994).
UNCTAD (1990) suggested that effective transfer occurs when the technology is requested, transmitted, received, understood, applied, diffused widely and improved. Construction technology may be transferred through joint ventures between foreign and local companies which may either be project specified or of a long-term nature (Ofori, 1994). International joint ventures and subcontracts employed in building material and construction can be serious vehicles of TT provided it contributes to provide high quality inputs into the wider industry (Ganesan and Kelsey, 2006). Shrestha and Kumaraswami (2000) shown that technology transfer can takes place in various ways, such as direct use of technology, use of technology with modification, and the reverse use of technology.

1.2. Technology Transfer Barriers and Enablers

Technology transfer has been advocated as a catalyst of the change or improvement required in many construction industries; however free transfer of technology from one country or region or firm to another has been restricted by various barriers (Shrestha and Kumaraswami, 2000). They identified few technology transfer barriers such as: (i) Organizational culture, (ii) Lack of time, (iii) Capacities of individuals (e.g. Training skills) (iv) Attitudes of individuals (e.g. Reluctance), (v) Lack of clear policy, (vi) National /ethnic culture differences, (vii) Lack of clear agreements (viii) Lack of clear procedures, (ix) Lack of funding provisions, and (x) Language. According to their findings related to Hong Kong construction industry, organizational cultural barriers, lack of time, capacities and attitudes of the individuals and lack of clear policy are the major barriers.
Steward and Waroonkun (2007) developed a conceptual framework which included four technology transfer enablers namely, transfer environment, learning environment, transferor (foreigner) and transferee (host) characteristics. The performance and interrelationship between, these enablers contributes to the degree of value added to the host country construction sector in three main areas, namely, economic advancement, knowledge advancement and project performance as shown in Figure1.

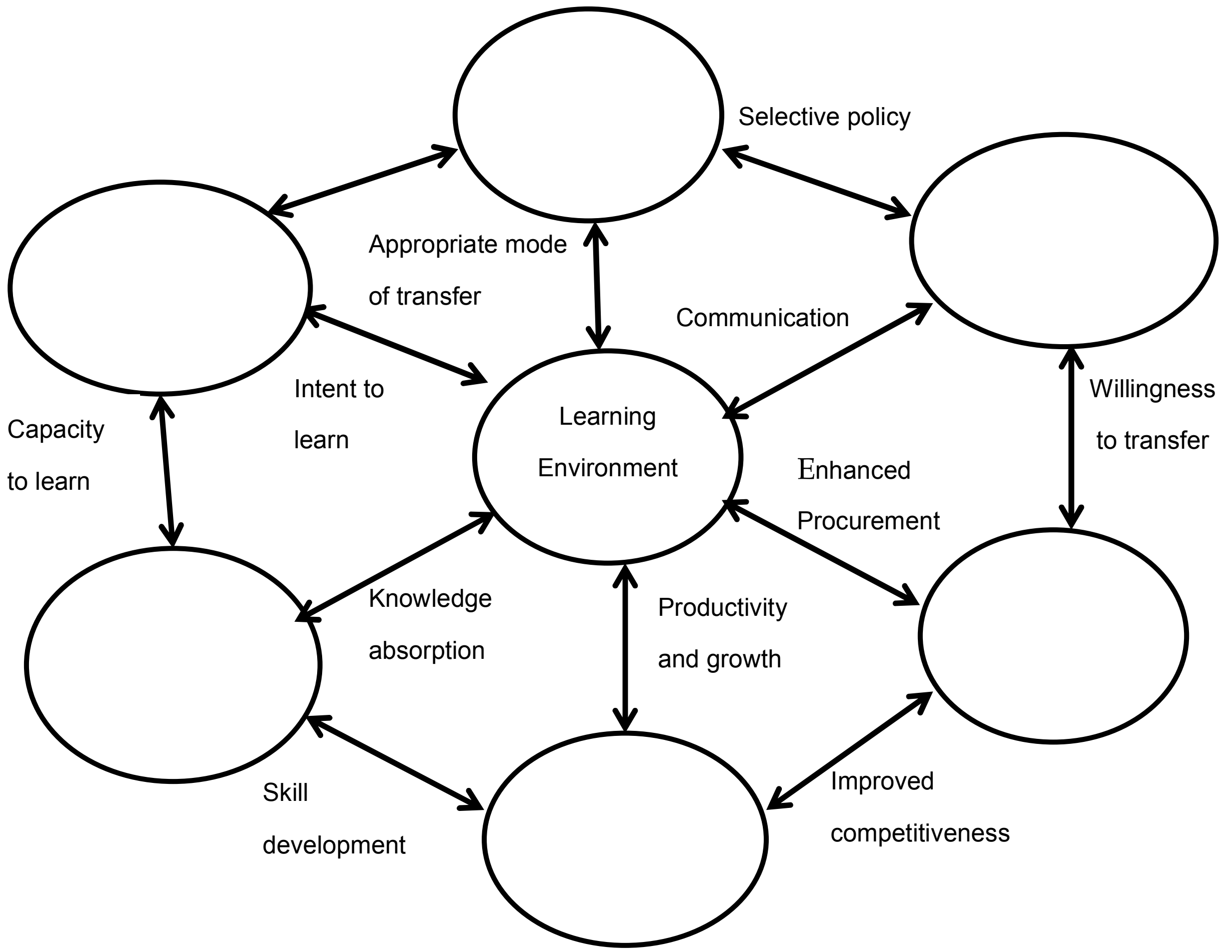

Figure 1: Seven proposed perspectives for benchmarking Technology Transfer (Source: Stewart and Waroonkun, 2007))

Transfer environment: Construction projects incorporating technology transfer are mainly produced by turnkey systems, direct licensing agreements, management contracting or joint ventures. For large or complex technology transfer projects alternative modes may be adopted such as licencing-cum joint ventures, turnkey-cum-licensing etc (Calantone et al, 1990). According to Calantone et al (1990), host government's policies, regulations and enforcement practices can impact greatly on the effectiveness of technology transfer initiatives, especially with their international political system and domestic political structure. Those government policies should provide initiatives to transferor to transfer technology pro-actively to local competitors (Kumaraswamy and Shrestha, 2002).

Learning Environment: Cultural characteristics and the distance between the technology transferor and transferee is a major concern of technology transfer process (Fisher and Ranasinghe, 2001, Kumaraswamy and Shrestha, 2002). Effective technology transfer process required the parties involved in the process to build a culture of mutual trust through effective communication (Malik, 2002). Devapriya and Ganesan (2002) suggested strong commitment from the senior management teams of both the host and foreign firms to establish this culture.

Transferor and Transferee Environment: To achieve a successful technology transfer the transferor should transfer the appropriate technology and the transferee should have every intension to adopt it. Moreover, the degree of international experience of both parties and the nature of experience can impact significantly on the technology transfer process. The capacity to transfer and adopt technology will also depend on each individual existing knowledge base and the gap between this knowledge level and the level required utilizing the transferred technology (Steward and Waroonkun, 2007). Kumaraswamy and Shrestha (2002) showed the appropriateness of the transferor and transferees culture for working in partnership to embarking on the TT process. Capacity of the local construction industry to assimilate, adapt, modify, and generate technology is critical to an effective transfer of technology. This capacity building cannot be done only by importing technologies from foreign countries but also through advanced man power and management training, domestic production of materials, tools and equipment, and individual innovation focused on construction industry. Ganesan and Kelsey, (2006) shown the need for opportunities for acquiring knowledge about designs and construction planning, and education and training programmes to formulate within construction joint ventures in order to train local firms on newly established construction technologies and construction knowledge. Participation of local design and or construction firms at early stages will facilitate the absorption and implementation of appropriate construction technology. Open and positive attitudes among the parties involved may encourage transfer technologies from foreign firms to local construction firms.

Economic Advancement: A predominant reason to encourage international TT initiatives by governments of developing countries is to improve living standards and economic prospect of people (Steward and Waroonkun, 2007). Therefore host workers and professionals should be performed at a higher level and become more competitive in the domestic and international market to achieve this objective (Fisher and Ranasinghe, 2001). It should be noted that to gain economic advancement transferred knowledge has to be absorbed and applied on a number of projects across the host country.

Knowledge Advancement: As sited by Gilbert and Cordey-Hayes, 1996, beyond the quantitative economic benefits achievable from TT, host construction firms may also experience knowledge advancement at the individual and organizational level (Steward and Waroonkun, 2007). This knowledge which can be transferred as implicit and tacit knowledge should lead to improve working practices in the immediate term and hopefully become the norm over the long term (Steward and Waroonkun, 2007)

Project Performance: Financial performance, schedule performance and quality performance are generally accepted as major objectives of a construction project. The effective TT should improve the performance of these key areas (Devapriya and Ganesan, 2002).

Language and cultural barriers seems to be significant factor which affect the level of TT to Sri Lankan construction industry. Most of the time host workers and foreign workers use different languages which decrease the mutual understanding between the parties due to lack of effective communication. Within the host country environment a system for effective communication should be established in order to channel the required construction knowledge and technology in an effective way. Capabilities of individual firms should be identified before form the agreements for joint ventures so that to understand which technology is to be transferred and which technology to be absorbed.

2. DISCUSSION

As sited by Ganesan and Kelsy (2006), the capacity to assimilate, adapt, modify, and generate technology is critical to an effective transfer of technology. This capacity building cannot be done only by importing technologies from foreign countries but also through advanced man power and management training, domestic production of materials, tools and equipment, and individual innovation focused on construction industry. If absorptive capacity of a certain country is higher that country has that country is more capable of assimilate the technology from foreign countries (Roshanthini et al, 2011). Therefore, a key to effective technology transfer is to develop absorptive capacity of Sri Lanka construction industry by adopting well established guidelines.

Planning for technology transfer in foreign funded projects is a critical need (Ganesan and Kelsey, 2006). Opportunities for acquiring knowledge about designs and construction planning, and education and training programmes should be formulated within construction joint ventures in order to train local firms on newly established construction technologies and construction knowledge. Participation of local design and or construction firms at early stages will facilitate the absorption and implementation of appropriate construction technology (Ganesan and Kelsey, 2006).

Clear policies and initiatives should be established by the Sri Lankan government to ensure the transfer of suitable technology and knowledge to the Sri Lankan construction industry from foreign construction firms. Therefore Sri Lankan government and responsible authorities should take initiatives to address and monitor the total process of technology transfer between local and foreign firms. It is evident that during last few decades Institution for Construction Training and Development has been developing proposals to upgrade domestic contractors, but only limited success in increasing work opportunities for domestic contractors (Ganesan and Kelsey, 2006). Therefor the proposals should be target the development of local construction industry in aid of technology transfer through foreign construction firms. Clear agreements should be established to form successful joint ventures and subcontracting agreements to have successful technology transfer to foreign firms. The diffusion of construction technology gained from foreign firms can be ensured by organizing training sessions for the local firms to make aware them on newly established technologies.

Planning for technology transfer to local construction through foreign funded, or through foreign contractors involved projects is a critical need. Clear agreements should be established to form successful joint ventures and subcontracting agreements to have successful technology transfer through foreign firms to local counterpart. Therefore, appropriate governmental organizations should develop and monitor the total process of technology transfer between local and foreign firms. Absorptive capacity of Sri Lankan construction industry also to be enhanced by adopting well established guidelines. Further research need to be done to make aware how local firms and authorities should promote commitment from partners to encourage the frank communication between parties, organize interactive training sessions to exchange and understand the language and culture between organizations. It is also time to realize the mechanism of diffusion of construction technology gained from foreign firms and adopt them for local use without restrictions. Unless industry is ready to take some risk and do experiments Sri Lankan construction industry never gain anything through technology transfer. It is time to review some unacceptable conditions and practices on pre-qualifications and experiences stipulated by certain foreign contractors before selecting them as contractor for certain construction projects by which acquisition and adaption of technology is encouraged.

3. ACKNOWLEDGEMENT

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Walkability Evaluation of Streetscapes: Development of Prediction Equations for Walking Needs of Tourists

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Abstract: *Work presented here is a part of a comprehensive study which aimed at calculating the walkability levels of streets. The walkability levels are to be calculated based on five walking needs which have been identified in previous stages. In the work presented here, equations which can predict five walking needs were developed based on independent variables which represent the effect of physical environment.*

Keywords: *Walking needs, Walkability Evaluation, Landscape, Tourists*

1. INTRODUCTION

Provision of walkable spaces has become an important consideration for urban planners and designers (Alfonzo, 2004). With the present research mainly focusing on regular walkers, limited attention has been paid to the tourists. In order to cover such a research gap this work has focused on the walkability of tourists. Among different efforts of creating walkable spaces, evaluation of walkability levels of streets has a prime importance. Researchers around the world have attempted to propose walkability evaluation instruments to evaluate influence of elements of the physical environment. The influence of the elements of the physical environment on a walking decision is mediated through perceived amenities (Ewing et al., 2006). Thus an accurate walkability evaluation method predicts walkability after considering such mediation process.

Previous works on walkability evaluation have proposed walkability evaluation methods either in terms of individual physical elements or in terms of amenities. In order to reduce the complexity, methods based on individual physical elements use only a limited number of highly influential elements without considering some other influential elements. Methods based on the amenities are criticized for being subjective since the amenities reflect subjectively measurable parameters. Yet if a proper set of predication equations are developed to predict the amenities then such amenities can be predicted objectively.

The work presented here is a part of a research work which attempts to predict walkability incorporating the mediation process based on a research framework shown in Figure 1. The initial work (Samarasekara et al., 2011b) indicated that when people evaluate walkability their decisions are influenced by the street type such as whether it is a residential street or a shopping street. Accordingly a street classification consisting of four streets namely, Type A -Mixed use – high destination streets, Type B- Shopping streets, Type C - Mixed use – low destination streets and Type D- Residential streets was finalized with and objective distinction criteria. In the next stage based on the outcomes of a psychological experiment, five walking needs which would influence the walking decision of a tourist were identified (Samarasekara et al., 2011a). The present work addresses the next part of such work. For each identified walking need prediction equations were developed to calculate such walking needs based on a set of pre- identified physical elements.

2. MATERIALS AND METHODS

2.1. Research Approach

Prediction equations for each walking need were derived by systematically variation of parameters pre-identified parameters and evaluating how the subjects would perceive the walking needs for different conditions. Due to the practical difficulties of systematically varying several parameters within several standard conditions, photo retouching was used to produce stimuli. Separate prediction equations were produced for different streetscape types.

Based on previous research work, a set of potential parameters were identified which could influence each walking need. The base images for each type of walking need and each streetscape type were taken in standard conditions. The images were taken in summer using an 18mm camera showing the viewpoint of a pedestrian from the left side of the street looking in the forward direction. The camera was set at a 1.5m height and at a distance of 1.2m from the boundary demarcating the private and public land.

Each of the denotative parameter was systematically varied using multiple levels. For each connotative parameter one element was selected as the standard and it was systematically varied with other denotative parameters. After selecting one occurrence of the standard element, a set of stimuli were prepared by the replacement standard by the other connotative elements. By comparing the ratings of such stimuli against the stimulus with the standard element, the contribution of other connotative elements were calculated as a weight. For the walking need Shade, only one connotative parameter (Area of non-shade provision trees) existed. Therefore it was systematically varied.

2.2. Experimental work & Data Analysis

73 Saitama University students participated in the experiment. Experimental conditions: Participants viewed the images projected on to a screen. Their seating position was fixed after considering the dimensions of the projected image and the viewing angles of the camera so as to provide a similar view to that seen on the real site. They viewed individual images and rated them for the relevant walking need semantic scales on a 9-point Likert scale of –4 to +4. In order to familiarize respondents with tasks, 8 practice images was used prior to experimental tasks.

For each image, data of each walking need semantic rating obtained from all the 73 respondents was averaged to get the average rating scale values for a particular image. The prediction equations were developed by applying Hayashi's quantification method I (for Traffic safety in walking area & Comfort of walking area & Shade) and multiple regression (for Environmental appearance & Activity potential).

3. RESULTS

3.1. Outcomes of the experiment: Physical elements influencing each walking need

This work identified specific parameters influencing each walking need while quantifying the contribution of influential parameter. Many researchers using top down approach start by hypothesizing the main walking needs followed by the hypothesizing the individual elements influencing each walking need. In establishing the construct validity, they tend to use the approach of checking the correlation of such parameters to walkability. But in the instances where high multicolinearity exists this approach may lead to the identification of inconsequential parameters as being consequential. Thus proper identification of parameters had to be done by the systematic variation of each parameter. Considering such, this work identified influential parameters for each walking need, based on the systematic variation of potential parameters.

Outcomes confirmed that each need is influenced by a limited number of parameters while the other proposed parameters were found to be inconsequential. Tables 1 to 5 shows the outcomes of the experiment in terms of influential elements and their relative contribution to each walking need.

Table 1: Prediction equations for Traffic safety in walking area – Hayashi's Quantification Method I

Parameter	Statistical measure	Category description for different streetscape types		Streetscape type			
				A	B	C	D
Model	a			0.979	0.954	0.976	0.935
Method of pedestrian separation	b			0.919	0.754	0.885	0.828
	p			0	0	0	0.00001
		Type A & B	Type C & D				
	cs	1- Separation by trees & Bollards	1- Separation by trees & Bollards or Only trees	1.168*	0.868*	0.674*	0.761*
	cs	2 Separation by trees	2 Separation by Bollards	0.633*	-0.206*	0.567*	1.226*
	cs	3-Seperation by Bollards	3- Mount up or Colored strip	0.828*	0.413*	-0.176*	-0.252*
	cs	4- Mount up or Colored strip	4-White line	-0.381*	-0.031*	-0.865*	-1.289*
	cs	5-White line	5-No separation	-1.075*	-0.556*	-1.572*	-1.071*
	cs	6-No separation		-1.336*	-0.757*		
Walking area width	b			0.958	0.931	0.955	0.825
	p			0	0	0	0.00001
		Type A,B,C & D					
	cs	1-Width below0.5m		-2.066*	-1.698*	-1.881*	-1.509*
	cs	2-Width 1m		-1.089*	-0.560*	-0.913*	-0.405*
	cs	3- Width 2m		0.310*	0.726*	0.074*	0.650*
	cs	4- Width 3m		0.896*	0.966*	0.941*	0.761*
	cs	5- Width 4m		1.261*		1.152*	
Number of lanes	b			0.314			
	p			0.0234			
		Type A					
	cs	1- 2 lane		-0.182			
	cs	2 -4 lane		0.182			
Pedestrian priority	b				0.146		
	p				0.36765		
	cs	Type D					
	cs	1- No pedestrian priority			0.064		
	cs	2- Pedestrian priority			-0.064		
Negative Connotation	b			0.182	0.163	0.243	0.326
	p			0.197	0.314	0.242	0.173
		Type A,B,C & D					
	cs	1- negative connotation : Absent		-0.102	-0.072	-0.080	-0.208
	cs	2- negative connotation : Present		0.102	-0.072	0.080	0.173
Positive connotations	b			0.197	0.406	0.230	0.026
	p			0.162	0.009	0.269	0.916
		Type A,B,C & D					
	cs	1- Positive connotation : Absent		-0.069	-0.188*	0.076	-0.016
	cs	2- Positive connotation : Present		0.069	0.226*	-0.076	0.013
Constant				0.751	0.544	0.960	0.378

a - Multiple correlation coefficient; b- Partial correlation coefficient; p – p value; cs – category scores; *- Value significant at 0.05 level; A: Mixed use High destinations; B:Shopping streets; C: Mixed use low destinations; D: Residential

Table 2 : Prediction equations for Comfort in walking area– Hayashi's Quantification Method I

Parameter	Statistical measure	Category description for all streetscape types	Streetscape type A	Streetscape type B	Streetscape type C	Streetscape type D
Model	a		0.998	0.900	0.991	0.993
Paving type	b		0.613	0.508	0.890	0.949
	p		0.023	0.134	0.000	0.000
	cs	1-Unit blocks	0.026*	0.413	0.415*	0.612*
	cs	2-Asphalt	-0.104*	-0.502	-0.508*	-0.286*
	cs	3-Concrete	0.098*	0.215	0.195*	-0.255*
Walking area width	b		0.998	0.849	0.989	0.992
	p		0..000	0.002	0.000	0.000
	cs	1-Width below0.5m	-2.443*	-0.665*	-2.004*	-1.391*
	cs	2-Width 1m	-0.962*	-1.468*	-0.857*	-0.284*
	cs	3- Width 2m	0.607*	0.244*	0.998*	0.869*
	cs	4- Width 3m	1.354*	2.111*	1.128*	1.270*
	cs	5- Width 4m	2.258*		1.404*	
Electric wires	b		0.933	0.547	0.736	0.570
	p		0.000	0.101	0.004	0.085
	cs	1-Level I (None)	0.086*	0.375	0.138*	-0.069
	cs	2-Level II	0.366*		-0.457*	0.146
	cs	3-Level III	-1.022*	-0.844	0.048*	-0.094
	cs	4 Level IV	-0.189*	-0.845	0.079*	0.048
Positive connotations	b		0.401	0.023	0.600	0.715
	p		0.174	0.949	0.030	0.020
	cs	1-Positive connotation : Absent	-0.043	-0.024	-0.175*	0.115*
	cs	2- Positive connotation : Present	0.043	0.011	0.136*	-0.185*
Constant			1.052	0.208	0.405	0.128

a - Multiple correlation coefficient; b- Partial correlation coefficient; p – p value; cs – category scores; *- Value significant at 0.05 level; A: Mixed use High destinations; B:Shopping streets; C: Mixed use low destinations; D: Residential

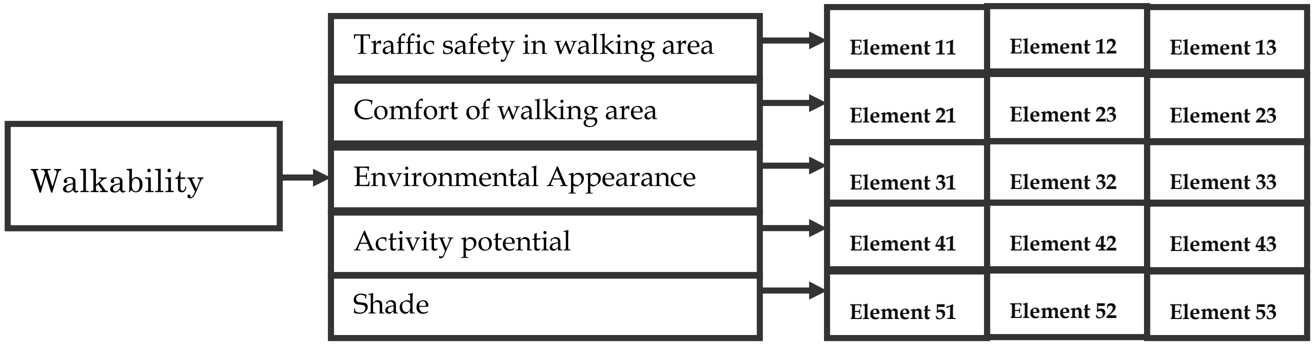

Figure 1: Conceptual framework for the evaluation of walkability

Table 3 : Prediction equations for Shade – Hayashi's Quantification Method I

Parameter	Statistical measure	Category description for all streetscape types	Streetscape type			
			A	B	C	D
Model	a		0.939	0.970	0.747	0.950
Volume of shade provision trees	b		0.934	0.969	0.713	0.949
	p		0.000	0.000	0.000	0.000
	cs	1-Level I (None)	-2.366	-2.257	-1.074	-2.323
	cs	2-Level II	-0.526	-0.149	-0.599	0.074
	cs	3-Level III	1.383	0.821	0.589	0.855
	cs	4 Level IV	1.509	1.586	1.083	1.394
Canopy shape	b		0.237	0.252	0.213	0.218
	p		0.113	0.091	0.156	0.146
	cs	1 - Pyramidical	0.208	0.006	0.096	-0.135
	cs	2 – Vase	-0.068	0.082	0.201	0.126
	cs	3 – Round	0.053	-0.127	0.001	0.078
	cs	4 - Columnar	-0.193	0.069	-0.299	-0.068
Positive Connotation – Volume of home vegetation	b		0.613	0.441	0.395	0.452
	p		0.000	0.002	0.007	0.002
	cs	1-Level I (None)	-0.666	-0.083	-0.498	-0.257
	cs	2-Level II	0.293	-0.167	0.115	-0.063
	cs	3-Level III	0.372	0.249	0.383	0.321
Constant			0.482	0.421	0.378	0.515

a - Multiple correlation coefficient; b- Partial correlation coefficient; p – p value; cs – category scores; *- Value significant at 0.05 level; A: Mixed use High destinations; B:Shopping streets; C: Mixed use low destinations; D: Residential

Figure 2: Sample Images

Table 4: Prediction equations for Environmental Appearance –Multiple Regression

Parameter	Statistical measure	Streetscape type A	B	C	D
	P value -model	0.018	0.009	0.004	0
	Adjusted R^2	0.54	0.367	0.849	0.900
	Constant	0.564	0.589	0.143	0.304
Trees	β	0.599	0.476	0.783	0.875
	B	2.44E-07	2.94E-07	7.02E-07	9.72E-07
Electric wires	β	-0.306	-0.285	-0.452	
	B	-2.62E-04	-3.98E-05		
Exposed service boxes	β				
	B				
Exposed Service lines	β				
	B				
Garbage	β	-0.373	-0.334	-0.218	-0.379
	B	-2.82E-06	-3.64E-06	-6.03E-06	-3.33E-06

β ; Standardized coefficient; B ; Unstandardized coefficient; Note: β values are shown only for those parameters contributing at 0.05 significance level ;A: Mixed use High destinations; B:Shopping streets; C: Mixed use low destinations; D: Residential

Table 5 Prediction equations for Activity Potential–Multiple Regression

Parameter	Statistical measure	Streetscape type A	B	C	D
	P value -model	0.028	0.022	0.047	0.005
	Adjusted R^2	0.846	0.642	0.381	0.885
	Constant	0.037	-0.267	-0.431	0.051
Destinations	β	0.778	0.727	0.515	0.903
	B	4.34E-07	6.66E-07	5.26E-07	9.74E-08
Intermediate spaces	β	0.432	0.402	0.446	0.291
	B	-5.27E-07	1.40E-06	5.40E-07	7.75E-08
Positive Connotation	β				
	B				
Office	β				
	B				
Industry	β	-0.253			
	B	-1.40E-06			
Parking Area	β				
	B				

β ; Standardized coefficient; B ; Unstandardized coefficient; Note: β values are shown only for those parameters contributing at 0.05 significance level ;A: Mixed use High destinations; B:Shopping streets; C: Mixed use low destinations; D: Residential

Results related to traffic safety reveals identified width of walking area as the most influential parameter across all streetscape types. Method of pedestrian separation was also identified as highly influential across all types of settings. Also as expected in type A streetscapes, safety feeling decreased with the increase of number of lanes. Although positive connotative suggestion of presence of crosswalks could influence the safety feeling in type B streets, it did not have any influence in any other streetscapes.
Feeling of comfort was also highly influenced by walking area width across all streetscapes. Paving type while being influential on all setting types, were highly influential in type A and D streetscapes. Within the individual categories, the asphalt paving was disliked in all settings while the unit blocks were preferred over concrete paving in all settings except for type A. With comfort having a dominant influence on a walking decision, this shows that presence of unit blocks can be quite important in a encouraging a walker. Presence of electric wires exerted a higher influence on streetscapes C and D.
Among the three parameters investigated within shade, canopy shape was proved to have insignificant influence while the other two parameters had significant influence across all types of streetscapes. Both of the influential parameters expressed volume of vegetation. As expected the denotative parameter volume of shade provision trees had the highest influence on walkability across all types of settings. In addition the connotative parameter, non-shade provision trees also had a considerable contribution across all settings. This suggest that even those trees which may not provide any shade can still encourage a person to walk through the suggestion of shade.
The results for Environmental appearance show that trees, electric wires and garbage to have significant influence for all types of settings except for the non-significant influence of electric wires in type B streets. Outcomes related to vegetation here is easily explainable and is in line with many previous research works. Presence of electric wires had a significant influence on both Environmental appearance and Comfort of walking area.
The prediction equations related to activity potential had a heavy influential power from destinations and strong yet secondary influence by the intermediate spaces. Within the type D settings intermediate spaces showed influential power quite close to that of destinations. Improvement to activity potential through increase of destination may be less feasible in residential areas. In such a context the intermediate spaces provide some potential to improve activity potential.

4. CONCLUSIONS

The outcomes of this experiment identified set of influential elements for each walking need in the context of different streetscape types. Nevertheless results revealed a tendency for a similar set of physical elements to influence a particular walking need irrespective of the streetscape type. In general safety from traffic was influenced by method of pedestrian separation, and width of the walking area. Comfort was influenced by type of paving, width of walking area and the presence of electric wires. Shade was influenced by the presence of shade provision trees and non-shade provision trees. Environmental appearance was influenced by Presence of trees, electric wires and garbage. Activity potential was influenced by the presence of destinations and intermediate spaces. Connotative suggestions had a limited role except in the case of shade. Using the results a set of prediction equations were developed to calculate the values of each walking need based on the physical elements present in the streetscapes.

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Section III- Advances in Environmental Engineering and Hydraulics

Effect of river bed slope and particle size distribution on washing out condition of trees in rivers

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Abstract: *In previous studies, washing out of trees can be evaluated using Breaking or Overturning Index(BOI) and Washing-Out Index(WOI). However, the applicability of those indices were only validated some middle-stream rivers where the bed slope is not so steep. For validating the indices at different river condition from past studies, this survey carried out at 7 rivers with different bed slopes. Accordingly, a relationship between WOI and particle size distribution became clear. Bed slope is found to have no influence on the critical condition of BOI, but particle size distribution can affect the critical wash-out condition for WOI.*

Keywords: *tree trunk breakage, tree wash out, breakage or overturning index(BOI), wash-out index (WOI), particle size distribution of river bed materials*

1. INTRODUCTION

Forestation inside a river (Johnson, 1994) sometimes causes a problem, not only because it reduces river flow capacity downstream, but because debris by floods, i.e., broken tree trunks and branches, increase the drag force on bridge piers in rivers. The debris attached to and accumulates around a pier causes a large scour hole around it and sometimes breaks the pier (Melville and Dongol, 1992). In addition, excessive forestation by a single tree species or an invasive exotic tree species sometimes affects the biodiversity of a river ecosystem (Stokes, 2008). For rejuvenation of a gravel bed river, it is necessary to get the information on how floods affect the formation of a plant community in a river. In particular, the characteristics of sand deposited by the flood event (i.e., particle size, nutrient content) (Oswalt and King, 2005), flood disturbance frequency (Gilvear and Willby, 2006) and intensity (Vervuren et al., 2003), and bed degradation due to flooding (Kamrath et al., 2006) are reported to be important factors that affect the plant community.

Considering the above situation, it is necessary to know the wash-out and breaking conditions of river vegetation due to floods. Conditions of plants uprooted by strong wind or flooding have been analyzed mainly in terms of drag moment acting on the plants (TRCRD, 1994; Gardiner et al., 2000) or bed shear stress (dimensionless shear stress) (Temple, 1980; Egger et al., 2007). Recently, Tanaka and Yagisawa (2009) conducted field investigation in Tamagawa river for a flood in 2007 and reported breaking and washing out of trees can be evaluated using Breaking or Overturning Index(BOI) and Washing-Out Index(WOI), respectively. However, the applicability of those indices was only validated for some middle-stream rivers where the bed slope is not very steep.

Therefore, the objective of this study was to clarify the effect of river bed slope and particle size distribution of river bed materials on critical value of BOI and WOI. To fulfill the objective, field investigations were conducted in the Arakawa River, Miyagawa River (in Mie Pref), Miyagawa River (in Gifu Pref.), Fujigawa River, Kamanashigawa River, Ooigawa River and Abegawa River for finding out the breaking and wash-out conditions of trees by floods.

2. MATERIALS AND METHODS

2.1. River flow analysis

The basic equations included in the hydrodynamic model are the conservation of fluid mass equation and momentum equations (Reynolds equation). The basic governing equations in Cartesian coordinate are shown as follows:
Continuity equation:

$$\frac{\partial h}{\partial t}+\frac{\partial M}{\partial x}+\frac{\partial N}{\partial y}=0 \tag{1}$$

Momentum equations in x and y directions:

$$\frac{\partial M}{\partial t}+\frac{\partial Mu}{\partial x}+\frac{\partial Mv}{\partial y}=-gh\frac{\partial Z_s}{\partial x}-\frac{\tau_{0x}}{\rho}-\frac{f_x}{\rho}+\frac{\partial}{\partial x}\left(-\overline{u'^2}h\right)+\frac{\partial}{\partial y}\left(-\overline{u'v'}h\right) \tag{2}$$

$$\frac{\partial N}{\partial t}+\frac{\partial Nu}{\partial x}+\frac{\partial Nv}{\partial y}=-gh\frac{\partial Z_s}{\partial y}-\frac{\tau_{0y}}{\rho}-\frac{f_y}{\rho}+\frac{\partial}{\partial x}\left(-\overline{u'v'}h\right)+\frac{\partial}{\partial y}\left(-\overline{v'^2}h\right) \tag{3}$$

where t is the time; h is the water depth; u and v are the depth-averaged velocities in x and y directions, respectively; $M=uh$ and $N=vh$ are the flux in x and y directions, respectively; τ_{0x} and τ_{0y} are the bed shear stress in x and y directions, respectively; f_x and f_y are the drag forces per unit area in x and y directions, respectively; $-\overline{u'^2}$, $-\overline{u'v'}$, and $-\overline{v'^2}$ are depth-averaged Reynolds stresses; g is the gravitational acceleration, h is the water depth, ρ is the fluid density; and Z_s is the water level. Depth averaged Reynolds stresses are calculated by the same method with Hosoda (2002) as follows:

$$-\overline{u'^2}=2D_h\left(\frac{\partial u}{\partial x}\right)-\frac{2}{3}k \tag{4}$$

$$-\overline{uv'^2}=D_h\left(\frac{\partial u}{\partial y}+\frac{\partial v}{\partial x}\right) \tag{5}$$

$$-\overline{v'^2}=2D_h\left(\frac{\partial v}{\partial y}\right)-\frac{2}{3}k \tag{6}$$

$$D_h=\alpha h u_* \tag{7}$$

where, D_h is the kinetic eddy viscosity, k is the depth averaged turbulent energy, α is constant (α is set as 0.3 in this study), u_* is the friction velocity. k is given as shown below equation (Nezu and Nakagawa(1993)).

$$k=2.07{u_*}^2 \tag{8}$$

For grid transformation from the Cartesian coordinates to generalized coordinate system, the method by Hosoda et al. (1996) was applied. Finite volume method was used to solve the partial differential equations. The applicability of the model was validated for river flow (Tanaka and Yagisawa 2009).To analyze the moment acting on a tree, M, we considered the drag force, F, including the tree stand structure (Tanaka et al., 2006) was considered as shown in Eqs. (9) and (10), and bed shear stress, τ, was evaluated by Eq. (11).

$$(f_x,\ f_y)=(u,v)\times\frac{1}{2}m\rho C_{d-ref}d_{BH}\sqrt{u^2+v^2}\int_0^h\frac{d(z)}{d_{BH}}\frac{C_d(z)}{C_{d-ref}}dz=(u,v)\times\frac{1}{2}m\rho C_{d-ref}d_{BH}\sqrt{u^2+v^2}\int_0^h\alpha(z)\beta(z)dz \tag{9}$$

$$M=\frac{1}{2}\rho C_{d-ref}d_{BH}\left(u^2+v^2\right)\int_0^h z\alpha(z)\beta(z)dz \tag{10}$$

$$(\tau_{0x},\ \tau_{0y})=(u,v)\times\frac{\rho g {n_b}^2\sqrt{u^2+v^2}}{h^{1/3}},\quad \tau=\sqrt{{\tau_{0x}}^2+{\tau_{0y}}^2} \tag{11}$$

where z (m) is the vertical axis from the ground where trees are vegetated, m (number of trees/m^2) is the tree density per unit area, $C_{d\text{-}ref}$ is the reference drag coefficient (=1 considering a circular cylinder in this study), $C_d(z)$, $d(z)$ is the drag coefficient, cumulative width of tree trunks, and branches (m) at height z, respectively, d_{BH} is the tree trunk diameter at breast height (m), n_b (m$^{-1/3}$s) is the Manning roughness coefficient without vegetation, $\alpha(z)$ is an additional coefficient expressing the vertical tree structure, and $\beta(z)$ is an additional coefficient representing the effect of leaves and the inclination of branches (for details, see Tanaka et al. (2006)).

2.2. Study area and field investigation

Before and after typhoon No.9 on 2007, field investigation was conducted on gravel bars (AR and KU) in Arakawa River, Japan. For other rivers represented in Figure1, field investigations were conducted before and after typhoon No.12 and 15 on 2011. In each site, (1) particle size distribution of river bed material (d_{50} and d_{90}), (2) tree characteristics (as like trunk diameter at breast height (d_{BH}) and tree

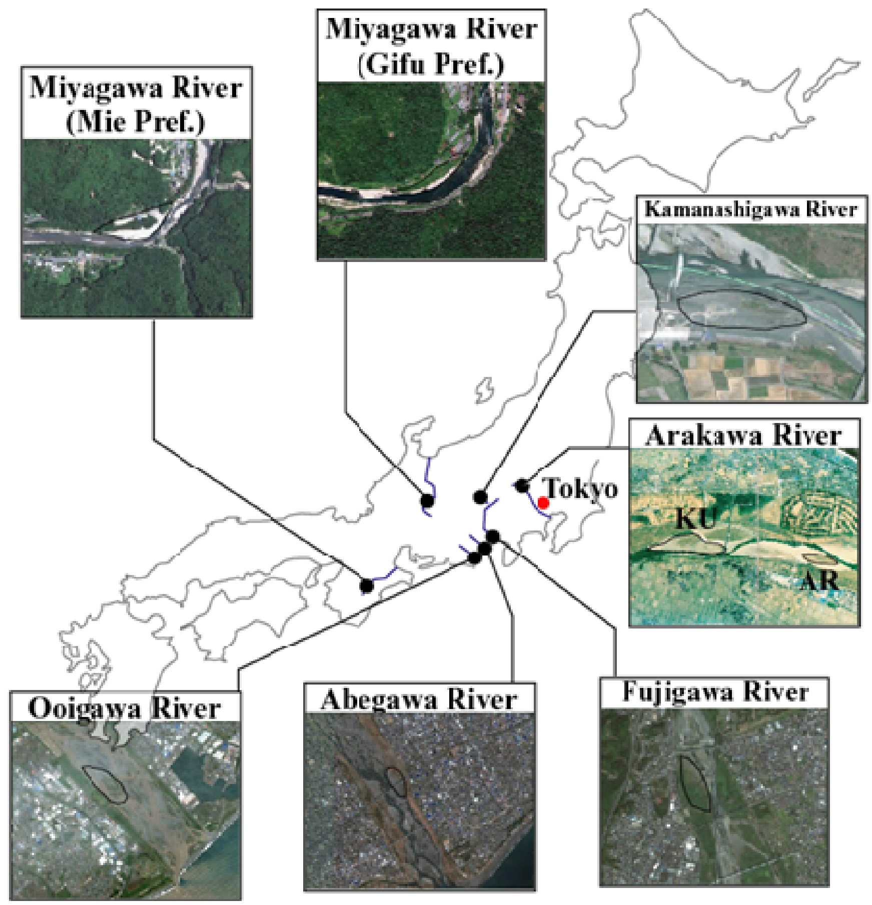

Figure 1 Locations of studied rivers/study areas

Table 1 Characteristics of investigated rivers

River	Distance from river mouth (km)	River bed slope	Particle size of river bed material		Tree characteristics	
			d_{50} (cm)	d_{90} (cm)	Trunk diameter at breast height d_{BH} (cm)	Tree height H_t (m)
Miyagawa River (Mie Pref.)	72 - 78	1/16 - 1/175	18 - 100	20 - 129	2 - 85	2 - 23
Miyagawa River (Gifu Pref.)	58 - 75	1/76 - 1/132	20 - 53	26 - 66	2 - 88	2 - 21
Kamanashigawa River	81	1/97	8 - 21	12 - 36	5 - 17	4 - 7
Ooigawa River	2	1/156	6 - 13	9 - 21	4 - 18	3 - 6
Fujigawa River	2	1/156	9 - 23	11 - 35	6 - 20	3 - 9
Abegawa River	4	1/284	4 - 13	6 - 19	4 - 17	2 - 6
Arakawa River	78 - 80	1/375	2 - 11	3 - 14	2 - 15	2 - 13

height(H_t)), (3) trees breaking or washed out situation due to flood and (4) maximum trace water depth at each flood event (H_{max}) were measured. Particle size distributions of river bed materials at each site were determined. d_{50} and d_{90} were estimated by two methods in this study. One was to take a photo of the river bed and conduct image analysis to determine the particle size distribution when the particle size is large and the screening test cannot be used. The second was to sample bed materials from the river bed surface to 5 cm depth and screen them by using five sieves with 31.7 mm, 19.1 mm, 9.52 mm, 5.66 mm, and 4.00 mm mesh to obtain the particle size distribution. In this study, second method was applied only few locations in Arakawa River.

2.3. Critical shear stress estimation for d50 and d90

To evaluate the shear stress acting on the grain, τ_{*i}, the non-dimensionalized Shields parameter that are usually used for considering 'the gravity force (slope direction)' over 'the weight of the grain in water' were used as below:

$$\tau_{*i} = \frac{\rho g H I_b}{(\rho_s - \rho) g d_i} = \frac{H I_b}{\left(\frac{\rho_s}{\rho} - 1\right) d_i} \tag{12}$$

where ρ_s and ρ are the density of the particles and water, respectively; g is the gravitational acceleration; d_i is the grain diameter at which i% volume passes through the sieve, and I_b is the bed slope. The critical shear stress of d_{50} for the initiation of motion, τ_{*c50} can be approximated from the Shields diagram as:

$$\frac{\tau_{*c50}}{(\rho_s - \rho) g d_{50}} = 0.06 \tag{13}$$

To calculate the effects of the grain size distribution, the critical shear stress of each grain size i, τ_{*ci}, as proposed by Egiazaroff (1965), was:

$$\frac{\tau_{*ci}}{(\rho_s - \rho) g d_i} = \frac{0.1}{\left[\log_{10} 19 (d_i / d_m)\right]^2} \tag{14}$$

where d_m is the medium grain size τ_{*50} and τ_{*90} are derived by substituting d_i = d_{50} or d_{90} in Eq. (12), respectively. τ_{*c90} is derived by substituting d_i = d_{90} in Eq. (14).

2.4. Definition of the Breakage or Overturning Index (BOI) and Wash Out Index(WOI)

In this study, BOI and WOI were defined same with Tanaka and Yagisawa(2009) as shown in below equations.

$$BOI = \frac{d_{BH\max}}{d_{BH}} \tag{15}$$

$$WOI = \frac{\tau_{*90}}{\tau_{*c90}} \tag{16}$$

Where d_{BH} is the tree diameter at the flood event, d_{BHmax} is the maximum d_{BH} that the flood can break the trunk, τ_{*90}, τ_{*c90} is non-dimensionalized shear stress and non-dimensionalized critical shear stress of d_{90}, respectively.

3. RESULTS

3.1. Comparison of simulated and observed water levels for the September 2007 flood in Arakawa River

Contour maps of the simulated velocity and water depth at the peak discharge of the flood are shown in Figure 2(a). The inundations of flood areas “A (island at KU)” and “B (flood plain)” in Figure 2(c) were reproduced well in the simulation (Figure 2(b)). However, it was difficult to confirm that the result of the simulation was good enough to evaluate the situation of the flood as a two-dimensional expression. Therefore, validation of the numerical simulation was conducted by comparing the calculated and observed peak water levels (Figure 2(d) and (e)). The maximum water level at the flood event was determined by the Ministry of Land, Infrastructure, Transport and Tourism of Japan (MLIT) by measuring the height at which debris was attached. The peak water level in the flood simulation was used as the maximum value at each calculation grid, and the calculations agreed well with the observed peak water level as a whole.

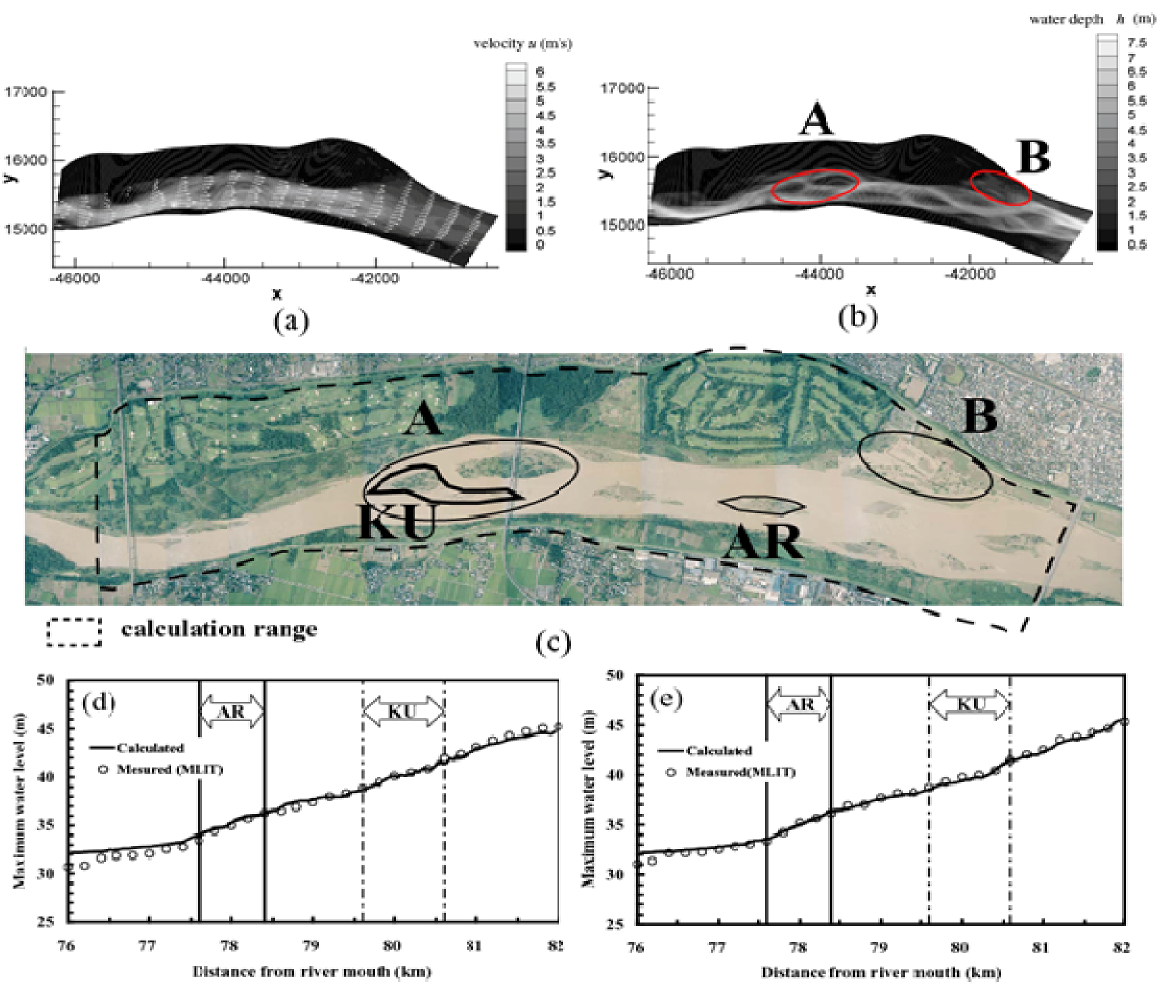

Figure 2 Flow situations of Arakawa River between 76 km to 82 km at the maximum discharge of the September 2007 flood: (a) contour and vector map of simulated velocity, (b) contour map of simulated water depth, (c) aerial photo (provided by Kanto Regional Development Bureau, Ministry of Land, Infrastructure, Transport and Tourism, Japan). (d) and (e) show comparison of simulated and observed maximum water levels along the Arakawa River during the September 2007 flood on the right bank and left bank, respectively. *x* and *y* represent the axes in the orthogonal coordinate system. The two elements in Figure 3(c) representing the inundated areas are “A (island at KU)” and “B (flood plain)”.

3.2. Effect of river bed slope and particle size distribution of river bed material on BOI and WOI

Figure 3(a) shows the comparison between BOI values obtained from drag moment by numerical simulation for Arakawa River and destruction situation of trees due to a flood observed from field investigation. In this study, 4 regions (Region A-D) were defined depending on BOI and WOI values as shown in Figure 3(a). The most of broken trees due to the 2007 flood were categorized in Region B. In contrast, most of trees not received damage were plotted in Region A. However, some data representing broken trees were plotted in Region A. Regarding these data, it was confirmed by the field investigation after the flood that these trees were broken due to attach the large amount of debris. In numerical simulation, the effect of attached debris on the increment of drag force acting on trees was not considered. Therefore, it is supposed BOI values of these trees were underestimated.

Figure 3(b) shows the comparison between WOI values calculated for the flood event and washed out situation of trees observed from field investigation in the Arakawa River. Some data representing overturned trees were plotted in Region C and D. If WOI exceeds 1, most of trees can be washed out in the region BOI > 1. However, not washed out trees were also categorized in Region C even when WOI exceeds 1. This result indicates that critical value for WOI increases with decreasing BOI in

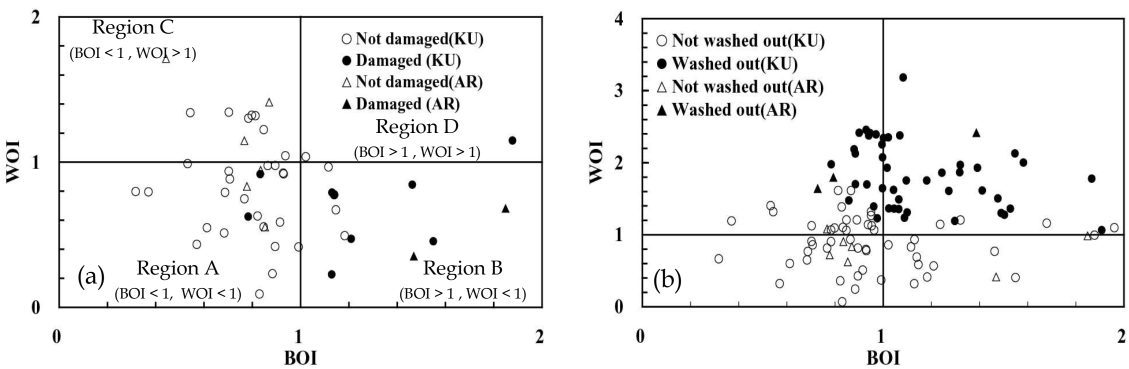

Figure 3 Tree damage situation in relation to the critical value of trunk breakage(BOI) and washed out of trees(WOI) based on the results of numerical simulation for Arakawa River (a) BOI , (b) WOI

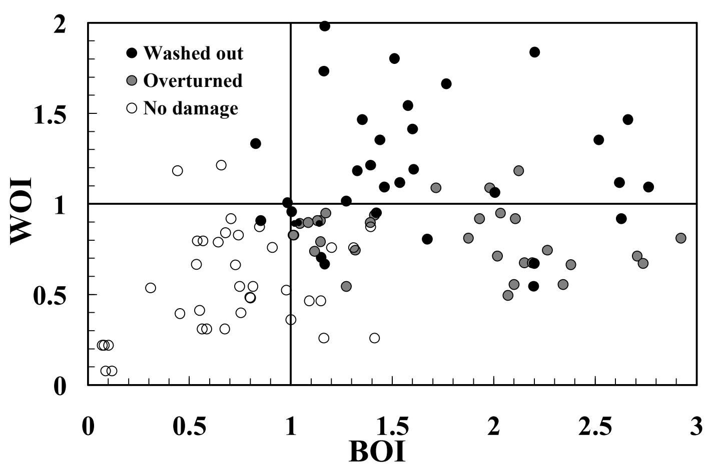

Figure 4 Tree damage situation in relation to the critical value of trunk breakage(BOI) and washed out of trees(WOI) based on the results of field investigations for Miyagawa River (Mie and Gifu Pref.), Fujigawa River, Kamanashigawa River, Ooigawa River and Abegawa River

Region C. In case of small BOI, to break the tree roots become difficult. Therefore, it is thought that the trees were not washed out.

In Figure 4, BOI and WOI values of investigated rivers except for Arakawa River are plotted. In comparison with BOI value for overturned tree and not one, BOI can distinguish well whether trees are broken or not. Similar tendency can be obtained from Arakawa River case (Figure 3(a)). These results indicate that BOI is not affected by river bed slope and particle size distribution. On the other hand, unlike with result of Arakawa River (Figure 3(b)), some washed out trees can be plotted In Region B. It is supposed that large fluid force acted on upper ground part of trees and uprooting of tree-roots occurred even when d_{90} did not move.

4. DISCUSSION

For elucidating the effect of particle size distribution of river bed material on critical value of WOI, relationship between d_{90}/d_{50} and WOI are summarized in Figure 5. In this figure, WOI values plotted in Region B of Figure 4 are represented. Dotted line and line were decided by WOI values for downward limit of washed out trees and upper limit of remained trees, respectively. According to these two lines, it can be found that critical value of WOI increase with increasing d_{90}/d_{50}. When the large size particles are mixed with the small one, movement of the small particles should be prevented by sheltering effect of large particles. Therefore, river bed materials become hard to move with the increase of d_{90}/d_{50}. This result suggests that effect of d_{90}/d_{50} on critical condition of washing out trees needs to be considered in Region B.

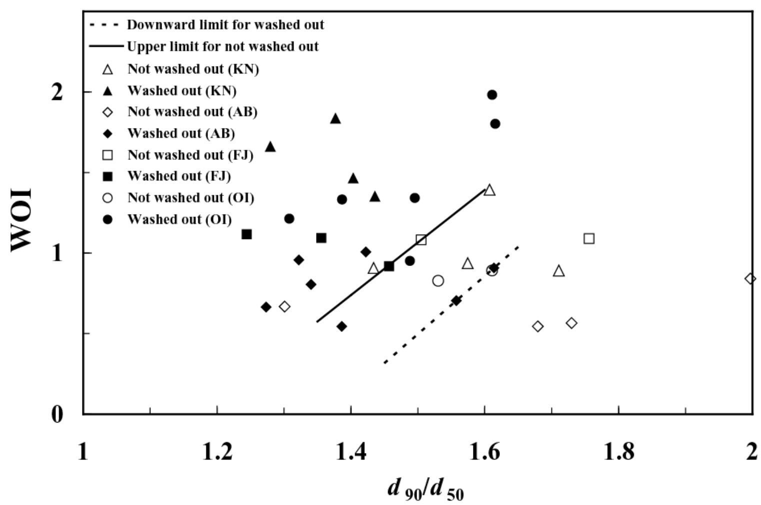

Figure 5 Effect of particle size distribution (d_{90}/d_{50}) on the critical value of WOI
KN : Kamanashigawa River, AB : Abegawa River, FJ : Fujigawa River, OI : Ooigawa River

5. SUMMARY

The following conclusions and recommendations were obtained by this study:
1) Bed slope is found to have no influence BOI, but particle size distribution affects the critical wash-out condition for WOI.
2) Critical value of WOI increases with decreasing BOI in Region C.
3) Effect of d_{90}/d_{50} on critical washout condition of trees needs to be considered in Region B.

6. ACKNOWLEDGMENTS

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Evaluation of Evapotranspiration Methods to Replace Penman-MonteithMethod in the Absence of Required Climatic Data in order to have a better Irrigation Scheduling

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Abstract: *Evapotranspiration (ET) can be identified as a key function of vegetation which develops interaction in between climatologic, hydrological and terrestrial environments. Though there are number of climatologic methods are available to calculate ET such as Penman-MonteithMethod(PMM), Blany-Criddle Method(BCM), Radiation Method (RM), Hargreaves- Samani Method(HSM), Linacre Method(LM), Makkink Method(MM) and Turc Method(TM),in practice Penman-Monteith Method is used since it is likely to provide the most satisfactory result. But, it is not easier to obtain all types of climatic data since malfunctioning ofweather stations.However, the method selected to estimate ET should produce reasonable result with minimum number of climatic data. Thus, this attempt was taken in order to identify the most suitable method to utilize in various months to calculate ET considering the quality of result and input data requirement.According to the obtained results both Hargreves- Samani and Linacre Methods have shown reasonable values to replace PMM. Therefore Irrigation demand can be predicted and will be able to plan fairly successful irrigation scheduling in the field.*

Key words: *Evapotranspiration, Penman-Monteith Method, Climatic data, Mean Absolute Error*

1. INTRODUCTION (CASE STUDY FOR WALAWE BASIN)

The ET is a combination of two separate processes namely evaporation and transpiration. Evaporation is a process in which water is evaporated or lost to the atmosphere from the wet soil. Transpiration is a process in which water is respired or lost to the atmosphere from small openings on the leaf surface. In practice there is no easy way to distinguish those two processes separately. Normally in the initial stage of the cultivation larger fraction of ET represent by the evaporation. But when the crop canopy shaded more and more area with the time, more than 90% of ET comes from transpiration (Allen *et al*. 1998)

Thus in irrigation schemes mainly water losses occur due to ET. Therefore total crop water requirement is directly proportional to ET. Hence ET is typically known as the term of Crop Water Requirement. Therefore in designing and maintaining irrigation schemes, knowledge about ET becomes more important. For an example if the extent is known, required amount of water can be calculated. Then it will be easy to predict whether there would be any deficit or excess in demand via that early precautions can be taken. The Evapotranspiration from a reference surface (a hypothetical grass reference crop with specific characteristics), not short of water, is called the Reference Evapotranspiration and is denoted as ET_0(Allen *et al.*1998).

Recently Food & Agriculture Organization (FAO) has introduced software known as "ET_0 calculator" which calculates ET_0 once required climatic data are fed, in this programme ET_0 is being calculated using Penman-Monteith Method (PMM). But for this methods number of climatic data required is comparatively high. In practice it is not easy to obtain all the parameters from one weather station as most of those stations are not either fully equipped or not function well. On the other hand climatic data are not available free of charge any more.

Therefore it is not easy to strict to a one method. Hence this study was conducted to find out which method is the most appropriate to replace with PMM in each month.

2. METHODOLOGY

2.1. Selection of Study Area

As the study area, considering the cultivation extent it was decided to select Walawe basin (Cultivation extent nearly298 km^2) as it is one of the major basins where cultivation extent is high. There are two Agro-meteorological centers which are maintained by Meteorological Department in Angunakolapelessa and Weerawila close to Walawe basin. Since there is no any other observation center with in the Walawe basin it was decided to use climatic data which were obtained from Angunakolapelessa observation station. A detailed lay out of the basin is shown in Figure1.

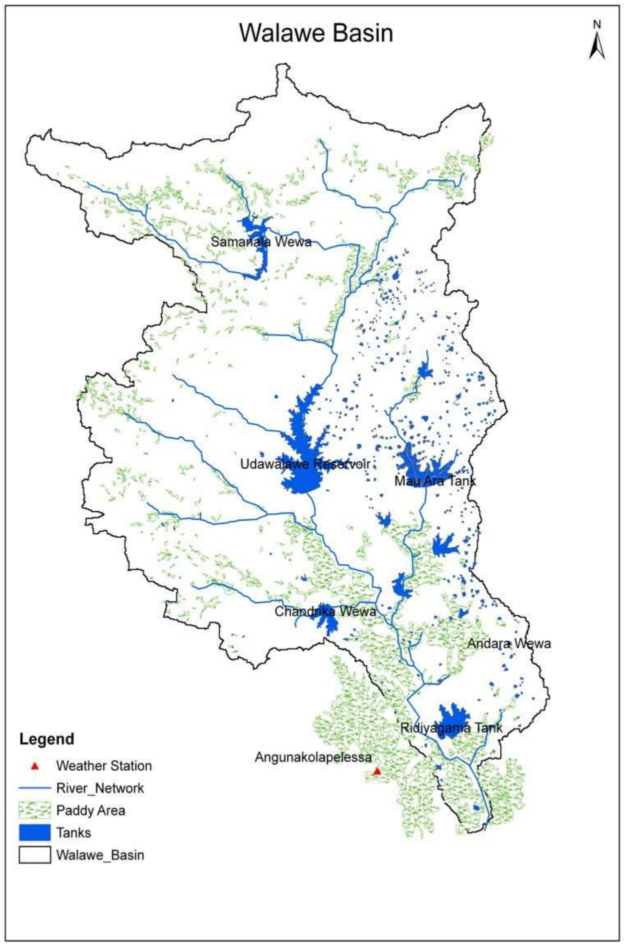


Figure 1 General Lay Out of Walawe Basin

2.2. Climatic Data Collection

For the ET_0 analysis climatic data such as maximum and minimum temperature, maximum and minimum relative humidity, sunshine hour, wind velocity from 2001 to 2005 were collected. Extra-terrestrial radiation was calculated according to the FAO Irrigation & Drainage paper No 56 (Allen *et al.* 1998). In Table 1 it is shown averaged monthly figures of aforesaid climatic data.

Table 1 Averaged Monthly Climatic Data (From 2001 to 2005) from Angunakolapelessa, Sri Lanka

Month	$Temp_{Max}$(^{0}C)	$Temp_{Min}$(^{0}C)	Sunshine Hrs	Wind Speed (km/h)	RH_{max} (%)	RH_{min} (%)
January	32	23	6.6	4.73	85.21	71.11
February	32	22	8.5	5.04	82.46	68.34
March	33	23	8.3	4.36	81.27	68.51
April	33	24	7.0	3.39	83.61	74.78
May	33	25	7.3	6.36	82.95	73.58
June	33	25	7.7	6.98	81.52	71.74
July	33	25	7.4	7.52	78.23	66.99
August	33	25	8.5	7.62	76.89	64.46
September	33	24	7.4	6.57	77.81	68.02
October	32	24	6.9	5.05	80.81	71.94
November	31	24	4.9	3.37	86.08	79.20
December	31	23	6.2	4.19	84.83	74.99

2.3. Estimation of Evapotranspiration

The choice of ET_0 estimation method depends on its suitability in the particular region and the availability of data. Following mentioned are the widely used methods to calculateET_0 (Race *et al.*2009, Hargreves *et al.2003)*.

i. Penman-Monteith Method (MPM)
ii. Blaney- Criddle Method (BCM)
iii. Radiation Method (RM)
iv. Hargreaves- SamaniMethod (HSM)
v. Linacre Method (LM)
vi. Makkink Method (MM)
vii. Turc Method (TM)

The meteorological data required for each method is different to each other. In Table 2 it is shown the data requirement for each climatologic method (Kassan et al. 2001 and Raes et al .2009).

Table 2 Climatic Data Requirement

Climatologic Method	Temperature	Humidity	Wind Speed	Radiation	Sunshine Hrs
Penman-Monteith Method (PMM)	√	√	√	√	√
Blaney- Criddle Method (BCM)	√	√	√		√
Radiation Method (RM)	√	√	√	√	
Hargreaves-Samani Method (HSM)	√			√	
Linacre Method (LM)	√	√			
Makkink Method (MM)	√			√	
Turc Method (TM)	√	√		√	

2.4. Result Analysis Procedures

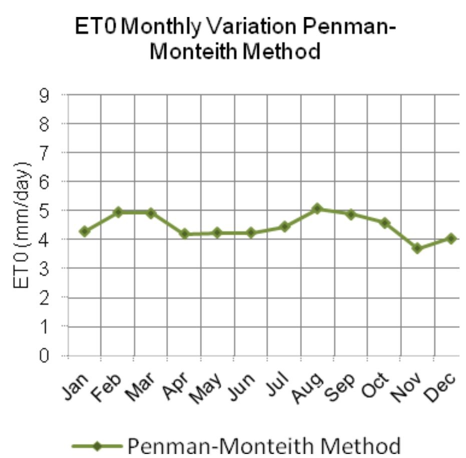
There are several statistical routines are available for this type of result analysis. In this study the criteria onwhich mainly considered was the amount of deviation from the PMM. Therefore Mean Absolute Error (MAE) method was used for analysis (Willmott, 1982).

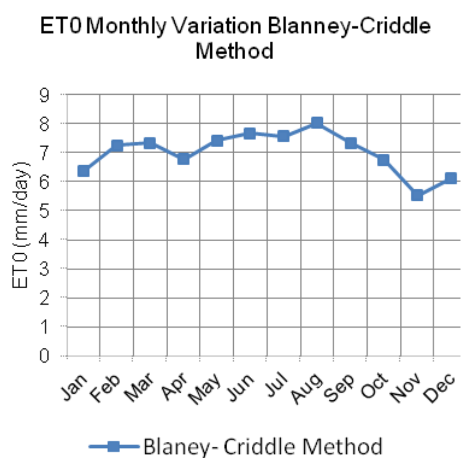
Mean Absolute Error (MAE) = $N^{-1}\sum_{i-1}^{N}(|Pi - Oi|)$

Where N is the number of valid data points (in this case from N=1, since it is considered individual months), P is the test climatologic method ET_0 value and O is the value obtained from PMM.

3. RESULTS AND DISCUSSION

The calculated ET_0 values for each month using various climatologic methods are plotted in Figure 2.





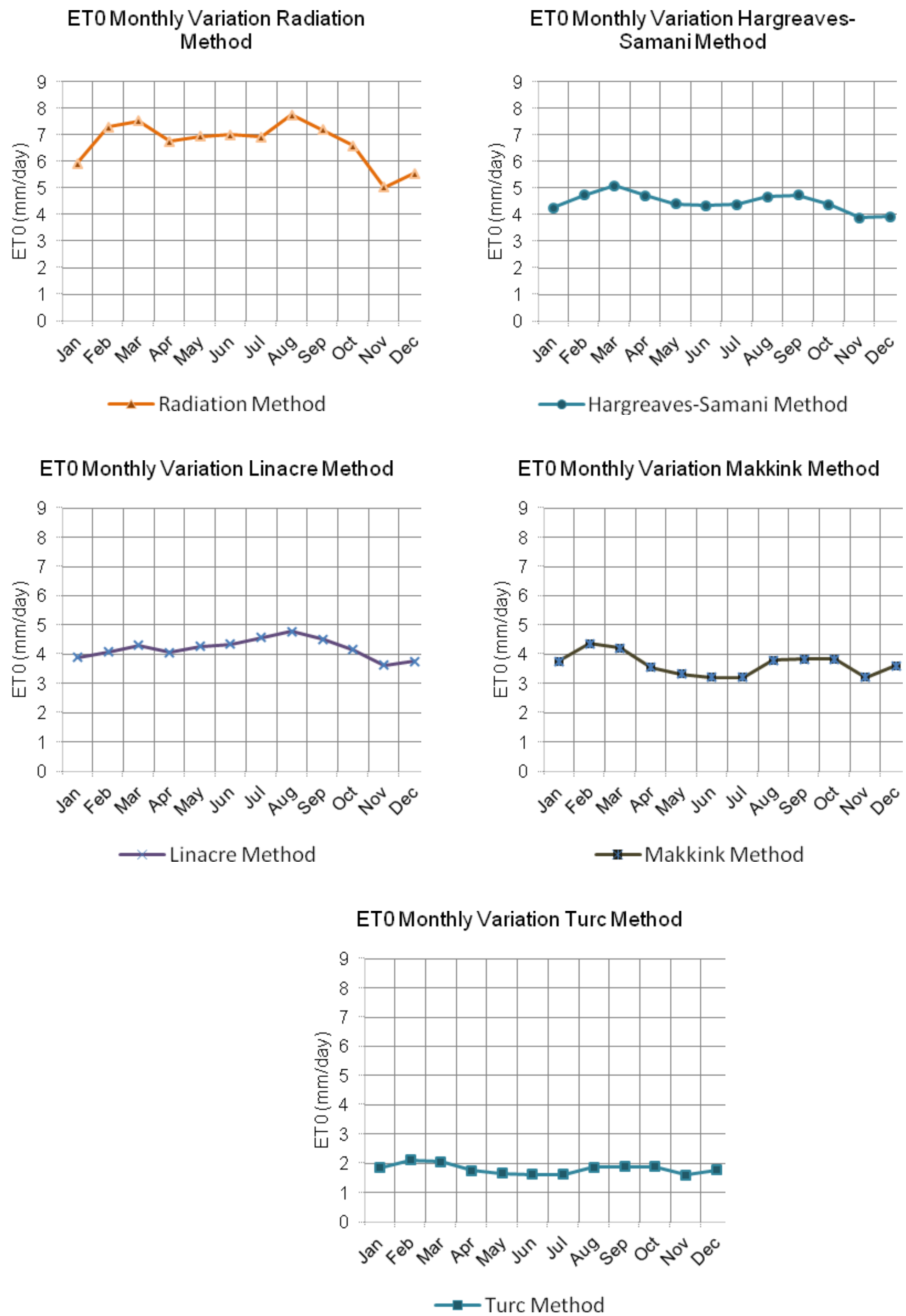


Figure 2 ET0 Variations in Different Climatologic Methods

Generally ET_0 is more sensitive to relative humidity and it has a negative correlation with ET_0 (Yu-Min Wang *et al.*2011). This relationship can be clearly visible from the obtained ET_0 results. ET_0 is comparatively high in February, March, August and September where relative humidity is less and less ET_0 in October, December and January where relative humidity is high.
The calculated MAE for each method in each month is listed out in Table 3.

Table 3 Calculated Mean Absolute Error for Each Method

Month	Blaney-Criddle Method	Hargreaves-Samani Method	Linacre Method	Makkink Method	Radiation Method	Turc Method
January	2.07	0.03	0.40	0.53	1.64	2.44
February	2.31	0.20	0.86	0.58	2.36	2.84
March	2.43	0.18	0.62	0.71	2.59	2.87
April	2.59	0.52	0.12	0.65	2.56	2.43
May	3.18	0.17	0.02	0.93	2.70	2.58
June	3.41	0.08	0.10	1.03	2.75	2.63
July	3.13	0.05	0.14	1.23	2.49	2.82
August	2.96	0.39	0.27	1.27	2.70	3.20
September	2.45	0.16	0.38	1.06	2.31	3.01
October	2.16	0.19	0.43	0.74	2.01	2.70
November	1.84	0.21	0.06	0.49	1.35	2.08
December	2.08	0.12	0.29	0.44	1.52	2.26

Since the objective of the study is to identify the most suitable method to replace with PMM in monthly basis, in Table 4 it is ranked from lowest MAE value to highest (Rank 1 implies the lowest MAE value while Rank 6 implies the highest) for each month.

According to the obtained data, Hargreves- Samani and Linacre methods give closest answers to PMM while Blaney Criddle and Turc methods deviate from PMM. According to the obtained result Hargreves Samani method canbe used for January, February, March, June, July and December months and Linacre method canbe used for April, May, Auguest and November in the absence of required climatic data for PMM .

Table 4 Ranking According to the MAE value

Month	Rank 1	Rank 2	Rank 3	Rank 4	Rank 5	Rank 6
January	HSM	LM	MM	RM	BCM	TM
February	HSM	MM	LM	BCM	RM	TM
March	HSM	LM	MM	BCM	RM	TM
April	LM	HSM	MM	TM	RM	BCM
May	LM	HSM	MM	TM	RM	BCM
June	HSM	LM	MM	TM	RM	BCM
July	HSM	LM	MM	RM	TM	BCM
August	LM	HSM	MM	RM	BCM	TM
September	HSM	LM	MM	RM	BCM	TM
October	HSM	LM	MM	RM	BCM	TM
November	LM	HSM	MM	RM	BCM	TM
December	HSM	LM	MM	RM	BCM	TM

In Normal case, results obtained from Blaney Criddle method closly vary with result obtained from PMM. But in this case it does not show such variation. It is because , BCM is not accurate in extrem climatic conditions such as windy, dry and sunny conditions(C Bruwer *et al*). On the Other hand Radiation method shows a significance deviation. It is because, the performance of Radiation method in arid zone is erratic (Allen *et al* 1998). Genarally, Turc method should agree with PMM. But the reliability of this method depends on the wind speed (Trajkovic *et al.*2007). Since the Angunakolapelessa weather station is closer to coastal area this type of variation can be expected due to high wind speed. Therefor in oder to have a reasonable result from Turc method, a wind speed adjustment factor should be applied to the equation (Trajkovic *et al.*2007*).*

In this research it was used only five years of climatic data. The significance of result can be improved with more than five years data. On the other hand result can be improved further more if it is possible to take the daily climatic data for the calculations.

In general , the result presented in this paper provides a basic idea for the agricultural sector which method will give better estimation of ET_0 for Walawe basin (as well as the sourthern regions where the same climatic conditions are available) in the absence of climatic data for PMM. The obtained result will help to irrigation sector for irrigation scheduling and there by increasing productivity,profit and reducing public conflicts on irrigation water supply.

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Assessment of Spatial Variation in Water Quality: A Case Study at Three Surface Water Bodies in Galle Municipal Council Area

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Abstract: *This study was conducted in two phases. In the first phase, a water quality analysis was conducted on Moragoda Ela, Kepu Ela and Mahamodara Lake. The results revealed that the water quality of the three water bodies was lower than the demanded quality of water for many usages. Moragoda Ela was identified to be the most polluted one. In the second phase, a detailed water quality analysis was carried out on Moragoda Ela. Mean values of COD and BOD_5 were 127.5mg/L and 14.8 mg/L, respectively. Mean values of fecal and total coliforms were 47 and 23471 colonies/100 mL, respectively. Prevailing BOD_5, COD and the total and fecal coliform content exceed the ambient water quality standards for inland waters in Sri Lanka. It is concluded that the water quality deterioration of all three water bodies may be attributed to the large number of industries, institutions and commercial establishments, high population density and the illegal community activities.*

Keywords: *Water quality parameters, standards, effluent discharge.*

1. INTRODUCTION

The apparent water quality of the surface water bodies such as Moragoda Ela, Kepu Ela and Mahamodara lake in Galle Municipal Council (GMC) area is not satisfactory. Most of them receive wastewater discharges from different sources. Rapid urbanization and industrialization have accelerated the water quality deterioration of these water bodies. GMC possesses industries and institutions number of which is enough to cause surface water deterioration. These wastes contain different organic and inorganic pollutants and various pathogenic microbes. The industrial discharges cause euthrophication that leads water bodies to become dead. This may lead to a chain of adverse effects on the aquatic environment and the human beings. Kepu Ela and Mahamodara lake are located in the heart of the GMC area and are at high risk of receiving various types of waste discharges. The catchment of Moragoda Ela covers an area of 6.5 km^2 serviced by a mixed sewer system. The stream receives the effluent discharged by about 50 000 population equivalent. It is surrounded by a number of industries, institutions, commercial and residential establishments, and cultivated areas. These sources pose immense water quality deterioration. In addition urbanization is very rapid in the area.

A vital tool for assessing the extent of contamination in water is water quality monitoring in that the level of pollutants present in water is determined. Water quality monitoring is imperative because the results of such a study can determine the degree of usability of surface water bodies. Ultimately the results would help take preventive actions against the existing water pollution.

The aim of this study was to evaluate the water quality in three urban streams, namely Moragoda Ela, Kepu Ela and Mahamodara Lake in the GMC area with respect to several physical, chemical and biological water quality parameters. The objectives included investigating the variation of water quality parameters along these streams according to land use pattern, general hydraulic conditions such as flow pattern, direction and apparent velocities, the locations of sewer outfalls and abandoned waste disposal sites along the streams.

2. METHODOLOGY

2.1. Phase 1: Water Quality Analysis of Three Water Bodies

Moragoda Ela, Kepu Ela and Mahamodara Lake were selected for the study. Industrial, institutional, commercial, domestic and agricultural areas at the sites of the selected water bodies were identified and marked on a map of the Galle town. Then the selected streams were divided into several segments anticipating that the water quality of each segment would be different. One sampling point was selected from each segment. There were 5, 3 and 2 sampling points in Moragoda Ela, Kepu Ela and Mahamodara Lake, respectively. Sampling points were established within about 3.65 km and 2.25 km from the outfall of Moragoda Ela and Kepu Ela, respectively. The area of the Mahamodara Lake is about 0.125km^2. Water samples were analysed for several water quality parameters, namely temperature, pH, turbidity, solids (Total Suspended Solids, Total Solids, Fixed Suspended Solids) dissolved oxygen (DO), chloride (Cl^-), nitrate nitrogen (NO_3^--N), total nitrogen (TN), sulphate (SO_4^{2-}), alkalinity, Biochemical Oxygen Demand (BOD_5), Chemical Oxygen Demand (COD) and coliform organisms.

2.2. Phase 2: Detailed Water Quality Analysis of Moragoda Ela

In the second phase a detailed water quality analysis was carried out on Moragoda Ela. Based on a site survey 14 sampling points (SP1, SP2, ..., SP14) of different potentials for getting polluted were identified. Figure 1 shows the selected sampling points along Moragoda Ela. Table 1 gives a description of the selected sampling locations.

Table 1 Sampling points of Moragoda Ela

Sampling Points	Land usage and geological features	Bank condition	Flow pattern
SP1	Galle Harbour (outfall)	Both sides covered by gabions	Both upward and downward due to tidal effects
SP2	Bend	Both sides covered by gabions	Both upward and downward due to tidal effects
SP3	Highly residential area	Both sides covered by gabions	Both upward and downward due to tidal effects
SP4	School and Coir factory	One side covered by gabions	Slower flow due to the SP4 located in a bend
SP5	Coir factory	One side covered by gabions	Both upward and downward due to tidal effects
SP6	Residential area	One side covered with gabions and other side covered with vegetation	Resultant flow to the down ward
SP7	Slaughter house	One side covered by gabions	Resultant flow to the down ward
SP8	Pulp and paper industry	Both side covered by gabions.	Resultant flow to the down ward
SP9	Ginger cultivation	No proper bank protection	Resultant flow to the down ward
SP10	Highly residential area	No proper bank protection	Slower flow due to SP6 located in a bend.
SP11	Highly residential area	One side covered with gabions	Resultant flow to the down ward
SP12	Water surface is covered with aquatic plants	No proper bank protection	Resultant flow to the down ward
SP13	Residential area	Gabion wall up to some distance	Resultant flow to the down ward
SP14	Residential area, Cement block industry, Car wash center	No proper bank protection	Resultant flow to the down ward

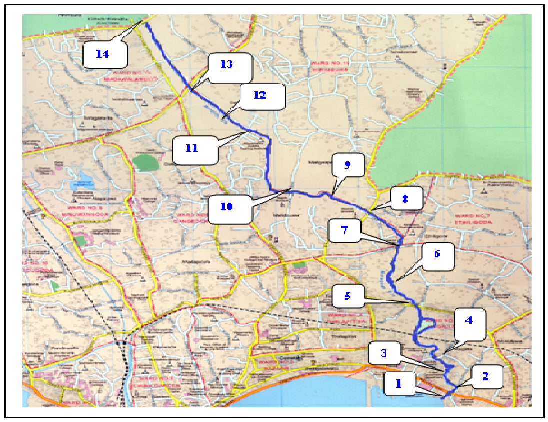

Figure 1 Sampling points {SP1 (at the mouth) ……SP14 (furthest point upstream)}

The following criteria were considered in identifying the sampling points: land usage such as industrial, institutional, commercial, residential and agricultural areas, general hydraulic conditions such as flow patterns, directions (Direction may change due to effect of sea wave) and apparent velocities, the locations of sewer outfalls along the stream, sanitary bypasses and the locations of solid waste deposits with a high content of organic silt and matter. Water samples collected from all the sampling points were analysed for several water quality parameters. Statistical and graphical analyses of the values of water quality parameters were used to quantify the extent of pollution of the stream. Principal Component Analysis (PCA) was used to identify the multi-variation of the water quality and obtain the correlation matrix of all the parameters. The same sampling protocol was used for all the parameters in order to maintain the consistency of sampling and to make all the results accurately comparable.

3. RESULTS AND DISCUSSION

3.1. Water Quality Analysis of Three Water Bodies

Discharges of sewage or industrial waste and the presence of large numbers of microorganisms cause high turbidity. Turbidity in Moragoda Ela and Mahamodara Lake was lower than Kepu Ela. According to David H. Liu et. al. (1997), the three water bodies are not suitable to be used as a source for drinking water and recreational purposes when considering the turbidity level. The prevailing turbidity level may also be harmful to the wildlife propagation.

There is a noticeable increment of total solids in Moragoda Ela. The allowable optimum and maximum limits of TS (Total Solids) for domestic purposes are 500 mg/L and 1500 mg/L, respectively and both Kepu Ela and Mahamodara Lake can be used for domestic purposes, bathing and swimming in terms of solids concentration (David H. Liu. et. al., 1997). The solids concentration of these two water bodies is also within the tolerance limit to be suitable for boating and fishing purposes. pH value changes within 6.2-7.5 and it does not show a significant varation. Alkalinity is higher in Moragoda Ela. In Moragoda Ela, upstream water may not be suitable for drinking and recreational purposes as the pH level is not within the acceptable range (David H. Liu et. al., 1997). Mahamodara Lake satisfies the permissible range of pH.

COD and BOD_5 are two of the most common generic indices used to assess aquatic organic pollution. According to Figure 2, BOD_5 concentration is extremely high in all three water bodies. Figure 2 shows that COD concentration in Kepu Ela has a significant increment due to the presence of high organic waste. This may be an evidence of illegal untreated wastewater discharge from the Mahamodara teaching hospital. COD and BOD_5 values of all three water bodies are higher than the critical values which are 0.5-30 mg/L for BOD_5 and 20-30 mg/L for COD. Therefore the degree of pollution of these water bodies is extremely higher when compared with the permissible concentrations in a surface water body. Minimum DO range that should be in a water body for its water to be used for domestic purposes, recreation and wildlife propagation is 2 mg/L (David H. Liu. et. al., 1997).

The relative low DO values are indicated in Moragoda Ela and it can happen due to the utilization of oxygen for the degradation of organic matters present in the stream. In Mahamodara Lake, DO concentration is relatively high.

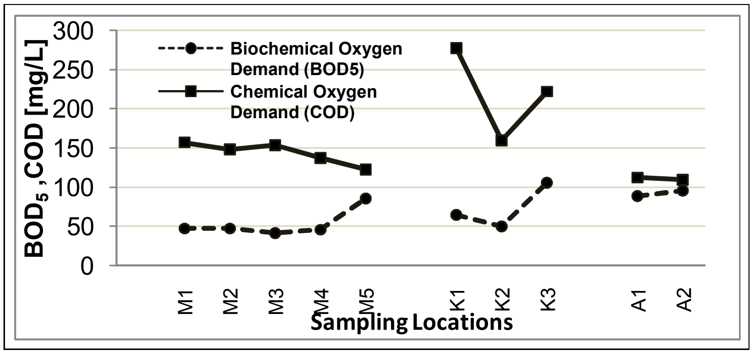

Figure 2 Spatial Variation of BOD_5 and COD at three water bodies

(M-Moragoda Ela, K-Kepu Ela and A-Mahamodara Lake)

Chloride enters waterways through erosion and leaching from soil. Other sources of chloride include seawater intrusion, human and animal wastes, industrial wastes and fertilizers. Chloride is responsible for sewage pollution because of the chloride content in urine. At the upstream area of Moragoda Ela Chloride concentration is in the allowable limit for domestic purposes and wildlife propagation according to David H. Liu. et. al.(1997). Chloride concentration of Kepu Ela and Mahamodara lake have reached an alarming level as the allowable range is 750-2500 mg/L for domestic purposes and wildlife propagation.SO_4^{2-} concentration at the sampling location where the cement block industry is nearby is the highest. It may be due to the illegal discharge of effluent from the cement block industry. The presence of shells and sea water may also cause concentration of SO_4^{2-} to increase.

TN and NO_3^- -N are important parameters since biological reactions can only proceed in the presence of sufficient nitrogen. NO_3^- is an essential nutrient for plants. Excess NO_3^- into waterways may lead to eutrophication, which is the excess growth of algae and aquatic plants. NO_3^--*N* and TN concentrations are higher in Moragoda Ela due to the presence of cultivated areas around the lake. According to the WHO guidelines, the maximum permissible NO_3^- -*N* concentration that should be in potable water is 10 mg/L. Since NO_3^- -*N* concentrations in all segments of Moragoda Ela and two segments of Kepu Ela are less than 10 mg/L, the quality of this water in terms of NO_3^- -N concentration is satisfactory.

The presence of fecal and total coliform organisms in a water sample indicates that water might be harmful to the human health. Very high levels of fecal and total coliform can give water a cloudy appearance, cause unpleasant odors and increase oxygen demand. According to David H. Liu. et. al. (1997), maximum coliform limits are 50, 100 and 10 000 colonies/100ml for domestic purposes, bathing and swimming, and for boating, fishing and aquatic life propagation, respectively. In accordance with Figure 3, the highest coliform concentration exists at the sampling location M4, which is at a slaughter house and a very less concentration of coliforms is shown at the sampling location M3, which is near a ginger cultivation.

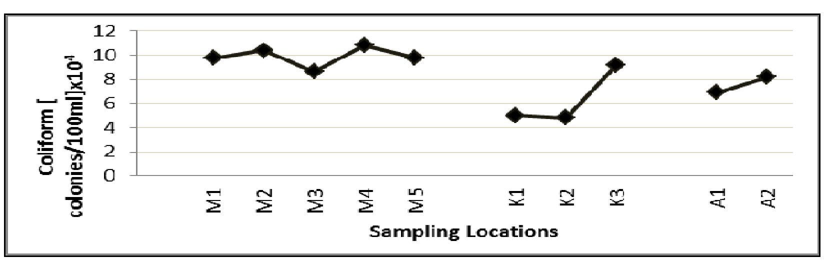

Figure 3 Spatial Variation of Total Coliform Concentrations at three water bodies

(M-Moragoda Ela, K-Kepu Ela and A-Mahamodara Lake)

Overall Moragoda Ela was identified to have got more deteriorated than the others when measured water quality data were compared with the ambient water quality demanded for various usages.

3.2. Detailed Water Quality Analysis of Moragoda Ela

Table 2 shows the mean and standard deviation values for several measured parameters and the compliance status of Moragoda Ela in each parameter with the ambient water quality standards demanded for different usages. The ambient water quality standards enacted by Central Environmental Authority (CEA) were obtained from Priyanka *et al*. 2007.The PCA plot in which the angle between two variables is inversely proportional to the correlation coefficient between those variables indicates that all water quality parameters of SP4, SP6, SP12 are scattered. It implies that those three locations have a relatively similar pattern of water quality variation.

Table 2 Mean and standard deviation (SD) values and compliance status of Moragoda Ela in each parameter with the ambient water quality standards demanded for different usages

Parameter	Mean value	SD	Drinking water with simple treatment	Bathing	Fish and aquatic life	Drinking water, conventional treatment	Irrigation and agricultural	Minimum quality other uses
Turbidity (NTU)	34.5	10.11	√	-	-	-	-	-
Conductivity(μs)	888.3	875.7	-	-	-	-	×	-
pH	7.2	0.5	√	√	√	√	√	√
Minimum DO(mg/L)	5.7	1.4	×	√	√	√	√	√
SO_4^{2-} (mg/L)	404.3	282.7	×	-	-	×	√	-
Cl^- (mg/L)	2248.4	4186.3	×	-	-	×	×	-
Total Hardness(mg/L)	875.7	1515.0	×	-	-	-	-	-
BOD_5(mg/L)	14.8	3.6	×	×	×	×	×	×
COD(mg/L)	127.3	49.3	×	×	×	×	-	×

√ - Satisfy the standard quality × - Not satisfy the standard quality

Temperature along the stream lies within a range of 25.9-29.6 ^{0}C.Temperature does not vary significantly along the stream. According to the PCA analysis, Temperature negatively correlates with BOD_5, COD, salinity, alkalinity and pH, while it positively correlates with turbidity. Pradhan et al. (2003) and Das .(2000) showed a positive correlation between temperature and each of turbidity and DO.

The electrical conductivity values of first two sampling points which are close to the sea are over twofold higher than those of other locations. There is a notable correlation between conductivity and salinity with a correlation coefficient (R) of 0.95 (Figure 4). R between the conductivity and chloride, and salinity and chloride are 0.913and 0.903, respectively. The PCA analysis shows that conductivity is positively correlated with TS,TSS, alkalinity, Ca hardness, total hardness, chloride, sulphate with R of more than 0.8. Kataria and Jain (1995) stated that the conductivity positively correlates with the parameters that directly or indirectly add up to available ions in water. Singh and Kalra (1975) and Lystrom et.al. (1978) used conductivity either generally or specifically to estimate salinity in natural water. Measuring salinity of water is important to irrigation (Metcalf and Eddy 2003).

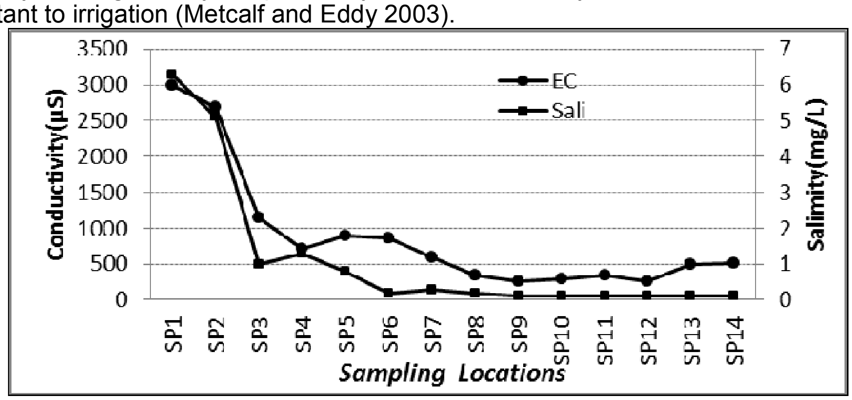

Figure 4 Spatial Variations of Conductivity and Salinity

Turbidity in water is caused by suspended matter such as clay, silt, and organic matter and by plankton and other microscopic organisms that interfere with the passage of light through the water (American Public Health Association, 1998). Turbidity is closely related to total suspended solids (TSS), but also includes plankton and other organisms. Figure 5 shows the variation of turbidity and TSS along the stream. Except three sampling locations (SP4, SP6 and SP7) there is the same variation pattern between turbidity and TSS. Hence turbidity can be used as a surrogate parameter for TSS analysis for Moragoda Ela. Samples were collected from SP3 and SP6 after a heavy rainfall. However the high rainfall has not affected much on the turbidity of the stream because there are sampling points with more turbidity than those two locations. The solid content in a stream largely depends on how frequently the water body gets affected by soil erosion and floods, and the amount of receiving decayed plants and animals. SP4 and SP13 have significant lower values of solids compared to the other sampling points. This is probably due to the fact that gabions have been placed in the stream banks and the area has a good vegetation cover.

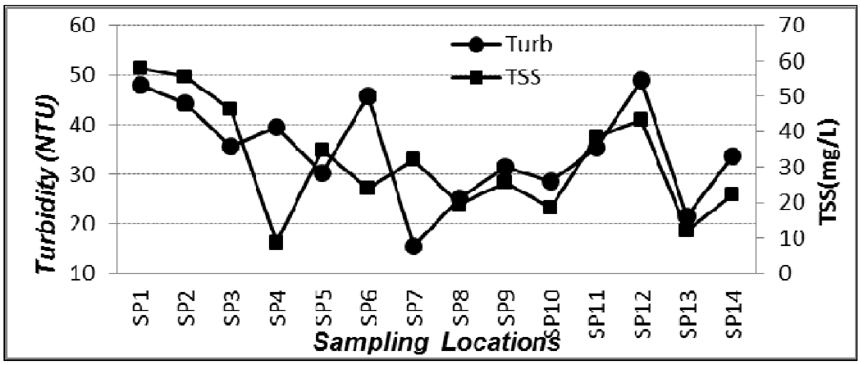

Figure 5 Spatial Variations of Turbidity and Total Suspended Solids

pH changes within 6.5-8.3 along the stream and it does not show a significant variation with the type of surrounding area. Alkalinity level lies within 90-365 mg/L. The highest value of pH is 8.30. Water with a pH of less than 4.8 or greater than 9.2 can be harmful to aquatic life, and alkalinity of 100-200 mg/L will sufficiently stabilize the pH in a stream (Mitchel, M.,2003). The alkalinity value of Moragoda Ela lies within this range and then scaling or corrosion potential of water is low. The drinking water standard for pH and the range of pH demanded by irrigation water lies within 6-8.5. Significantly high values of alkalinity were observed throughout the investigation period at sampling point 1 and 2. Das and Pandey (1978) stated that high alkalinity indicates pollution.

The permissible concentration of chloride for drinking water with simple treatment is 200mg/L and that for agricultural irrigation is 100mg/L (Priyanka *et al*. 2007). Hence the water in the reaches from SP6 to SP14 is suitable for agricultural irrigation in terms of chloride concentration (Figure 6).Sulphates of Moragoda Ela vary between 160-970mg/L (Figure 6). The maximum standard value of sulphate in drinking water with simple treatment is 250mg/L and for agricultural irrigation is 1000mg/L (Priyanka *et al*. 2007).The high sulphate concentration at the upstream of Moragoda Ela may be due to the discharge of detergent-laden sanitary water directly into the stream by the surrounding residential areas.

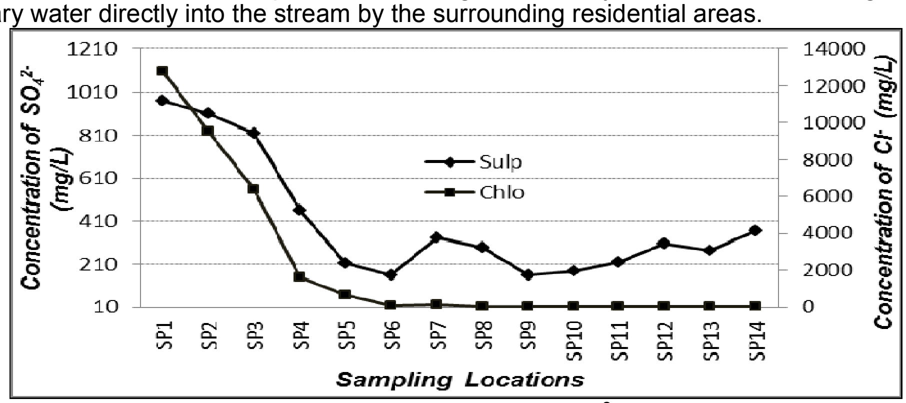

Figure 6 Spatial Variations of SO_4^{2-} and Cl^-

Minimum DO range that should be in a water body for its water to be used for domestic purposes, recreation and wildlife propagation is 2 mg/L (David H. Liu. et. al., 1997). BOD_5 is often used to evaluate the biodegradable fraction, and COD is the total organic pollution load of waters contaminated by reductive pollutants. DO concentration varies from 8.55 mg/L to 3.49 mg/L. SP7 experienced the lowest DO concentration. The same sampling location has the second highest BOD_5 concentration (Figure 7). Hence the low DO level may be attributed to the illegal discharge of slaughter house wastewater which is rich in biodegradable organic matter and suspended solids.

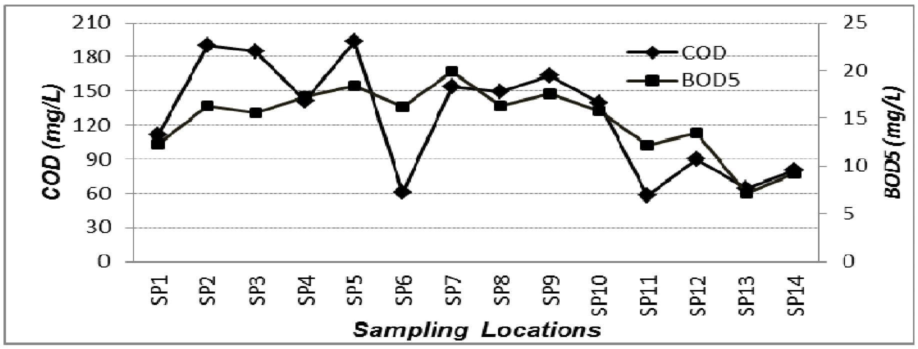

Figure 7 Spatial Variations of Biochemical Oxygen Demand and Chemical Oxygen Demand

The COD and BOD_5 values of all sampling points except few are higher than the critical values which are 10 mg/L for BOD_5 and 20 mg/L for COD. Therefore the degree of pollution of this stream is high when compared with the permissible concentrations in a stream. Mean value of COD and BOD_5 are 127.5mg/L and 14.8mg/L, respectively. Prevailing BOD_5 and COD in Moragoda Ela exceed the ambient water quality standard for inland waters in Sri Lanka. According to Figure 7, COD concentration varies from 193.84 mg/L to 58.38 mg/L. SP2 shows the highest COD value. This point is at a bend of the river. It can reasonably be argued that this may be due to the accumulation of organic suspended solids at the bend. Both BOD_5 and COD are in a trend of decreasing toward the upstream. The lower COD value at SP14 implies that the illegal discharges from the cement industry may not have caused much effect on the increase of organic load into the stream. SP8 which is close to the pulp and paper industry has relatively high BOD_5 and COD values compared to the locations of its upstream area. The pulp and paper industry discharges wastewater rich in COD and deficient in BOD_5. Therefore there may be another source of biodegradable organic matter at that point such as the discharge of domestic sewage. High values of COD due to accumulation of domestic sewage were reported by Mohan et al. (2007) ranging between 88 - 535 mg/L. Sharma and Gupta (2004) also found that the municipal waste water is responsible for maximum organic pollution, resulting in an increased BOD_5. There is an inverse relation between DO and oxygen utilization in terms of BOD_5 and COD (Das, 2000). BOD gives a complete picture of the nature and extent of pollution and about the water quality (Kumar and Sharma, 2002).

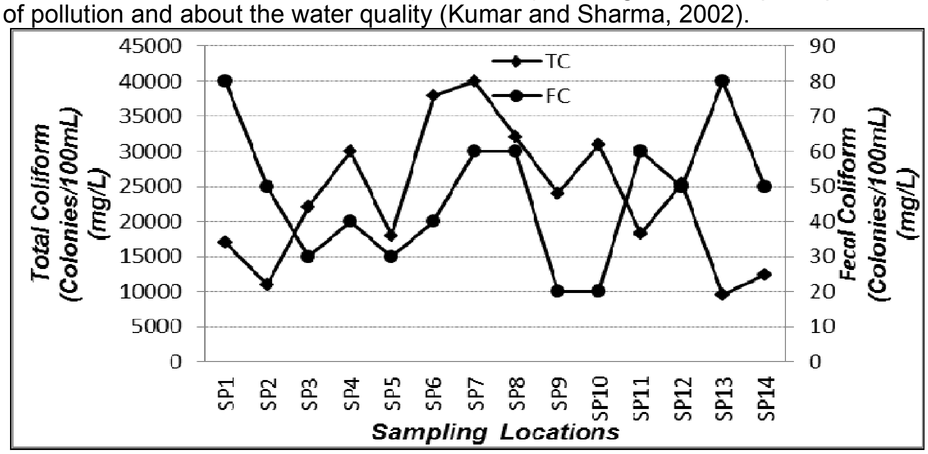

Figure 8 Spatial Variations of Total and Fecal Coliform Concentrations

Figure 8 shows the variation of total coliform organisms. Mean values of fecal and total coliforms are 47 colonies/100mL and 23471 colonies/100mL, respectively. Prevailing total coliforms in Moragoda Ela exceed the ambient water quality standard for inland waters in Sri Lanka Priyanka *et al*. 2007).The mean fecal coliform value does not satisfy the ambient water quality standards for activities other than irrigation and agricultural purposes. The highest total coliforms content is seen at SP7 which is closer to the slaughter house.

4. CONCLUSIONS

It can be concluded that the water quality of Moragoda Ela, Kepu Ela and Mahamodara Lake is lower than the demanded quality of water that is used for domestic purposes, bathing and recreational activities. They are also not suitable to be used as water sources for water treatment plants. The study revealed that Moragoda Ela has relatively a high pollution level. The degree of pollution decreases along the river from downstream to upstream. Different segments of Moragoda Ela have got polluted to different degrees. The level of pollution is highly dependent on the parameter of concern. Especially first three sampling locations which are near the sea experience most of water quality parameters exceeding the demanded quality of water that is used for many activities. Those 3 locations posses elevated values for chloride and sulphate concentrations, conductivity, salinity, alkalinity and hardness. There is a notable correlation between conductivity and salinity with a correlation coefficient (R) of 0.95. Conductivity is also positively correlated with TS, TSS, alkalinity, Ca hardness, total hardness, chloride, sulphate with R of more than 0.8. Turbidity can be used as a surrogate parameter for TSS analysis for Moragoda Ela because there is a good correlation between the two parameters except at three sampling locations.The stream receives a high organic loading from the surrounding industries and residential and commercial establishments. This may be the major source of pollution in the stream.The poor implementation of rules and regulations on wastewater discharge and less contribution by the government to maintain and clean those water bodies may also contribute to the existing pollution.

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Estimation of drag coefficient of trees considering the tree bending or overturning situations

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Abstract: *Drag coefficients of a real tree trunk and the sheltering effects of an upstream trunk on a downstream one in a linear arrangement with different spacings and inclinations were investigated in detail. In addition, for elucidating the change of drag coefficient for an overturned tree, drag force acting on a real tree with roots was also measured in this study. For the measurement of drag force with different inclinations, Terminalia Cattapa and Albizia sp., vegetated in Sri Lanka, were selected in this study. Drag coefficient of inclined tree trunk has the similar tendency in relation to the Reynolds number with that of vertical standing tree investigated in Tanaka et al.(2011). For the vertical tree trunk with rough surface, drag coefficient of rear-side tree trunk was decreased with decreasing L/d (where, L is spacing and d is the diameter of trunk). In addition, as a result of mutual interference experiment of two inclined tree trunk, the drag coefficient of rear-side trunk decreased with the increase of the inclination. Under the influence of the increment of projected area due to existence of roots and shear force acting on tree trunk surface, the drag coefficient of a tree with roots became similar value (1.0-1.2) comparing with that of a vertical standing tree.*

Keywords: *drag coefficient, Reynolds number, bending, overturning, tree roots, sheltering effect*

1. INTRODUCTION

In recent years, forestation in a river becomes a big problem in Japan. The trees in a river increase the drag force to the flow, rise the water level at flood events when they are dense and not broken. When the tree is broken, it produces floating debris and causes a damage to downstream bridges and other structures in a river. Therefore, it is very important to manage trees in a river, and it is also essential to estimate the drag characteristic of trees with high-accuracy for the design and management of river.
Previous studies usually placed smooth circular cylinders (drag coefficient around 1-1.2) on a flume bed in a submerged or emergent condition and investigated the effects on flow (Li and Shen 1973, Petryk and Bosmajian 1975, Baptist et al. 2007). However, the drag coefficient or flow structure of real trees depends on various factors, including velocity (Mayhead 1973, Kouwen and Fathi-Mogfadam 2000, Armanini et al.l 2005), velocity and canopy morphology (Su et al. 2008), flexibility (Schoneboom and Aberle 2009, Wunder et al. 2009), the presence of leaves in emergent or submerged condition (Wunder et al. 2010), and plant density (Nepf 1999, Takemura and Tanaka 2007, Tanaka and Yagisawa 2010). Moreover, Tanaka and Yagisawa (2009) investigated the breakage pattern of trees by field investigation after a large flood event, and classified the unwashed-out breakage pattern as trunk bending or overturning. If a tree is bent, it is important to know the drag coefficient of a bent trunk or branches. When a tree is overturned,

the knowledge of a drag coefficient of uprooted roots is necessary. However, much details/information about the drag coefficient is not available under these situations.

Figure 1 shows the schematic diagram of our study regarding the tree drag characteristics. Tanaka et al.(2011) have already conducted Experiment 1 and Experiment 2 in Figure 1. Therefore, the objectives of this study were to elucidate the drag characteristics of real trees especially paying attention on 1) inclination to the flow (Experiment 3), 2) sheltering effect by frontal trees on rear-side tree (Experiment 4) and 3) overturned condition (Experiment 5). For their objectives, wind tunnel experiments and towing tank experiments were conducted.

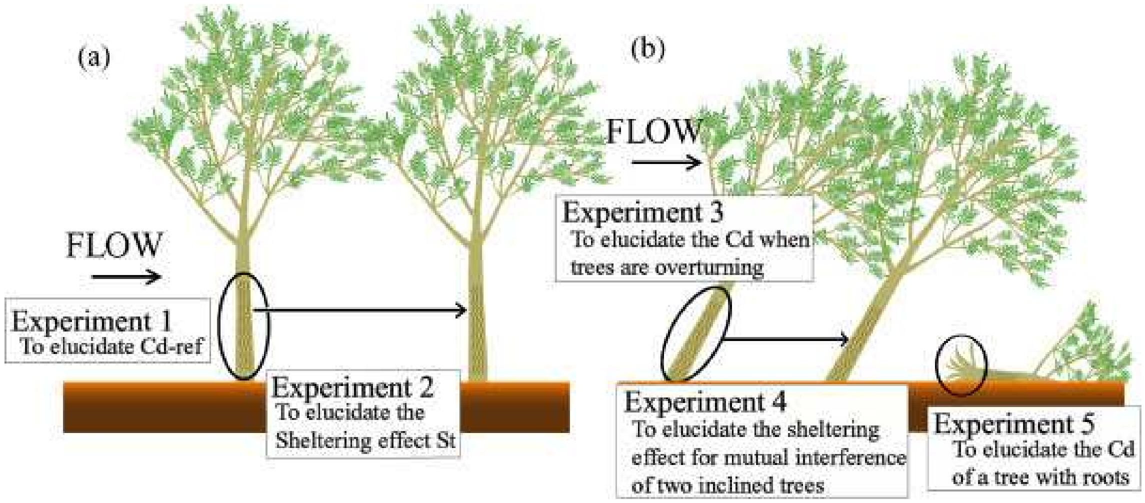


Figure 1 Schematic diagram

(a) Drag characteristic of vertical tree (b) Drag characteristic of inclined tree

2. MEASUREMENT METHOD

In this study, the following three experiments were conducted.

1) Experiment 3: estimation of the drag coefficients of an inclined tree.
2) Experiment 4: the sheltering effect due to mutual interference of two inclined trees
3) Experiment 5: the drag coefficient of trees with roots

However, comparison with results of our study (Experiment 3-5) and that of Tanaka et al.(2011) is needed in section 3(Results and Discussions). Therefore experimental method and results of Tanaka et al.(2011) are also shown in this study.

Experiment 1 and 2 were carried out at an Eiffel type wind tunnel facility with a 0.5 m × 0.5 m cross section at Saitama University (Figure 2). The setup for Experiment 1 is shown in Figure 2b. Figure 3 shows the cross sections and side views of trunk models used in Experiment 1. In this study, *Robinia pseudoacacia* (rough surface; hereafter, Trunk-R), willow(*Salix* sp.; hereafter, Trunk-W), a circular polyvinyl chloride cylinder(smooth surface; hereafter, Cylinder-S) and a different type of circular cylinder (roughness surface; hereafter, Cylinder-R) were used for Experiment 1. Cylinder-R had a different aspect ratio and non-dimensionalized equivalent roughness height as Trunk-R.

First, the drag force acting on Cylinder-S was measured directly, and the drag coefficient was calculated. Then, the relationship between Reynolds number ($R_e = ud/\nu$, where u is the reference velocity, d is a diameter of the tree trunk or the cylinder, and ν is kinematic velocity) and drag coefficient was derived using Trunk-R, Trunk-W, and Cylinder-R to elucidate the effects of different surface roughness conditions on drag characteristic. Trunk-R, Trunk-W, Cylinder-S, and Cylinder-R were 0.35 m in length. Two rods were inserted into the cut ends of the specimen to minimize the effect of three-dimensional separation of the boundary layer (the so-called horse-shoe vortex) at the upper and lower walls. Trunk-R, Trunk-W, Cylinder-S, or Cylinder-R with rods were separately installed at 4.3 m from the inlet of the wind tunnel. Two Load-cell instruments (the product made by AIKOH: RX-10, the measurement maximum load is

100N, the resolving power is 1/1000) were set at the upper and lower ends of each rod attached to the model, and the drag forces acting on the upper (*Fu*) and lower (*Fl*) parts of the model were measured.

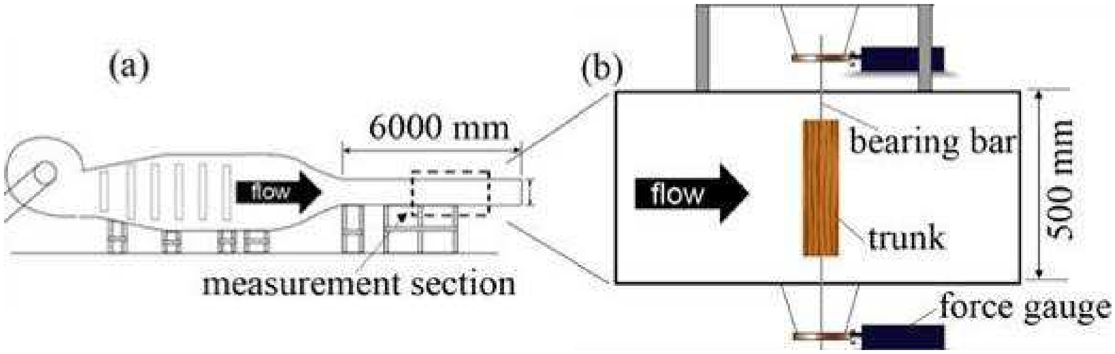


Figure 2 Experimental apparatus, (a) Wind tunnel, (b) set-up for Experiment 1 (Tanaka et al.(2011))

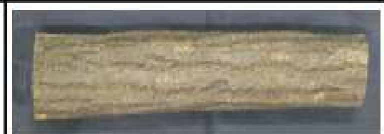
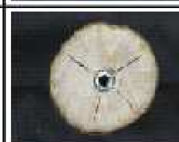
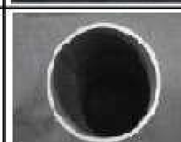
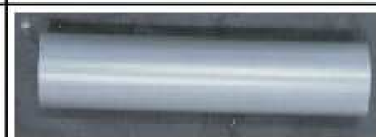
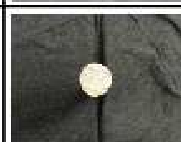
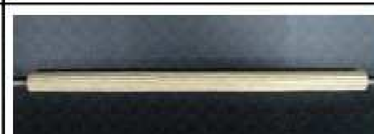
	d (cm)	k_s/d $\times 10^{-3}$	cross section	side view
Trunk-R	7.8	45		
Trunk-W	7.7	11		
Cylinder-S	7.6	–		
Cylinder-R	2.0	45		

Figure 3 Surface roughness characteristics of trunk model.
The cross section and side views of Trunk-R, Trunk-W, Cylinder-S, and Cylinder-R.
The maximum k_s/d (d : trunk diameter, k_s : equivalent roughness height) value of each model is shown in the table (Tanaka et al.(2011))

The drag coefficient Cd of the model by the total drag force F (F = Fu + Fl) was calculated from the following equation.

$$C_d = \frac{2F}{\rho u^2 A} \quad (1)$$

Where, F is the drag force (N), ρ is the density of water (kg/m3), u is velocity (m/s) and A is a project area (m^2).

A towing tank facility in the University of Peradeniya was also used in this study. The towing tank has a carriage which is installed on the channel with the length of 50 m, the width of 2 m, and the depth of 2 m (Figure 4).

In each above-mentioned towing tank experiment, tree models are installed in the rear side of the carriage. Drag force was measured by Load cell as the same one used in Experiment 1 with running the carriage. The drag force (*F*) was calculated by the measured value of *F2* in the experiment and substituted in the drag coefficient equation considering the balance of a moment as bellow:

$$F3 = \frac{F2 \times L}{L + y} \quad (2)$$

Where, F2 and F3 are the drag at the point of P2 and middle of tree in the figure 4,respectively. *y* is the distance from the point where *F2* acts to the half of water depth (m), *L* is the distance from the point where *F1* acts to the point where *F2* acts (= 1.042 (m) in this study).

The experimental conditions of Experiment 3 are shown in Table 1(a). The inclinations of trees in the experiment were set in two cases as θ = 0 (vertical) and 30 degrees. The range of Reynolds number R_e is from 2.0×10^4 to 9.0×10^4. *Terminalia Cattapa* with a mean diameter of 3.5cm was used as the trees model. The reason why this species was chosen as roughness condtion of tree trunk surface is similar to that of *Willow.*

The experimental conditions of Experiment 4 are shown in Table 1b. In Experiment 4, two inclined tree models were arranged in the towing direction. The same *Terminalia Cattapa* as Experiment 3 with 3.5 cm-diameter (*d*) was used for a rear-side tree. The trees model with 4.0cm diameter tree was used for a front side tree. Under the condition, the sheltering effect of drag force by front side tree on rear side tree was measured. The inclination of the trees model were conducted two cases as θ = 0°, and 30°. In addition, non-dimensional length *L/d* (where, *L* is the distance between trees) was set two cases (4 and 10) in each inclination. Then, a total of 6 cases were conducted in Experiments 3 and 4.

Finally, the experimental conditions of Experiment 5 are shown in Table 1-(c). *Albizia* of which trunk diameter is 5.0cm and mean root diameter is 32.0 cm was used in the Experiment 5. The tree model with roots was installed as θ = 90 degrees (horizontal) and roots were set at the front side to the flow considering the overturned situation. Moreover, the range of a Reynolds number R_e was set as the same range with the Experiment 1 (2.5×10^4 to 1.2×10^5) by using 5.0 cm-diameter tree in order to compare with θ = 0 degree case.

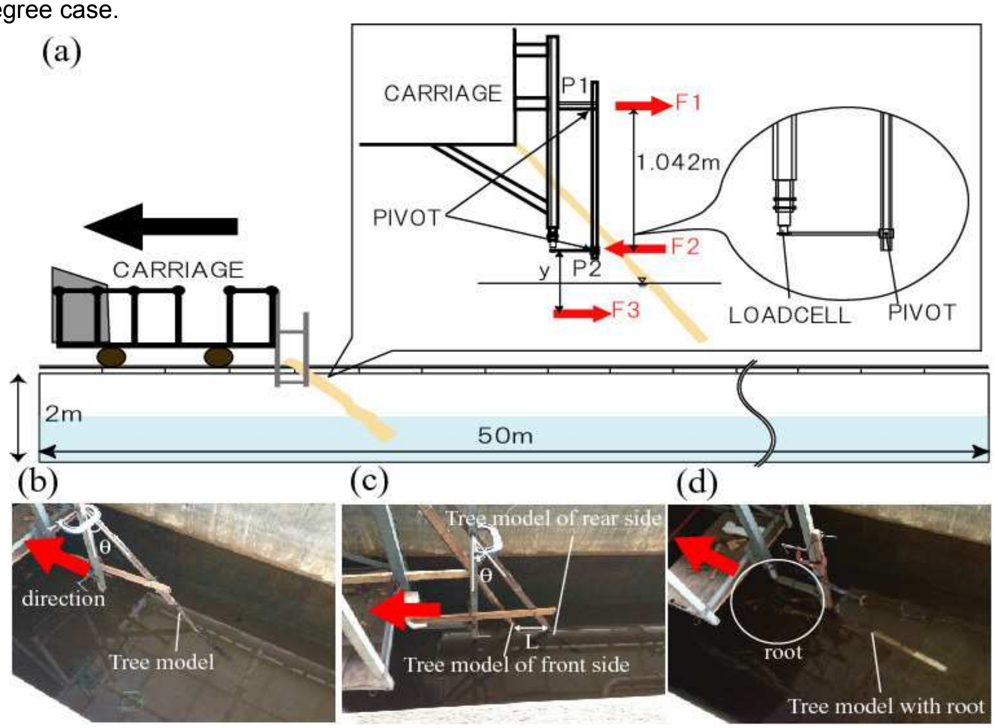


Figure 4 Experimental set up
(a) Towing Tank (b) set up of an inclined tree
(c) set up of two inclined trees (d) set up of a tree with roots

Table 1 Experiment condition and Material.
(a) Experiment 3 (b) Experiment 4 (c) Experiment 5

(a)

Tree Species	diameter d (m)	angle θ (°)	velocity u (m/s)
Terminalia Cattapa	0.035	0	0.50, 0.75, 1.00,1.25, 1.50, 1.75,2.00, 2.25
		30	

(b)

<table>
<tr><th>Tree Species</th><th>diameter d (m)</th><th>angle θ (°)</th><th>L/d</th><th>velocity u (m/s)</th></tr>
<tr><td rowspan="2"><Front side> Terminalia Cattapa</td><td rowspan="2">0.04</td><td rowspan="2">0</td><td>3.7</td><td rowspan="4">0.50, 0.75, 1.00, 1.25, 1.50, 1.75, 2.00, 2.25</td></tr>
<tr><td>10.6</td></tr>
<tr><td rowspan="2"><Rear side> Terminalia Cattapa</td><td rowspan="2">0.035</td><td rowspan="2">30</td><td>3.6</td></tr>
<tr><td>11.4</td></tr>
</table>

(c)

Tree Species	diameter d (m)	angle θ (°)	velocity u (m/s)
Albizia	0.05	90	0.50, 0.75, 1.00, 1.25, 1.50, 1.75, 2.00, 2.25

3. RESULTS AND DISCUSSION

3.1. Effect of surface roughness of tree trunk and roots on drag coefficient (Experiment 1 and 5)

Figure 5 shows the variation of drag coefficient with Reynolds number (R_e) for the real tree trunk (Trunk-R, Trunk-W), circular cylinders (Cylinder-S, Cylinder-R) and tree with roots. Although the drag characteristics of real trunks have been investigated in wind tunnel experiments, the characteristics can also be applied to the vegetative drag of trees in water flow. The Mach number ($M_a = U/a$, U is the relative velocity of fluid, a is the acoustic velocity) in this study was around 0.02‑0.07 and was less than 0.3, which is often used as a maximum Mach number for incompressible gas flow (Schlichting 1979, Potter and Foss 1982) The drag coefficients of Trunk-R, Trunk-W, and Cylinder-S were smaller than those obtained in previous research (1.0‑1.2) (Wieselsberger 1921). The aspect ratio is supposed to affect the drag coefficient. It is well known that the drag coefficient of a circular cylinder with a smooth surface decreases with decreasing aspect ratio (Okamoto and Yagita 1973, Uematsu and Yamada 1995). The aspect ratio of Trunk-R, Trunk-W, and Cylinder-S was about 4.5, and the aspect ratio was supposed to greatly affect the drag coefficients because the effect of the detour flow from the gaps at the top and bottom of the cylinder became large.

A different tendency can be seen between the drag coefficient of Trunk-R and Trunk-W. The percentage of the standard deviation of the drag force on Trunk-W, which has small roughness, and the average drag force on it was around 2-8 %. In contrast, the percentage for Trunk-R, which has large surface roughness, was around 5-20 %. In the case of Trunk-R, the boundary layer of the trunk surface may become turbulent because Trunk-R had a large k_s/d than that of Achenbach (1971). Therefore, the flow had already become supercritical, and the drag coefficient of Trunk-R showed a constant value without depending on R_e. In contrast, the drag coefficient of Trunk-W decreased gradually with increasing R_e. In the case of Trunk-W, the boundary layer of the trunk surface may range within a transition region from laminar to turbulent because the k_s/d of trunk-W was similar to the condition of $k_s/d = 1\times10^{-3}$-9×10^{-3} in Achenbach(1971). The results of the trunk experiments in this study did not show a large change of drag coefficient with changing R_e as shown in previous research (Wieselsberger 1921, Achenbach 1971) because the surface roughness of real tree trunks is irregular.

On the other hand, the drag coefficient of Cylinder-R with a large aspect ratio was evaluated to be around 1.0. This value is similar to that of the smooth cylinder with the same aspect ratio. In addition, the drag coefficient of a tree with roots was almost same value for Cylinder-R at low Reynolds number (R_e) and for Trunk-R, Trunk-W and Cylinder-S at high Reynolds number (R_e). In case of a tree with roots (model for overturned tree), the projected area was decreased in comparison with vertical standing tree. However, drag coefficient of a tree with roots is similar (1.0-1.2) to that of a vertical standing tree.

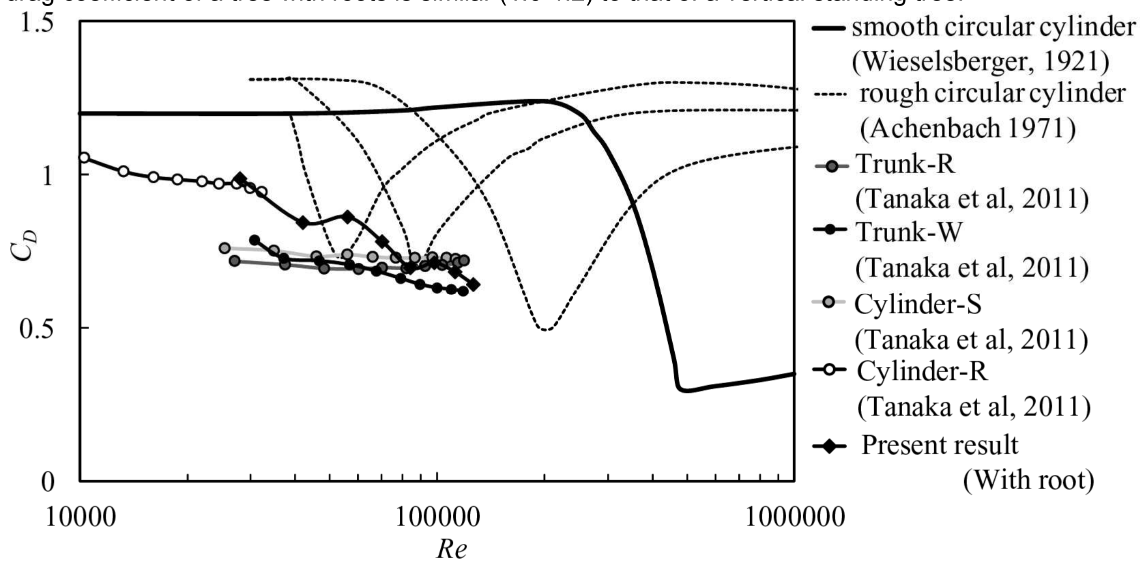


Figure 5 Variations of drag coefficient (C_D) of a tree with roots

3.2. Influence of mutual interference of two trunks and an inclined tree and two inclined tree on drag coefficient (Experiment 2, 3 and 4)

Using two trunk models, the variation of drag coefficient was measured by changing R_e and L/d. The drag coefficient of rear-side model was decreased with decreasing L/d for both surface conditions of the trunk models (Trunk-R and Trunk-W). However, it shows quite different tendency with Reynolds number because the drag force decrement for the frontal trunk at high R_e depends on the surface condition, as shown in Figure 6. In case of Trunk-W, boundary layer of the trunk surface may be ranged within transition region from laminar to turbulent as mentioned in 3.1. Therefore, the wake region becomes narrower with increasing R_e in comparison with laminar or turbulent boundary layer, and approach velocity is supposed to be larger with increasing Reynolds number. The drag force and coefficient of Trunk-W was then increased with increasing Reynolds number.

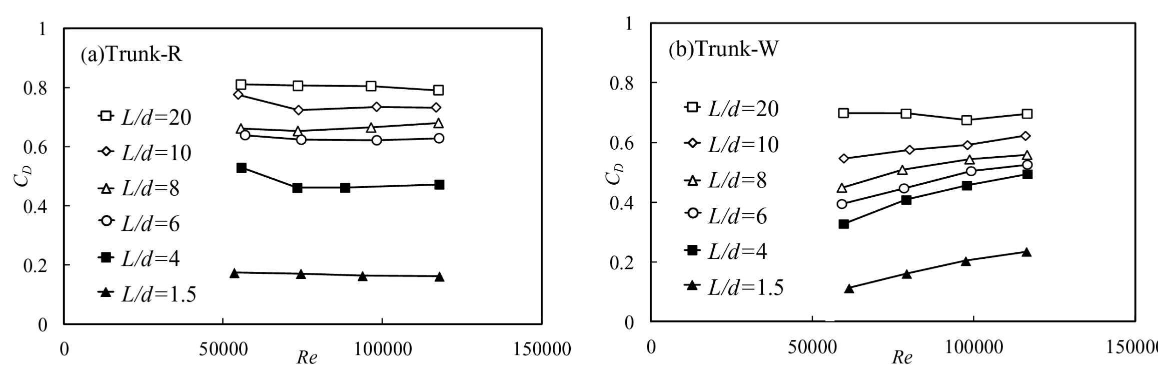


Figure 6 Variations of drag coefficient (C_D) of rear-side models with Reynolds number (a) Trunk-R, (b) Trunk-W (L is the distance between two trunk models, d is the diameter of trunk model) (Tanaka et al.(2011))

In case of Trunk-R, boundary layer of the trunk surface may be shifted to turbulent enough, and wake region suppose to be not changed with increasing R_e. Therefore, the drag coefficient of Trunk-R becomes almost constant for the investigated R_e. The results for Trunk-R and Trunk-W with different surface roughness shows different tendency. However the trunk surface of both tree species become like Trunk-R when the trunk diameter becomes large with aging. Therefore, when the drag coefficient of real trunk is applied, it is better to apply the drag coefficient of Trunk-R than that of Trunk-W.

On the other hand, Figure 7 shows the drag coefficient of an inclined tree and two inclined trees. The drag coefficients of a single tree of θ = 0° and 30° was 0.86 and 0.68, respectively. In addition, *d/H* of an inclined tree of θ = 0° or 30° was 0.1 or 0.08, respectively. The drag coefficient of an inclined tree of θ = 0° was similar to the approximated line for Uematsu and Yamada (1995). Therefore, the method of measuring drag force used in this study was judged to be appropriate. The result for an inclined tree of θ = 30° was smaller than those of θ = 0° by 20 percents. Since the value of *d/H* was almost the same, the difference was supposed to be caused by the inclination of a tree.

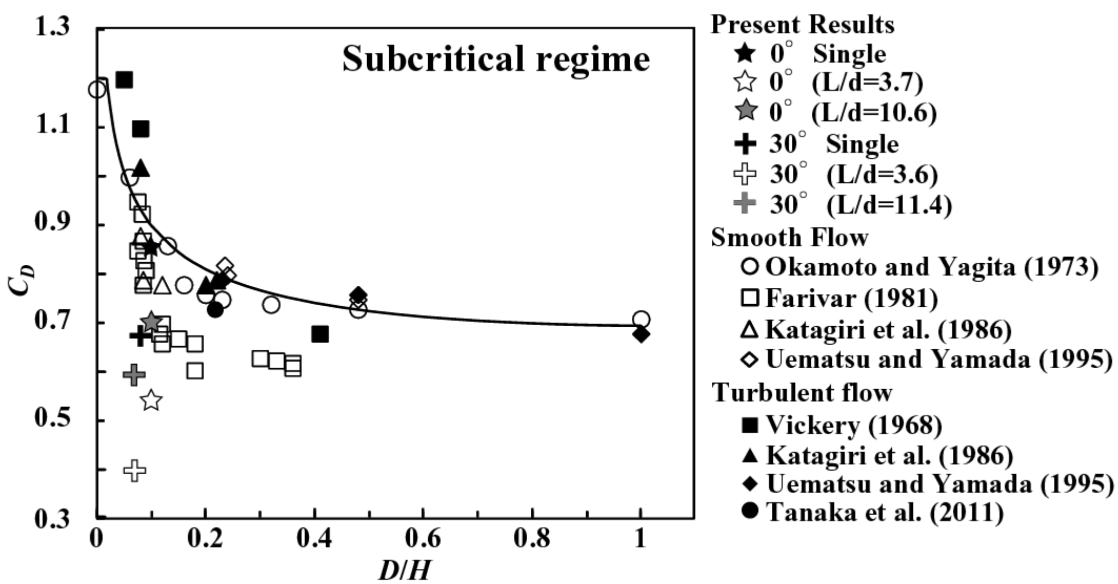


Figure 7 The relationship between aspect ratio (*d/H*) and drag coefficient (C_D) under the subcritical regimes. (This figure is modified from Uematsu and Yamada (1995)).

In case of two inclined trees experiments, drag coefficient of the rear-side tree trunk was decreased with decreasing *L*/d**.** Similar tendency can be confirmed in case of the vertical standing tree (Tanaka et al. (2011)). In addition, drag coefficient of the rear-side tree trunk was decreased with increasing inclination. In case of single trunk, drag coefficient of vertical (θ=0°) and inclined (θ=30°) tree trunks were 0.85 and 0.68, respectively. On the other hand, in case of two trunks (*L/d*=3.7), drag coefficient of vertical tree trunk (θ=0°) and inclined one (θ=30°) was 0.54 and 0.41, respectively. These results indicate that the influence of *L/d* on reduction of C_d value is larger than that of inclination.

4. CONCLUSIONS

The following conclusions were obtained in this study:
1) When tree is inclined, the drag coefficient was decreased about 20% for both of an inclined tree and two inclined trees.
2) The drag coefficient of a tree with roots is almost the same value to Trunk-R and Trunk-W. Therefore, the value of C_d=1.0-1.2 can be given as using projected area of roots when a tree is overturning in a flood event. So the estimation of a projected area of root zone is important.

5. ACKNOWLEDGMENTS

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Optimization of Multipurpose Reservoir Operation Using Game Theory

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Abstract *This paper introduces the reader to the concept of game theory and illustrates how it can be efficiently applied to optimize the allocation of water resources in multipurpose reservoirs. First part of the paper discusses the fact that life is full of conflicts, and these conflicts can be modelled as games. The second part discusses the mathematical modelling of a game. The basic characteristic of a game is that each player has a set of strategies and players do not know what strategy the other players are going to use. However, payoff to each player depends not only what he or she does, but also on the strategies chosen by other players. Final section of the paper develops a multipurpose reservoir operation as a game. In this game the two players are the reservoir operator and nature. Objective of the model is to optimize the benefit to the reservoir operator irrespective of what nature does..*

Keywords: *Optimization, Reservoir operation, Game theory*

1. INTRODUCTION

Life is full of conflict and competition. Some examples are cricket, war, political campaigns, advertising and marketing by business firms. Game theory is a mathematical theory that deals with conflict situations.

Water resources engineers in developed countries have been using various mathematical techniques to optimize the benefit from available water resource (Harboe 1997). Today they have ability to plan and operate water resources projects much more efficiently than what they did 10 years ago. For this purpose different mathematical models have been developed and relevant algorithms are used to optimize the benefits (Hall 1961).

The situation is different in developing countries. They have acquired the computational power to match those of developed countries. However, they are far behind in application of optimization techniques in planning and operating of water resource systems (Block 2011).

The situation is not different in Sri Lanka. Almost all organizations who are involved in managing water resources have acquired sophisticated computer equipment. This equipment is not used to optimize the benefits. They are primarily used to simulate the hydrologic processes and management of irrigated water.

1.1. Basic Characteristics of a Game

In mathematics a game is defined as a situation where two or more players are in conflict with each other. Players can be people or organizations. Basic properties of a game are listed below (Taha 2003).

- There are at least two players.
- Each player wants to win.
- The winner will get a payoff.
- Players compete with each other.
- Cooperation is not an advantage.
- Rules of the game are clearly defined and known to all players, in advance.
- Each player has a finite set of possible strategies.

- The outcome of the game depends on the strategies chosen by each player.

A game is played by a set of moves taken one at a time. This collection of moves is called a strategy. A player's optimal strategy is the one that gives maximum benefit to a player irrespective of what the other player does.

Game theory is the study of how players should rationally play games. Each player would like the game to end in an outcome which gives him as large payoff as possible. The four main components that interact with each other in a game are players, strategies, outcome and payoffs.

Games are defined by,

i) The number of players. When there are two players it is called a two person game. When there are n players it is called an n person game etc.

ii) The net winnings of the game. A zero sum game is where the net winnings is zero. For example in a two person zero sum game what one player wins the other looses.

iii) Fairness of the game. A game which is not biased toward any player is called a fair game.
A game in which a given player can always win by playing correctly is therefore called an unfair game.

5.1.1. Payoff table

A payoff table is a table showing payoff to a given player for different strategies of players(Hillier 2002). Sample pay off table for player A in a two person game is given below

Table 1: Payoff table for a two person game

		Player B		
		x_1	x_2	x_3
Player A	y_1	a_{11}	a_{12}	a_{13}
	y_2	a_{21}	a_{22}	a_{23}
	y_3	a_{31}	a_{32}	a_{33}

In this example player A has three strategies y_1, y_2 and y_3 while Player B has three strategies x_1, x_2 and x_3. According to this example payoff to player A is a_{13} if Player A plays strategy y_1 and Player B plays strategy x_3.

Let us consider a simple numerical example to illustrate the solution procedure.

Table 2: Pay off table for a game with pure strategy

		Player B		
		x_1	x_2	x_3
Player	y_1	3	2	4
A	y_2	6	9	5
	y_3	1	3	2

The methodology of obtaining the optimal strategy for the two players is explained below. The mathematical techniques involved are outside the scope of this paper. Interested reader can find them in (Hillier 2002)

For player A: Find the maximum entry in each column. Get the minimum of these maxima.
For Player B : Find the minimum entry in each row. Get the maximum of these minima.

Optimal strategy for A is minimax and that of B is maximin. In this case maximin is equal to minimax.
When minimax is equal to maximin there is a pure strategy. Optimal strategy of A is y_2 and that of B is x_3.

It was easy to solve this problem because there was a pure strategy. However, most of the problems that are encountered in real life do not have a pure strategy. They need to be solved using linear programming techniques (Saiseni 1969), which is explained later.

1.2. Application of Game Theory in Reservoir Operation

In United States the capacity of multipurpose reservoirs are generally large (Hall 1961). Most of their capacities are equivalent to about five times the annual inflow. Operators of these reservoirs are tempted to keep the reservoirs about half full most of the time. In the event of a drought there is sufficient water in the reservoir to meet the irrigation demand. On the other hand if a flood arrives there is sufficient capacity to accommodate the flood.

In developing countries, multipurpose reservoirs are relatively small. Their capacity is equivalent to about annual inflow. When the reservoir is half full the operator is in a dilemma. If he releases the water for other uses he might run out of water for irrigation. On the other hand if he retains the water there might not be sufficient capacity to accommodate a flood.

1.3 Similarities Between Multipurpose Reservoir Operation and a Game

Operation of a multipurpose reservoir has all the characteristics of a game. There are players. In this case two. One, the person who operates the reservoir and the other 'nature'. Let us look at the problem from the reservoir operators angle. He does not know what the nature is going to do next. However, payoff to him depends not only what he does but also what 'nature' does. For example let us assume that currently the reservoir is full and the operator decides to keep it full without releasing any water. If a flood arrives at this stage there could be a big damage. On the other hand if he decides to release as much water as possible, the damage will either be mitigated or eliminated.

Now that it is established that reservoir operation can be modelled as a two person game, theory of games can be used to optimize the benefits.

1.3.1. Preparation of a Payoff Matrix.

First step in the development of the model is to prepare a pay off matrix (Romp 2003). Let us prepare the payoff matrix for the reservoir operator. At each stage reservoir operator has several strategies and nature has several strategies.

In this paper a hypothetical reservoir is modelled to illustrate the solution procedure. For simplicity, the reservoir is assumed to have five different levels. At each level operator can take several decisions, in this case they are called strategies. Let us assume that a maximum of 2000 units of water can be released from the reservoir. If we assume that the water is released in blocks of 500 units of water, following are five possible strategies for the operator.

Strategy 1 – No release
Strategy 2 – Release 500 units
Strategy 3 – Release 1000 units
Strategy 4 – Release 1500 units
Strategy 5 – Release 2000 units.

Similarly we can define a set of strategies for the nature. For simplicity let us limit the number of strategies to five. Each strategy represents the inflow to the reservoir. The range will be the expected minimum to the maximum inflow.

Next process is the preparation of a payoff matrix for each pair of strategies of the two players. Each release is associated with a net benefit to the operator which is the sum of benefits from all uses, in the case of a multipurpose reservoir. This benefit will have to be evaluated in monetary terms. For example if the inflow is less than release more volume is available for flood control which will increase the flood control benefits. On the other hand it will lower the water level of the reservoir which will reduce the recreational benefits. This will increase the risk of water shortage. The net benefit is the sum of all benefits, some of which are negative.

To illustrate the computational procedure, a hypothetical pay off table given below was prepared. In this example five strategies are available for both players. x_1, x_2, x_3, x_4, and x_5 are the strategies available for the nature and y_1, y_2, y_3, y_4 and y_5 are the strategies available for the operator. The payoff table given below represents the payoff to the operator. Now the problem is to find out the best operational policy which will optimize the benefit to the operator.

Table 3: Pay off table for the Operator vs Nature Game

		Nature				
		x_1	x_2	x_3	x_4	x_5
	y_1	*12*	*35*	*24*	*-13*	*18*
Operator	y_2	*21*	*-19*	*16*	*19*	*27*
	y_3	*20*	*17*	*31*	*18*	*22*
	y_4	*14*	*13*	*28*	*18*	*24*
	y_5	*19*	*21*	*22*	*16*	*26*

Let X=(x_1,x_2,x_3,x4,x5) and Y=(y1,y2,y3,y4,y5) represents the optimal mixed strategies forthe operator and nature respectively. The probability that operator plays strategy i is x_i, and the probability that the nature plays j is y_j

The gain to operator at each play is a random variable α and the expected value *E* of a play to the operator is given in eq.(1)

$$E(\alpha; X, Y) = \sum_{ij} a_{ij} x_i y_j \quad (1)$$

Where a_{ij} is the gain to operator when operator plays strategy i, and nature plays the strategy j.

Operator wishes to choose X so that regardless of the nature of Y, his expectation at each play exceeds some amount v. Operator wishes v to be as large as possible.

Analysing on similar lines, expected gain for 'nature' is,

$$E(-\alpha; X, Y) = \sum_{ij} -a_{ij} x_i y_j \quad (2)$$

Nature will play so that the expected value of his gain exceeds some number v.

These conditions will yield the following equations.

$$12y_1 - 21y_2 + 20y_3 + 14y_4 + 19y_5 <= v \quad (3)$$
$$35y_1 - 19y_2 + 17y_3 + 13y_4 + 21y_5 <= v \quad (4)$$
$$24y_1 + 16y_2 + 31y_3 + 28y_4 + 22y_5 <= v \quad (5)$$
$$-13y_1 + 19y_2 + 18y_3 + 18y_4 + 16y_5 <= v \quad (6)$$
$$18y_1 + 27y_2 + 22y_3 + 24y_4 + 26y_5 <= v \quad (7)$$
$$12x_1 + 35x_2 + 24x_3 - 13x_4 + 18x_5 >= v \quad (8)$$
$$21x_1 - 19x_2 + 16x_3 - 19x_4 + 27x_5 >= v \quad (9)$$
$$20x_1 + 17x_2 + 31x_3 + 18x_4 + 22x_5 >= v \quad (10)$$
$$14x_1 + 13x_2 + 28x_3 + 18x_4 + 24x_5 >= v \quad (11)$$
$$19x_1 + 21x_2 + 22x_3 + 16x_4 + 26x_5 >= v \quad (12)$$

In addition we have two equations indicating that sum of all probabilities equal to 1.

$$x_1+x_2+x_3+x_4+x_5 = 1 \quad (13)$$
$$y_1+y_2+y_3+y_4+y_5 = 1 \quad (14)$$

Solving these equations as a linear programming problem using Lingo software, yields the following result.

```
Global optimal solution found.
  Objective value:                              21.17647
  Infeasibilities:                              0.000000
  Total solver iterations:                            10


                       Variable           Value        Reduced Cost
                              V        21.17647            0.000000
                             Y1       0.6639469            0.000000
                             Y2       0.2009488            0.000000
                             Y3        0.000000            0.000000
                             Y4       0.1351044            0.000000
                             Y5        0.000000            0.000000
                             X1        0.000000            6.000000
                             X2        0.000000            5.235294
                             X3       0.5294118            0.000000
                             X4        0.000000            22.88235
                             X5       0.4705882            0.000000
```

[X]= {0,0,0.53,0,0.47}
[Y]={0.66,0.20,0,.14,0}

V = 21.2

As such the operator should select his strategy according to probability distribution [Y]. When he does this he is guaranteed a payoff of 21.2, irrespective of what nature does.

2. CONCLUSION

In mathematics a game is defined as a situation where two people who are called players are in conflict. Each player has a set of strategies. Each one wants to win. The one who wins get a payoff. The outcome of a game depends not only what one player does, but also what the opponent does.

Reservoir operation has all the characteristics of a game. The two players are the reservoir operator and the nature. The operator does not know what nature is going to do in hydrological terms. Hence reservoir operation can be modelled as a game. Game theory can be used to optimise the benefits from reservoir operation irrespective of what the nature does.

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Online Learning Process Management and Learners' Motivation: A Case Study of University of Yamanashi Virtual Academy Programme

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Abstract: *This paper presents a computer-based online learning strategy to assist in introducing and teaching hydrological modeling to different stake holders (Basic degree holders to postgraduates and also practicing engineers). As part of the online learning strategy, an interactive computer-based instructional aid was specifically developed to assist distance learners to set up, run and analyze the output from a hydrological model developed by University of Yamanashi, Japan for prediction of river discharge and other hydrological outputs relevant to a river basin. The online learning strategy comprised with an internet based course lessons which include helpful text, graphics, and links to publicly available relevant data resources. An anonymous after lesson questionnaire survey was introduced to assess participants' perceptions towards the adopted online learning strategy. ARCS (Attention, Relevance, Confidence and Satisfaction) principles were used to gauge participants' learning motivation based on the questionnaire outcomes. Alterations to the online programme are underway to enhance the quality of the programme based on lessons learned through the feedback*.

Keywords: *ARCS principles, Learning motivation, Online learning, YHyM-BTOPMC.*

1. INTRODUCTION

The benefits of using hydrological models as an aid in understanding the catchment response for hydrological cycle and its components have been recognized within the water-related professions for more than two decades. Hydrological models can be used to find the best answers to both existing and potential hydrological problems. Additionally, they have also been resourceful in demonstrating solutions at high level meetings by way of interactive graphics and animations, educating the most inexperienced of observers. An online structured learning programme has been developed to enhance the understanding of hydrological modeling while preparing prospective participants for the type of problems they may encounter in the workplace and assisting them in the teaching/management of their educational and research activities. This paper presents a description of the online learning programme including the model used and results of a formal anonymous questionnaire survey undertaken to gauge participants' perceptions towards the effectiveness of the program.

1.1. Hydrological Model

The Block-wise use of TOPMODEL and Muskingum-Cunge method (BTOPMC) is the core module of the University of Yamanashi Distributed Hydrological Model (YHyM); here after refer as YHyM-BTOPMC. This model has been developed by the University of Yamanashi, Japan (Takeuchi, Ao & Ishidaira, 1999), (Ao, Takeuchi & Ishidaira 2000), (Zhou, et. al, 2006) and (Takeuchi, Hapuarachchi, Zhou, Ishidaira, & Magome, 2007) and introduced to this online learning programme. This model has already been successfully applied to many basins, large to small, temperate to tropical throughout the world (Hapuarachchi et.al., 2004), (Shrestha et. al., 2007) and (Silva, Magome & Ishidaira, 2010). YHyM-BTOPMC includes four main sub-models (topographic, runoff generation, flow routing and parameter identification). The structure and parameterization of the model lead to the advantage of both lumped and distributed approaches in hydrological modeling process.

1.2. Virtual Academy

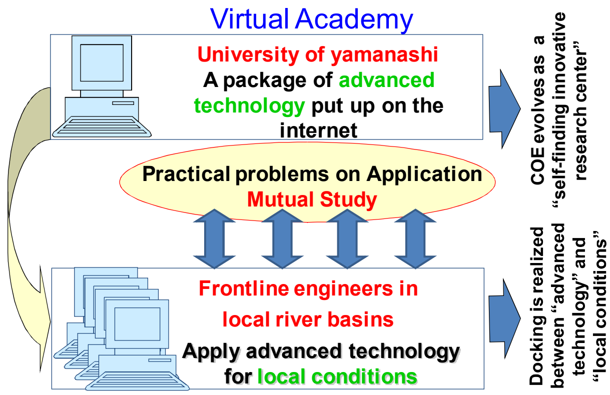


Figure 1 Purpose and Goal of Virtual Academy

The University of Yamanashi, Virtual Academy (VA) was inaugurated in 2004 as one of the main educational features under 21st Century Center of Excellence programme and presently continuing under the Global Center of Excellence programme. The objective of the VA is to encourage international collaboration in research and exchange of information and expertise relate to river basin management with mutual study (Figure 1). VA programme guide the participants with theoretical and practical expertise necessary for local applications, through a structured curriculum offered via internet. Here, an advanced Learning Management System (LMS) has been used. All lesson contents, grades, reports, Q&A and bulletin board are managed and opened to participants and instructors through LMS.

The prime focus of present VA programme is on YHyM-BTOPMC distributed hydrological model simulation to obtain hydrological outputs at a given watershed. The VA course lessons consist of basic theory as well as all necessary procedural steps including supplementary software tools that are developed to extract various data from publicly available information. These tools help enrolees to prepare the input datasets for YHyM-BTOPMC model application in their chosen river basins. The lessons comprised many figures, tables and web links in addition to traditional text-lessons to enhance effective learning. The curriculum provides all necessary expertise for basic theory, setup and execute of the model, and interpret the output (Figure 2), using the Fujikawa river basin, Japan as an example application. Over the years, the number of participants who apply for VA programme has been increasing and the composition of participants becomes more diverse. In year 2007, about one third of the registered participants were from public institutes. About a half of the total applicants were postgraduate students and the rest of the applicants were from diverse status. Therefore, since 2007 VA started to offer two courses (Beginners Course and Advanced Course) to cater to different levels of participants. Also these two courses are designed with quarter system and the end quarter assignments to further enhance the learning efficiency.

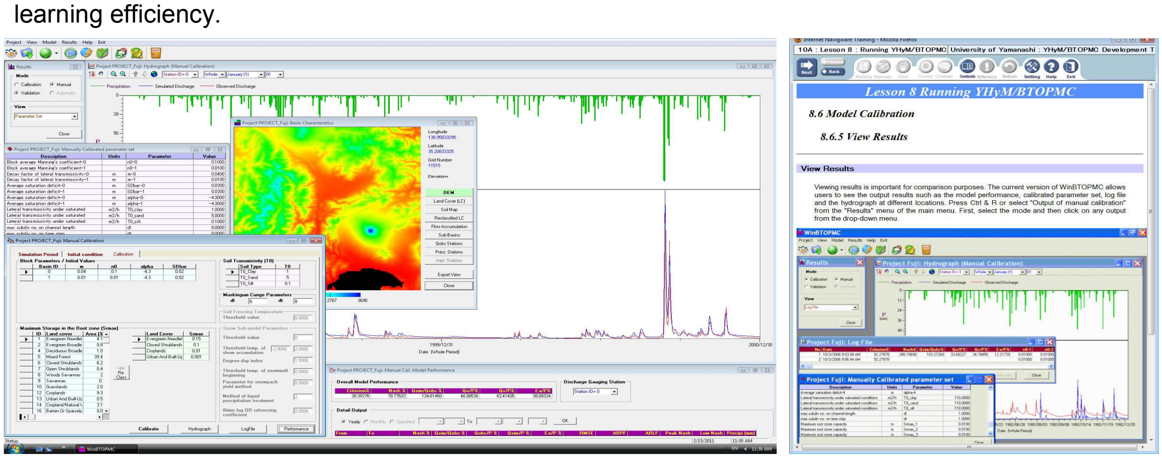


Figure 2 Graphical User Interface for YHyM-BTOPMC model and sample lesson page of VA

2. LEARNING MOTIVATION

Motivation is a complicated psychological attribute, basically classified as extrinsic (coming from outside the learner and target on final grades) or intrinsic (genuine interest in learning originating from within). Extrinsic motivation more closely associate with traditional class-room learners. Intrinsically motivated learners have several strengths including sustain desirable learning behaviours. Hence, intrinsic motivation is identified as much relevant with distance (online) learners (Patricia, Nancy & Brian, 2002). Keller, 1987 presented four key principles namely Attention, Relevance, Confidence, and Satisfaction (defined as ARCS model) for planning and implementing distance learning courses if learners are to stay motivated. In the current study, ARCS principles have been investigated through an online course evaluation questionnaire, which is designed to gauge participants' perception towards the VA programme. The questionnaire results presented in this research article are from 20 highly motivated VA enrolees (2007 - 2009) who had responded at the end of 13-week VA course. Table 1 summarizes the used and evaluated ARCS principles under different strategies. The last column gives the arithmetical mean and standard deviation for each investigated learning motivation strategy based upon responses from VA participants. Figure 3 illustrates Likert scale radar-plots for course and instructor evaluation by VA participants which clearly demonstrate the level of participants' agreement to each target aspect. Most of the aspects have been positively agreed upon with a satisfactory level and the details of each are discussed in the next section.

Table 1 ARCS principles for evaluating learning motivation

Key principles	Principles used	Principles evaluated	Mean ± SD
Attention Strategy			
Enhance enthusiasm	Lessons were prepared to introduce both modeling principles and applications.	Written materials were of high quality and suitable for the course.	4.5 ± 0.7
Communication support	Interactive communication was expected through Bulletin Board facility.	Opportunity for interaction among participants.	3.4 ± 1.3
Diversity	Learners were encouraged to research on different case studies.	NA	
Relevance Strategy			
Control and relevance	Participants were guided through level based learning.	Contents were appropriate to the allocated time.	4.5 ± 0.7
Confidence Strategy			
Minimize anxiety	Instructor showed enthusiasm and empathy.	Instructor has a thorough knowledge of the subject.	4.5 ± 0.7
	Enrollees are provided with clear, organized and effective instructions.	Instructor was organized and prepared.	4.5 ± 0.7
Challenge level	Lessons were planned according to pre-defined course goals.	Contents had adequate, clearly identifiable goals.	4.8 ± 0.4
	The assignments were designed to increase the level of challenge as the course progressed.	Assignments were appropriate.	4.7 ± 0.6
Satisfaction Strategy			
Effective evaluation	Learners are provided clear, encouraging feedback.	Instructor used effective teaching strategies.	4.3 ± 0.7
Multidimensional tasks	Multiple skills were tested through case studies.	NA	

Notes: 5 = Strongly agree to 1 = Strongly disagree; SD = Standard Deviation; NA = Not assessed

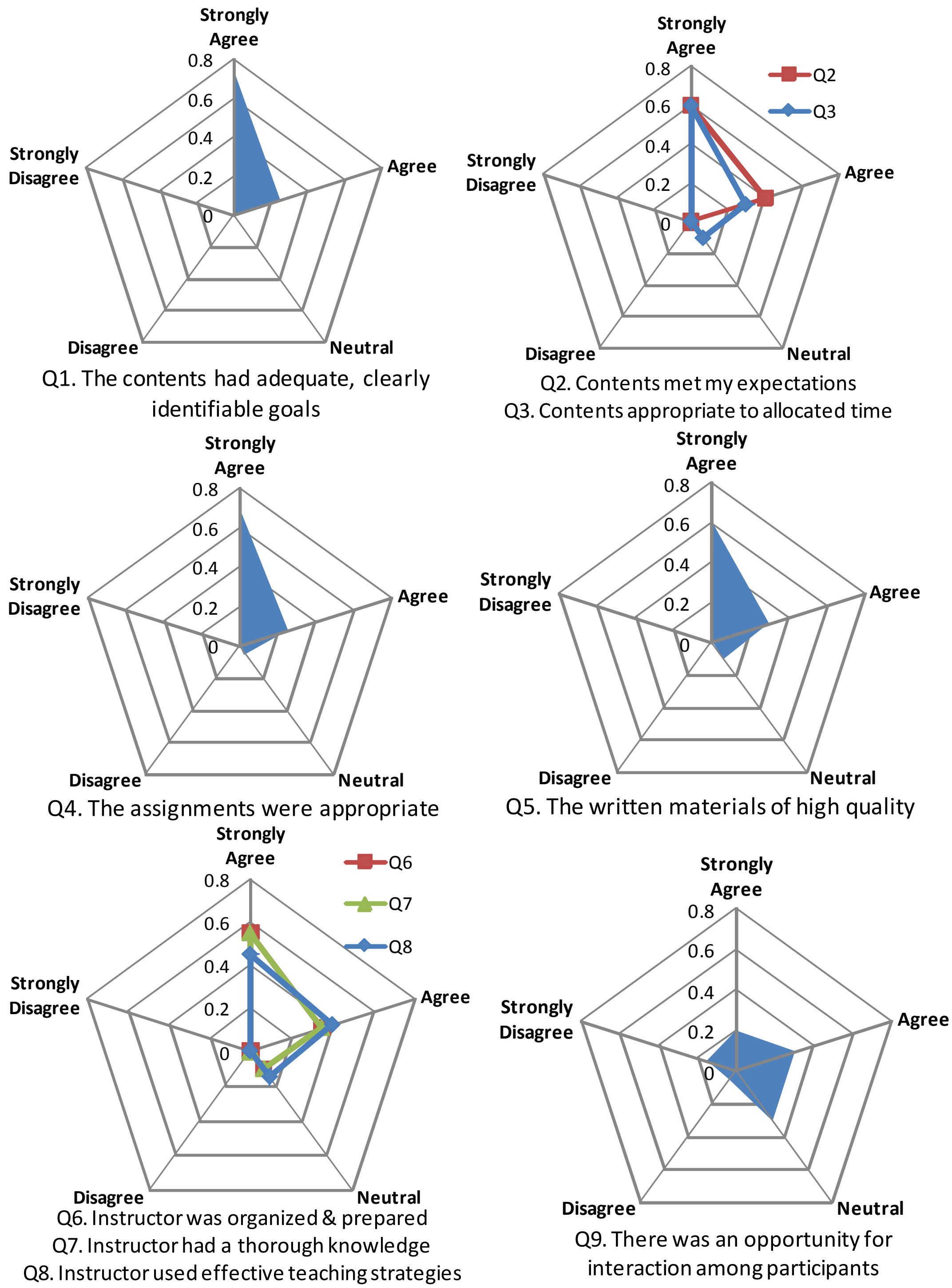


Figure 3 Summary of course and instructor evaluation by VA participants

2.1. Accomplished ARCS principles

2.1.1. Attention strategy

Enhance enthusiasm: VA lessons include introduction of YHyM-BTOPMC model as well as governing hydrological principles pertaining to modeling. The Lessons are presented through well structured level based learning from which learners would be able to extract the essence of hydrological modeling. Enrolees found that the lesson contents were of high quality (Figure 3-Q5 and Table 1).

Communication support: When designing assignment, choosing technology, and providing feedback, interaction among participants should be considered. In VA, Bulletin Board facility (Figure 4) was provided from which participants can either post a question or search for a similar inquiry previously posted. Through this facility, it is expected that participants can exchange issues related to different lessons within the programme. In VA, participants seldom read others' posting and found the Bulletin Board moderately useful (Figure 3-Q9 and Table 4).

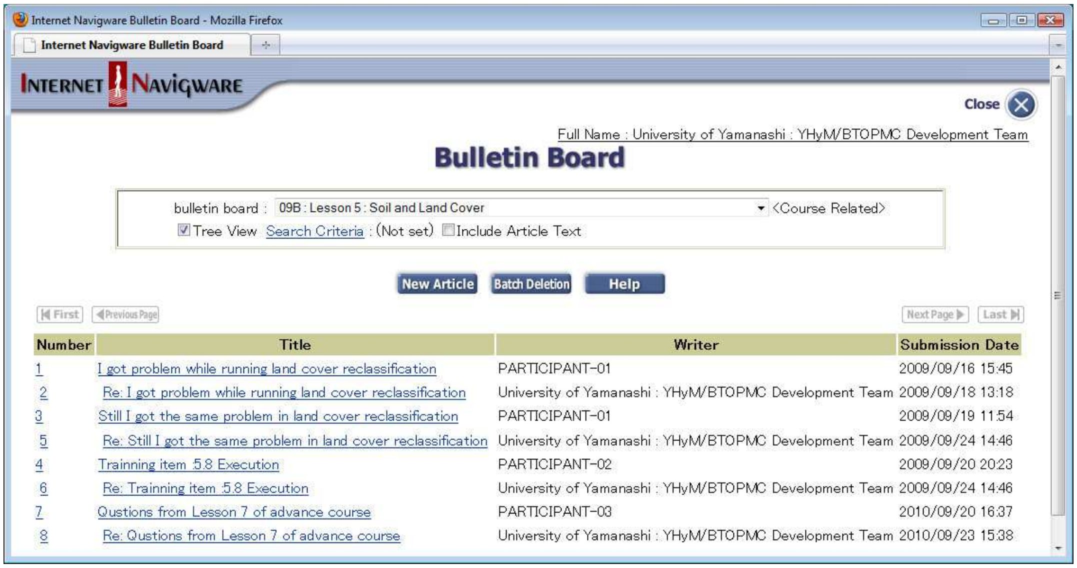


Figure 4 Bulletin Board

Diversity: Stipek, 1993 warns against predictability, in repeatedly using the same teaching method. Instead, diversity is recommended to maintain participants' curiosity. However, in online learning it is understood that predictability in different lesson contents may optimize course organization and clarity while decreasing anxiety of participants. VA has been designed to balance the need for consistency without sacrificing diversity. The present course evaluation questionnaire does not cover this aspect and it will be included in future programmes.

2.1.2. Relevance strategy

Control and relevance: Another strategy to enhance intrinsic motivation is to let participants choose activities that are personally meaningful and relevant. In this programme, enrolees can choose their own target river basin for their application. Except precipitation and discharge data sets, the YHyM-BTOPMC model can be set up and simulated with publicly available datasets. This promotes participants to select any basin according to their desire, interest, and enthusiasm. Thus, participants may feel that they have more control over learning application. VA offer flexibility in how participants completed their assignments. Although due dates are needed to ensure coherence in the overall programme, participants are given ample time to post reports as the assignments questions are made available at the beginning of each quarter. Participants found that the time allocation for different lessons were adequate (Figure 3-Q3 and Table 1).

2.1.3. Confidence strategy

Minimize anxiety: Participants new to distance learning may feel a level of anxiety different from that felt in the traditional face-to-face classroom. Concerns about assignments, motivating themselves without a designated meeting time and place, and interacting with classmates at a distance may generate anxiety which can hamper effective learning. VA instructors use communication strategies to continue to minimize participants' anxiety by adopting careful, specific, informative, and frequent feedback (Figure 5). At the end of each quarter, an encouraging comment and feedback are posted along with the assigned grades for the assignment. When the enrolees work is past due, the instructor makes a gentle reminder which is appreciated by many participants. Praising participants for successful progress and avoiding strictures in communication enhance confidence. While communicating, instructors composed clear, respectful messages to demonstrate knowledge, understanding and enthusiasm. The tone of any messages should be friendly and humour has been used appropriately. Enrolees felt that VA instructors effectively cleared up points of confusion (Figure 3-Q7 and Table 1).

Challenge level: The challenge level of course assignments is designed in order to enhance intrinsic motivation, and to complement the extrinsic motivation associated with grades. Moderately challenging course materials are accepted more enthusiastically than difficult, easy, or repetitive materials. Structuring course lessons with increasing difficulty and attainable challenges helps build confidence of participants. VA course contents at the first quarter are simpler, specific, and directed than those used in later quarters. The first quarter assignment is based on participants' experience rather than new knowledge. In midcourse assignments participants are directed to download and process public domain datasets using data-extraction tools developed by University of Yamanashi. Participants are also expected to obtain pre-processing model outputs related with topography, precipitation, discharge, evaporation, snow melt, and other hydro-meteorological parameters. Quarter-4 assignment is more challenging with the full model set up and parameterization for their chosen basin. Enrolees rated the lesson contents useful and the course assignments moderately challenging (Figure 3-Q2 and Table 1).

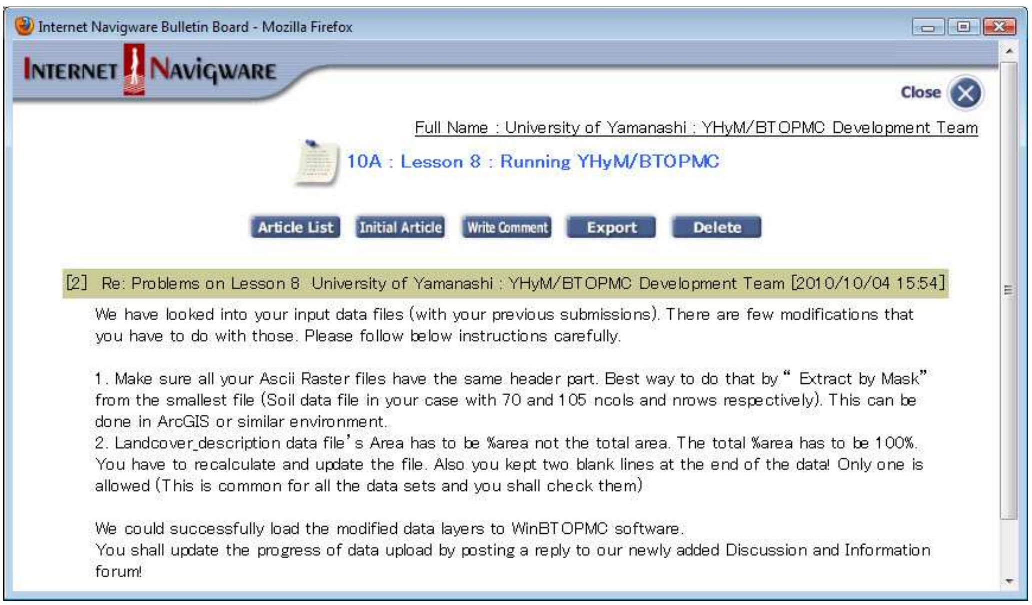


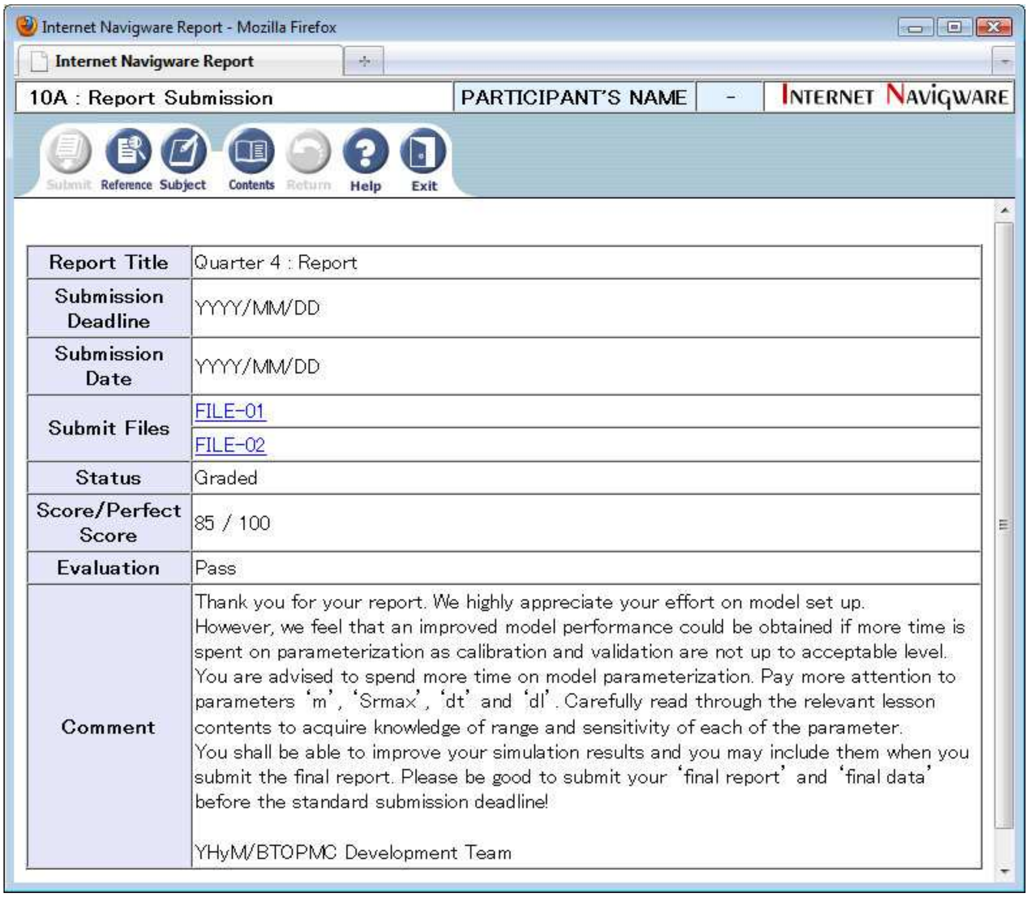
Figure 5 A typical VA feedback

2.1.4. Satisfaction strategy

Effective evaluation: In the ARCS model, personal attention, informative feedback, and praise are examples of techniques that increase participant satisfaction. During VA evaluation, in addition to final marks, enrolees received clear, encouraging feedbacks (Figure 6). Stipek, 1993 recommends that distance learner evaluations not be made public. Evaluation marks for VA are uploaded to the system and automatically generated e-mail notifies each participant. Individual participant can have access to his or her own evaluation page which is protected by own password. Participants rated instructor feedback useful (Figure 3-Q8 and Table 1).

Multidimensional tasks: Intrinsic motivation is also enhanced by multidimensional tasks, which use sustained effort over time, require more than one skill, and produce a tangible product or achievement. In quarter-4 report, VA participants are expected to use knowledge and critical thinking skills gained during the course to set up the YHyM-BTOPMC model using public domain and local data.

Subsequently, they are guided to prepare a comprehensive report combining the entire study outcome in a standard research article format.



Report Title	Quarter 4 : Report
Submission Deadline	YYYY/MM/DD
Submission Date	YYYY/MM/DD
Submit Files	FILE-01
	FILE-02
Status	Graded
Score/Perfect Score	85 / 100
Evaluation	Pass
Comment	Thank you for your report. We highly appreciate your effort on model set up. However, we feel that an improved model performance could be obtained if more time is spent on parameterization as calibration and validation are not up to acceptable level. You are advised to spend more time on model parameterization. Pay more attention to parameters 'm', 'Srmax', 'dt' and 'dl'. Carefully read through the relevant lesson contents to acquire knowledge of range and sensitivity of each of the parameter. You shall be able to improve your simulation results and you may include them when you submit the final report. Please be good to submit your 'final report' and 'final data' before the standard submission deadline! YHyM/BTOPMC Development Team

Figure 6 Sample evaluation sheet

3. CONCLUSION

This paper discusses the use of a structured approach to provide training for theoretical and practical expertise on distributed hydrological modeling for distance participants. An interactive online learning system was specifically developed to assist participants to set up, run and analyze a hydrological model which is introduced through the VA distance learning programme. The system comprises multi-lessoned modules and quarter system that support learners a good foundation to expand their knowledge and experience relevant to principles and applications of hydrological modeling. Level based teaching and evaluation are arranged with basic exercises with an example river basin and more detail and self oriented applications based on the participants' chosen river basins.

The computer-based teaching and learning strategy for distributed hydrological modeling was evaluated using an anonymous questionnaire to assess the participants' perception towards the developed approach. Adopted strategy was found to be successful for most of the strategies in relation with ARCS principles. A new VA-communication-space is developed to enhance participants' interaction as an improvement for present VA system (Figure 3-Q9). After completing the programme on-site workshops are conducted (4 times during last 7 years) for invited participants from on-line distance learning programme. These workshops have been designed to enhance the participants' understanding towards the hydrological modeling and also to improve the quality of the VA programme. These follow-up activities have been mutual beneficial for all VA stake holders to enhance and expand VA network. Use of advanced information technology to hold workshops online catering to a much wider audience is under consideration. The VA with YHyM-BTOPMC model is available for training at http://www.gcoe.yamanashi.ac.jp/e/va.html.

4. ACKNOWLEDGMENTS

Sincere gratitude goes to Global Centre of Excellence, University of Yamanashi, Japan for providing the opportunity to conduct this study. Contributions from all the VA participants by way of feed-back are also acknowledged with appreciation. The project success is due to the trainee, set up of the YHyM-BTOPMC hydrological model, web-based lesson contents and the commitments from programme developers and instructors. All the 21st Century Center of Excellence programme and Global Center of Excellence programme students, researchers, supporting staff and professors who have contributed in different capacities for the success of VA are also remembered and their committed contributions are sincerely acknowledged.

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Overflow Pattern and the Formation of Scoured Region by the Tsunami Propagated in River Channels in Great East Japan Earthquake

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Abstract: *The tsunami caused by the Great East Japan Earthquake on 11 March 2011, with a magnitude of 9.0, caused catastrophic damage to people and buildings in the Tohoku and Kanto regions of Japan. A field survey was conducted to elucidate the damage to river embankments and their hinterlands (residential area) by tsunami propagation in river channels and overtopping of embankments. Three, three, and four rivers in Iwate Pref., Miyagi Pref., and the Kanto Region, respectively, were selected for the field investigation. In the hinterlands, the tsunami came from coast and river, and the situation, including the evacuation of people, became complex. Tsunami inundation patterns were classified by the river capacity and whether a river or sea embankment was breached or not. This will provide useful information for making new hazard maps and planning new cities.*

Keywords: *tsunami propagation in river channels, overtopping flow, scoured region, erosion of embankment, meandering of river channel, Great East Japan Earthquake*

1. INTRODUCTION

The Great East Japan Earthquake at 14:46 JST on 11 March 2011 had a magnitude of 9.0 and an epicenter 129 km east of Sendai, and it was followed by a large tsunami that broke many of the sea walls (tsunami gates, large embankments)(Takahashi et al., 2011) and coastal forests (Tanaka, 2012), causing catastrophic damage to people and buildings in the Tohoku and Kanto regions of Japan.

Tsunamis can cause catastrophic damage to both human life and socioeconomic property. Extensive experimental (Peregrine, 1967; Madsen & Mei, 1969) and analytical (Benjamin, 1972) studies have shown that tsunamis are also propagated far upstream in a straight channel of uniform depth and width because a solitary wave like a tsunami propagates without changing its shape and speed. In an actual tsunami, river morphology greatly affects the propagation. Although the propagation of solitary waves through curved shallow water channels was investigated by numerical simulations and the deformation of the wave at the outer bank has been described (Shi et al., 1998; Yuhi et al., 2000), the disastrous results of propagation of an actual tsunami in a curved channel were not reported in previous research.

In addition, it is very important to elucidate the role of inland embankments of roads, railways, and channels along the coast in mitigating the tsunami as it inundates the inland, and the relationship between the tsunami propagating from the sea and the flow overtopping the banks of a river.

Therefore, the objectives of this study were: 1) to investigate the interactions between a tsunami propagating from the sea and the flow overtopping from a river, 2) to elucidate the effects of river morphology on the tsunami overtopping the embankment, and 3) to determine the effects of inland embankments on tsunami propagation in the hinterlands of the coast or river.
For that objective, field investigations were conducted of three rivers (Heiigawa, Omotogawa and Sakarigawa Rivers) in Iwate Prefecture, three rivers (Abukumagawa, Old and New Kitakamigawa Rivers) in Miyagi Pref., and four rivers (Kujigawa, Nakagawa, Tonegawa, and Mikawa Rivers) in the Kanto Region

in April and May 2011. Figure 1 shows the location of the mouth of each river.

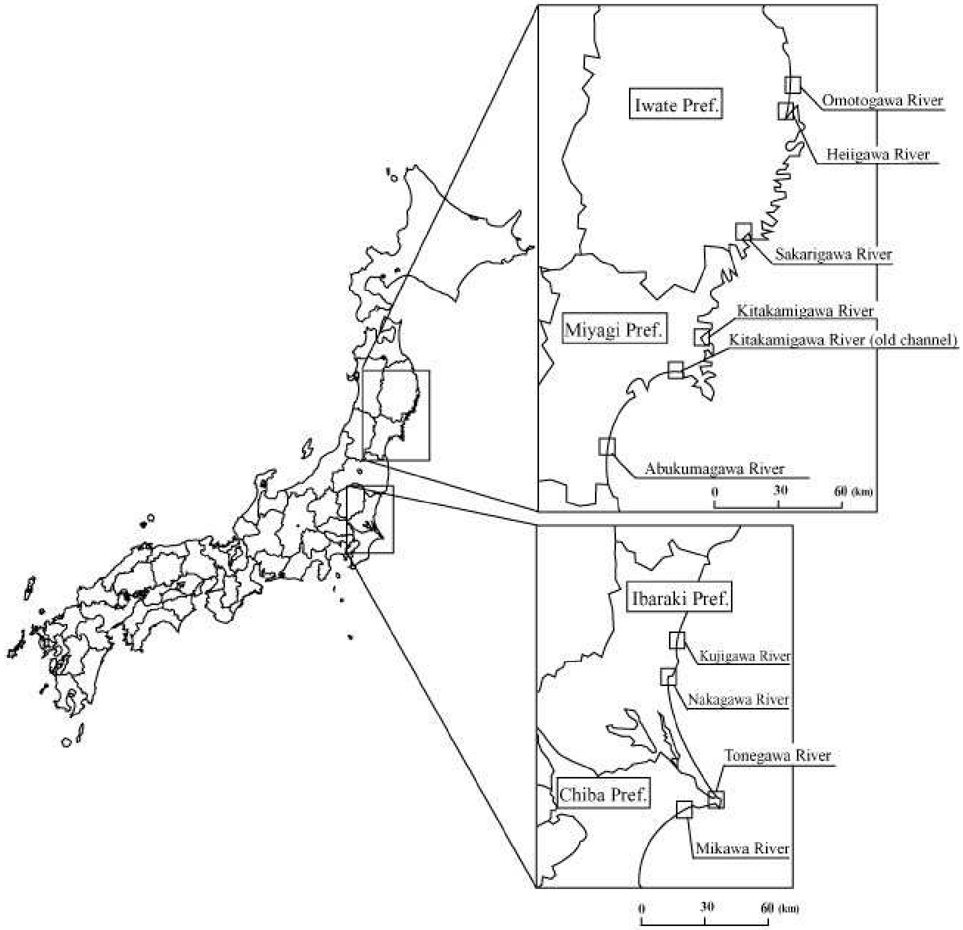

Figure 2 Location of investigation sites (locations of the mouth of each investigated river are shown in this figure)

2. SITE LOCATIONS AND MEASUREMENT METHOD

Table 1 shows the river width at the river mouth, tsunami water depth, and height of the sea/river embankment at each investigation site. The tsunami water depth was obtained from "The 2011 Tohoku Earthquake Tsunami Joint Survey Group" (http://www.coastal.jp/tsunami2011/). As for the tsunami damage, the width of sea/river embankments that were breached and the width/length of the regions that were scoured by overtopping flow are also shown in Table 1.

The tsunami water depth at each site was determined by the height of scars made by collisions of debris with tree trunks or broken branches, water marks, e.g., collision traits, on the walls of damaged houses, marks on broken roofs, or debris located on roofs. The tsunami directions were analyzed by the directions trees and fences were bent and the location of broken houses and scour regions behind embankments or houses. In addition, estimated tsunami water depths of the river or on the embankment and that in the hinterlands were compared to judge the dominant tsunami direction.

Table 1 Characteristics of rivers investigated

Location	Name of River	River width at mouth (m)	Tsunami water depth* (m)	Embankment height		Breach width		Scoured region due to overtopping flow	
				sea embankment (m)	river embankment (m)	sea embankment (m)	river embankment (m)	Width** (m)	Length** (m)
Iwate Pref.	Omotogawa River	200	6.9	9	4	0	230	70.0 - 100.0	14.0 - 15.0
	Heiigawa River	160	8.9	-	3	-	-	-	-
	Sakarigawa River	150	9.3	4	3	0	0	3.5 - 15.0	2.5 - 6.0
Miyagi Pref	New Kitakamigawa Riv	600	7.4	3	3	680	2100	3.0 - 39.0	2.0 - 5.0
	Old Kitakamigawa Rive	200	7.3	4	3	-	0	15.0	4.0
	Abukumagawa River	900	8.8	5	5	260	200	1.5 - 50.0	1.0 - 19.5
Ibaraki Pref.	Kujigawa River	200	4.2	4	4	0	0	-	-
	Nakagawa River	300	3.3	3	3	0	0	-	-
Chiba Pref.	Tonegawa River	700	3.0	5	5	0	0	-	-
	Mikawa River	25	3.4	2	-	0	-	-	-

*Tsunami water depth data were obtained from "The 2011 Tohoku Earthquake Tsunami Joint Survey Group" (http://www.coastal.jp/tsunami2011/)
**The width and length of regions scoured by overtopping flow are shown in Figure 2

3. RESULTS

3.1. Tsunami propagation and overtopping from river embankment (without tsunami gate at river mouth)

The direction and water depths of the tsunami inundation around the Abukumagawa River are shown in Figure 2. At location A, breaching of the sea embankment was observed. Behind the sea embankment, large areas were scoured by the tsunami overtopping of sea embankment. Near this region, the tsunami water depth was around 5.5 m and the embankment height was 4.8 m. At location B, the overtopping water depth at the top of embankment was estimated to be around 1 m based on the debris attached to the fence on the embankment. Just downstream, the river embankment was also breached by direct attack of the tsunami. The overtopping from the river to the hinterland was severe at location C, but was a little less severe upstream at locations D and E. However, the overtopping became severe again at location F because it was located on the outer-bank side of the river. The extent of overflow was judged from the erosion of the river embankment slope and scoured regions around houses and the broken or washed-out condition of houses. The elevation of the road along the river was higher around location F than in locations D and E, so the difference between the ground level of houses and the road was greater. In location D and E, the road in front of houses had a role to prevent erosion, however location D, downward flow still continued and caused erosion. In that case, the overtopping flow caused scoring and local scour around houses that combined to generate large scour area (Figure 3). Thus, houses around location F were completely washed out, not by the tsunami propagated from seaward, but by the tsunami overtopping the river. Similar overtopping from outer bank of a river was also observed in old Kitakamigawa River.

In case of Heiigawa River (Figure 4), a railway bridge which is 1.5 km upstream from the river mouth was washed out after the damming of flow by debris including a ship (Location G). People who temporally escaped on the railway embankment needed to escape far inland by the destruction of the bridge and railway. Most of the tsunami from seaside was stopped at the railway embankment (Location H), but tsunami also propagated by sewage pipe line, inundated in the inland region of the embankment, and overflown from the upstream river embankment when the railway bridge was broken. This case shows that; 1) bridge has some possibility to be a trigger of the overflow from embankment, and 2) tsunami also propagated from sewage line. This should be considered for the tsunami simulation and hazard mapping.

Figure 2 Damaged situation in Abukumagawa River Basin

Figure 3 Large scoured region by overtopping flow (Right hand side of Abukumagawa River)

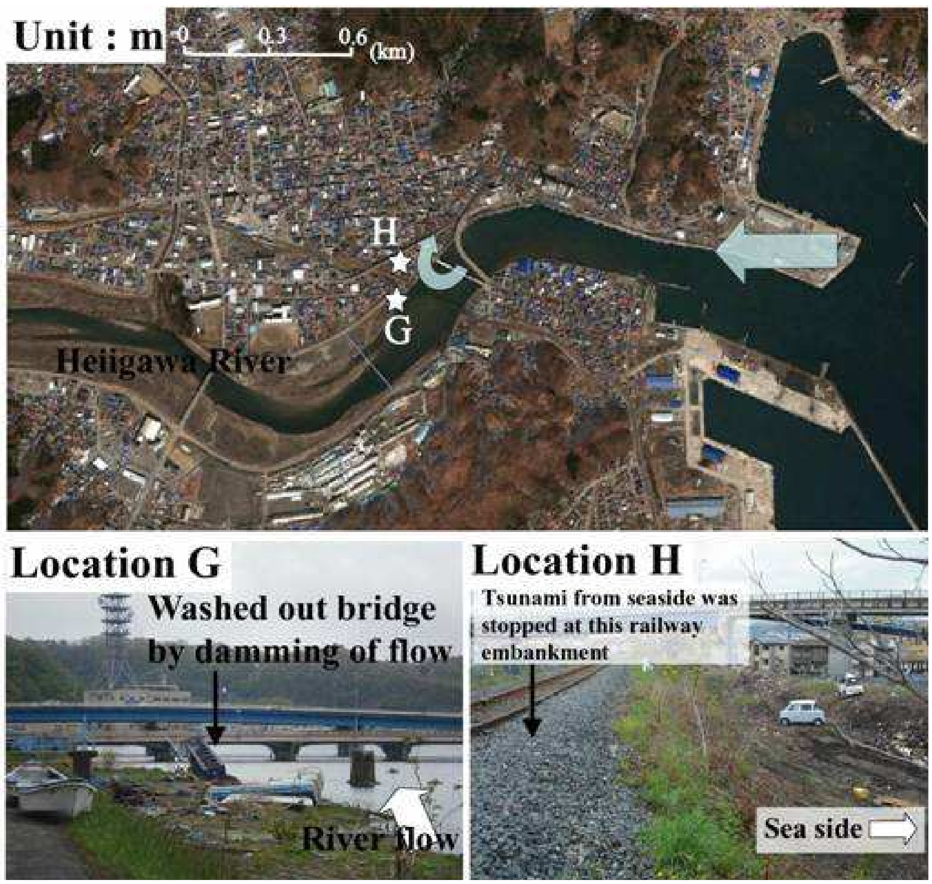

Figure 4 Damaged situation in Heiigawa River Basin

3.2. Change of tsunami propagation pattern with the tsunami gate at river mouth

The Omotogawa River (Figure 5) has a tsunami gate and high sea embankment (around 10 m from the ground on the hinterland side). The overtopping tsunami depth from sea embankment was assumed to around 1.6 m. Many large concrete blocks in front of the sea embankment had been transported by the tsunami, a large scoured region had been generated behind the sea embankment, and the pine trees near the region were overturned (Location I). Most of the houses near the sea and river embankment were washed out by the overtopping flow (Location J). From large scoured regions on floodplain of the river bed, and a scattered region of a broken parapet on the embankment, the breaching was supposed to be occurred from the inland side to the river. Even if a tsunami gate had existed, the tsunami would have been higher than the tsunami gate and the sea embankment; in fact, the tsunami inundation occurred mainly from the high embankment with high potential energy, and washed out or broke the houses. On contrary, in case of a small river in Chiba prefecture, the tsunami was stopped at the gate, because the tsunami height was low in comparison with Iwate Prefecturte.

Figure 5 Damaged situation in Omotogawa River Basin

3.3. Changes of tsunami inundation due presence of road, train embankment, sanitary channel, or mountain

In case of the Sakarigawa River basin (Figure 6), the presence of a road and train embankment changed the direction of most of the tsunami flow intruded from seaward. The tsunami passed through only the culvert of the embankment from seaward to inland, but it continued to overtop the river embankment. Thus, the people who lived upstream of the road and railway embankments received tsunami inundations from two directions. However, in this case, the tsunami overflow from river embankment itself was not large compared with that of the Abukumagawa River. The inland embankment for railway or road is very useful in some cases and needs to be utilized more, considering the tsunami inundation pattern.

In the Kanto Region, the tsunami height was lower than that in the Tohoku area. The Kujigawa, Nakagawa, and Tonegawa Rivers had sufficient capacity to absorb the tsunami. Thus, the areas inundated by the propagated tsunami in the three rivers were restricted. Most of the tsunami intrusions occurred around branches of the river and drainage channels connected to the river. Even when a gate existed between main River and its branches, inundation was also occurred because the earthquake caused a gate trouble by an electricity failure of the system. In the Mikawa River, most of the tsunami was stopped by a sand dune on the coast, but the river itself was open to the sea and a tsunami could easily intrude into and overflow the hinterland from the river. This kind of problem in the gap of an embankment or vegetation barrier was already discussed (Mascarenhas and Jayakumar, 2008; Thuy et al., 2009; Tanaka, 2009, 2011). The river mouth problem is very difficult to mitigate because if a gate is constructed, it may change the tsunami inundation pattern, as in Omotogawa River when the tsunami exceeded the designed gate level.

Figure 6 Damaged situation in Sakarigawa River Basin

4. DISCUSSION

As described in the previous section, overtopping of tsunami from the outer bank side of a river was severe. It is easily assumed from previous studies that the outer bank side is vulnerable to tsunami propagation (Shi et al., 1998; Yuhi et al., 2000). When overtopping occurred here, the scouring of roads in front of houses was not severe, and some houses remained standing although their walls were broken. In contrast, when the difference between the road height and the elevation of the house was large, the scouring became severe and the houses were washed out. The type of utilization of the riverside greatly affects the damage and needs to be studied in more detail in the future.

If a river embankment is not high enough to obstruct a tsunami or a city has rivers or creeks, tsunami inundation occurs not only from the sea but also from the rivers or creeks. In the Old Kitakamigawa River (old channel), or the Sakarigawa River, the tsunami propagated in the river, which is usually faster than a tsunami propagating over land, and the water overflow hit people from two directions.

Many patterns of tsunami propagation in rivers were observed, and they depended on 1) the river capacity (especially embankment height), 2) whether a river or coastal embankment was broken/breached or not, and 3) the existence of an inland embankment or area of high elevation, like a mountain, near the river. For the evacuation from the tsunami, revision of tsunami hazard maps, and new plans for a city design after a tsunami, this complex propagation pattern should be considered and informed to people. The knowledge in this study also needs to be considered in the design of tsunami protection and mitigation systems in a city.

5. SUMMARY

The following conclusions and recommendations were obtained by this study:
1) The flow overtopping embankments occurred mainly on the outer bank side of meandering river sections. Severe erosion occurred on the levee slope, and neighbouring houses were washed out by the scouring due to the overtopping flow.
2) In the hinterlands of coast and river embankment, it is necessary to identify locations where a tsunami can easily overtop for different tsunami conditions, and the information should be utilized for making next hazard map.
3) The tsunami inundation patterns were complex and could be classified based on the river capacity, the existence of gate, and whether a river or sea embankment was breached or not.

6. ACKNOWLEDGMENTS

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Consequences of Flow Turbulence: Biomass Partitioning and Plastic Responses in Morphology

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Abstract: *Water movement has a major influence on plant growth in aquatic ecosystems. Although the plants growing in shallow lakes and wetlands are not experiencing mean flow, they also experience water movement as flow turbulence. The objective of the current study was to observe the variations of morphology and biomass partitioning in Egeria densa and Chara fibrosa when exposed to three different turbulence levels. Chara fibrosa has been observed to have shorter internodal lengths, less number of internodes when exposed to increased turbulence, while reducing the lateral branching. Egeria densa has been observed to reduce biomass gain and lateral branching while increasing the shoot:root ratio. Morphological variations of C. fibrosa and E. densa are more or less similar while their responses to flow turbulence directed towards their survival in respective condition.*

Keywords: *Flow turbulence,Egeria densa,Chara fibrosa,Morphoplogy,Biomass*

1. INTRODUCTION

Understanding of the behaviour of aquatic species and their interactions with the environmental parameters are very important for assessing the impact of development projects on floral dynamics, for achieving the goal of sustainable development. Movement of water is a primary factor that regulates the growth and distribution of macrophytes (Madsen et al., 2001), as well as it experience many variations in proceeding development projects. Macrophytes growing in flowing water, such as rivers experience the forces caused by current velocity. Macrophytes grown in lentic environments such as shallow lakes, wetlands do not experience mean flow, however they experience turbulence caused by waves and wind-induced currents. Water movement has both beneficial and adverse effects on plant growth (Madsen et al., 2001). High current velocities and wave action has been observed to reduce plant growth and damage to existing colonisations. Water movement in small scale has been observed to reduce thickness of diffusion boundary layer hence increasing the nutrient uptake (Nishihara and Ackerman, 2006). Plants exposed to current have been observed to show different plastic responses, some of them are highly favorable for their survival at the respective environment. For instance, some species have been observed to be shorter in length, high in belowground biomass allocation in the exposure to current (Szmeja and Galka 2008).

The objective of the current study is to identify the morphological variations and growth in two different submerged aquatic species, *Egeria densa* and *Chara fibrosa*. In which *C. fibrosa* is a species of characean algae, while *E. densa* is an angiosperm.

2. MATERIALS AND METHODS

The experiments were conducted in 6-L (15.7x15.7x24.5 cm^3) microcosms with a water depth of 20.5 cm for a period of 12 weeks under controlled laboratory conditions. An experimental setup was consisted of four microcosms, among them three had different turbulent conditions and the other was the control. The experiment was conducted in duplicate with two exactly same experimental setups under same environmental conditions. The temperature was maintained at 23 ± 2 ^{o}C in a room with fluorescent lighting. The light intensity ranged from 270-240 µmol m^{-2} s^{-1}, and all of the microcosms were subjected to a 12 h/12 h light/dark period.

2.1. Turbulence generation and quantification

Turbulence was generated by vertically oscillating horizontal grids at three oscillating frequencies: 1, 2 and 4 Hz (O'Brien et al. 2004; Hondzo and Lyn 1999). The grid spacing (M) was 2.5 cm and the grid was made of 5-mm square Plexiglas rods (Hondzo and Lyn 1999), with a resulting solidity of 38% (De Silva and Fernando 1994). The stroke length (S) was 3 cm, and the grids were oscillated from the top of the tank the mean grid position being 2 cm below the tank top.

The horizontal velocity profile of the tanks was measured with a two dimensional current meter (SF-5712, Tokyo- keisoku Corporation, Japan). The water velocity was measured at nine different points, which were symmetrically distributed over the area; additionally, the velocity profile was measured at four depths (6, 9, 12 and 15 cm from tank top) for each of the nine points. All of the nine measurements were averaged to calculate the turbulence of velocity in each depth in each tank. Because oscillating grids generate nearly isotropic and homogeneous turbulence, the vertical component should be the same as the measured horizontal component and was thus not measured (De Silva and Fernando 1994). The time constant of the current meter (response time) was 0.05 s, and the measurements were carried out at a 10 Hz frequency for 1 min. The voltage signal was converted to velocity by a calibration graph after the data were extracted with the GL200_800-APS software Version 1.01 (Graphtec Corporation, Japan).

2.2. Plant material and growth conditions

The apical tips obtained from a laboratory culture were planted in the tanks; eight tips were positioned in each tank on peaty sediment that was collected from a nearby pond (Akigasei park, Saitama, Japan). The sediment was passed through a 1-mm sieve to remove debris and floating matter, and a 4-cm layer of sediment was added to each tank. The sediment was used with the purpose of supporting anchorage of plants and providing micronutrients to the media. The plants were allowed to grow in the experimental tanks for two weeks prior to the start of the experiments.
The water column nutrient concentrations were maintained at 0.143 ± 0.014 mM Nitrate and 0.016 ± 0.003 mM Phosphate through the addition of $NaNO_3$ and K_2HPO_4 (Analytical grade reagents, Wako, Japan). The nutrient concentrations were measured weekly according to the standard methods listed in the APHA (1998), and the concentrations were adjusted to fall within the appropriate range by addition and gentle mixing of the previously mentioned stock solutions. The dissolved inorganic carbon in the water was measured with a total organic carbon analyzer (TOC 5000 A, Shimadzu Co. Ltd., Japan), and ranged from 0.467 ± 0.016 mM. The water ammonia concentration was analyzed with the phenate method (APHA 1998), and was determined to be negligible.

2.3 Sampling and laboratory analysis

Plants were harvested at the end of the experiment period (12 weeks). Plant length, branching patterns, length of the branches were measured for *E.densa,* while the number of internodes, internodal length were measured for *C. fibrosa*. The plants were oven dried at 70 °C for 72 h and the dry weight (DW) of the shoots and roots was measured separately.

2.4. Statistical analysis

All of the data are presented as the mean ± SD. The homogeneity of variance test and Levene's check for equality of variances were performed on the data sets prior to the statistical analysis to verify the assumptions of normal distribution and homogeneity of variances. Differences among the various groups were analysed using one way ANOVA to check for significance with post-hoc Tukey's test. For all of these analyses, the SPSS for Windows (Release 13, SPSS Inc., Chicago, IL, USA) statistical software package was used.

3. RESULTS AND DISCUSSION

Measured turbulence velocity fluctuations at each tank are shown on Table 1. Control had no turbulence variation. Turbulence velocity fluctuations were chose to be in the range of turbulence velocity fluctuations observed in natural environment.

Table 1 : Measured turbulence velocity fluctuations (u') at each depth for each tank

Depth (cm)	Turbulence of velocity, u' (cm s^{-1})		
	High	Medium	Low
6.0	2.86 ± 0.79	1.86 ± 0.79	0.81 ± 0.16
9.0	2.64 ± 0.99	1.41 ± 0.58	0.68 ± 0.15
12.0	1.61 ± 0.53	1.30 ± 0.41	0.65 ± 0.08
15.0	1.62 ± 0.44	1.36 ± 0.27	0.67 ± 0.12

Average biomass of the plants exposed to turbulence was significantly reduced in *E. densa* (ANOVA, P<0.05) and the weight to length ratio indicated that biomass gain of the plants were significantly retarded by exposure to flow turbulence (Table 2). The linear weight was 0.57 times lesser in the exposure to high turbulence compared to the plants in the control tank. Mean while shoot:root ratio was decreased in the exposure to high turbulence, showing that biomass allocation for roots become increasing to increase its anchorage capacity, which is a common observation for most plants (Szmeja and Galka, 2008).

Table 2: Average shoot biomass per plant, shoot:root ratio and weight:length ratio of *E. densa* exposed to different turbulent levels, at the end of experimental period

Level of turbulence	Average shoot biomass (g DW/plant)	Shoot: root ratio	Weight/ Length ratio (mg DW/ cm length)
High	0.06 ± 0.004	6.05 ± 0.06	2.52 ± 0.38
Medium	0.23 ± 0.023	5.79 ± 0.16	3.37 ± 0.33
Low	0.28 ± 0.016	7.76 ± 0.18	2.91 ± 0.04
Control	0.37 ± 0.058	9.90 ± 0.34	4.36 ± 0.20

Chara fibrosa showed clear morphological variations when exposed into turbulence. Number of internodes per plant reduced significantly (ANOVA, P<0.05) while a reduction of internodal length also was observed (Table 3). Usually the maximum internodal length was observed in the basal part of the shoot, while the maximum internodal length was more than two times lesser in the exposure to high turbulence. Same has been observed in other studies also with shorter internodal lengths at basal parts in shallow lakes (Asaeda et al., 2007).

Table 3: Number of internodes, average number intermodal length and maximum intermodal length of *C. fibrosa* at the end of the experiment period

Level of turbulence	Number of internodes	Average internodal length	Maximum internodal length
High	2.3 ± 0.6	1.13 ± 0.06	1.60 ± 0.10
Medium	5.7 ± 2.1	1.40 ± 0.68	1.85 ± 0.50
Low	7.0 ± 1.0	1.46 ± 0.76	2.45 ± 0.07
Control	9.3 ± 0.6	2.32 ± 1.09	4.00 ± 0.87

Plastic responses of plant morphology has been observed to reduce the force on the plant when they are exposed to current by many authors (Puijalon and Bornette, 2006; Shutten and Davy, 2000). Clonal growth patterns have an influence in current velocity and vise versa. Both species showed (Table 4) results agreeing for these observations in the exposure to the flow turbulence also. Lateral branching of the plants were reduced and the branch length was also reduced. They were more preferred to be singular units at the exposure to the turbulence may be because lateral branching may enhance drag force they experience.

Table 4: Average number of branches per plant and average branch length (cm) of *E. densa* and *C. fibrosa* exposed to different turbulent levels, at the end of experimental period

	High	Medium	Low	Control
Average number of branches per plant				
E. densa	1.00 ± 0.00	1.75 ± 0.50	2.00 ± 0.82	3.50 ± 1.73
C. fibrosa	0.00 ± 0.00	0.75 ± 0.50	2.75 ± 0.96	3.25 ± 0.96
Average branch length (cm)				
E. densa	3.28 ± 0.95	4.95 ± 1.33	5.44 ± 1.45	7.54 ± 2.34
C. fibrosa	0.00 ± 0.00	0.88 ± 0.65	1.78 ± 0.39	5.13 ± 2.60

Plants exposed to water movement has been observed to show different plastic responses and observed to be depending on their own morphology. Species such as *Myrophyllum spicatum*, *Elodea canadensis* has been observed to have more streamlined shapes, when living in moving water and it has been observed that there is a tendency to reduce the drag force by this shape (Sand Jensen, 2008). These morphological variations have been observed to be outcome of variations of plant hormone metabolism and catabolism as well as stress responses of the plants (Ellawala et al., 2011a Ellawala et al., 2011b).

4. CONCLUSIONS

Plants exposed to turbulence show significant morphological variations and biomass partitioning as results of the exposure to flow turbulence. They may reduce the growth and biomass gain of the plant. In fact it is beneficial for the survival of the plant in that respective environment, since it can reduce the force exerted on the individuals.

5. ACKNOWLEDGEMENTS

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The Current Status of Density Stratification of Koggala Lagoon

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Abstract: *A field observation was conducted on 22nd November 2011 to estimate the present state of density stratification of Koggala lagoon. Vertical variation of salinity, water temperature and dissolved oxygen were measured with neap and ebb. The same measurements were taken from inflow streams. The observations indicate that main water body of the lagoon is salinity stratified despite the strong or partial mixing at the mouth. Particularly, strong saline stratification is prominent in the deep central part of the lagoon. Firstly, the current hydraulic state and then bulk parameter of Koggala lagoon is analysed. The relationship between the mixing state and the values of parameters of Koggala lagoon agree with Fisher's plot. The effects of modification of lagoon mouth are discussed based on these parameters. Temporal and spatial complexities of lagoon mouth affect transport and mixing of saline water and intrusion into the lagoon. In future, a modified bulk parameter will be needed for better understanding of stratification behaviour of the Koggala lagoon.*

Keywords: *Koggala lagoon, salinity, density stratification, lagoon mouth, estuary Richardson number.*

1. INTRODUCTION

Density stratification in coastal lagoons caused by saline water intrusion is one of the important factors determining various physical, chemical, and ecological processes occurred in the water body. The system of Lagoon is likely to consist of various elements such as catchment area, ocean conditions and lagoon mouth characteristics. The effects of these various factors surrounding a lagoon would influence mixing processes. Those various phenomena occurred in the lagoon affect social and economic activities around it. Conversely, various impacts by humans influence the coastal lagoon. Therefore, density stratification is an important issue to be considered for proper management of a lagoon.

Koggala lagoon is one of the forty-three coastal lagoons encircling the coastal belt of Sri Lanka. Extraction of substantial amount of sand from the sand bar at the mouth leads to a strong erosion. To prevent that a rock arm has been constructed causing lagoon mouth open all the time. The salinity level of this lagoon has increased due to large amount of seawater intrusion. These physico-chemical changes lead various problems such as socio-economical problems as well as natural ecosystem degradation in and around the lagoon. Thus, countermeasure for this issue is required. Previous studies have pointed out the importance of lagoon mouth morphology for restoration of lagoon environment (Priyadarshana et al. 2007, Gayana et al. 2009, 2010). By using hydrological parameters, an improved lagoon salinity level will be obtained by a new rubble mounde structure proposed by the authors. With the modifications it is expected that the ecosystem and the water quality of the lagoon would reverse to a more freshwater-oriented system. However, generally human impact will cause various unexpected effects on water bodies particularly complex systems like lagoons.

Amarasekara et al. (2011) reported that catches of the green chromide after groyne construction markedly decrease because of increase in salinity level. Some studies about past salinity condition of Koggala lagoon have been done (Suneetha and Chandana 2006, Priyadarshana et al. 2007) However, salinity stratification of Koggala lagoon has not been surveyed. Therefore the objective of this study is to estimate current states of density stratification of Koggala lagoon. This preliminary study would be useful for future research and management of Koggala lagoon as well as other coastal lagoons in Sri Lanka.

2. MATRERIALS AND METHODS

2.1. Study sites

Koggala lagoon is located on the southern coast of Sri Lanka (Fig. 1). Hydro-catchment at the lagoon outlet is about 55 km^2, of which about 15% is the lagoon area. It is estimated to have further 15% of paddy fields or low lying areas (Priyadarshana et al., 2007). The water depth ranges from 1.0 to 3.7 m (IWMI, 2006). The coastal lagoon is essentially fed by rain and a number of streams connected to it. Warabokka stream (Koggala-oya) enters the lagoon from the north-west. Kerena anicut was built combining both the streams named as Mudiyansege stream and Thithagalla stream. Heen stream contributes slightly to the water inflow. Apart from these three streams, Kahanda stream, Gurukanda stream, and Thelambu stream were contributors for inflow but presently these three are abandoned and have become marsh lands with almost zero water flow due to overgrown vegetation. The only outlet of the lagoon is Pol-oya located at the southeast corner; a narrow 300 m long canal connects the lagoon with the sea.

2.2. Field observations

Field observations to estimate present states of density stratification of Koggala lagoon were conducted (Table 1).

Figure 1 shows the survey points on Koggala lagoon. Generally vertical density stratification of the lagoon differs depending not only on temporal conditions but also on spatial conditions. In addition, river-inflow and intrusion of saline water affects on the stratification through the lagoon mouth from sea. Considering such lagoon characteristics, survey points were decided as shown in Figure 1.Two main river-inflows were selected as observation sites; Warabokka and Heen streams. The density profiles were measured both longitudinally and vertically from Warabokka to the lagoon mouth through the lagoon water body. The same measurements were also taken in specific locations in the lagoon mouth area. Table 1 shows the detailed conditions of each survey point.

A water quality measuring equipment (multi probe) YSI Model 55 was used to obtain vertical profiles of temperature, salinity, and dissolved oxygen (DO) (approx. 0.5 m intervals).

Table 1 Outlines of survey points

Group	Stations	Distance [km]*1	Survey time	Depth [m]	z_s[m]*2
Ocean	O−1	−	16:10	−	−
	O−2	−	16:20	−	−
Mouth	M−1	0	15:40	1	−
	M−2	0.1	11:00	2.5	−
	M−3	0.35	11:15	1.5	1.2
	M−4	0.6	11:28	1.5	1.3
Lagoon	L−1	1.2	11:45	3.5	1.3
	L−2	1.7	12:15	1.5	0.9
	L−3	2.3	12:55	1.5	0.8
	L−3'	2.4	15:10	1.5	−
Inflow stream	W−1	−	17:30	2	−
	H−1	−	17:50	−	−
	K−1	−	18:10	−	−

*1 Distance from mouth end to upstream
*2 Transparency

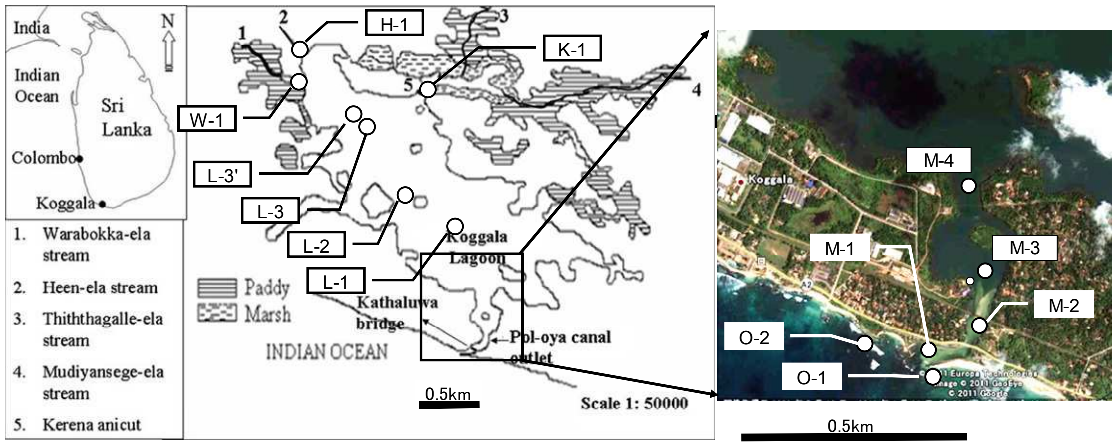

Figure 1 Koggala lagoon and survey points

Field observations were conducted on 22 November 2011. The lagoon receives fairly high rainfall from southwest monsoon from May to September. Although the survey was done on early northwest monsoon season (from October to February), it had already started raining before the survey day. Tidal condition of survey day was neap. The survey from the mouth area to the upstream station was done during 11 am to 5 pm period. Detailed time for each point is shown in Table 2.

3. RESULTS AND DISCUSSION

3.1. Density stratification of Koggala lagoon

In this section, the characteristics of density stratification and other related phenomena are described for each area.

3.1.1. Lagoon mouth area

Figure 2 shows the vertical profiles of each measured parameter of the mouth area. These measurements were made at the late flood tidal periods. The salinity level in the mouth area was low compared to that of the sea and it ranged from 10 to 20 ppt. Because the vertical nonuniformity increases at upstream and the lagoon mouth area is a transient zone which is influenced from strong mixing to partial mixing. Water temperature and DO exhibit similar vertical trends. M-4 station only shows the difference in results between the surface and the bottom. These results show that the bottom layer of M-4 remained without mixing for a certain period.

3.1.2. Koggala Lagoon

Figure 3 shows the vertical profiles of some parameters measured in Koggala lagoon. For comparison, the measured values at the upstream end of the mouth area (M-4) and the representative inflow stream (W-1) are also shown. The vertical profile of salinity shows similar trend for each point. Salinity level of the surface was about 10 ppt. For all points below 1m depth, halocline was confirmed. At the centre area of the deep layer, a high salinity bottom layer exists. Water temperature and DO exhibit similar trend. Below 1m depth high water temperature and relatively low DO concentration were confirmed. This means that deep layer water parcels remain without strong mixing with surface layer for a certain period.

The salinity level of the lagoon surface is similar to that of the inflow stream except for the stream surface. Brackish water from some streams through out the year would be the main source of lagoon surface water parcels.

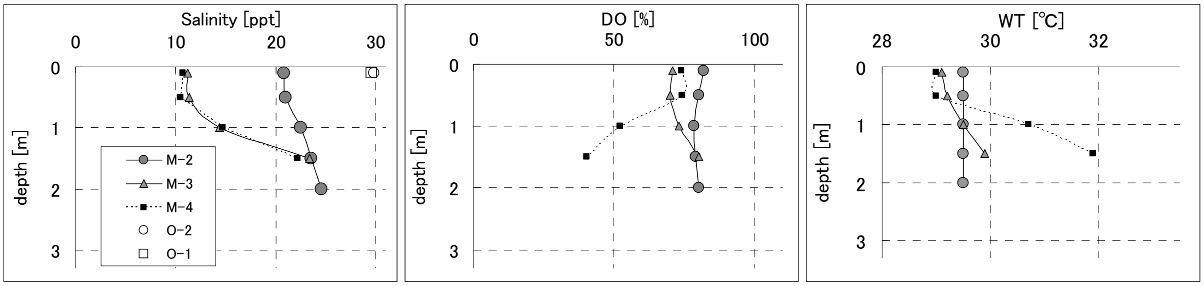

Figure 2 Vertical profiles of each measured parameter in the lagoon mouth area. The symbols in each legends mean survey stations (see Table 1).

3.1.3. Inflow streams

As shown in Figure 4, the inflow streams exhibit stratification of salinity and water temperature. The range of surface salinity was 0.8~4.4ppt depending on each stream. On the other hand, below the surface depth, the same salinity level was measured for each measured station in the streams. The brackish water supply from inflow streams corresponds to the effects of permanently open-mouth to Koggala lagoon as reported by Priyadarshana et al. (2007). The important point is salinity stratification of inflow streams. Raining before this survey is to be the reason to have fresh water layer in the stream. However, the low level of salinity in the surface water body which is less than 10 ppt was not found in the lagoon. This means probably that surface low salinity water of inflow stream would be mixed as surface plumes by any mixing factor such as shear stress by wind. However, there are no detailed evidences for such explanation. This is also one of future problems unresolved.

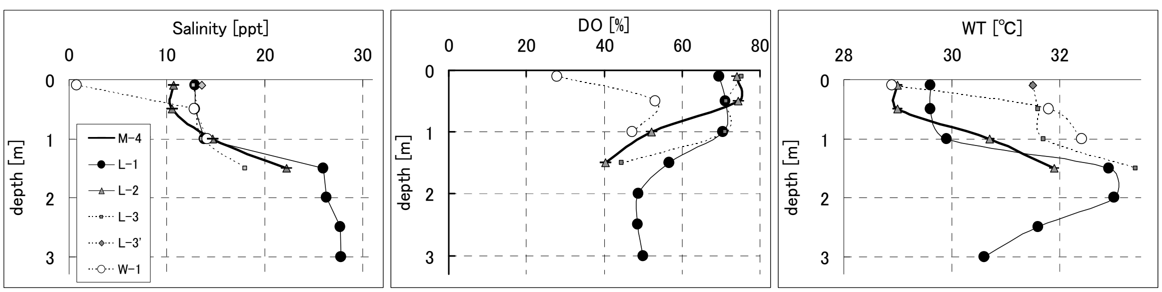

Figure 3 Vertical profiles of each parameter of Koggala lagoon.

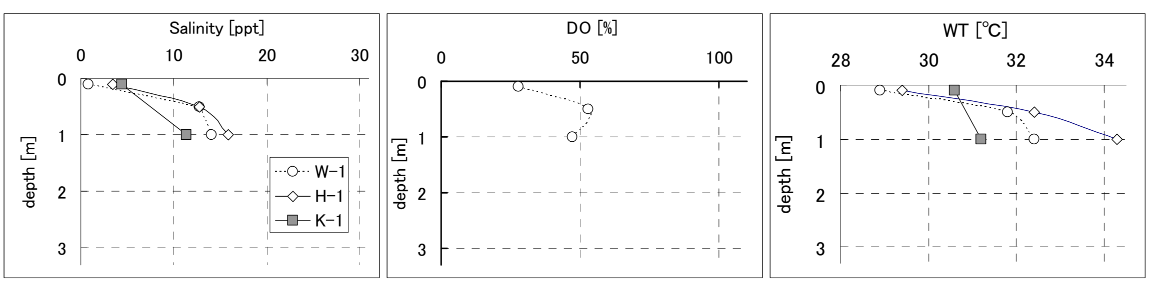

Figure 4 Observation results of inflow stream

3.2. The current states of mixing conditions and related phenomena of Koggala lagoon

Based on the field observation results described above, the characteristics of hydraulic and related phenomena in Koggala lagoon can be estimated as follows (Figure 5).

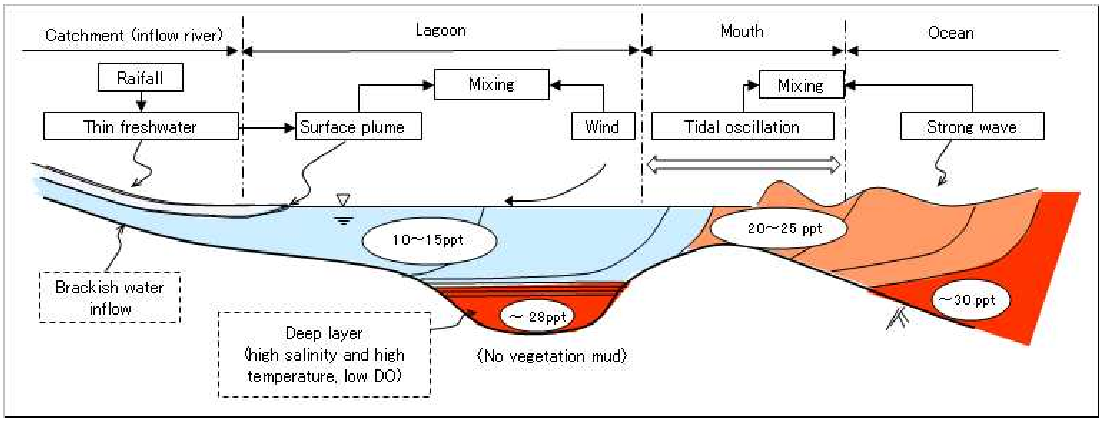

Figure 5 Synoptic representation of density stratification and related phenomena of Koggala lagoon of neap and high water. Lines in water bodies represent isohalines. Values in circles are salinity level.

Firstly in the lagoon mouth area, high saline brackish water parcels longitudinally oscillate as strong mixing state. However, in the upstream area of the mouth, mixing state transits to partial mixing. High salinity in deep water exists in the centre area of the lagoon. Because the survey was done at neap tidal condition, salinity level of this area is possibly higher than the level we measured in more strong tidal conditions such as in spring tide.

Secondly, internal water body of the lagoon consists of brackish surface layer with half salinity level of sea and high salinity warm deep layer. The depth of surface layer is about 1m and halocline exists up to 1.5m depth. Below this depth, the deep layer exists. Water temperature of the deep layer is higher than that of sea water (~32℃). Considering that the transparency is higher than 1m, the deep layer suffers solar radiation. Thus, the water parcel remained for a certain period after entering from sea to the lagoon in the strong tide condition before the measurements were taken. Generally, high temperature makes saturation concentration of DO lower level. Moreover, DO consumption rate is high at high temperature. Therefore, the deep layer has the potential of making anoxic condition. Submerged plants grown in the lagoon bottom were disappeared after removal of sand bar (Priyadarshana et al. 2007). This condition will lead more rapid decrease in DO concentration.

Finally, thin surface freshwater parcels of inflow streams could not be measured in the lagoon and even in the upstream station (L-3'). This means that surface vertical or horizontal mixing by wind stress can not be neglected in Koggala lagoon.

3.3. Bulk parameter analysis and estimation of future stratification states of Koggala lagoon

Salinity stratification occurred in coastal lagoons is influenced by various and complex factors. However, for engineering point of view, simple bulk parameters are useful for estimation of the effects of human intervention on natural phenomenon. In this section, classical bulk parameters related to estuary mixing are applied to the present states of Koggala lagoon. Using the parameters, the effects of lagoon mouth modification on future stratification was discussed.

1.1.1. Parameter description

Generally, salinity stratification of estuaries including coastal lagoons can be estimated by bulk parameters such as estuary Richardson number (R_{iE}) (Fisher 1972) and densimetric Froude number (F_m) (Hansen and Rattray 1966).

$$Ri_E = \frac{(\Delta\rho/\rho)\, g\,(Qf/b)}{U^3} \tag{1}$$

$$Fm = \frac{Qf/bd}{\sqrt{gd\,\Delta\rho/\rho}} \tag{2}$$

Where $\Delta\rho$ is the density difference between inflow freshwater and sea water, g is the acceleration of gravity, Qf is the discharge of fresh water into the estuary from tributaries, b is the width, U is the r.m.s. tidal velocity, d is the depth. Ri_E expresses the likelihood that a buoyant discharge mixes vertically in a river flow. If Ri_E is very small it is expected that the estuary to be well mixed. From observations of real estuaries, the transition from a well mixed to a strongly stratified estuary occurs in the range of $0.08 < Ri_E < 0.8$ (Fisher 1972). In addition to above relationship, large Fm makes δS (salinity difference between surface and bottom) increase.

3.3.1. Application of bulk parameters to Koggala lagoon

These parameters were estimated by using both field observation data and previous data on Koggala lagoon (Gunaratne et al. 2010, 2011). Table 2 shows these values. The values of bulk parameter are estimated as Ri_E 2.42, Fm 0.15 respectively. The values of $\delta S/S$, from both the observed and estimated data, were plotted according to original Fisher's diagram. As shown in Figure 6, the condition of present salinity stratification of Koggala lagoon agrees with the bulk parameter estimation. However, similar to the original diagram, there is a certain difference between the observed and estimated $\delta S/S$. These bulk parameters are concluded to be useful for roughly estimating the salinity stratification of Koggala lagoon from small number parameters.

Table 2 Values used for calculation of the Ri_E, Fm and $\delta S/S$

Parameters		Values	Unit	Description
Qf	Freshwater inflow discharge	9.478[*1]	m^3/s	Mean annual total stream water inflow
$\Delta\rho/\rho$	Density difference between inflow water from catchment and ocean	0.018	-	Estimated from observed salinity and water temperature obtained by this survey
b	Width of mouth	85[*2]	m	Existing lagoon mouth width
d	Maximum depth of mouth	1.0	m	Existing lagoon mouth topography
U	r. m. s. tidal velosity	0.2	m/s	Observed value at mouth end[*2]

[*1] Gunaratne et al. (2010) Table 3
[*2] Gunaratne et al. (2011b)

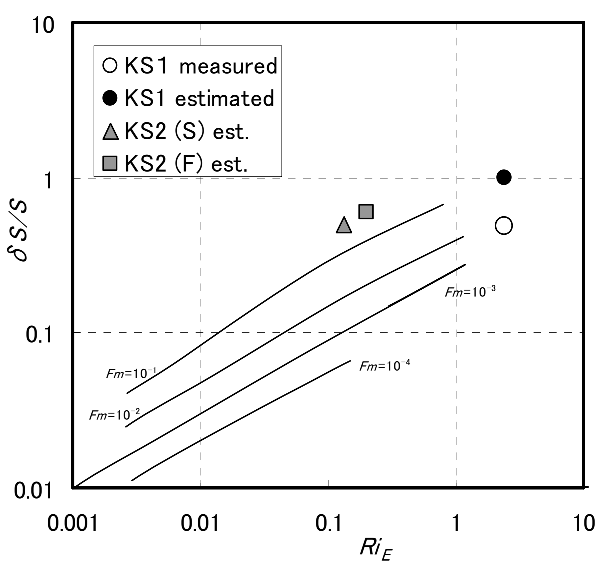

Figure 6 Present and future conditions of Koggala lagoon in Fisher diagram. ○, measured salinity difference;●, ▲ and ■, estimated value based on Ri_E and Fm

3.3.2. Problems of applying Fisher's model to a choked coastal lagoon

Fisher's model is directed to wide range estuaries including different types of water body such as river mouth, coastal lagoon and fjord. Additionally, there are three types for coastal lagoon such as choked, restricted and leaky lagoon (Kjerfve & Magill 1989). Hume et al. (2007) reported that many factors such as climate, oceanic, riverine, and catchment property determine the physical and biological characteristics of estuaries. Particularly, for a choked lagoon, the topology of lagoon mouth is complex. Intermittent mouth closing will have any effect on the stratification. Such temporal and spatial complexities of lagoon mouth will affect transport and mixing of saline intrusion. In future, a modified bulk parameter including such coastal lagoon characteristics will be needed. Particularly, mixing in the mouth region should be investigated because of the importance for stratification occurred in chocked lagoon, such as Koggala lagoon.

3.3.3. Future viewpoint of research on Koggala lagoon for proper management

Gunaratne et al. (2011b) stated the possibility of improving salinity condition of Koggala lagoon by modifying the lagoon mouth (see scenario KS2 in Gunaratne et al. 2011b). Table 3 shows the anticipated hydrodynamic changes by improving the lagoon mouth (Gunaratne et al. 2011b). In the table, we added the calculated bulk parameters related to density stratification. For the calculation, two future conditions are assumed. One is only mouth modification (KS2(S)). The other is that inflow stream water quality becomes freshwater (KS2(F)). Figure 6 also shows the estimated *δS/S* from these parameters although the validation of applicability of the model to Koggala lagoon is not sufficient. There is a potential for remaining any stratified condition of Koggala lagoon (Figure 7).

Table 3 Comparison between hydrodymanic and mixing parameters of Koggala lagoon between the Scenarios[*0]

Parameters	KS1	KS2 (S)	KS2 (F)
B	85m[*1]	20m[*1]	
Fs[*2]	0.68[*1]	0.54[*1]	
$T_{50\%}$[*3] [h]	9 – 37 [*1]	12 – 72 [*1]	
EF[*4]	17 [*1]	40.77 [*1]	
Salinity of inflow stream [ppt]	12	12	1
R_{iE}	2.42	0.13	0.20
F_m	0.15	0.62	0.50

[*1] Gunaratne et al. (2011b)
[*2] Salting factor (Chubarenko et al. 2005)
[*3] Flushing half time (Gunaratne et al. (2011b)
[*4] Evacuation factor (Haines et al. 2006)
[*5] Salinity level of inflow for estimating inflow density, ρ

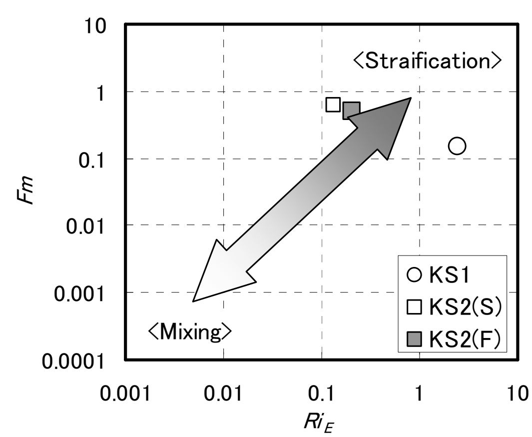

Figure 7 Bulk parameters of present and future of Koggala lagoon

Needless to say, returning the salinity level to past condition is important. However, in these processes, various unexpected phenomenon would occur. Becker et al. (2009) reported that fish-kill by anoxic conditions occurred by closed mouth in temperate coastal lagoon. As shown in Figure 5, one of the physio-chemical characteristics of Koggala lagoon is “high saline and high temperature bottom layer”. Generally, high water temperature leads low DO level because of low saturated DO concentration. Thus, if the decrease in exchange between the deep layer and oxic water parcels such as in surface layer or saline intrusion from sea occur, DO level will immediately become anoxic condition. It is not immediately evident whether such phenomenon occurs or not in the future. However, for proper future management of Koggala lagoon, the research focusing on density stratification will be needed. Furthermore, the knowledge about the relationship between the conditions of lagoon mouth and internal processes of lagoons likely density stratification will be needed for sustainable development of coastal areas in Sri Lanka.

4. ACKNOWLEDGMENTS

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Study on the efficiency of removing some environmental pollutants in yellow water by using sand and brick powder

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Abstract: *Urine and flush water is known as yellow water in general. Releasing of untreated urine to the environment causes bad odor, aquatic and terrestrial pollution also. The main focus of this research is to investigate the usage of sand and red brick powder to remove environmental pollutants in yellow water. Filtration column used for the study was packed by using sand or brick particles in the range of 0.270-0.355 mm diameter. Continuous flow filtration through column of sand or brick powder was investigated for removal of total nitrogen, color, conductivity and COD caused by urine. Whilst brick powder demonstrates good filtration efficiency, sand is found to be inefficient for the intention. Initial efficiencies demonstrated by brick powder for removal of total nitrogen, color, conductivity and COD were 93%, 92%, 55%, 65% respectively and decreased upon continuous filtration except for total nitrogen. During the study, total nitrogen removal efficiency remained constant.*

Keywords: *Yellow water, Urine, Brick, Sand, Total nitrogen, COD*

1. INTRODUTION

Urine and flush water, known as yellow water, is the nutrient richest component of domestic waste water output whilst gray water and brown water also contribute on nutrient content of domestic waste water. Waste water form kitchen and bath is known as gray water. Brown water may contain feces, toilet papers and flush water (UN 2006). Since one liter of urine averagely contains 14 g of nitrogen, 0.8 g of phosphorous and1.7 g of potassium (Putnam 1971), yellow water remains the nutrients richest even after being diluted with flush water. There are many environmental problems related with urine. Average urea content of urine is 16.3 g/L^{-1} and hydrolysis of urea in urine gives an unpleasant odor (Udeart *et al.* 2003). Although urine is expelled from the body as a sterile liquid, urine provides necessary nutrients and good environment for microorganism growth (Medes and Lynch 1976). Therefore, many pathogenic microorganisms can be proliferated on urine. It has been reported that large number of hydrophilic pharmaceuticals residuals are excreted with human urine. Often they are excreted only with slightly or no transformation (Heberer 2002). Synthetic estrogen compounds, the active ingredients of contraceptives, has been found in human urine at higher concentrations (Pacakova 2009). Although the environmental concentrations of estrogens are very low, their adverse effect on the reproduction of wildlife and humans is significant. Berkowitz (1941) has reported that, inhibition of development of secondary sex characteristics of male fish upon exposure to estrogen. Also such treated fish, took on female appearance regardless of their gender. This clearly indicates that, allowing urine to reaching aquatic bodies may have adverse effects on fish and other aquatic organisms. Also these pharmaceutical residuals may undergo different chemical reactions upon chlorination, which is used for disinfection of drinking water in Sri Lanka. Mixing yellow water and other domestic waste water and releasing to aquatic bodies is one of major reasons of their eutrophication (Tidakar 2003). A large number of researches have been carried out to investigate the use of urine as nutrient rich liquid fertilizer. Pradhan *et al.* (2007) has reported that human urine can successfully substitute for industrial chemical fertilizers. National Aeronautics and Space Administration (NASA) reports, recovering potable water by reverse osmosis of pretreated urine.

Currently in Sri Lanka, urine is allowed to mix up with household waste water and sent to septic tanks. Although illegal, in some cases combined household waste water is released to the environment without any treatment. Especially, in the case of mobile toilets which are mostly used at outdoor special occasions, untreated urine is released in to the municipal waste water even without dilution. In such incident, a large volume of urine is released to the environment within a short period causing bad odor and rapid pathogen proliferation. The main focus of this research is to investigate the usage of sand and red brick powder to remove environmental pollutants in yellow water.

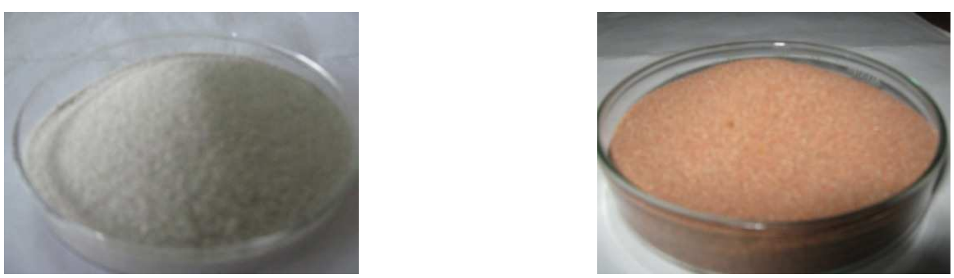
Kahn and Zareen (2006) have reported successful usage of granite sand for removal of anionic surfactants in water. Selvaraju and Pushpavanam (2009) has reported successful usage of fired red brick powder to remove chlorides, nitrates and phosphates in gray water. Also brick powder has shown good adsorption properties than sand having same particle sizes, due to its high surface area and increased wettability. Priyantha *et al.* (2011) has reported that the brick particle surfaces are negatively charged and having better adsorption properties of positively charged heavy metal ions. Carbon particle are formed in brick due to partially burnt organic matters at firing and they adsorbs both negatively charged and neutral species. Therefore, it is important to investigate red brick powder for pollutant removal of yellow water.

2. METHODOLOGY

Chemicals were obtained from Merck, Fluka, BDH, S.D. Fine-Chem Ltd. and Himedia. Analytical grade reagents were used whenever it was possible. pH measurements were carried out by Cyberscan 510 pH meter. Color was measured using DR/890 Data logging Colorimeter. TDS of samples were measured by Cpberscan PC 300 TDS meter.

2.1. Material preparation

Sand was collected from different areas of the country. Sand particles in between 0.270-0.355 mm in diameter was separated after sieving, washed with tap water, mud and dirt were removed, washed again with distilled water and allowed to dry at 80^0 C (Figure 1a).
Red brick were collected form Galle area and crushed by mortar and pestle. Brick particles in different size ranges separated after sieving, washed with distilled water until colorless washings and allowed to dry at 80^o C (Figure 1 b).



(a) (b)

Figure 1 (a) sand and (b) brick powder after preparation process

2.2. Filtration column preparation and use

Filtration columns were prepared with sand and or brick powder separately. An amount of 290g of sand or brick powder was used to pack a column with 3 cm diameter and 50cm bed height. Urine samples were collected from university students and working staff and samples were mixed. Urine filled to the column, allowed to flow through the column continuously and the flow rate was maintained at 50ml/hr.
The column effluent was collected and removed for analysis after each hour.

2.3. Chemical analysis

Chemical analysis of column effluent was carried according to ASTM waste water analysis procedures. COD was measured by closed reflux method. Total Nitrogen was analyzed by kjeldhal method.

3. RESULTS AND DISCUSSION

3.1. Efficiency of pollutant removal by sand

Removal efficiency (E) of all parameters (X) was calculated by;

$$E = \left(\frac{X_{Influent} - X_{Effluent}}{X_{Influent}} \right) \times 100\%$$

Color removal efficiency of different types of sand in the range of 0.270-0.355 mm in diameter is exploited in figure 3.1.

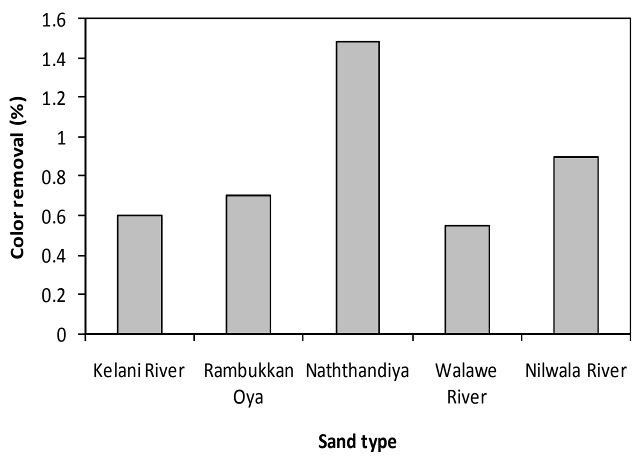


Figure 2 Efficiency of color removal by different types of sands

As exploited in figure 2 any type of sand is not efficient for removing color of urine. Color of urine is due to urobilin, which is a conjugated organic bio molecule (Lepp, 2006). COD and total kjeldhal nitrogen analysis revealed filtration through sands are having negligible effect on those parameters, confirming poor adsorption properties of sand. Although sand is reported to be efficient in raw water treatment (Fuhrman *et al,.2005)*, they are not effective for removing highly concentrated pollutants in urine. Out of five sand types Naththandiya sand, which are having fresh surfaces showed the best adsorption properties. Active surfaces of sand types collected from river beds may be already saturated with mud and other pollutants and may lead to poor adsorption properties.

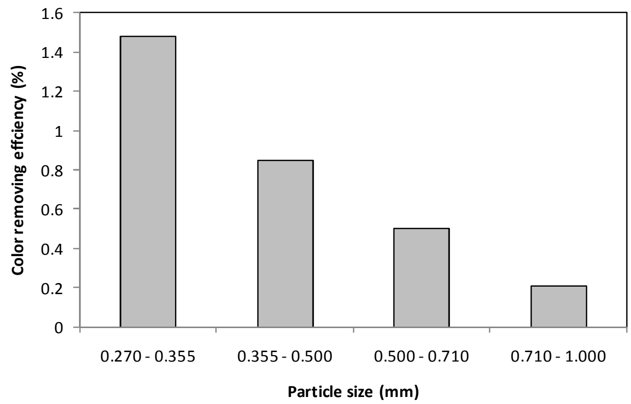
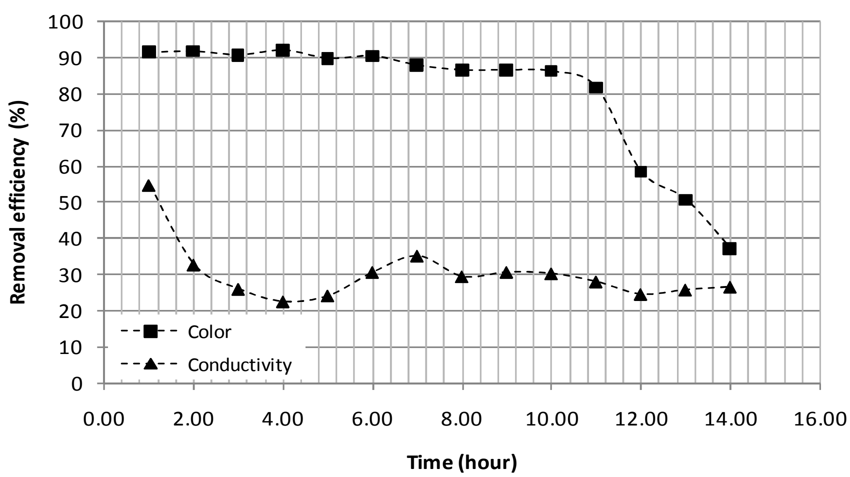


Figure 3 Color removal efficiency of Naththanadiya sand

Color removal efficiency of different particle sized of Naththandiya sand is shown in figure 3. It was observed that smaller sand particles which have higher surface area are more efficient in color removal. Sand particles are smaller than 0.270 were not suitable to be used as a filter material, because of making colloids in the solution and results in slow filtration process. Therefore optimum particle size was selected as 0.270-0.355 mm. Since all types of sand were not sufficient enough for pollutant removal in urine, red brick particles in the range of 0.270-0.355 mm were investigated to be used for the purpose.

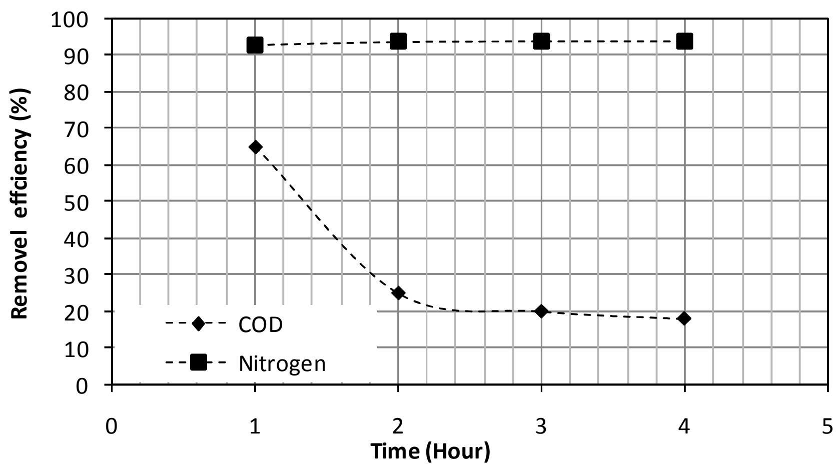
3.2. Efficiency of pollutant removal by red brick powder

Efficiencies of color and conductivity removing efficiencies posed by brick powder is exploited in figure 4 Conductivity reducing efficiency was gradually decreased with the time indicating the breakthrough of dissolved solids after 4 hours. Color removing remained at higher efficiency 10 hours and thereafter showed a sharp decrease may be due to formation of saturated monolayer.



(a)

Figure 4 (a) Color and conductivity (b) COD and total nitrogen removing efficiency by brick powder



(b)

Figure 4 Continued

COD and total nitrogen removing efficiencies by brick powder are shown in figure 4 (b). Chemical analysis was limited to four effluent samples collected through consecutive 4 hours, due to time constrains. Analysis were carried out in triplicate and averaged. Throughout the time brick powder has shown good ability to decrease the total nitrogen content of yellow water. COD of urine is reported to be a very high value (~8,000 mg/dm^3) and may cause to severe environmental stress (Putnam 1971). COD reducing efficiency also showed a sharp decrease after one hour indicating a formation of monolayer carbonaceous matter. Also the effluents were free of odor.

Brick powder can be used for mobile urinals to prevent the bad odor caused by yellow water. Since brick powder has a higher efficiency to retain nitrogenous compounds, exhausted filter materials can be used as a nitrogen rich fertilizer. Already urine is used a nitrogen rich liquid fertilizer. But, handling of liquids may be cumbersome than a solid.

For this research urine was used without any dilution, while yellow water contains few times diluted urine. Removal efficiencies will be higher for diluted urine than the reported efficiencies for undiluted urine.

4. CONCLUSIONS

Collected sand types are not efficient to remove the pollutants in yellow water. Brick powder has shown a good color and total nitrogen removing ability. Although brick powder has ability to decrease the COD and the conductivity of yellow water, the breakthrough is quick. Brick powder is suitable to be used a filter material in mobile urinals.

5. ACKNOWLEDGEMENT

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Suggestions to Implement Integrated Solid Waste Management Plan in Galle Municipal Council Area

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***Abstract**: The present practice of solid waste management (SWM) in the Galle Municipal Council (GMC) area is not adequate to meet the 18 tons of daily SW generation. Hence the suitability of implementing Integrated Solid Waste Management (ISWM) was studied by characterizing SW of the area and studying carefully the current practice. The study revealed that SW was mostly organic with a percentage ranging from 49 to 94 with relatively high moisture content. There was a notable decrease of organic content and an increase of the inorganic content in comparison to previous years. Though the highest calorific value (CV) which was 18 MJ/kg is less than that of normal fuels, fuel generation can be implemented through the production process of Refuse Derived Fuel (RDF). The key suggestions are source separation, introduction of composting and anaerobic digestion for all the biodegradable waste, introduction of recycling options for recyclables and an engineered landfill for the inert matter.*

Keywords: *Solid waste, biodegradable, non-biodegradable, composting, integrated solid waste management*

1. INTRODUCTION

Galle is the main city in the Southern Province of Sri Lanka with a population of 125,000 spreading in an area of 16 km^2. GMC has carried out the SWM of this area since more than a hundred years ago. Though there were all elements of SWM like collection, transport, treatment and disposal, it was evident that the existing system is not satisfactory. The municipality area is divided into 14 wards in which the generated waste is daily collected. The organic waste from 3 wards is used to produce compost and another small portion is used at the anaerobic digester at the GMC premises. The rest is dumped at a semi-controlled landfill at Heenpanthala. The energy from the anaerobic digester is used for some activities in the GMC and the produced compost is used at a farm and the rest is sold to the public. The compost is of low quality due to not maintaining the optimum conditions (moisture content, particle size) during the process and not using modern machineries like huller machines and cutters. Source separation is hardly practised and unsorted waste is dumped on a semi-controlled landfill with no engineering principles. The disposal site is adjacent to Ging River making the location highly unsuitable due to the threat of contamination of river water with highly polluted leachate.Therefore this study is significant because it is timely and crucial to implement ISWM in the GMC area. The quality, quantity and characteristics of SW are imperative for a sound SWM system that includes the selection of resources and energy recovery potentials. Through a thorough analysis of the quality and characteristics of SW in the GMC area, a proper solid waste practise can be implemented. The aim of this study was to investigate how to implement ISWM plan in the GMC area through an analysis of quality and characteristics of municipal solid waste (MSW).

2. METHODOLOGY

The study area (Figure 1) consisted of fourteen wards of the GMC, namely Fort, Bazaar, China Garden, Pettigalawatta, Magalle, Katugoda, Eliot Road, Kaluwella, Richmond Hill, Hirimbura, Ginthota, Kanampitiya, Market and Thalapitiya. The study was based on three main aspects which are waste characterization of the GMC area, the existing SWM practice and the public awareness and involvement.

For characterization of the SW, the waste collected from all fourteen wards were subjected to the following analyses: SW composition, bulk density, percentage moisture and total solids content, percentage volatile solids content and the energy content. Composition and bulk density was obtained by manually separating the waste in to categories and weighing and by filling a container of known volume with waste and weighing respectively. Percentage moisture content was calculated by weighing a portion of SW before and after keeping in an oven at 50^0C for 24 hours. Percentage of volatile solids was obtained by igniting the SW in a muffle furnace at 550^0C for 1 hour. Energy content was found by using Bomb Calorimeter test.

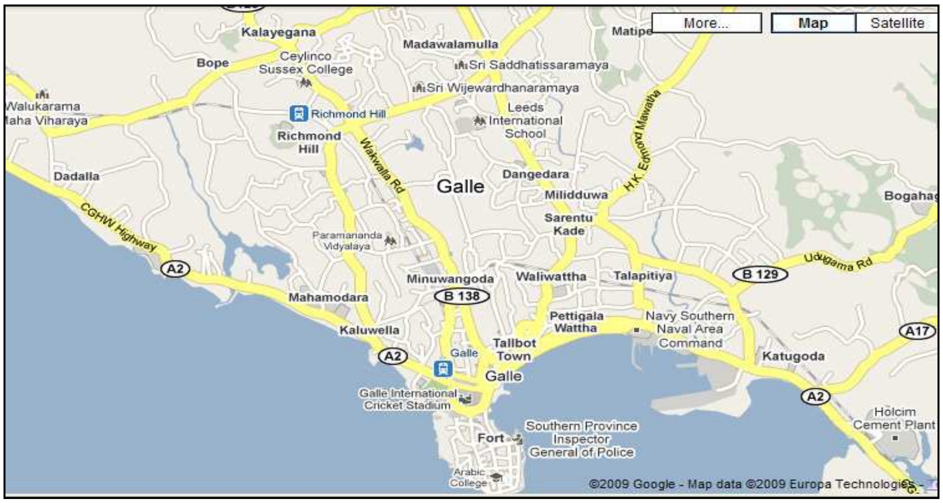

Figure 1 Study area

The study on the existing SWM practice was mainly focused on the composting and anaerobic digestion processes. In order to evaluate the performance of the composting plant, samples were collected from composting piles to represent each week of total 8-week composting period. They were analysed for the percentage moisture and volatile solids content.

In order to evaluate the public awareness on SWM issues and their involvement in related activities, a questionnaire was distributed among 100 households selected randomly covering all fourteen wards. The questionnaire included questions to determine their satisfaction or dissatisfaction over the existing practice of SWM, their views on weaknesses of the current practice and their suggestions to improve it.

In addition another questionnaire was distributed among 25 workers in the SWM sector of the GMC with the intention of collecting data on their capability, safety and requirements. Based on the output, their level of training, job satisfaction, awareness on safety and sanitation were understood.

Finally based on the results of the above mentioned categories, key components of an ISWM plan for GMC were suggested.

3. RESULTS AND DISCUSSION

3.1 Composition of Solid Waste

Figure 2 shows the composition of SW in each ward. It can be seen that the SW from all wards contains a very high organic content. The wards contain an average of 79%, 8% and 9% of polyethene, plastic and paper respectively. Due to the high organic content, the best treatment for this type of SW would be composting. SW from Dewata, Thalapitiya and Hirimbura have the highest percentage of organic content, 94%, 85% and 84% respectively. Therefore SW from these three wards would give the maximum outputs in composting.

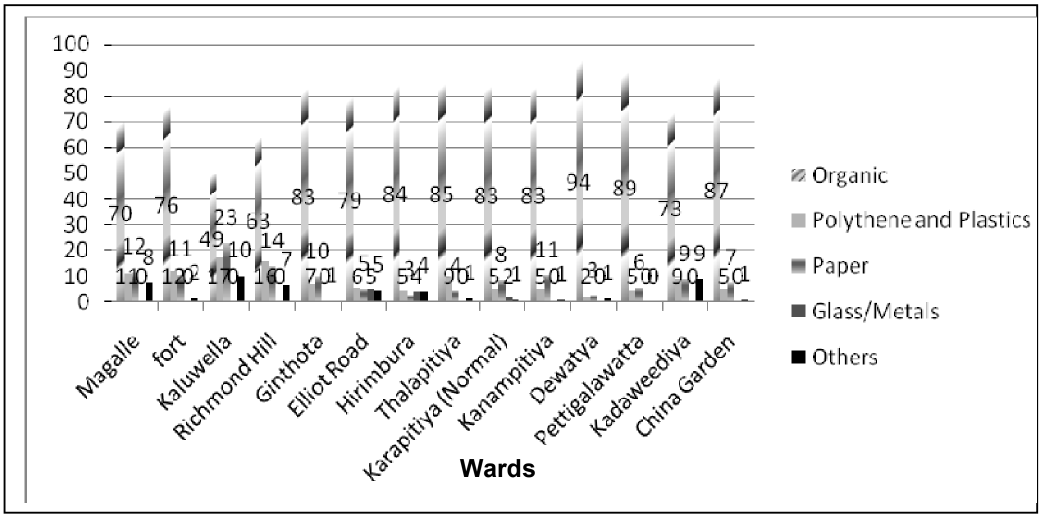

Figure 2 Composition of solid waste

Figure 3 shows the variation of the average composition of SW with time. Comparing with the values of the year 2006 (Karunasiri, 2006) and the year 2005 (Todd, 2005), there is a clear decrease of organic fraction. The developmental activities and the changes in lifestyles of people can be reasons for the reduction of organic content by 7 % in three consecutive years. On contrary the amount of polythene, plastic and paper have increased in a significant amount. It may be due to the extensive utilization of plastic and polythene. Modern technological advances in packaging goods create a constantly changing set of parameters for the designer of SW facilities. Of particular significance are the increasing use of plastics and the use of frozen foods, which reduce the quantity of food wastes at home but increase the quantities at agricultural processing plants (Tchobanoglous et.al., 1993).

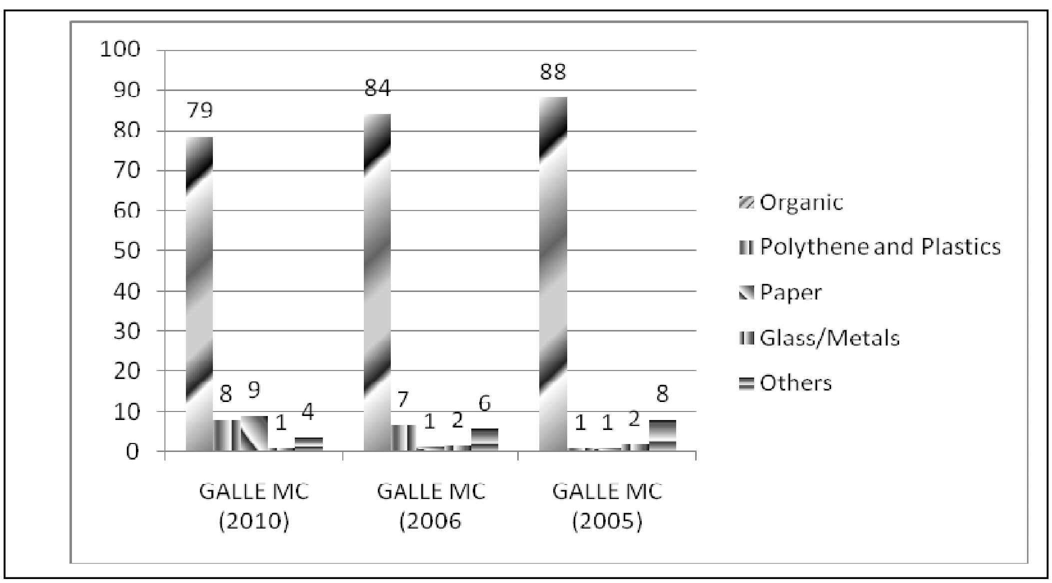

Figure 3 Variation of the average composition of GMC-SW with time

3.2. Moisture Content of Solid Waste

All the wards excluding Kaluwella, Ginthota and Elliot Road contain more than 50% of moisture which can be clearly seen in Figure 4. In many wards the moisture content is more than the solids content. Thus it is very impractical to use methods like incineration to treat the waste. The disposal of waste with no treatment can be highly harmful to the environment as the leachate generation will be high due to the high moisture content.

Landfill- leachate contains a variety of chemical constituents derived from the solubilization of the materials deposited in the landfill and from the products of the chemical and biochemical reactions occurring within the landfill (Tchobanoglous et.al., 1993). Hence many adverse effects can occur due to the mixing of leachate with the water bodies and soil. As the location of GMC- landfill is on a river bank, the adverse effects could be very much high.

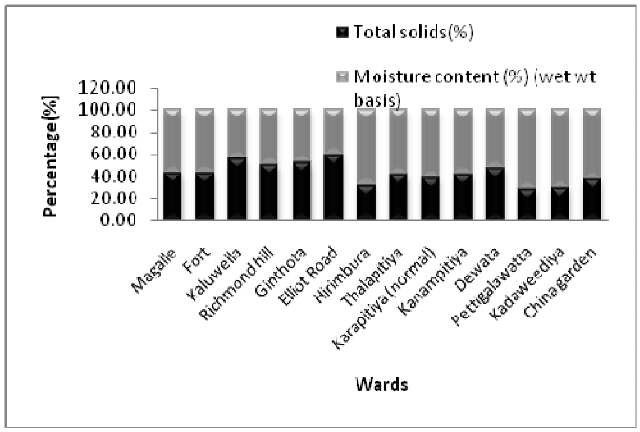

Figure 4 Total solids and the moisture content of solid waste

3.3. Fuel Value of Solid Waste

When comparing the calorific values of SW (Figure 5) produced in the GMC area with those of some fuels, it can be seen that the GMC-SW has very low values. Incineration will not be a suitable treatment technique due to the high organic and moisture contents. If the waste contained high amounts of paper, polythene and plastic, high calorific values could be expected.

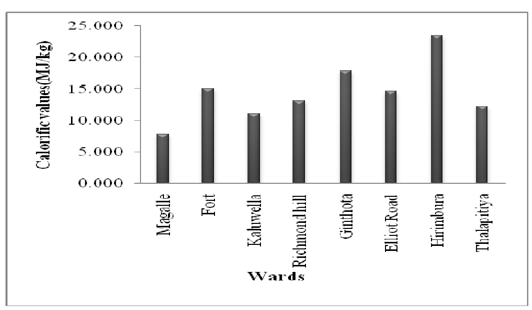

Figure 5 Calorific Values

3.4. Questionnaire Survey among General Public and Workers

According to the results obtained from the questionnaire survey done in 100 households, the public are aware of the SWM practice in GMC area. They appreciate the daily collection. Nearly 100% of the public give their cooperation by placing the garbage bins outside at the correct time for collection by the GMC workers. Some households do source separation of SW as biodegradable and non-biodegradable wastes. Separate containers have been provided to them by the GMC. About 37% from the surveyed community dispose separated waste. However it can also be stated that the public awareness on waste transformation and waste disposal is very less. Less than 10% is interested in home based composting. Thus it can be stated that the public awareness on waste collection is satisfactory, while the awareness on transformation, disposal have to be improved. By the survey done among the workers of the landfill site, composting plant and collection scheme, it can be said that workers are enthusiastic about their profession even under limited facilities. 86% of the laborers satisfy with their job and 86% have received training on SWM prior to work. However their concern on safety is very less. Only 21% use boots; 28% use gloves and none uses masks. Among the problems they face is not receiving vaccination at regular intervals and not being provided with sanitary facilities. They require proper uniforms and rain coats suitable to their work type and the climate. By providing those will increase their efficiency and enthusiasm about the job.

3.5. Present Practice of Solid Waste Management in GMC Area

Only the wastes from 3 wards are currently used for producing compost. Figure 6 shows the variation of the percent moisture content of the composting pile with time. The optimum moisture content for composting is 55 % or at least between 50-60 %. For most organic wastes, once the moisture content is brought to a suitable level (50-60%), the microbial metabolism speeds up (*Tchobanoglous et.al., 1993*). Hence the optimum moisture content is not maintained even in the initial piles. The lack of water added to the piles can be identified as the cause for this problem. Because the water to the site is given by the National Water Supply and Drainage Board, the officers are reluctant to use a large amount of water for the composting piles.

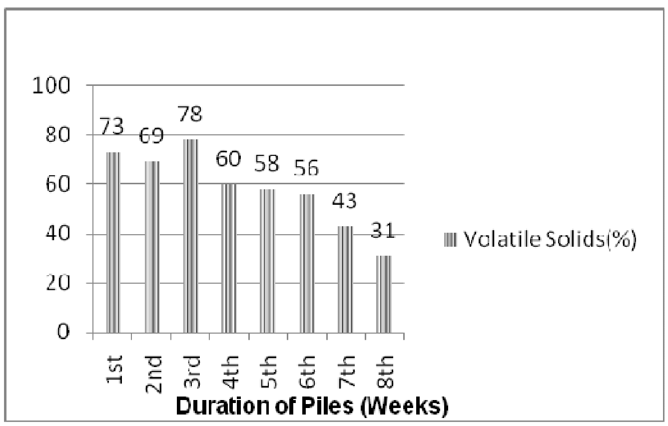

Figure 6 Variation of percent moisture content of the composting pile with time

Figure 7 shows the reduction of the volatile solids content with time. This indicates the efficiency of the plant. The overall reduction of volatile solids during composting is about 40 %. The composting piles showed signs of lack of moisture. The piles of the latter weeks had white fungus such as Aspergillus fermigatus,which is a threat for the human health. This fungus is believed to be responsible for causing respiratory problems if inhaled. Most fungi have the ability to grow under low moisture conditions, which do not favour the growth of bacteria (*Tchobanoglous et.al., 1993*). The compost produced in the Heenpanthala plant is not efficient as a soil conditioner, and there is also no clear characterization of the compost. This can be due to not maintaining the optimum conditions of temperature, moisture content and pH.

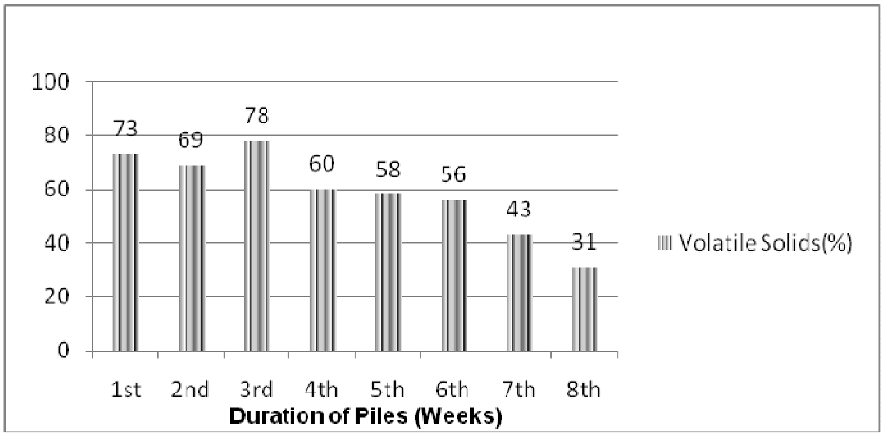

Figure 7 Variation of percent volatile solid content of the composting pile with time

3.6. Suggestions for Implementing ISWM in GMC Area

The key suggestions are based on the following functional elements of ISWM: waste handling and separation, storage and processing at the source; collection; separation, processing and transformation of SW; transportation; and disposal. Source separation of SW is of major concern. The SW collectd from GMC would be categorized into organic waste, reusable or recyclable waste and unusable items and different bins would be provided to dispose those. A suitable colour code can be introduced for easy identification. Households can be advised to implement home based composting systems for organic waste. Several households can get together and maintain an anaerobic digester for their organic wastes. The gas from digesters can be used for cooking purposes. The compost produced at Heenpanthala compost yard can either be used or sold to outsiders. Reusables and recyclables can be collected by the GMC staff once a week. The waste should be transported to the treatment facilities or disposal sites by covered vehicles, thus the odour and vector attraction would be minimal. The vehicles provided for waste collection and transport by GMC should fit to transport waste with high density and moisture content. The vehicles selected should be more suitable for roads of Sri Lanka and the type of SW generated. Heavy vehicles like the compactor truck used in the present practice create a huge traffic congestion in narrow roads and also not easy to be repaired.

The biodegradable waste form GMC should be sent for either composting or anaerobic digestion. The non-biodegradable waste can again be divided into recyclables and non-recyclables. Recyclables like polyethene, plastic, paper, cardboard can be sent to the relevant recycling centers. The non- recyclables can be dumped on an engineered landfill. As home based composting is a key strategy, the composting plants managed by the GMC may not be very large. Turning of piles, moisture content and C:N ratio should be properly maintained in the Heenpanthala composting plant, thus the best quality compost is produced, which is sold to the public at various outlets around Galle.

The gas from the anaerobic digesters at MC office can be used for energy recovery purposes and the slurry can be sold as a fertilizer to the farms. Recycling shops have to be established so that the people can give their paper, metal, glass for recycling or reusing and earn some money. The remaining waste after all the treatment should be disposed in an engineered landfill. The existing semi-controlled landfill at Heenpanthala should be improved to an engineered landfill by providing a proper liner, landfill gas and leachate collection systems and daily and final soil covers.

The waste produced by the Karapitiya and Mahamodara hospitals of GMC area will be separated as hazardous and non-hazardous waste. The hazardous waste can be incinerated within the hospital premises and the other waste have to be collected by the GMC and directed to the common waste treatment stream. The ash or residue from the incineration should be disposed in a separate area.

Another option for this type of SW would be to utilize it in energy generation by converting it to RDF. Though the initial cost of such a facility would be high, it would be possible to implement that technology to all type of SW without going for separate methods such as composting, anaerobic digestion and land filling. Thus it would be advantageous when the long term benefits are taken into consideration. However, the typical moisture content of RDF is 7-28 %(Gendebien et.al. 2000), thus a drying procedure will have to be done for the SW prior to the generation of RDF. By utilizing this technology, the energy problem will be solved by some fraction and the large areas required for land filling and the large number of workers needed for every stage of SWM will be reduced.

4. CONCLUSIONS

Rapid urbanization and industrialization have increased the generation of SW and changed the conventional SW composition in the Galle municipality. Therefore it is timely strategy to develop an ISWM plan for the GMC area.

The results of the SW characterization in the GMC area indicate that the waste is mostly organic and contains high moisture contents. Therefore the treatment methods used by the GMC such as composting and anaerobic digestion are suitable. The waste cannot be directly used for fuel generation. It was observed that the GMC has adopted reasonably good collection, treatment, disposal methods. However the efficiency of those services is not to the satisfactory level. The safety and health condition of the waste handling personals are also not given a proper attention.

The key suggestions to implement an ISWM plan for the GMC are source separation, introduction of composting and anaerobic digestion for all the biodegradable waste, introduction of recycling options for recyclables and an engineered landfill for the inert matter. The production of refused derived fuel is another option to utilize the fuel value of solid waste. It is concluded that if the public support can be obtained through good awareness, and if the current services and facilities are improved, ISWM could be successfully implemented in the GMC area.

5. ACKNOWLEDGEMENT

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Effects of the coastal forests, sea embankment and sand dune on reducing washout region of houses at the tsunami caused by the Great East Japan Earthquake

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Abstract: *The tsunami caused by the Great East Japan Earthquake on 11 March 2011, broke most of the sea embankment and coastal forests, and caused dreadful damage to people and buildings in Tohoku and Kanto districts of Japan. This study hypothesized that the coastal forest had a tsunami mitigation effect even when the coastal vegetation was bent down, because most of the vegetation was not washed out, hence could acts as a dense roughness element. Therefore, the objective of this study is to evaluate the vegetation effect on reducing the washout region of houses under severe tree breaking phenomenon using numerical simulation and data from field investigation in April and May 2011. Numerical simulations estimated the effects of a 640m-coastal forest, sea embankment around 5.4m in height or sand dune (2m increase) on reducing the washout region of houses by around 100 m, 600m and 600m for 10m height tsunami at coast. It was observed/concluded that although the quantitative effect of coastal forest is smaller than sea embankment, the coastal forest and sand dune is not a negligible component of the mitigation measures when a large tsunami occurs and overflows the sea embankment.*

Keywords: *tsunami, coastal vegetation, inland embankment, critical moment of washout of houses*

1. INTRODUCTION

Tsunamis can cause massive destruction to both human life and socioeconomic property both on the coast and in the hinterlands. The importance of further mitigation techniques were recognized to be constructed after the Indian Ocean tsunami. Mitigation techniques are broadly categorized into two types. These include hard solutions utilizing large embankments and tsunami gates, and soft solutions utilizing a natural buffer zone of coastal vegetation, and sand dunes. Research on the effectiveness or limitations of coastal vegetation has accelerated since the 1998 Papua New Guinea tsunami, and the Indian Ocean tsunami on 26 December 2004.

The tsunami caused by the Great Japan Earthquake at 14:46 JST on 11 March 2011, which had a magnitude of 9.0 and epicenter 129km east of Sendai, broke most of the sea walls (tsunami gates, large embankments) and caused dreadful damage to people, buildings, and coastal forests in the Tohoku and Kanto districts of Japan. In particular, the tsunami passed through sand dunes planted with coastal vegetation (mainly pine trees) and completely washed out the houses behind the forests for 0.4–1.6km and partly destroyed them for 1.4–5.2km, especially in the Sendai Plain.

Several of our previous studies (Shuto (1987); Tanaka et al.(2007)). have discussed the effects of vegetation on tsunami mitigation based on numerical simulation results. However, the effect of tree breakage was not considered in most of these studies except for Yanagisawa et al.(2009), Tanaka et al.(2010) and Thuy et al.(2011). Yanagisawa et al. performed field surveys and proposed a fragility function for mangrove trees (*Rhizophora* sp.) to describe the relationship between the probability of damage and the bending stress caused by the maximum bending moment that was based on the field studies. Authors also analyzed how the breaking of trees in a forest affects tsunami disaster mitigation effects using a numerical model based on two-dimensional nonlinear long-wave equations which calculate the breaking condition of sand dune vegetation directly considering the tsunami force and bending moment of trees (Tanaka et al.(2010); Thuy et al.(2011)).

Even though the coastal vegetation was bent down by the Japanese tsunami, most of the vegetation was not washed out, hence could acts as a dense roughness element. Therefore, the objective of this study was to evaluate the effect of vegetation in reducing the damage to houses when severe tree-breaking phenomena occur. To fulfill these objectives, a field investigation was conducted in the coastal zone of the Sendai Plain, Japan, and quantitative information on the effects and limitations of coastal vegetation was evaluated by a numerical simulation.

2. SITE LOCATIONS AND MEASUREMENT METHOD

2.1. Information of sites

Field investigations were carried out in April and May of 2011 in the tsunami-affected forests in the Tohoku area of Japan (see **Figure.1**). The representative vegetation was mainly *Pinus densiflora* Siebold and Zucc. and *P*. *thunbergii* Parlat. In each location, tsunami water depth, damaged situation of sea embankment, trees (diameter, height and density) and houses were investigated. The tsunami water depth at each site was determined by water mark or evidence of collisions.

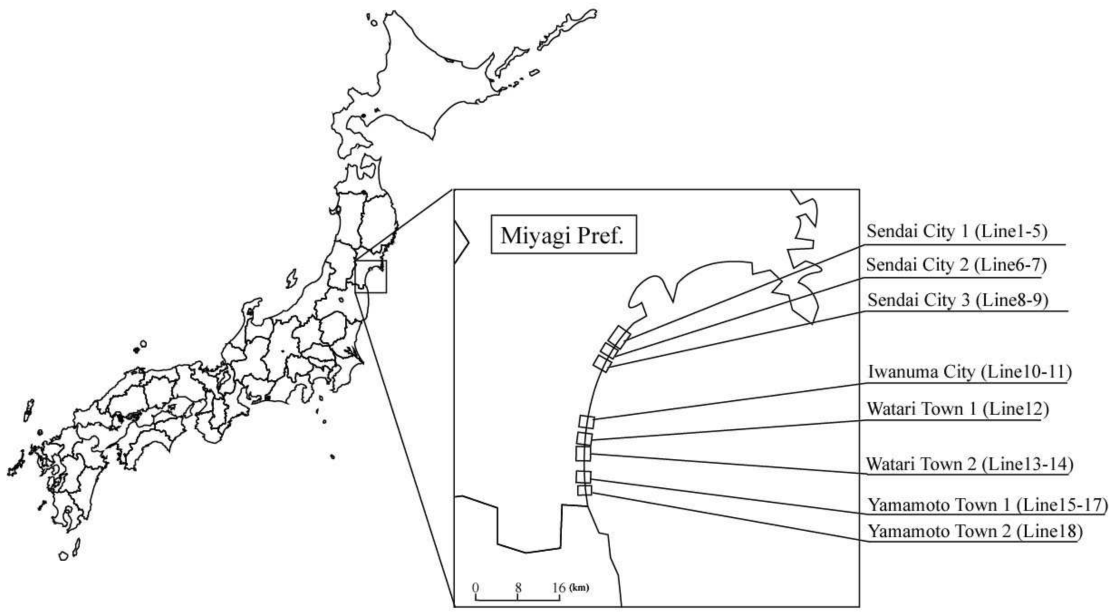


Figure.1 Location of investigation sites

For analysing the effect of coastal forest, sea embankment and sand dune, 18 locations were selected (**Table.1**). Damage situation of inland by the Japanese tsunami was very complex, since the sea embankment was breached in some area and coastal forest was mostly broken. Among 18 locations, one location (Wakabayashi District as shown in **Figure.2**) was selected considering that; 1) sea embankment existed and it was not washed out, 2) there were no inland embankment such as road embankment that affects tsunami inundation pattern greatly, and 3) the density of houses were not so large because washed out houses produced large quantity of debris and it affected the energy dissipation process and washout condition of houses. After validating the numerical model using the information of tsunami water depth measured at the post tsunami survey in many locations in this site, the effect of coastal forest, sea embankment and sand dune for reducing the washout region of houses were compared with each other.

Table.1 Information on the field investigation sites

Line No	City or town	Location (latitude, longitude) N	E	Length of washout region from coast (m)	Condition of embankment after the tsunami	Washout condition of coastal forest	Existence of houses near sea embankment or on the sand dune region	Existence of large building within the washed out region of houses
1	Sendai	38°15'20"	141°0'43"	2100	WE	WS	EE	not existed
2	Sendai	38°15'15"	141°0'39"	1400	NW	NF	EE	existed
3	Sendai	38°14'56"	141°0'39"	800	WE	NS	NH	existed
4	Sendai	38°14'33"	141°0'21"	1800	WE	NS	NH	not existed
5	Sendai	38°14'8"	141°0'1"	1400	WE	WO	NH	not existed
6	Sendai	38°13'2"	140°59'12"	1800	NW	NS	ES	not existed
7	Sendai	38°12'50"	140°59'3"	2000	NW	NS	ES	not existed
8	Sendai	38°12'24"	140°58'42"	1900	NW	NS	NH	not existed
9	Sendai	38°10'51"	140°57'50"	1900	NE	NF	ES	not existed
10	Iwanuma	38°4'42"	140°55'34"	500	NW	WS	NH	not existed
11	Iwanuma	38°4'22"	140°55'31"	800	NW	WS	NH	not existed
12	Watari	38°2'21"	140°55'19"	1200	WE	NT	ES	not existed
13	Watari	38°1'3"	140°55'6"	900	NW	WS	NH	not existed
14	Watari	38°0'1"	140°54'59"	1000	WE	WS	NH	not existed
15	Yamamoto	37°58'17"	140°54'55"	600	NW	WS	NH	not existed
16	Yamamoto	37°57'53"	140°54'58"	1200	NW	WS	NH	not existed
17	Yamamoto	37°57'32"	140°54'59"	1000	WE	WS	NH	not existed
18	Yamamoto	37°65'26"	140°55'12"	1600	WE	WS	NH	not existed

Note: WE: Washed-out embankment, NW: Broken but not washed-out embankment, NE: No embankment, ES: There was houses in sand dune region, EE: There was houses near the embankment, NH: There were no houses near the embankment and/or on sand dune, NS: Tree existed, but there were no scour region and trees were not wahed out, NF: There was no forest, NT: Scour region was genearted but there were no trees in the scour region, WS: Washed out from the scoured region, WO: Washed out not only by scouring but also by mainly overturning

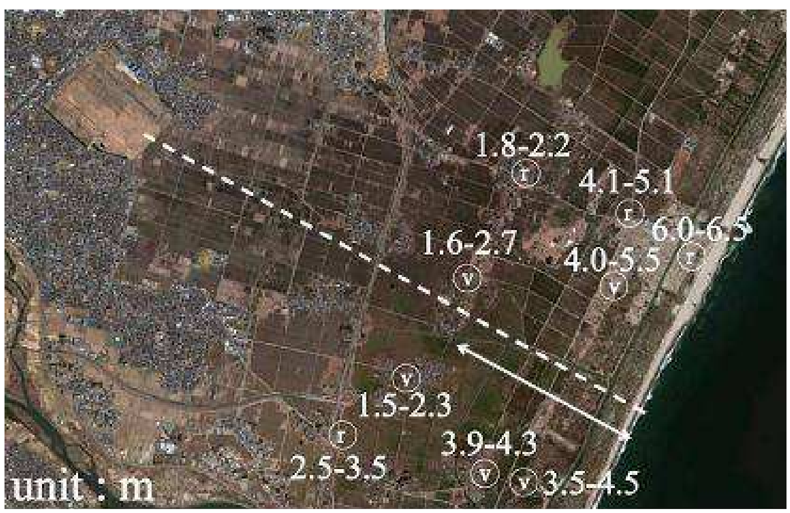


Figure.2 The location of analyzed area at Wakabayashi District , where numerical values show the observed tsunami water depth, *v* means the data for model validation, *r* means reference data, dotted line shows inundated area, and line with an arrow means the washout region of houses.

2.4. Numerical simulation

To elucidate the mitigating effect of a coastal forest quantitatively, numerical simulations were conducted using the model developed by Thuy et al.(2009) used, which is formulated by two-dimensional nonlinear long-wave equations (continuity equation: Eq.(1), momentum equations: Eqs.(2) and (3)) and an Sub-Depth Scale (SDS) turbulence model.

$$\frac{\partial \zeta}{\partial t} + \frac{\partial (hV_x)}{\partial x} + \frac{\partial (hV_y)}{\partial y} = 0 \tag{1}$$

$$\frac{\partial V_x}{\partial t} + V_x \frac{\partial V_x}{\partial x} + V_y \frac{\partial V_x}{\partial y} + g\frac{\partial \zeta}{\partial x} + \frac{\tau_{bx}}{\rho h} + \frac{F_x}{\rho h} - \frac{E_{Vx}}{h} = 0 \tag{2}$$

$$\frac{\partial V_y}{\partial t} + V_x \frac{\partial V_y}{\partial x} + V_y \frac{\partial V_y}{\partial y} + g\frac{\partial \zeta}{\partial y} + \frac{\tau_{by}}{\rho h} + \frac{F_y}{\rho h} - \frac{E_{Vy}}{h} = 0 \tag{3}$$

where

$$\vec{\tau}_b = \frac{\rho g n^2}{h^{1/3}} \vec{V}\left|\vec{V}\right| \tag{4}$$

$$\vec{F} = \gamma \frac{1}{2} \rho C_{D-all} b_{ref} \vec{V}\left|\vec{V}\right| h \tag{5}$$

$$E_{vx} = 2\frac{\partial}{\partial x}\left(h\nu_e \frac{\partial V_x}{\partial x}\right) + \frac{\partial}{\partial y}\left(h\nu_e \frac{\partial V_x}{\partial y} + h\nu_e \frac{\partial V_y}{\partial x}\right) \tag{5}$$

$$E_{vy} = 2\frac{\partial}{\partial y}\left(h\nu_e \frac{\partial V_y}{\partial y}\right) + \frac{\partial}{\partial x}\left(h\nu_e \frac{\partial V_x}{\partial y} + h\nu_e \frac{\partial V_y}{\partial x}\right) \tag{6}$$

x and y are the horizontal coordinates; V_x and V_y are the depth-averaged velocity components in x and y directions, respectively; t is the time; h the total water depth ($h = h_0+\zeta$); h_0 the local still water depth (on land, the negative height of the ground surface); ζ the water surface elevation; n the Manning roughness coefficient; and γ the tree density (number of trees/m^2). C_{D-all} is the depth-averaged equivalent drag coefficient considering the vertical stand structure of the trees, which was defined by Tanaka et al.(2007) as:

$$C_{D-all}(h) = C_{D-ref} \frac{1}{h}\int_0^h \frac{b(z_G)}{b_{ref}} \frac{C_D(z_G)}{C_{D-ref}}) dz_G \tag{7}$$

where $b(z_G)$ and $C_D(z_G)$ are the projected width and drag coefficient of a tree at height z_G from the ground surface, and b_{ref} and C_{D-ref} are the reference projected width and reference drag coefficient, respectively, of the trunk at z_G=1.2m in principle. The eddy viscosity coefficient v_e is expressed in the SDS turbulence model.

To clarify tree breaking, the models of Tanaka et al.(2010) and Thuy et al.(2011), which consider the breaking condition of tropical sand dune vegetation, were adapted to pine trees. Moment acting on the tree trunk at ground height (Eq.(8)) and critical bending moment of trees (Eq.(9)) are used for judging tree trunk bending as below.

$$M = F \times \frac{h}{2} = \frac{1}{2} C_{D-all} \rho u^2 h d_{BH} \frac{h}{2} = \frac{1}{4} C_{D-all} \rho u^2 h^2 d_{BH} \tag{8}$$

$$M_{bcri} = kD_{BH}{}^3 \tag{9}$$

where u (m/s) ($= \sqrt{V_x{}^2 + V_y{}^2}$) is the velocity; C_{D-all} drag coefficient (=1 before breaking, because the pine trees at the site doesn't have many branches); ρ (kg/m^3) density of fluid; h (m) tsunami water depth; d_{BH} and D_{BH} (=100d_{BH}) tree trunk diameter at breast height in m and cm unit, respectively; k dimensional constant (=2 or 3 for hard trunk and elastic trees, respectively (Tanaka & Yagisawa, 2009). When M is larger than M_{bcri}, the tree is judged to be bent down and the drag coefficient is changed from 1.0 to 0.2. Considering fluid force (F) and moment by drag force (M), the fluid force index (u^2h), and the moment index (u^2h^2) were defined. At Wakabayashi district, pine trees with 15cm in diameter and 9.6m in height were mostly bending. The threshold momentum index is around 180m^4/s^2.

A set of the model equations was solved by the finite-difference method of a staggered leap-frog scheme, which is used widely in numerical simulations of tsunamis. A sinusoidal incident tsunami was given as a time-dependent boundary condition at the most offshore side of the wave-generation zone. The initial conditions were given for a waveless state in the computational domain including the wave-generation zone. In the numerical simulation, a uniform grid size of 10m was applied considering available elevation data and CPU time. The Manning roughness coefficient n was set as 0.025s/m$^{1/3}$ for a relatively bare rough ground. Forest length and tree density was set as the condition of the site being evaluated.

A uniform coastal topography with a cross-shore section perpendicular (x-axis) to a straight shoreline, as shown in **Figure.3**, was selected as a model case. The density of houses here was low compared with other districts, so the effect of impact force by floating debris could be considered small in this area. The offshore water depth at an additional wave-generation zone with a horizontal bottom was 200m below the datum level of z = 0. The direction of the incident tsunami was perpendicular to the shoreline. In the present paper, the run-up of only the first wave is discussed. The width of the coastal forest was 640m, and it started at x = 80m from the shoreline. The forest was assumed to extend finitely in the direction of the shoreline (y-axis). The maximum tsunami water depth at the shoreline was set at 10m, which is the average value in this area.

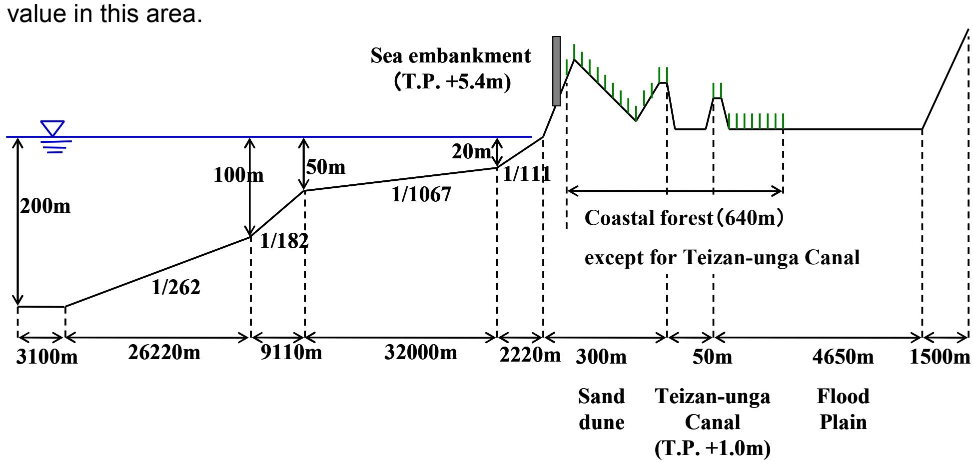


Figure.3 Schematic of topography for numerical simulation area

3. RESULTS AND DISCUSSION

3.1. Verification of numerical simulation including vegetation breaking model

The Japanese tsunami broke coastal forest for long distance, so the validation of not only tsunami water depth but also the breaking length of the coastal forest by the tsunami is also required. Vegetation can reduce the velocity, water depth inside a forest, and timing of the tsunami front arrival.

Figure.4 shows a comparison of the numerical simulation data with the observed tsunami characteristic (maximum water depth). In this figure, three simulation results are shown; using a non-breaking model in which C_d is set at the before-the-tsunami condition of 1.0), a non-breaking model (in which C_d is estimated in the after-the-tsunami condition by the ratio of broken trees to all trees of 0.24), and a breaking model (in which C_d is changed with time: when a tree is judged 'broken', the value is reduced from 1.0 to 0.2). From the results, the breaking model can be seen to reflect the tsunami water marks well. The breaking length of coastal forest by the simulation was 420m and it was a similar value with the actual length. Thus, the effectiveness of our model was validated for this actual tsunami.

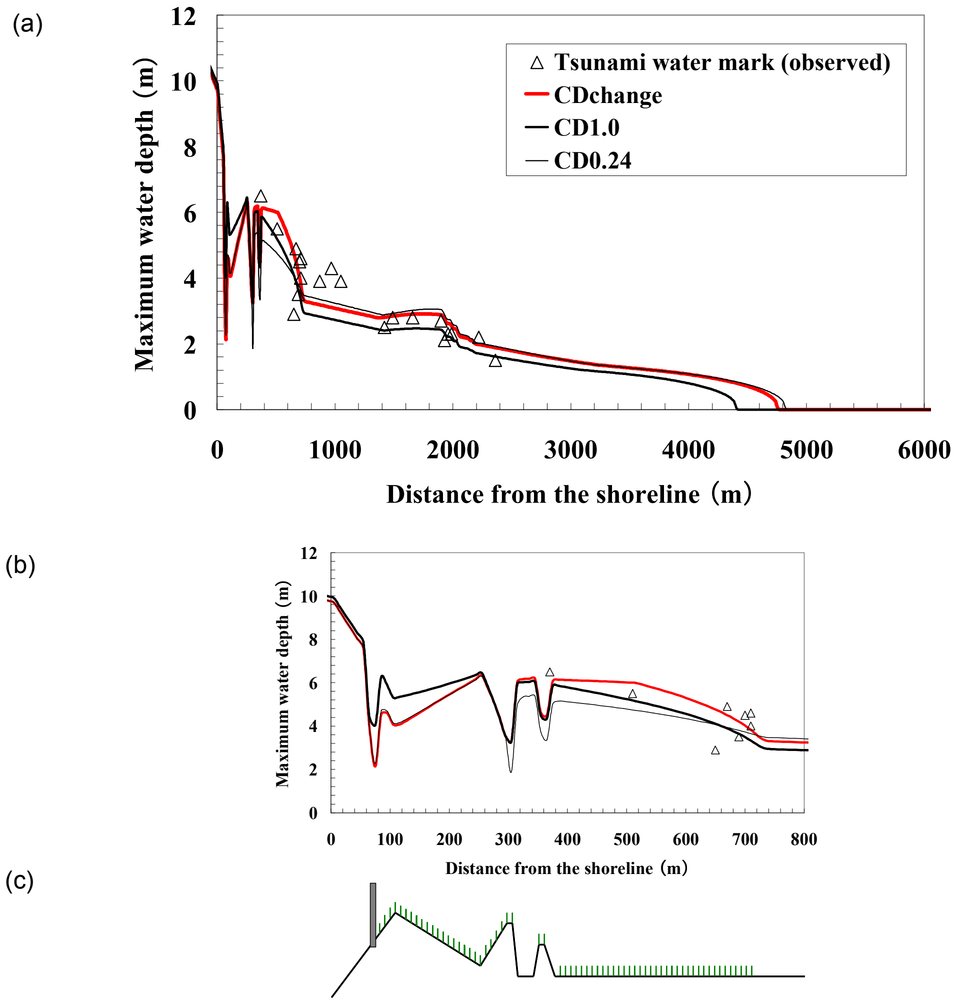
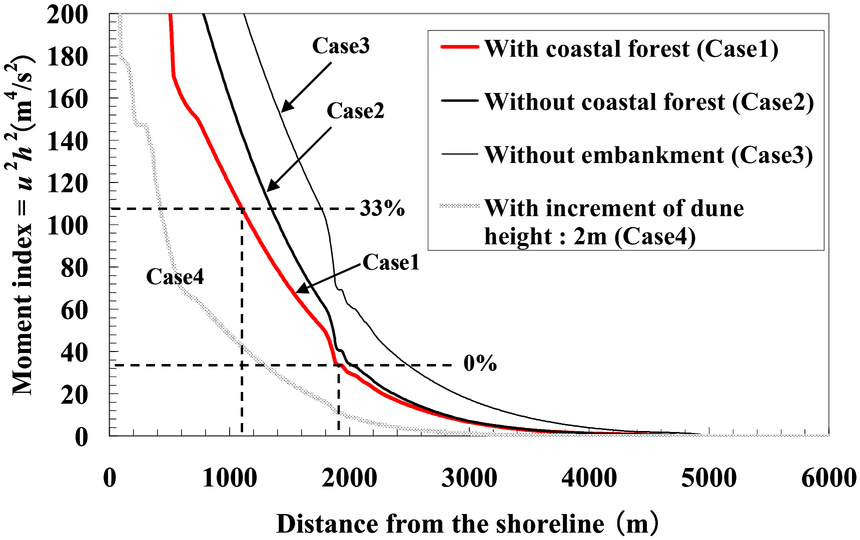


Figure.4 Comparison with calculated maximum water depth and observed values
(a) Validation of maximum water depth for three models. CD change: with breaking model, CD1.0: $C_{D\text{-}all}$ is a constant of 1.0 (before the tsunami), CD0.24: $C_{D\text{-}all}$ is a constant of 0.24 (after the tsunami), (b) the graph which enlarged the range of 0-800 m in Figure.4(a), (c) topographical shape corresponding to the range of Figure.4 (b).

3.2. Effectiveness of coastal vegetation in reducing fluid force.

Figure.5a and **b** show the fluid force index (u^2h, where u and h are the tsunami velocity and water depth, respectively) and the momentum index (u^2h^2) for the present case (with vegetation and an embankment: Case 1), without vegetation (Case 2), without an embankment (Case 3), and with higher sand dune (in this case sand dune height is increased 2m: Case 4). The critical value of the moment index (M_{cr}) was around 76m^4/s^2 according to a real-scale experiment (Takahashi et al.(1985)). For the fluid force index (F_{cr}), the study by Hatori (1985) showed that most houses were washed out when F_{cr} exceeded 100m^3/s^2, and one-third of houses were lost when F_{cr} was around 15m^3/s^2. Based on the washout situation observed at the present study site and the simulation result, the M_{cr} values for 33% and 0% washout were 109m^4/s^2 and 34m^4/s^2, respectively. This is similar to the value that Takahashi et al. reported in 1985. Moreover, F_{cr} values for 33% and 0% washout values were 41m^3/s^2 and 18 m^3/s^2, respectively, which is slightly larger than the figures of Hatori (1985). Based on a comparison of Cases 1 and 2, the vegetation could be assumed to decrease the washout region by around 110m. On the other hand, an embankment could reduce the washout region by 590m. In addition, the comparison of Cases 1 and 4 shows that the sand dune (2m height increment) could be assumed to decrease the washout region of forest around 420m and thus decreased the washout region of houses by around 590m. From the results, the vegetation effect is not large in comparison with the effect of the embankment or sand dune, but it is not negligible either.

(a)



(b)

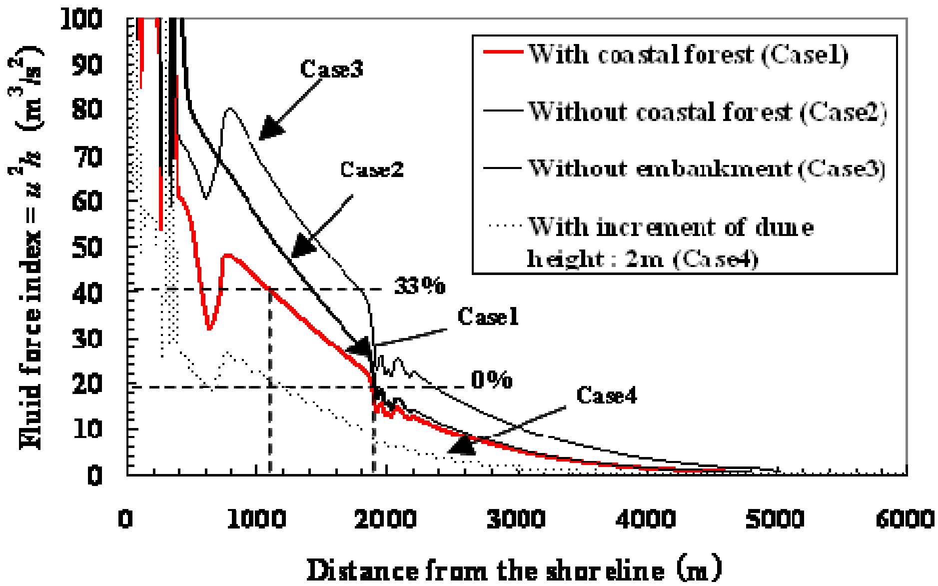


Figure 5. Differences between cases with/without coastal forest and embankment, and sand dune height (Case 1: with forest (present case), Case 2: without vegetation, Case 3: without an embankment, Case 4: with higher sand dune (in this case sand dune height is increased 2m): (a) moment index. 0% and 33% mean percentage of houses washed out at the location. Figure.5 (b) fluid force index.

4. CONCLUSION

Tree damage is directly related to the tsunami force, but the effects of tree breaking on numerical simulation results were not directly discussed in previous studies except for a study we published recently. We validated our numerical model with field measurement data on the threshold water depth for tree breaking, then breaking length, and finally reduction of water depth. This study demonstrated that the breaking phenomenon decreases the effect of vegetation, but it also has some role in reducing the fluid

force and moment by drag force when trees are not washed out. Therefore, construction of a bioshield in an appropriate area (a region that will not be scoured) is very useful for mitigating the disaster caused by an extremely large tsunami that overflows or destroys an embankment.
A numerical simulation estimated the effects of a coastal forest and embankment on reducing the washout region of houses by around 100m and 600m, respectively. The effect of vegetation is small compared to that of the embankment, but it is not negligible in the mitigation when a large tsunami arrives and the sea embankment overflows.

5. ACKNOWLEDGMENTS

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Characteristics of municipal solid wastes from selected plots located at up-slope and down-slope of the dumping site: A case study in Udapalatha/Gampola

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Abstract: *Open dumpsite has become the most widely used and common practices in most of the developing countries in waste management due to their lack of will, technology, capital and virtuous attitudes. This has elevated the risk to the ecosystem and humans derived by the improper dumping of waste in bare lands, sloping lands (valleys, river banks), wet lands etc. Characterization of waste is an important primary stage in studying dumpsites in sloping lands to identify its effects and potential remedial actions, and to develop new technologies to reduce their adverse effects on the environment and society. In this research, waste characteristics of some selected plots located at up and down slope of a dumpsite were studied to identify the possible difference and their relationships with the slope. The results revealed that waste characteristics such as ash content, combustible content, ignition loss and aggregated soil content vary in the plots located at up and down slope, suggesting that the rolling and sliding of larger particles in high sloping dumpsites has a profound effect in varying waste characteristics. This was further confirmed by the particle size distribution of both plots.*

Keywords: *Waste characteristics, sliding and rolling of waste, up and down slope, waste characteristics, open dumpsites.*

1. INTRODUCTION

In developing countries municipal solid waste management has become a serious issue due to their inability to manage their waste in an environmentally and human friendly manner so far. As a result the implementation of improved land disposal practices is succeeding at varying rates depending upon the available resources and national regulatory standards for municipal solid waste management. The need to improve land disposal practices is being forced along by highly concentrated populations, where rural residents are moving to cities resulting in rapid urban population growth which has created an increasing demand for the development of new technologies for better waste management and disposal methods. However, when the developing countries cannot afford such an advanced and new methods to mitigate the environmental and social effects, illegal open dumping dominates the scenario.

Open dumpsites, being the most common and immediate solution to dispose the waste generated by the municipalities of the developing countries induced by their lack of will, technology, capital and virtuous attitudes to perceive waste as resource, has been unable to diminish the environmental and social issues caused by the discarded waste. The opposition from the general public for the naissance created by the open dumpsites, has forced the administrative parties to localize the dumpsites in remote areas where less inhabited by humans. This has lead the dumpsites to be created in remote areas generally where the population is less and thus generated many environmental issues in these areas. Hence, most of the waste generated by the developing countries is being openly dumped (World bank, 2011) in bare lands, valleys, river banks, wet lands etc. threatening the ecosystem to be degraded in an increasing rate.

Characterization of waste is the process of determining the chemical and physical characteristics of waste which is an important primary stage of studying dumpsites to identify their effects and potential remedial actions and to develop new technologies to reduce their adverse effects on the environment and society. Characteristics of waste in dumpsites depend on various aspects including its geophysical nature and environmental conditions. Slope is an important geophysical factor in dumpsites which has a profound effect on waste characteristics. As per Johannessen and Boyer (1999) most of the dumpsites located in sloping lands and the hypothesis of this study is that the up and down slope waste characteristics could be different in such dumping in sloping lands.

So that study of the variation of the characteristics of waste dumped in sloping lands is of great significance in identifying the important physical and chemical processes occurring in the dumpsites which ultimately causes the environmental degradation. The objective of this study was to identify the possible variation of waste characteristics of up slope and down slope in open dumping in slopping land.

2. METHODOLOGY

2.1. Site selection

The selected dumpsite was located near the right bank of Mahaweli river where a steep slope of 36 % exists in Udapalatha Pradeshiya Sabha (PS) in the central province of Sri Lanka which has been used by both Gampola Urban council and Udapalatha PS for around seven years to dump their waste and it has been abandoned six months earlier before waste sampling on December 2011, so that dumpsite conditions do not change during the study period. This dumpsite is located in the wet zone of Sri Lanka where the average annual rainfall is above 2000 mm with an average annual temperature of 24.7 ^{0}C (Statistical Abstract, 2010).

2.2. Sampling

Samples were obtained from easily accessible two plots from the middle transect of this abandoned site to represent the sloping topography. One sample was taken from the surface of up slope (Top - N 07^{0} 08' 34.2'', E 80^{0} 34' 42.1'') and the other from the surface of down slope (Bottom - N 07^{0} 08' 34.4'', E 80^{0} 34' 40.9'') (Figure 1). The distance between the sampling points was 50 m with an elevation difference of 18 m accounting for 36 % of slope. The samples were collected into air tight polythene bags with an air trap and brought to the laboratory for the analysis.

2.3. Laboratory analysis

The samples were immediately prepared for the drying by mixing the sample on the floor with a shovel followed by composite sampling (Figure 2) and then a part of the sample containing approximately 5-10 kg, was selected for drying at 110 ^{0}C for 48 hours.

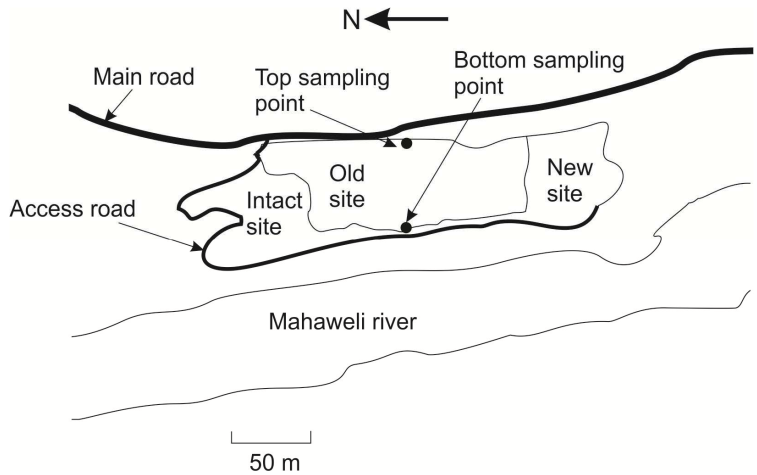

Figure 1 Sampling points of the Udapalatha/Gampola dumpsite

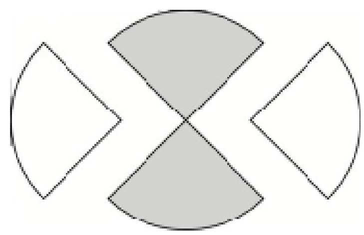

Figure 2 Conceptual diagram of divided sample into four segments

Bulk density and the moisture content were determined. The dried samples were then categorized into different waste types such as Kitchen waste, Paper, Hard plastic, Soft plastic, Metal, Glass, Ceramic, Leather & Rubber, Textile, Grass & Wood, Rock, Cemented material, Aggregated soil and Others to obtain the composition of the waste followed by the determination of ash content of each category by ignition at 800 ºC for two hours. Particle density of the samples was obtained according to the JIS A 1202 Japanese standards (Equivalent to ASTM D854-10) for the dried samples followed by the particle size distribution (JIS A 1204; Equivalent to ASTM D422-63). Raw parts of the samples were subjected to the analysis of pH, EC and liquid limit & plastic limit test according to the JIS A 1205 (Equivalent to ASTM D4318-10). All these parameters were selected to identify all the possible differences due to geophysical conditions and the processes.

After the determination of ash content of the sample, combustible content (C) was calculated as follows:

$$C = 100 - M_w - A_{wet} \quad (1)$$

where M_w is moisture content (%) and A_{wet} is the ash content (%) wet basis. Lower calorific values were calculated by proximity analysis as shown in Eq. 2 and 3 as explained by Watanabe (2000) and the reference values were obtained from a previous JICA report (2003):

$$H_{total} = \sum ((A(i) + V(i)) / (W \times H(i)) \quad (2)$$

$$H_{available} = H_{total} + 0.6 \times (\sum (M_{air}(i) + M_{ult}(i)) + 18/2 \text{ (hydrogen)}) / W \quad (3)$$

where A(i) is the ash content of i^{th} waste component, V(i) is the volatile content of i^{th} waste component, H(i) is the heat value of i^{th} waste component, $M_{air}(i)$ is the air dryable moisture content of i^{th} waste component, $M_{ult}(i)$ is ultimately dryable moisture content of i^{th} waste component and W is the total weight. Ignition loss (L_i) is calculated as follows:

$$L_i = 100 - A_{<2mm} \quad (4)$$

where $A_{<2mm}$ is the ash content (%) of the particles of size less than 2 mm diameter.

3. RESULTS AND DISCUSSION

Variation of the measured parameters of the two sampling points is shown in Table 1 which can be used to compare the parameters between the two plots to identify the effect of slope on waste characteristics.

As shown in table 1, most of the parameters were same for both plots located at up and down slope except ash content, combustible content, ignition loss, pH and aggregated soil content. Ash content is higher in the plot at down slope and so that combustible content is lower in the plot at down slope than that of the plot at up slope as these two parameters are inversely related. Ignition loss is lower in the plot at down slope and this may be due to the elevated ash content in the plot at down slope. When considering the waste composition, aggregated soil percentage is higher in the plot at down slope than in the plot at up slope (Figure 3).

Table 1 Comparison of data between up and down slopes

Parameter	Up slope	Down slope
Moisture content (%)	26	26
Ash content (%)	199	622
Combustible content (%)	577	14
Unit volume mass (kg m^{-3})	5.71	6.00
Lower heating value (kcal kg^{-1})	739	613
Particle density of < 10 mm (g cm^{-3})	2.61	2.59
Particle density of < 2 mm (g cm^{-3})	2.31	2.29
pH	12.8	7.9
EC (μS cm^{-1})	267	224
Ignition loss (%)	90	7
Liquid limit (%)	58.0	42.1
Plastic limit (%)	57.6	41.8
Plasticity index	0.4	0.3

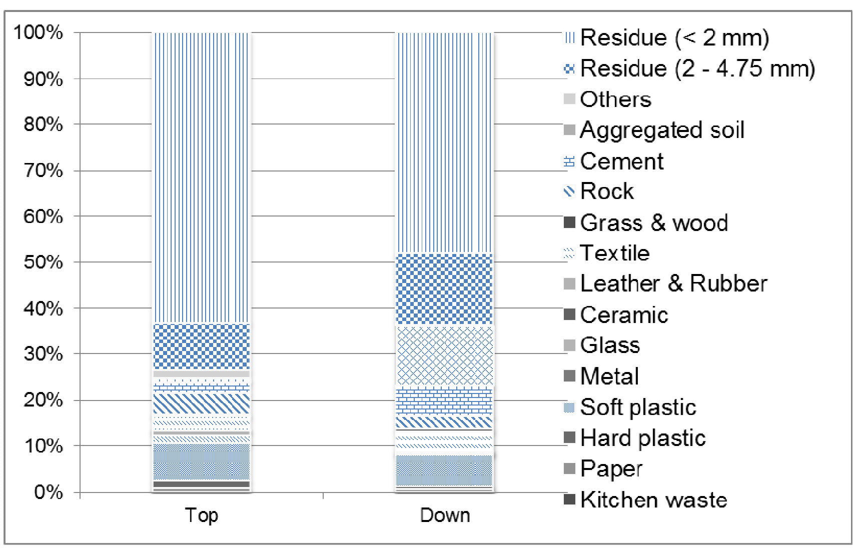

Figure 3 waste composition of up and down slope

The particle size distribution (Figure 4) of these samples clearly shows a difference of distribution of both of the plots compared to that of the intact soil. The plot at down slope has more larger particles compared to the plot at up slope within the range of 1 – 10 mm of particle size where more smaller particles are retained in the plot at up slope. This situation can be further explained as a result of settling of waste induced by three main phenomenon i.e. rolling, sliding and erosion (Figure 5).

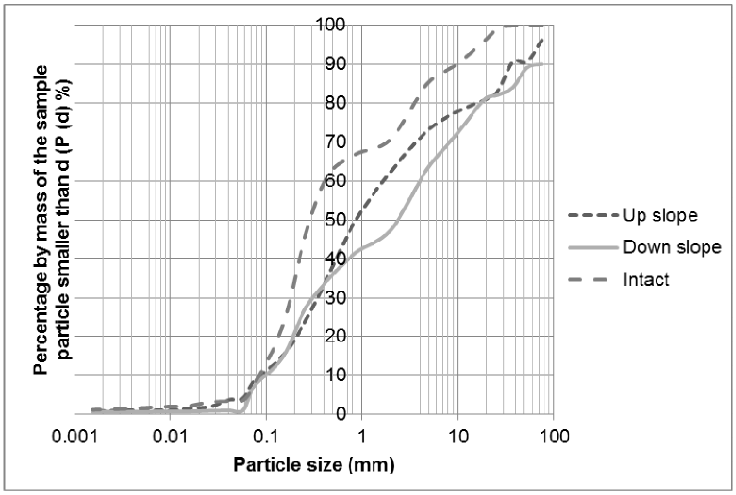

Figure 4 Particle size distribution of up and down slopes

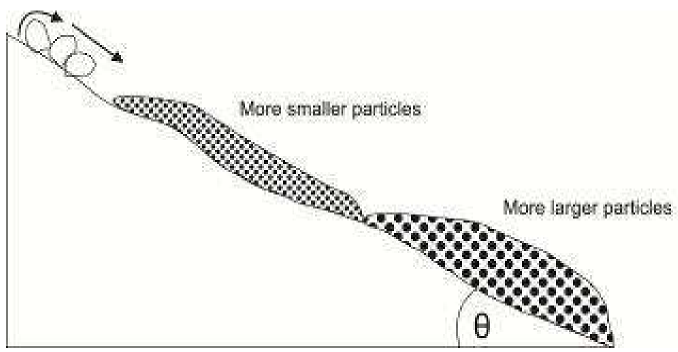

Figure 5 Conceptual diagram of sliding and rolling of waste leading to settle larger particles in down slope

According to Xu *et al.* (2008) the peak friction angle of waste is 18.6^o and this is the maximum angle in which waste can be retained without rolling and sliding. As the slope of the studied dumpsite is 36 % (θ = 19.8^o) which is larger than the peak friction angle, waste is subjected to rolling, sliding and erosion which ultimately results in settlement of larger particle in down slope and smaller particles retained in upslope (Figure 5). The particle size distribution of up and down slopes clearly suggest this settlement of larger particles in the down slope as the larger particles are higher in down slope and so that it is obvious that the rolling and sliding have become significant than the erosion in this site otherwise more smaller particles would have been eroded and settled in the down slope.

4. CONCLUSIONS

Characteristics of waste in some selected plots located at up slope and down slope are different in ash content, combustible content, ignition loss and aggregated soil content. This is mainly due to accumulation of larger particles including aggregated soil in down slope as a result of rolling, sliding and erosion of waste dumped in up slope. Particle size distribution of these locations clearly shows the rolling and sliding of larger particles in high sloping dumpsites, where the slope is higher than the peak friction angle of 18.6^0. This will be helpful to properly select sampling locations in dumpsites for characterisation studies.

5. ACKNOWLEDGMENTS

This work was supported by the SATREPS project funded by Japanese International Cooperation Agency and Japan Science and Technology Agency. The authors thank all the staff members of the project and the academic and academic support staffs of the Environmental laboratory and Geotechnical laboratory of the Faculty of Engineering, University of Peradeniya, Sri Lanka for their support in the laboratory analysis and specially Ms. B.M.R.S. Balasooriya who gave immense support in laboratory analysis of the samples.

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Landfill Gases at an Abandoned Open Dump: A Case Study at Udapalatha/Gampola Site in the Central Province of Sri Lanka

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Abstract: *There are very limited studies on landfill gas on uncontrolled open dumps in developing countries. In this study, landfill gas samples at 1 m depth from an abandoned open dump (AOD) in the Central Province of Sri Lanka (N 7º 09', E 80º 35') were collected and the typical landfill gas composition such as O_2, N_2, CH_4, CO_2, H_2, H_2S, and N_2O were measured. Buried waste samples at 1 m depth were also taken from the site and organic carbon and nitrogen contents in the residue (< 2 mm) were measured. The samples were taken from some marked plots inside the dump with waste ages of around 0.5 and 7 years ($AOD_{0.5}$ and AOD_7) and outside intact (AOD_{int}).*

Measured CH_4 concentration for $AOD_{0.5}$ and AOD_7 ranged in 19–58 % and 0–12 %, respectively, suggesting that the dumped waste at 1 m depth was in the process to be the 'stabilization phase' at least 7 years after dumping. This is likely to be a much shorter time period to reach the phase after dumping than those in mid-latitude regions (typically in several decades). The carbon contents in the waste residue in $AOD_{0.5}$ and AOD_7 were 151±67 and 29±7 mg g^{-1}, respectively, implying that high waste decomposition and leaching of organic compounds might have been enhanced due to high temperature and precipitation at the site. A further study for the landfill gas and waste quality in the deeper layer is required to judge whether whole of the dumpsite had reached the stabilization phase rapidly.

Keywords: *Landfill gas, nutrient leaching, open dump, organic carbon, Sri Lanka*

1. INTRODUCTION

Haphazard dumping of the municipal solid waste is mostly observed in developing countries, where the waste is dumped in an uncontrolled manner. Such inadequate waste disposal creates serious environmental burden which affects health of humans and animals and causes serious economic and other welfare losses (Zurbrügg, 2003).

Landfill gas is an important factor which causes odor and firing and provides us information about the stability of the waste in a landfill. Figure 3 shows a conceptual model for variation in landfill gas composition over time (Rees, 1980). There are five phases of waste condition in a landfill, mentioned as follows: (I) Aerobic biodegradation phase: the organic matter in the waste is aerobically degraded by oxygen (a few hours), (II) Anaerobic acid fermentation phase: volatile organic acid, CO_2, and H_2 are generated by anaerobic decomposition of biodegradable organic matter in the waste (a few months – a few years), (III) Methanogenic phase I: the volatile organic matter changes into CH_4 and CO_2 (a few months – a few years), (IV) Mehanogenic phase II: CH_4 is generated by anaerobic decomposition of refractory organic matter in the waste and ranges from 45 to 60 % of the landfill gas (several decades), and (V) Stabilization phase: the organic matter in the waste decrease and N_2 and O_2 concentrations in the landfill gas increase due to the invasion of the air into the waste (more than 100 years).

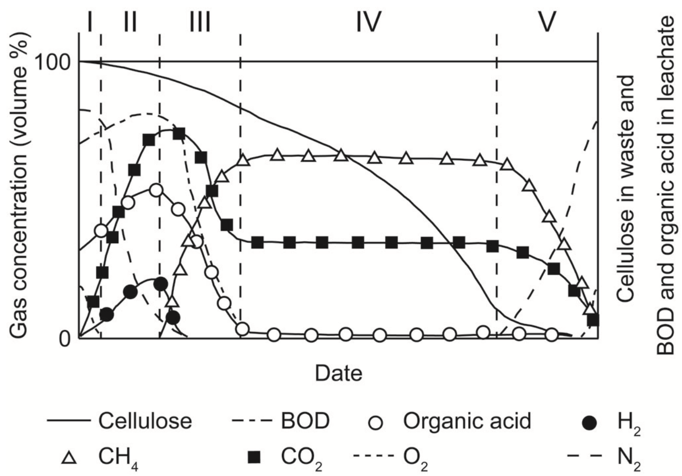

Figure 3 Conceptual model for variation over time in landfill gas composition (Rees, 1980, modified)

Despite the importance to observe landfill gas, there are very few studies on the uncontrolled landfills. The objective of this study is to observe typical landfill gas composition in an abandoned open dump which has different ages of dumped waste inside to evaluate time-dependent change in it. For comparison, landfill gas samples were also taken from another open dump and an engineered landfill those are under operation, in the same province.

2. MATERIALS AND METHODS

2.1. Site description

The present study was conducted between 28th November and 2nd December 2011 in the Central Province of Sri Lanka. Three different types of waste disposal sites were selected as study sites (Figure). An intensive observation was operated in an abandoned open dump (AOD) located at Udapalatha Pradeshiya Sabha (N 7º 09', E 80º 35'). The average annual rainfall is above 2000 mm with an average annual temperature of 24.7 ºC (Statistical Abstract, 2010).The samples were taken from three marked plots at regular intervals on transects along the slope (Top, Middle, and Bottom) each inside the dump with waste ages of around 0.5 and 7 years ($AOD_{0.5}$ and AOD_7) and outside intact (AOD_{int}). The samples were also taken from another open dump, Gohagoda (OOD; N 7º 19', E 80º 37') and an engineered landfill in Nuwara-Eliya (OEL; N 6º 58', E 80º 48'), those are under operation.

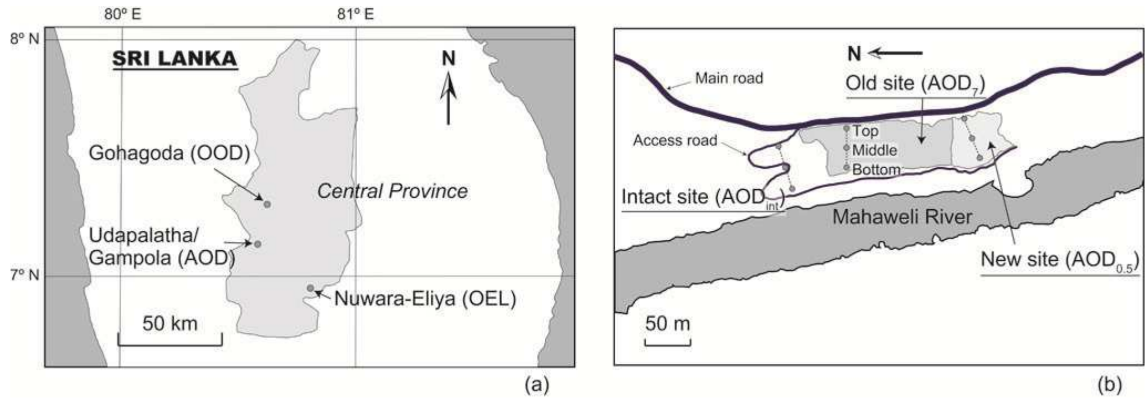

Figure 2 (a) Location of the sites investigated, (b) Map of Udapalatha/Gampola site (AOD). The slope faces into west in this site.

2.2. Sample analyses

Either perforated PVC tubes, 13.2 mm in diameter, or metal tubes, 4.1 mm in diameter, were installed at depths of 100 cm (sometimes it was shallower than 100 cm when it was failed to be installed at the depth) in each plot 30 minutes before the sampling to collect air samples from the dumped waste. The tubes were sealed by three-way cocks to allow the gas concentration in the tubes to equilibrate with the landfill gas. From each of the tubes installed, air samples (500 mL) were taken into an air-tight aluminium bag. The samples were transferred to the laboratory in Center for Environmental Science in Saitama, Japan to analyze the concentrations of typical landfill gases such as O_2, N_2, CH_4, CO_2, H_2, H_2S, and N_2O. O_2, N_2, CH_4, CO_2, and H_2 concentration in the gas samples were analysed using gas chromatography (GC) equipped with Thermal Conductivity Detector (GC-14A, Shimadzu, Kyoto, Japan and 6890 series, Agilent, Santa Clara, USA). H_2S was analysed using GC equipped with Flame Photometric Detector (5890 series II, Agilent, Santa Clara, USA). N_2O was analysed using GC equipped with Electron Capture Detector (Shimadzu 14B, Shimadzu, Kyoto, Japan).

Waste samples were collected from 100 cm depth (if it’s not possible, shallower than 100 cm) in each plot. The samples were dried at 110 ºC for 48 hours and sieved by 2 mm sized sieves to get the residue in the Laboratory in University of Peradeniya, Sri Lanka. The residue samples were transferred to the laboratory in Saitama University, Japan to analyze the carbon and nitrogen contents in it (MT-5, Yanaco, Kyoto, Japan).

Two-way ANOVAs were carried out to judge the significance of difference in CH_4 concentrations in the landfill gas and carbon contents in the waste residue obtained from each waste age and slope position in AOD. The statistical analyses were conducted by R (ver. 2.11.1; R Development Core Team).

3. RESULTS

The measured typical landfill gas concentrations for $AOD_{0.5}$, AOD_7, AOD_{int}, OOD, and OEL were presented in Figure . Measured O_2 and N_2 concentration in the landfill gas ranged in lower values at $AOD_{0.5}$ (2-4 % and 6-58 %, respectively) and higher values at AOD_7 and AOD_{int} (3-18 % and 65-79 %, respectively). Measured O_2 and N_2 concentration in the landfill gas for OOD and OEL ranged in similar low value as $AOD_{0.5}$ (2-3 % and 6-16 %, respectively).

Measured CH_4 and CO_2 concentration in the landfill gas ranged in higher values at $AOD_{0.5}$ (19-58 % and 19-35 %, respectively) and lower values at AOD_7 and AOD_{int} (0-13 % and 3-20 %, respectively). Measured CH_4 and CO_2 concentration in the landfill gas for OOD and OEL ranged in similar high value as $AOD_{0.5}$ (56-57 % and 24-36 %, respectively).

Measured H_2 concentration in the landfill gas was very low in each plot (0.00-0.02 %).

Measured H_2S concentration in the landfill gas were high (23-41 ppmv) at a few of the points which high concentrations of CH_4 and CO_2 while they were almost zero at the other points in AOD site. Measured H_2S concentration in the landfill gas for OOD and OEL ranged in high values of 4-24 ppmv.

Measured N_2O concentration in the landfill gas for the top and middle point in AOD_7 ranged in relatively high value (39-95 ppmv) while those in the other plots ranged in low (0-4 ppmv).

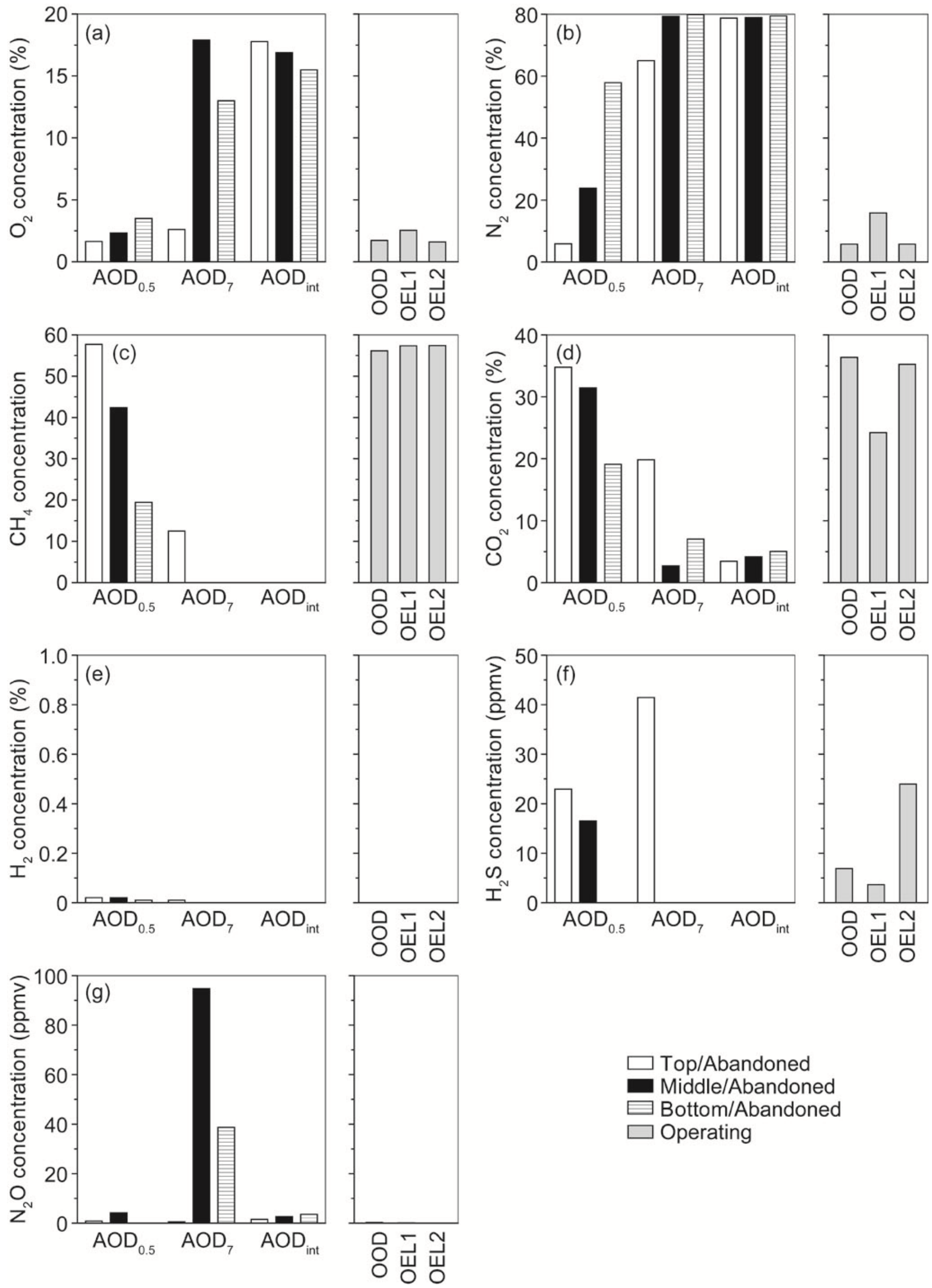

Figure 3 (a) O_2, (b) N_2, (c) CH_4, (d) CO_2, (e) H_2, (f) H_2S, (g) N_2O concentration in the landfill gas in AOD, OOD, and OEL sites.

Measured carbon and nitrogen content in the waste residue ranged in higher values at $AOD_{0.5}$ (93-224 and 8-14 mg g^{-1}, respectively; Figure) and lower values at AOD_7 and AOD_{int} (5-37 and 0-3 mg g^{-1}, respectively). Measured carbon and nitrogen content in the waste residue for OOD and OEL ranged in 30-79 and 3-7 mg g^{-1}, respectively. Calculated CN ratio in the waste residue ranged in 6-16 and there was no site specific tendency.

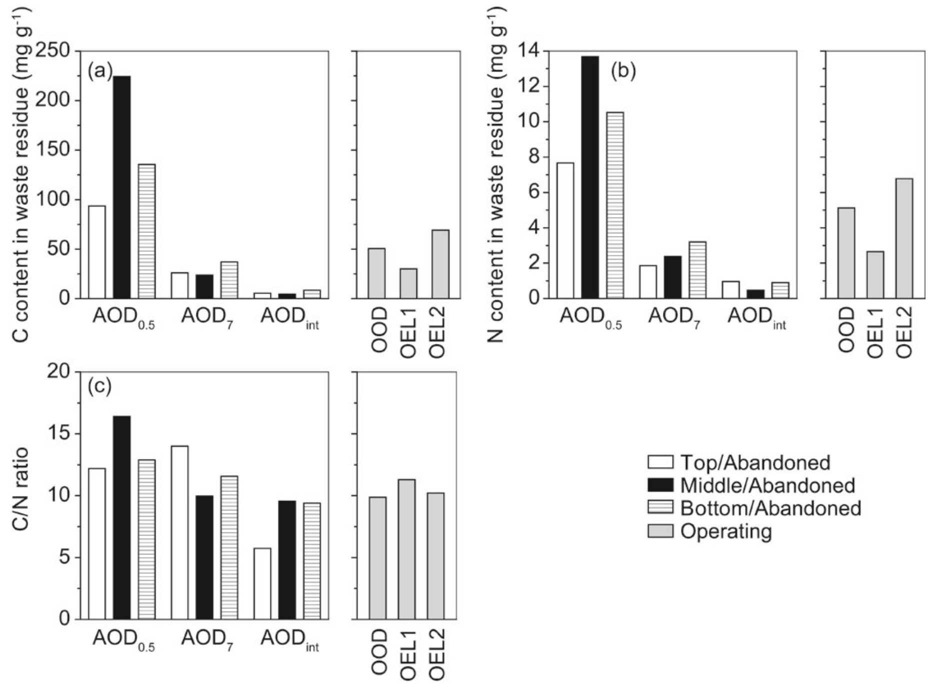

Figure 4 (a) Carbon and (b) nitrogen content and (c) C/N ratio of the residue of waste in AOD, OOD, and OEL sites.

4. DISCUSSION

The 2-way ANOVA for CH_4 concentration in the landfill gas revealed that there is a significant variation in the CH_4 concentration among the sites those have different age of waste ($P < 0.02$; Table 1), indicating temporal-dependent decrease of it (Figure c). The CH_4 concentrations of the landfill gas for AOD_7, at which seven years old of waste is buried, were lower than theoretical CH_4 concentration in 'Methanogenic phase II', which is said to continue for several decades (Rees, 1980). This would be due to high waste decomposition and leaching rate of organic compounds enhanced by high temperature and precipitation at the site ($P < 0.02$; Figure a, Table). Additionally, high concentration of O_2 and N_2 in the landfill gas at AOD_7 (Figure a, b) implies invasion of the air into the waste. The CH_4 concentrations in the landfill gas were high and the N_2 and O_2 concentration were low in the other waste disposal sites under operation (OOD and OEL; Figure a, b, c), indicating CH_4 was continuously generated using flesh organic matters in anaerobic condition in those sites. These indicates that the buried waste at 1 m depth in AOD_7 had already been in lack of organic matter that could be changed to CH_4 and had reached 'stabilization phase' nevertheless it had passed only 7 years since the stopage of dumping. However, we investigated only the surface layer of the open dump, where the oxidization is easier than that in deeper layers. A further study for the landfill gas and waste quality in the deeper layer is required to judge whether whole of the dumpsite had reached the stabilization phase rapidly.

The effect of slope position on CH_4 concentration in the landfill gas was not significant (Table 1). However, there is a tendency that higher CH_4 concentration in the landfill gas was observed at upper slope (Figure c). Lohila *et al.*, (2007) suggested that the higher emissions from the summit area are due to the uncovered surface and the daily deposited new waste. Although the studied site has no difference in cover treatment between the summit area and slope area and no daily deposited new waste, the waste buried in deeper layers may act some role which effect on the above phenomenon. A further investigation in the deeper layer will help us obtain valuable information on it.

Table 1 The ANOVA Table for CH_4 concentration in the landfill gas in AOD.

	SS	d.f.	MS	F value	P value
Waste age	36455	2	18228	11.56	0.02
Slope position	2728	2	1364	0.87	0.49
e	6305	4	1576	-	-
Total	45489	8	5686	-	-

Table 2 The ANOVA Table for carbon contents in the waste residue in AOD.

	SS	d.f.	MS	F value	P value
Waste age	2877	2	1439	13.90	0.02
Slope position	430	2	215	2.08	0.24
e	414	4	103	-	-
Total	3721	8	465	-	-

Besides, relatively high values of N_2O concentration were observed in some plots at AOD_7 (Figure g), suggesting that nitrification was stimulated due to time-dependent aerobic conditioning in the 1 m depth (measured O_2 concentration for the plots ranged in 13–18%). This indicates that nutrient leaching through runoff and surface water might give an impact to groundwater environment at open dump sites even in the 'stabilization phase'. Long term observation would be required to assess the environmental impacts of an open dump.

5. CONCLUSION

Observation for the typical landfill gas composition in an abandoned open dump located at the Central Province, Sri Lanka reveals that the surface layer of the dumped waste reached 'stabilized phase' only seven years after the disposal. A further investigation in deeper layer of the dump site is required to judge whether whole of the dumpsite had reached the stabilization phase rapidly. Furthermore, long term observation on nutrient leaching from an open dump is required even after reaching 'stabilized phase' because invasion of the air will enhance nitrification inside of the waste.

6. ACKNOWLEDGMENTS

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Section IV-Sustainable Built Environment

Reconstruction vs Retrofitting of a Bridge for Sustainability

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Abstract: *Bridges face the risk of being damaged by natural and manmade disasters. Old bridges are more vulnerable in such situations. The common practice is removing these damaged bridges and constructing new ones. However, repairing, retrofitting and reusing damaged bridges could be economical and less time consuming and hence more sustainable than building new bridges. There are various methods to assess the possibility of improving old and / or damaged bridges using modern day techniques. This paper is a case study for using one of such assessment procedures on a damaged railway bridge in Puttalam, Sri Lanka. The bridge concerned is 34m long, single spanned, double lattice girded, wrought iron Railway Bridge, which was built about 40 years ago and damaged and displaced from its abutments by floods. The paper discusses the method used to determine the possibility of reusing the bridge by conducting a series of tests on the temporarily erected bridge and using a finite element model. It also presents results of tests carried out after the bridge was repaired and retrofitted. The results show that retrofitting has made substantial improvements to the bridge.*

Keywords: *truss bridge, condition assessment, finite element model, validation, fatigue life, retrofitting*

1. INTRODUCTION

Bridges constructed over waterways and the sea face the risk of being damaged by floods and water waves. Since flooding has become more severe and common due to climate change etc., the risk of damages to bridges too has increased. Therefore, finding methods to repair and reuse bridges damaged by floods has become imperative.

There are methods to assess the possibility of improving damaged bridges and these methods mainly depend on the future fatigue life of bridge elements. Most widely used code of practice, BS 5400: Part 10: 1980 is one of the sources for the procedure of fatigue life assessment. However, this code has been prepared for the UK and with the use of statistics of roads and railways of the UK. Therefore using such codes without due care and modifications for local conditions may provide misleading results. There are other methods such as the method introduced by *Siriwardane et al* (2009). The principal behind this new method is, predicting past stress histories of critical bridge elements using present day measured strains and applying damage indicator based sequential law to estimate the fatigue life. The fatigue life estimate in this study was done by using a combination of BS 5400: Part 10: 1980 and the new method mentioned above.

This study concerns a 34m long, 5.2m wide, single spanned, double lattice girded, wrought iron railway bridge, located at Puttalam (Bridge No. 02 on the railway track between the Puttalam Cement Factory and Limestone Quarry, used for limestone transport by trains) which was built about 40 years ago and damaged and displaced from its abutments by floods. The bridge was then placed on temporary timber abutments for several years.

With the increase in cement production, the owners of the company wanted to use heavier engines (locomotives) on this railway track with increased number of trips. Therefore, there was a need of an assessment for the bridge in order to determine whether the bridge can be used further or should be demolished and a new bridge built in its place.

In order to do the assessment, a condition survey was carried out and then an analysis was done by modeling the bridge by using SAP 2000 program and validating the model by conducting loading tests on the temporarily erected bridge. Both static and dynamic loading tests were carried out by using an M2 locomotive with 6 numbers of 13.16 ton axles for 5 different loading cases to measure the displacement, strain and acceleration at pre-determined (critical) members of the bridge. Using the validated model, the ability of the bridge for higher loading situations was verified and the future fatigue life of the bridge was found to be 30 years for critical members. Therefore it was decided that rehabilitation of the bridge with necessary retrofitting works will be more sustainable than demolishing it and constructing a new one. The bridge is now in use after being repaired, retrofitted and placed on new abutments.

2. CONDITION SURVEY

During the condition assessment, the bridge and individual elements were measured including the sizes of webs and flanges and their current thicknesses, thicknesses of gusset plates, sizes of rivets and bolts etc., (i.e. main girders of the bridge are of equal span, each 33.65 m end to end, cross girders are placed at the panel points of the main girders and also at the centres of each panel, there are two longitudinal girders connected between cross girders in each panel and there are bracings between main girders, the rail is of gauge 1.75m and etc.).

Several deficiencies were identified during visual inspections such as corroded places in the bridge deck and in load carrying members, missing bracings, missing timber sleepers, improper arrangement of sleepers, improper alignment of the rail track on the bridge deck, increased thicknesses of the members due to oxides, missing rivets, replacements of rivets by nuts and bolts and etc., as shown in figure 1.

Corroded load carrying elements

Defective connections

Missing elements and rivets

Temporary abutment

Figure 1 Defects observed during condition survey

3. ASSESSMENT

3.1. Modelling the bridge using ASP 2000

The bridge was modelled using SAP 2000 as a three dimensional frame element model. Effective measured thicknesses were applied for the members of the model as much as possible. Rivet connections were assumed as fully fixed and rotational stiffness behaviour with a magnitude of 18,200 kNm/rad was assumed at connections of cross girders and bottom chord of main truss girder (Siriwardane et al., 2009). Figure 2 shows the view of the finite element model.

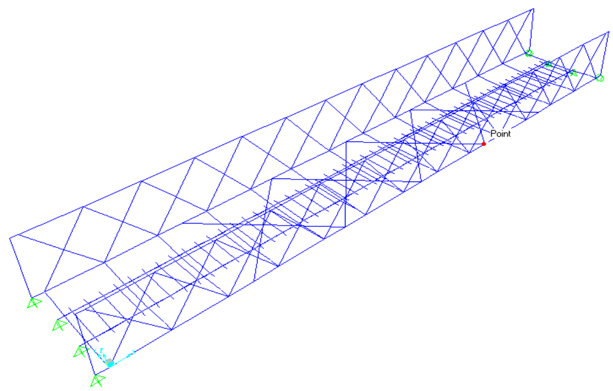

Figure 2 Finite element model of the bridge

3.2. On site loading tests

A series of tests were carried out on site in order to determine the behaviour of the bridge under loading, dynamic factors and for the validation of the finite element model.

An M2 engine (6 Nos., of 13.16 ton axels) was used for loading the bridge at 5 separate loading cases; the center of the engine stopped at 1/4 of the length of the bridge, stopped at mid span of the bridge, stopped at 3/4 of the length of the bridge, the train traveling at a speed of 10 km/hr and the train traveling at a speed of 20 km/hr.

In all the loading cases, displacement measurements (horizontal and vertical) of the mid span of the main girder were measured using 2 displacement gauges, strain measurements at four selected locations were obtained by using strain gauges and acceleration at the mid span using an acceleration gauge.

3.3. Validation of the model

A comparison between the onsite measurements and measurements obtained from the model for similar loading cases are given in table 1. As the readings from onsite displacement and strain gauges and the numerical results obtained from the model are quite close, it was concluded that the model would behave in a similar way to the actual bridge. Therefore the model was used to find numerical results of anticipated loads of the bridge.

Table 1 Comparison of onsite measurements and measurement from the model for the centre of the M6 engine at mid span

Description	Onsite measurement	Measurements obtained from the model
Horizontal displacement	1.46 mm	0.86 mm
Vertical displacement	15.36 mm	12.77 mm
Strain gauge 1 reading	108.05 µe	99.56 µe
Strain gauge 2 reading	239.50 µe	241.94 µe
Strain gauge 3 reading	58.07 µe	48.62 µe
Strain gauge 4 reading	18.56 µe	11.53 µe

3.4. Checking the bridge for anticipated higher loading

The model was loaded with an engine of weight 100 tons (with 6 axels) which is the anticipated engine which will be used on the bridge in the future. Then the strains, stresses and displacements at critical members (elements) were observed. Figure 3 shows the critical members and table 2 shows the stresses developed in those members against the yield strength and hence the factors of safety of each member.

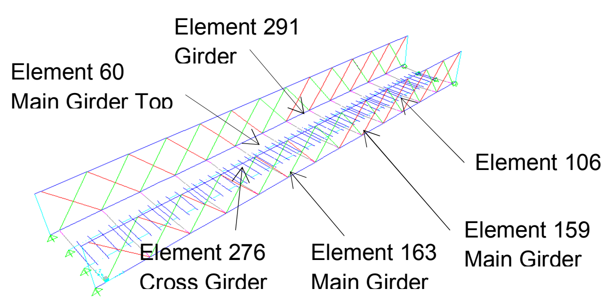

Figure 3 Critical elements of the bridge

As the factors of safety of all the critical members are within reasonable limits, it was decided that the bridge is safe for 100 ton engines subject to further analyses and improvements as mentioned in the next chapters.

Table 2 Factors of safety for critical members

Critical member	Yield stress / MPa	Developed stress/ MPa	Factor of Safety
276	245	97.953	2.501
291	245	96.781	2.531
60	245	68.916	3.555
106	245	60.515	4.049
159	245	96.313	2.544
163	245	64.968	3.771

3.5. Checking the future fatigue life of the bridge

In load carrying structural elements, due to repeated applications of stresses that are not sufficient to cause failures by a single application create cracks which propagate gradually until failure of the element. These failures are called fatigue failures.

Fatigue life estimation was done using the methods given in BS 5400: Part 10: 1988 with required modifications as described by *Siriwardane et al* (2009) such as dynamic factors for different engine types used in Sri Lanka (i.e. 1.30 for diesel engines). The S-N curve for wrought iron was taken from the report "District line fatigue of riveted under bridges, Infrastructures Consultancy Services, London Underground Limited, June 1998". It was assumed that the compressive stresses do not contribute to fatigue damage. The assessment was done only for the 3 most critical members.

Key parameters in fatigue life calculation are, f_{max} which is the maximum stress in a member during a particular stress cycle, f_{min} which is the minimum stress (due to dead load) in the member during the same stress cycle, were determined using the finite element model.

n (n_1, n_2,..) is the number of yearly repetition of stress ranges that the member is subjected to. The value of n during each time period was found using the daily frequency of trains obtained from the authorities (from train timetables).

N (N_1, N_2…) is the allowable number of repetition of stress cycles before failure. Using the f_{max} and f_{min} of a given cycle in a critical member, the value of N was obtained from the S-N curve for wrought iron mentioned before.

In the calculation of cumulative damage, a constant sequence of trains was assumed and the life span of the bridge was divided into 2 periods, from 1970 to 2000 (diesel engines M2, WDM 6 and carriages) and 2000 to 2010 (diesel engine M2 & carriages) depending on the trains that travelled on the bridge.

Miner summation for 2000 to 2010 for fatigue damage (α_1) for a 10 year period was obtained by the equation;

$$\alpha_1 = (\sum n/N) \times 10$$

Miner summation for 1970 to 2000 for fatigue damage (α_2) for a 30 year period was obtained by the equation;

$$\alpha_2 = (\sum n/N) \times 30$$

Then the cumulative fatigue damage for the period of 40 years from 1970 to 2010 (α) was obtained by the summation of α_1 and α_2;

$$\alpha = \alpha_1 + \alpha_2$$

Then the remaining fatigue damage is given by (1-α).

For the calculation of future life from 2010 upwards, an engine weighing 100 tons (16.7 tons/axel, 6 axels), WDM6 engine and carriages were assumed to be traveling per a given train timetable (anticipated train timetable from 2011) and the stresses in the critical elements were obtained from the finite element model. Then the fatigue damage per year (for future years) α_0 was calculated using the equation;

$$\alpha_0 = (\sum n/N)$$

Then the remaining life of each critical member was calculated using the equation;

$$\textit{Remaining fatigue life} = (1-\alpha)/(\alpha_0)$$

The remaining fatigue life thus calculated is subject to any changes of the train time table and engine (loading) weights.

In this assessment, it was found that the most critical member has a future fatigue life of 30 years.

3.6. Improvements

As per the assessment results, it was decided to reuse the bridge with necessary improvements. Accordingly, following improvements were recommended. (a). Placing the bridge on newly built reinforced concrete abutments, (b). Making the railway track straight near the bridge to reduce lateral stresses when the train travels on the existing curved railway track, (c). Cleaning the steel structure completely using sand blasting techniques, identifying the defective elements and replacing them (these improvements increase the fatigue life of these elements as well as the stiffness of the bridge), (d). Applying a corrosive resistant paint, (e). Placing new steel sections for missing elements and bolts for all missing rivets, (f). Aligning the railway track as smooth as possible to reduce vibration, (g). Placing sleepers at regular intervals, (h). Providing a proper river bank protection and (i). Regular maintenance. Figure 4 shows the bridge after improvement works.

Figure 4 Bridge after improvement works

4. VERIFICATION TESTS

Properties of individual elements as well as those of the whole structure may change due to various actions during its rehabilitation (and retrofitting) works. The bridge discussed in this paper was moved from its previous temporary abutment to new abutments without dismantling it. (Note: the bridge is assembled by rivets). In order to determine the status of the bridge after repair, it was decided to conduct non-destructive tests.

4.1. Verification test procedure

A series of loading tests on the bridge was carried out on 31/12/2011 by using an M2 engine (6 numbers of 13.16 ton axles) under different static and dynamic loading conditions as mentioned below. Deflections and strains at critical locations of the bridge were measured.

The loading cases used are; case 1: the center of the engine was stopped at 1/4 of the length of the bridge and measurements were taken, case 2: the center of the engine was stopped at the mid span of the bridge and measurements were taken, case 3: the engine was allowed to travel at a speed of 10 km/hr and measurements were taken and case 4: the train was allowed to travel at a speed of 20 km/hr and measurements were taken.

In order to take measurements, two displacement gauges were fixed at the mid span of a main girder; one below the flange of the main girder, fixed vertically to measure the vertical displacement and the other touching the web of the main girder, fixed horizontally to measure the horizontal displacement. Strain gauges were fixed at three pre-defined locations on critical elements to measure the strains for all four loading cases.

4.2. Analysis of verification test results

Maximum vertical and horizontal displacements of the bridge at static loading were observed as 6.40mm and 0.16mm respectively. Further at dynamic loading, the maximum vertical and horizontal displacements in the recent test were 6.55mm and 0.45mm respectively. Therefore, when comparing these new results with the displacement values obtained before improvement works (table 1), a clear reduction in the displacement measurements could be seen. Due to the retrofitting works (especially due to replacement of weak elements by new elements and by strengthening connections with the use of bolts where they were missing), the stiffness of the structure has improved.

For the 3 elements in which the strains were measured and stresses were calculated, it was found that the maximum stresses happen when the engine moves at 20km/hr (loading case 4). The factors of safety calculated for these 3 members for loading case 4 are 2.40, 2.28 & 18.04 and they are reasonably safe values.

As the verification tests have shown improvements with regard to deflections of the bridge and reasonable factors of safety for the three critical elements with regard to strains and stresses, it was concluded that the improvement works are successful.

5. CONCLUSIONS

Constructing a new bridge usually consumes more time and money than rehabilitating and retrofitting an existing bridge for reuse. There are ways to assess the possibility of reusing old and/or damaged bridges. This paper explains one such assessment method applied to a damaged steel railway bridge using onsite non-destructive tests, finite element models and principals of fatigue life. Further, the bridge so assessed and retrofitted was subjected to verification tests which have proved that the method used is successful.

Most of the old highway steel bridges and railway bridges in Sri Lanka are nearing the end of their design lives. Knowing the future life spans of these bridges is very important because, then the relevant authorities can plan for their rehabilitation or replacement in advance, thereby contributing to sustainable development. The method described in this paper can be applied practically in Sri Lanka to carry out life assessments of steel bridges.

6. ACKNOWLEDGMENTS

The authors acknowledge the collaboration of the team of experts who worked on the investigation of railway bridge No. 2, Puttalam and the kind support given by Holcim Lanka Pvt., Ltd. The authors also convey their gratitude to the National Research Council of Sri Lanka (Grant No. 11-106).

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Learning Tool for Reinforced Concrete Design

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Abstract: *Reinforced concrete design is considered as a vitally important discipline in the field of civil engineering. The design of reinforced concrete is done to various standards and well established code of practices. The British standard, BS 8110, has been followed for the design of reinforced concrete structures in Sri Lanka for almost three decades now. However, with the withdrawal of BS8110 by the United Kingdom to pave the way for European common standards, Sri Lanka has been compelled to use Euro standards or adopt other international standard for the design of reinforced concrete. American Concrete Institute ACI 318(2008), European Standard EC2 (BS EN 1992-1-2), Japan Concrete Institute JCI standards and model codes like CEB FIP Model Code 90 are the commonly used international standards for reinforced concrete design. It is also known that Sri Lanka are gearing to embrace the EC2 (BS EN 1992-1-2) in place of the withdrawn BS8110.The design approaches and design provisions of various standards are different form one another and results of various design approaches and their structural implications are continuously being debated in various forums. It is considered that the knowledge of different design approaches is essential for the better understanding of the behaviour of reinforced concrete. In addition, there is an especial requirement to train structural engineers in Sri Lanka to adopt Euro standards. Both of the above objectives can be achieved by a well design learning tool. Leaning tool allows the users to learn on their own phase. It is in this back ground that interactive learning tool "Recode" is formulated. This paper presents various features of Recode, the learning tool developed to teach beginners how to do design under different standards and the experienced, how to adopt to changing scenarios to which their designs have to be confirmed.*

Keywords: *Reinforced concrete, Design, Standards, Learning tool, Engineering education.*

1. INTRODUCTION

With the popularization of computer applications and internet applications, development of learning tools for various application and illustrations is getting popular by the day. There are few such attempts to use such applications for teaching reinforced concrete design as well. One such notable attempt is the development of "COMPACT" for EC 2(EC2, 2008). The main emphasis on such tools is normally to teach an establish code approach for the design of reinforced concrete and rarely have attempted to introduce different code approaches together. Therefore they have limitations to be used to compare result from different code approach and moreover to provide the advance learners the option to compare results under different code approaches.

It is in this background that "Recode", a learning tool that teach different codes and compare design philosophies of different code approaches has begun. In term of structural action Recode provide a platform to learn and compare major code approaches BS 8110(BSI, 1997), EC 2(EC2, 2008), ACI 318(ACI, 2008), JSCE (JCI, 1996), and the CEB-FIP Model code (CEB-FIP, 1993). In terms of element design, the Recode compares BS 8110(BSI, 1997) and EC 2(EC2, 2008) approaches.

2. SIGNIFICANCE OF THE STUDY

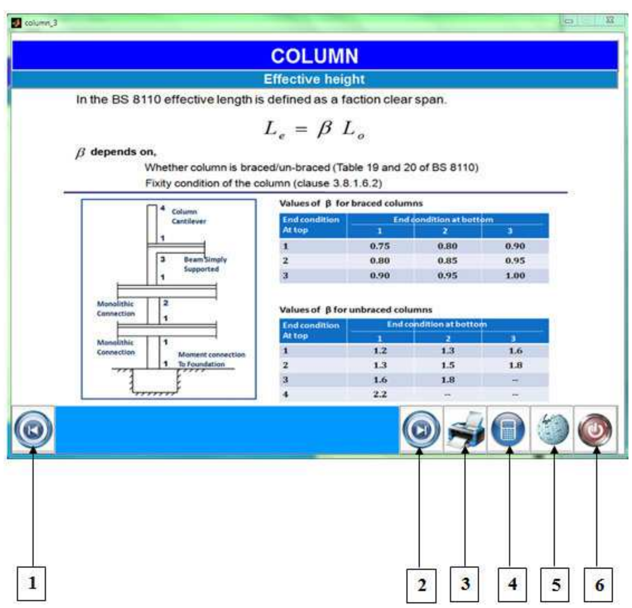
With the withdrawn of BS 8110, Sri Lanka is compelled to change for a code where latest developments and technological advances are continuously being upgraded. This learning tool provide a platform to teach design approaches for some of the notable codes namely ACI, BS 8110, EC 2, JCI, and the CEB

FIP in terms of major structural actions; bending and shear in the ultimate limit state and deflection and crack width in the serviceability limit state. Tool also offers provisions to element design. In terms of element design, the tool intends to cover BS 8110, EC 2 and ACI method and have so far covered column and beam design under the BS8110 and EC2.

3. METHODOLOGY

To achieve desirable outcomes as a leaning tool and comparison tool of different code approaches, "Recode" is design as an interactive program that allows user to familiarise with various design code approaches using interactive sessions, followed by guided examples and general examples. Guided examples provide user the ability to test their knowledge, while the general example provide user with the ability to define their own problems and find solutions for them. The user is also provided the ability to produce a report for the user defined example. Finally to assist the comparison, user is provided a comparative tool where solution can be found for user defined problems according to different code approaches. Notable variations of different code approaches are also highlighted in the tool to make the user aware of specific and generic variations of the different code approaches.

Given the complexity of mathematical manipulation required for putting all code approaches and design equation into a programmable platform graphical user interface provided in MATLAB 2010a was preferred for the programming. Main advantage of the use of MATLAB 2010a is its ability to tackle symbolic operations. Furthermore, the graphical interface allows the program develop as a a Windows application, which can be conveniently incorporated. Learning tool was also made available with inbuilt calculator, printing option and command buttons to easily navigate through tool. Fig. 1 explains the basic outlook of a tool window under general explanation illustrating how to find the effective length for columns according to BS 8110. Advance users are provided with the ability to navigate directly into the area they want to by providing a content page at the beginning of the module.



Numbers given to push buttons indicate the following,

1 – Navigate to previous window
2 - Navigate to next window
3 – Printer window
4 - Calculator
5 - Navigate to content page (home page)
6 – Discontinue a learning tool session

Figure 1: Overview of the typical arrangement of a window in the general explanation

4. COMMON FEATURES OF THE LEARNING TOOL

Arrangement of different modules of the learning tool can be summarised as follows,

1. Each module of the design tool will start with the contents page followed by the generic explanation on the basic philosophies for the design and other more specific design considerations, if any, required by the standard.

2. General explanation will always contain number of interactive sessions and a flow chart towards the end of the explanation to summarise how said code provision work in a given module to complete a given design aspect. This flow chart also outlines the way the learning tool will make its calculations for a particular aspect of design – Figure 2 shows the flow chart of the design of flange section according to the BS8110 as an example.

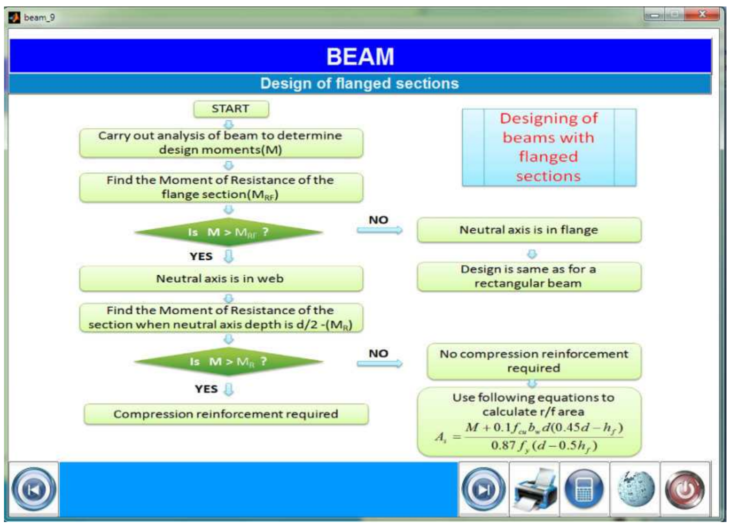


Figure 2: Flow chart used in the general explanation summarising the design of flanged beam

3. General explanation is followed by a guided example that will provide the learner with ability to check his understanding in the subject matter learned. At first the user will be able to work out the problem for which the results can be checked. Use of a guided example in the learning tool is illustrated in Figure 3.1 with Figure 3.2 showing the results at the end of the user interaction.

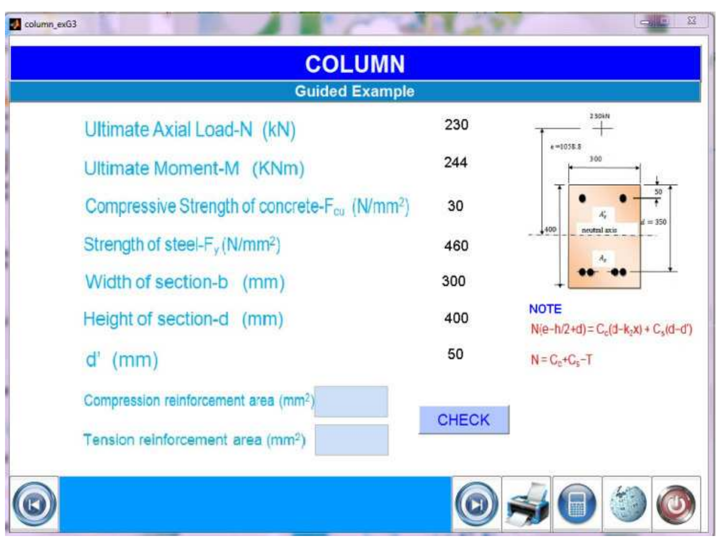


Figure 3.1: Guided example prompting the user to work out a question

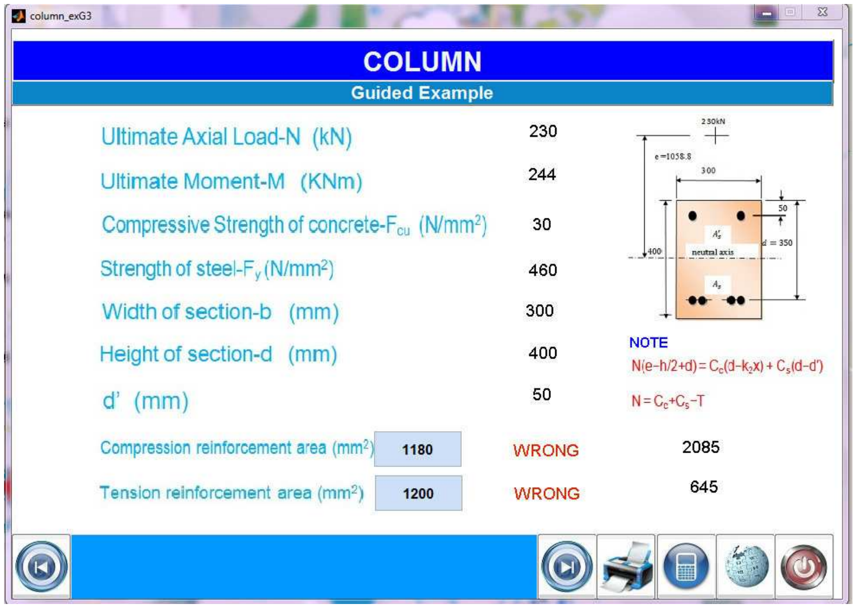


Figure 3.2: Results of the guided example after the user interaction.

4. Learning tool is also programmed to find the solution for user defined examples. Figure 4.1 and 4.2 show screenshots of a general example –Design of short column subjected to bending moment. Figure 4.1 is a screen shot that allows the user to define their own problem while the solution for the question formulated is shown in the Figure 4.2.

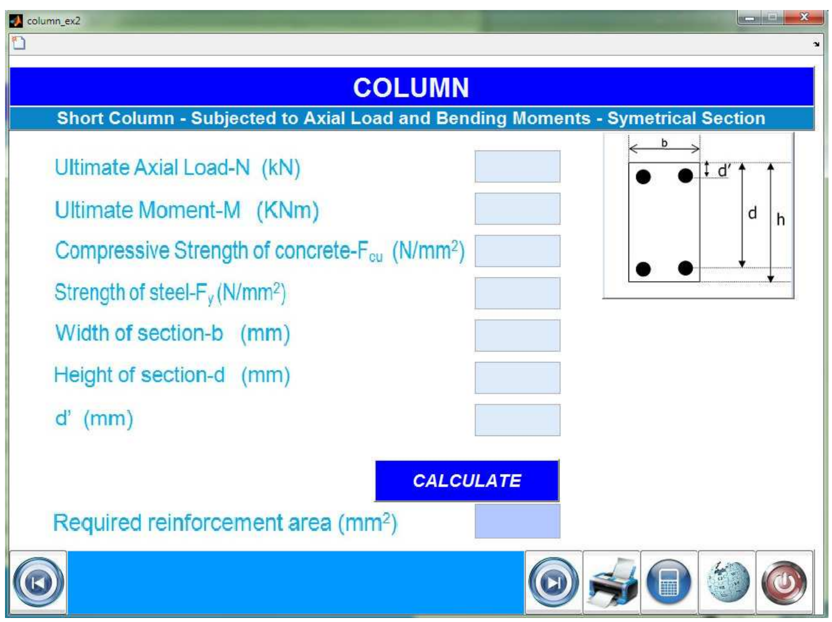


Figure 4.1: General example included in the learning tool (Before the results were obtained)

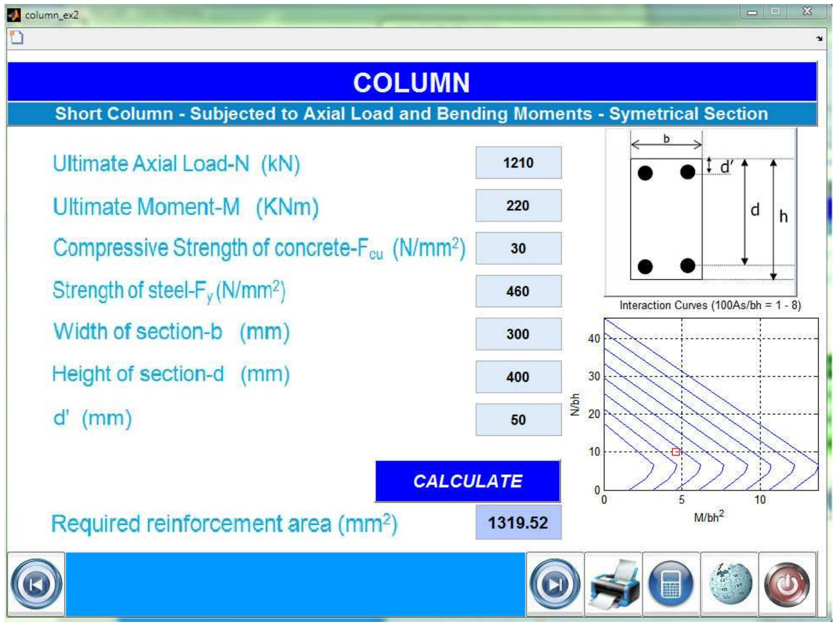


Figure 4.2: General example included in the learning tool (After the results were obtained)

5. Learning tool facilitate the advanced learners allowing them to compare the results of the user defined problem under different standards for the reinforced concrete design. Figure 5.1 and 5.2 shows the comparison tool for column design (Short column subjected to bending), checked under the design provision of BS 8110 and EC2. Figure. 5.1 prompt the user to defined the column while Figure 5.2 show the results under the different code approaches. Different reinforcement area resulted by the different standards highlight the different code provision and philosophies employed by the two standards for the design of reinforced concrete and in particular for the column design (Short column subjected to axial and bending deformations-asymmetric reinforcement arrangement).

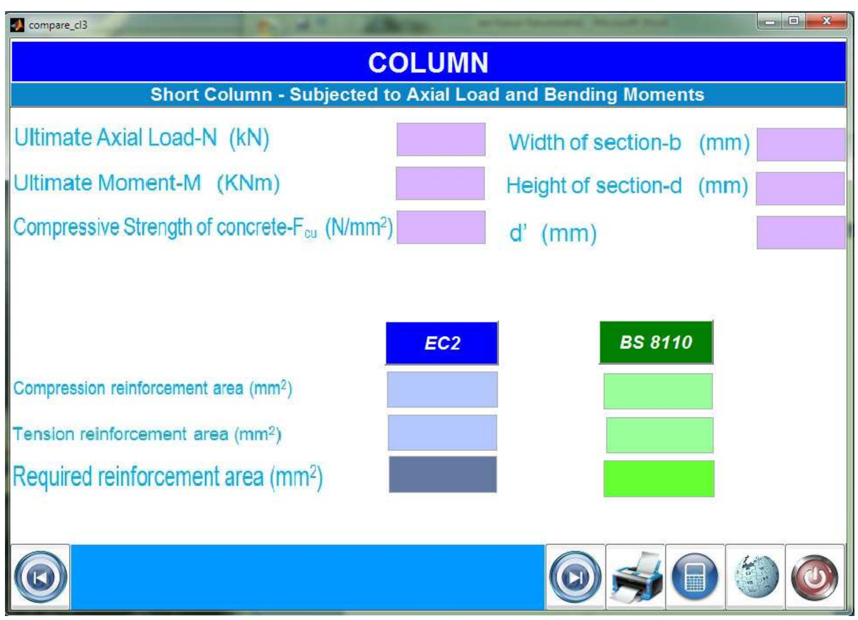


Figure 5.1: Comparison tool included in the program (Before the results obtained)

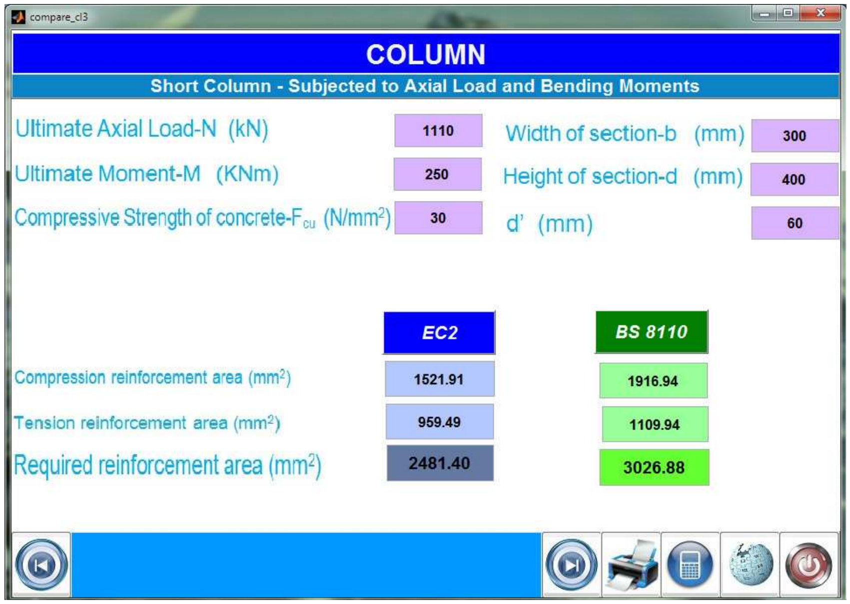


Figure 5.2: Comparison tool included in the program (After the results obtained)

5. DISCUSSION

The paper summarises learning tool devised to teach different code approaches for the design of reinforced concrete. Apart from the teaching the established design philosophies, the tool can be used in comparing results of various design standards among each other and against the actual experimental evidence to evaluate the relative merits and demerits of the different design approaches. At the moment the tool is limited for the structural actions bending, shear, deflection and crack width calculation for prominent code of practices namely BS 8110(BSI, 1997), EC 2(EC2, 2008), ACI 318(ACI, 2008), JSCE (The Japanese code)(JCI, 1996), and the CEB-FIP Model code (CEB-FIP, 1993) and element design for beam and column elements under BS 8110 and EC2 methods.

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Behaviour of Cantilever Slabs in Blast Environment and Strengthening Techniques

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Abstract: *Cantilever slabs are among the most vulnerable structural elements at blast loading. As they are indispensable to most structures, it is important to investigate the behaviour of cantilever slabs in a blast environment and possibilities of improving their blast resistant abilities. In this paper we are proposing simplified design envelopes drawn for steel to concrete ratio and effective depth of slabs against blast parameters for a common range of cantilever slabs. These design envelopes have been prepared using results of previous research. Using these envelopes, conclusions are made about the effect of slab thickness which is one of the major parameters for improving blast resistant ability. This paper also discusses the blast resistant ability of reinforced concrete cantilevers designed by using the code BS8110 and ways to improve such conventional designs to make them better resistant in a blast loading environment in an economical way.*

Keywords: *Blast resistance, cantilever slabs, structural designs, design envelopes, reinforced concrete, safety.*

1. INTRODUCTION

Even though blast resistant structural designs are becoming important due to the rise in blast risks throughout the world, high explosive blast loading has not yet been included directly in commonly used codes (High explosive blast design strategies are given in the codes UFC-04-10-01, UFC-03-340-02 etc., by the Department of Defence, USA). As the available codes are limited and the theories are complicated, it is worth preparing design guidelines for use by ordinary designers for application in their designs.

Considering the safety of structures and occupants, the EN 1990 and EN1991-1-7 Codes provide strategies for accidental design situations. However, for most common structures, the provisions of these codes are not applied. As the possibility of any structure facing blast loading cannot be predicted, it is important to know the blast resistant ability of vulnerable structural elements such as cantilevered slabs. Therefore, in this research, blast resistant design envelopes for cantilever slabs were developed by using results of previous research and current deign Codes. Using these envelopes, a detailed analysis on the strengthening techniques of cantilever slab by changing its effective depth and steel area is presented.

Strength (magnitude) of a blast depends on the weight of explosives and the effect of a blast on a structure depends on the distance from the blast to the structure. As the weight of explosives that can be brought close to a structure and the location (distance) of a possible explosion near a structure are controllable (i.e. by using appropriate site layouts, vehicle access control and restrictions, car parking away from the main structure, positioning waste bins away from critical structural elements, providing proper security fence (wall) etc.), attention must be drawn to such provisions during a design.

2. GENERATION OF PRESSURE AND ESTIMATION OF IT’S MAGNITUDE

As a result of the experiments done by various scientists since 1940s, there are theories developed mostly using empirical methods about the pressure generation due to a blast, the way this pressure acts on structures and acceptable methods to calculate blast wave parameters. Blast loading on a considered point on a structure is shown in figure 1.

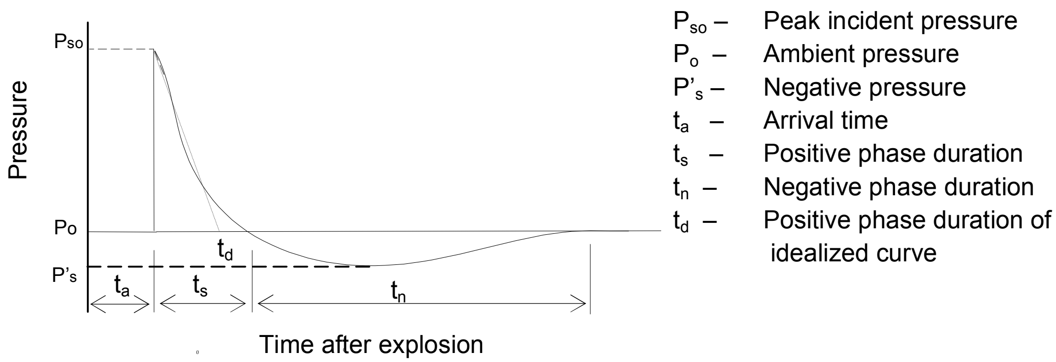

Figure 1 Variation of Pressure after a blast at a considered point

Specific energy of explosives is different from one explosive to another. Using TNT (tri nitro toluene) as the base material, equivalency factors have been estimated for the other explosives. Using the charge weight of TNT (W, measured in kg) and the distance from the centre of the blast to a considered point on the structure (R, also called the standoff distance, measured in m), the term scaled distance (Z) is defined which is;

$$Z = R/W^{1/3} \text{ (units: } m/kg^{1/3}\text{)}$$

Other important terms are; ground zero distance (R_g) which is the horizontal projection of R, and the angle of incidence (α) which is the angle between the vector from the point of blast to the point of concern and its horizontal projection. Pressure due to air blasts (i.e. blasts happened in the air) differs in some ways from that of surface blasts (i.e. blasts happened on the ground). The area of interest of this paper is the pressure on cantilever slabs due to surface blasts.

Blast loading parameters such as Incident Pressure (P_s), Reflected Pressure (P_r), Incident Velocity (U_s), Reflected Velocity (U_r), Incident Impulse (I_s), Reflected Impulse (I_r) and Positive phase duration (t_s) etc., are estimated with respect to Z. In order to estimate these parameters, the well known Kingery & Bulmash's empirical solutions are used.

3. BLAST RESISTANT DESIGN

The blast resistant design was done using the procedure introduced by Cormie *et al* (2009). This procedure has been prepared following the codes UFC-3-340-02, EN 1990, BSEN 1992 and BS 8110, Part 1 (1997), Part 2 (1985) etc.

3.1. Structural resistance

The structural resistance is determined using the link between the duration of loading of blast pressure on a structure and the natural frequency of the structure. Structural resistance can be divided into three regimes according the response of the structure when it is loaded by blast waves. These 3 response regimes are quasi-static, impulsive and dynamic. The response of the structure is quasi-static when $10T<t_d$ and $tm<0.3t_d$, impulsive when $t_d<0.1T$ and $3t_d<t_m$, dynamic when $0.1T<t_d<10T$ and $0.3t_d<t_m<3t_d$ where T is the natural period of vibration of the element (structure) and t_m is the time the element needs to reach its maximum deflection. For designs, quasi-static and dynamic regimes are combined to form one regime and impulsive regime is the other. The designs are done for ultimate limit state and for one occurrence of blast. Refer the illustration given in Figure 2.

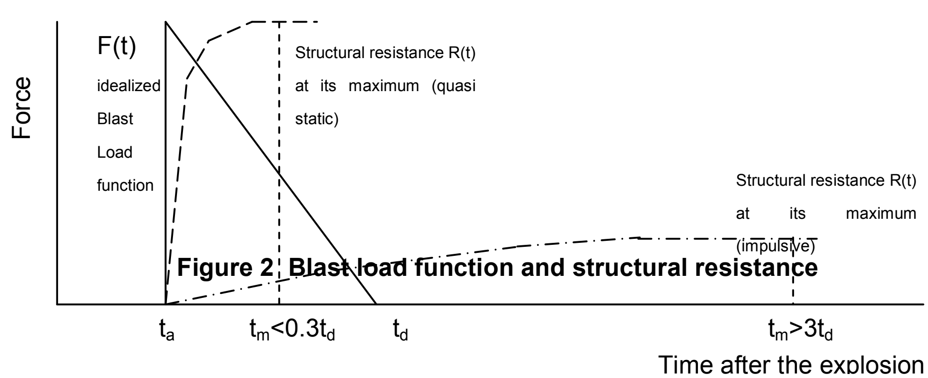

Figure 2 Blast load function and structural resistance

3.2. Protection categories

As described by Cromie *et al* (2009), two protection categories can be introduced for blast designs based on limits of deformation or deflection of the elements [support rotation (θ) and/or ductility ratio (μ) which is the ratio; total deflection (χ_m) / deflection at elastic limit (χ_e)]. Support rotation, $\theta \leq 2^o$ comes under protection category 1 which protects structural elements as well as occupants from blast loads. For $\theta \leq 2^o$, concrete cover at tensile side may be cracked but the cover on both tensile and compressive sides of the element is effective in resisting moments. Support rotation $\theta > 2^o$ comes under protection category 2 in which structural elements are protected from collapse (protection from collapse can be expected till $\theta = 4^o$). In this deformation region, concrete cracks at the tensile side and crushes at the compressive side. For $\theta > 2^o$, deformation limits imply plastic deformations of the element.

3.3. Factors for material strengths

Mechanical properties of steel and concrete change at rapid loading. Therefore static strengths of materials are converted to dynamic strengths by applying appropriate factors called dynamic increase factors (DIF). Further, according to EN 1992-1-1 (2004), accidental material factors (AMF) are applied on design strengths of materials to withstand accidental loads. Accordingly, nominal material strengths are modified using both DIF and AMF in blast designs. The values of DIF and AMF used in this research are mentioned at relevant chapters in this paper.

3.4. Loading diagram for a cantilever slab

As the cantilevered slabs are generally positioned at floors above the ground level and the blasts considered are surface blasts, the blast pressure should act at the underside of cantilevers whereas the dead and imposed loads act from the top of the cantilevers as shown in figure 3. Therefore there is a necessity of providing tensile reinforcement for dead and imposed loads at the top fibers and tensile reinforcement for blast loading at the bottom fibers.

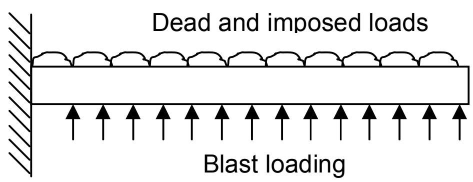

Figure 3 Loading diagram for a cantilever

3.5. Assumptions

As mentioned in the previous paragraphs, the important assumptions used are; idealized blast loading function (triangular pressure time function), idealized resistance deflection function, uniformly distributed loading, for quasi-static/dynamic regime t_d is longer compared to t_m ($t_m/t_d < 3$) and hence loading represents pressure (P) and for impulsive regime, t_d is shorter compared to t_m ($t_m/t_d \geq 3$) and hence loading represents impulse (I). Cantilevers are subjected to two loads from top and bottom sides at a blast as explained in chapter 3.4. The reduction of bending moment and shear forces due to dead and imposed loads (which are acting opposite to blast pressure) was not taken in to account depending on their magnitude compared with that of blast load.

3.6. Design for impulsive regime

The Impulsive regime is considered under protection category 2 which allows support rotations greater than 2° (up to 4°). The design resistant moment M_{Rd} (with dynamic design strengths) is given by;

$$M_{Rd} = [A_s f_{yd \cdot dyn}(z)]/b$$

where, A_s is the tensile reinforcement area, b is the width of the section, $f_{yd.dyn}$ (dynamic design strength of steel) is given by $1.2f_{yk}$ (static yield strength of steel) , z is the lever arm (distance between the tensile & compressive reinforcement). Ultimate resistance of the element R_m can be derived as a function of M_{Rd} and length (L) of the element and can be solved using:

$$I^2A^2/(2K_{LM}M) = (R_m\chi_e)/2 + R_m/(\chi_m\text{-}\chi_e)$$

where, I is the blast impulse, A is the loaded area, K_{LM} is the load mass factor, M is the mass of the element, χ_e is the elastic deflection and χ_m is the total deflection.

The relation between t_m/t_d vs t_d/T for triangular blast loading and the relation between the coefficient (f) for second moment of area for cracked section with equal reinforcement in opposite faces vs $A_s/(bd)$ are obtained from the charts of UFC-3-340-02. As the concrete in the compressive side of the element crushed due to allowed larger deflections in the impulsive regime, compression reinforcement is required. For cantilever slabs, the advantage is that tensile reinforcement provided at the top fibers to take dead and imposed loads can be improved to act as the compression reinforcement for blast loading.

3.7. Design for quasi-static/dynamic regime

Quasi-static/dynamic regime is the regime for protection category 1 designs where support rotation θ must be less than 2º. Simplified form of M_{Rd} (with dynamic design strengths) is found using;

$$M_{Rd} = [A_s f_{yd.dyn}(d\text{-}0.4x)]/b$$

where; d is the effective depth, x is given by $A_s f_{yk}/(0.59\ b\ f_{ck})$ and $f_{yd.dyn}$ is found using $1.2f_{yk}$.

R_m can be derived as a function of M_{Rd} and L using;

$$R_m = 2M_{Rd}/L$$

The natural frequency of vibration (T) is given by;

$$T = 2\pi\sqrt{(K_{LM}\ M/k_e)}$$

The relation between the coefficient (f) for second moment of area for cracked section with tension reinforcement vs $A_s/(bd)$, the relation between x_m/x_e and t_d/T and the relation between t_m/t_d vs t_d/T for triangular blast loading are obtained from the charts of UFC-3-340-02. Since concrete is effective in resisting moments at compression side, compressive reinforcement may be avoided in the quasi-static & dynamic regime.

4. METHODOLOGY

Use of the term $A_s/(bd)$ in reinforced concrete designs is common. When a design is done using BS8110 or similar codes, the ultimate result is the values for A_s and d. Z is the most convenience parameter in determining the effect of a blast. Therefore the requirement is to develop envelopes (design envelopes) for $A_s/(bd)$ vs Z covering all practical Z values, cantilever spans, effective depths and $A_s/(bd)$ values for both impulsive and quasi-static/dynamic regimes. Accordingly, in this research, the range of Z selected was $0.11\ m/kg^{1/3} \leq Z \leq 40.94\ m/kg^{1/3}$. The selected spans are 1.0m, 1.5m, 2.0m and 3.0m. Effective depths considered are within the rage 100mm ~ 350mm and $A_s/(bd)$ values are in the rage 0.05% ~ 2.00%. Using the method described in chapters 3.6 & 3.7, a series of numerical analyses were conducted using spreadsheets and the relationship between Z and $A_s/(bd)$ was obtained. The shear reinforcement was designed using BS8110, Part-1: 1997 with the use of DIF and DMF values which are described in chapter 3.3. The results were then plotted in 8 graphs (8 design envelopes), 4 of which contain 4 selected spans in the impulsive regime while the other 4 show the 4 spans in the quasi-static/dynamic regime. Cantilevers designed for BS8110 (conventional design) have also been plotted in these envelopes. The imposed load used for conventional design is 5.0kN/m2.

5. DESIGN ENVELOPES

Out of the 8 design envelops, 4 were selected for this paper. Figures 4 and 5 are the envelopes for spans 1.5m and 3.0m for impulsive regime. Figures 6 and 7 show the envelopes for spans 1.5m and 3.0m for quasi-static/dynamic regime.

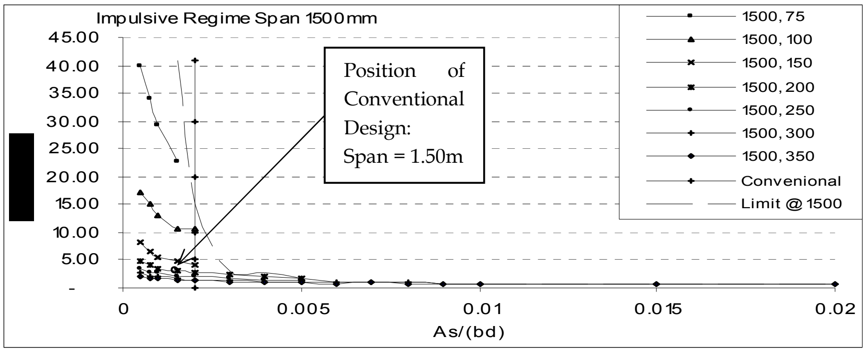

Figure 4 Z vs A_s/(bd) for span 1.5 m in the impulsive regime

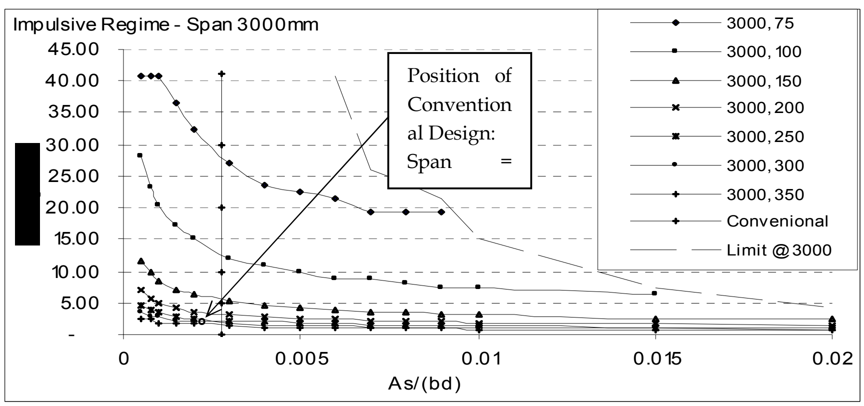

Figure 5 Z vs A_s/(bd) for span 3.0 m in the impulsive regime

The graphs for impulsive regime given in figure 4 and 5 show impulsive design limit which is one of the important observations in this research.

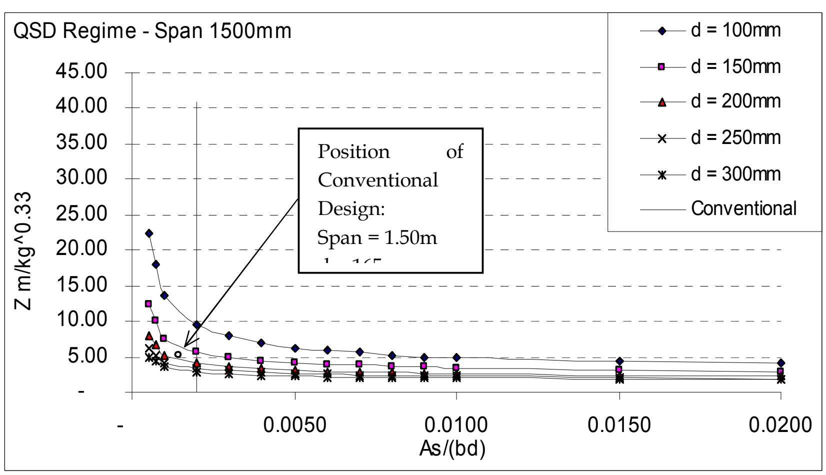

Figure 6 Z vs A_s/(bd) for span 1.5 m in the quasi-static / dynamic regime

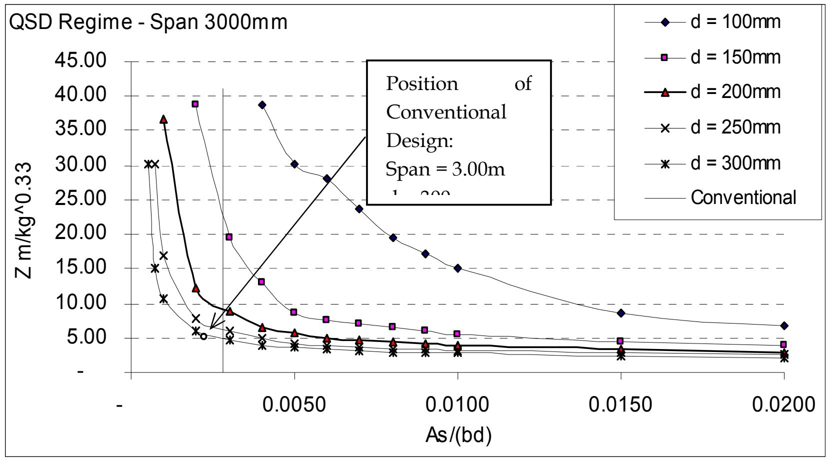

Figure 7 Z vs A_s/(bd) for span 3.0 m in the quasi-static / dynamic regime

Table 1 shows a comparison of shear reinforcement between impulsive and quasi-static / dynamic regimes for Z = 2.155 m/kg$^{1/3}$.

Table 1 Shear reinforcement for Z = 2.155 m/kg$^{1/3}$

Span (mm)	Z (m/kg$^{1/3}$)	Impulsive Regime		Quasi-static & dynamic regime	
		d / (mm)	Shear links (mm^2/m^2)	d / (mm)	Shear links (mm^2/m^2)
1000	2.155	169	0.00	215	3,934
1500	2.155	194	0.00	255	4,647
2000	2.155	215	0.00	280	3,203
3000	2.155	230	0.00	350	2,878

Figure 8 shows the shear reinforcement requirement for cantilevers 3.0m in span. It was observed that the effective depth d < 300mm cannot be designed for Z < 1.05 m/kg$^{1/3}$ and that shear reinforcement is not required for Z > 1.05 m/kg$^{1/3}$.

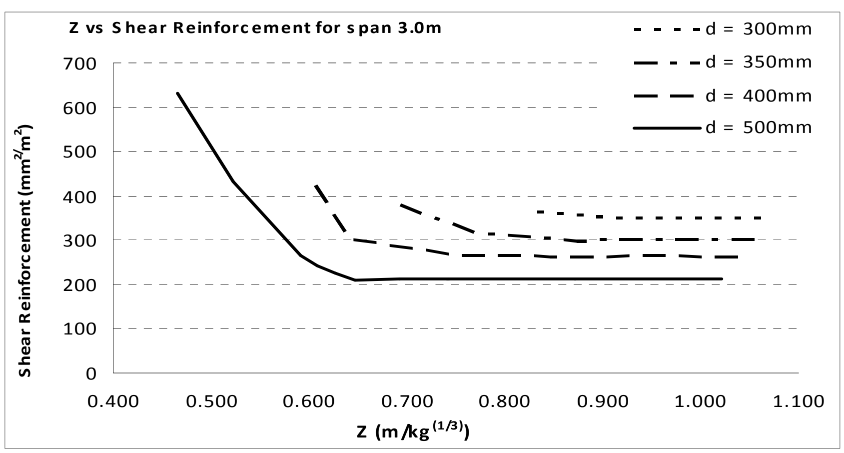

Figure 8 Shear reinforcement for span 3.0 m in the impulsive regime

6. ANALYSIS

Observing the graphical envelopes given in figures 4, 5, 6 & 7, for both impulsive and quasi-static/dynamic regimes, it can be seen that gradients of graphs get reduced with increasing $A_s/(bd)$. For a range of small $A_s/(bd)$ values, there is a faster reduction of Z values but with the increase of $A_s/(bd)$ the reduction of Z goes down. Therefore after a range of $A_s/(bd)$, increase of $A_s/(bd)$ has a minor influence in reducing Z. This means that, after a certain limit, the increase of A_s is not effective in improving blast resistant abilities. Studying the same graphs, it can be seen that by increasing effective depths d, Z can be reduced. The influence of d in reducing Z has spread along a wide range. The important observation is the effectiveness of increasing d for improving blast resistant abilities.

As mentioned in chapter 3.2 and 3.6, impulsive designs come under protection category 2 where no collapse is expected but not as safe as quasi-static/dynamic designs (chapter 3.7). As can be seen in table 2, there is a higher requirement of shear reinforcement for quasi-static/dynamic designs, but impulsive designs are possible without shear reinforcement up to a considerable limit of Z. Figure 8 shows impulsive shear requirement envelope for a cantilever slab (with a span of 3.0m) in which shear reinforcement is needed when $Z < 1.05$ m/kg$^{1/3}$ but in minor quantities compared to quasi-static/dynamic designs. Therefore, selecting impulsive designs with possible other safety measures as mentioned in the introduction may prove economical.

One of the important observations is the influence of impulsive limits for a design. As shown in figures 4, 5, there are maximum limits to $A_s/(bd)$ for cantilever slabs to be in the impulsive regime and the slabs move away from the impulsive regime when $A_s/(bd)$ goes beyond these maximum limits. For example, in figure 4, the conventional design is outside the impulsive limit for small d values and in figure 5, the design is within the impulsive limit for all considered d values. One of the reasons for this difference is the mass of the element (i.e. when the mass of the element is high, blast resistant ability is high too). An effective way to increase the mass is increasing the value of d. Further, the effects of T and t_m too have an impact. Increasing A_s (which increase $A_s/(bd)$) reduces T & t_m which in turn reduces the impulsive properties pushing the element away from impulsive limits.

It can be observed that unlike in the impulsive regime, there are no maximum limits for $A_s/(bd)$ for quasi-static/dynamic regime (i.e. any element if not in the impulsive regime should be in the quasi-static/dynamic regime). However, similar to impulsive regime designs, mass of the element plays a major role in the quasi-static/dynamic regime too and therefore, by increasing the value of d, blast resistant abilities can be improved effectively. However, in the quasi-static/dynamic regime designs, the quantity of tensile steel requirement is higher than that in the impulsive regime for a given value of Z (i.e., compare the impulsive envelope with quasi-static/dynamic envelope for similar d and Z). The shear reinforcement requirement in quasi-static/dynamic designs has already been discussed above.

Keeping appropriate values for $A_s/(bd)$ and d, cantilever slabs can be kept within the impulsive limits. If the limits are exceeded, the elements will not resist blast loading efficiently. The elements will then be in the quasi-static/dynamic regime and have less blast resistant abilities unless there are greater improvements to the elements.

It is to be noted that the envelopes show only tensile reinforcements (A_s). Equal reinforcement has been recommended for both tension and compression in the impulsive design and this compression reinforcement will act as the usual tensile reinforcement for dead and imposed loads if appropriate. No compression reinforcement is necessary for quasi-static/dynamic designs as per the principals of the design. However, for cantilevers, the tensile reinforcement provided for dead and imposed loads will act as compressive reinforcement during blast loading.

The positions of conventional designs (approximate values) have been plotted on each design envelop according to their spans. These are given as vertical lines in relation to $A_s/(bd)$ and the arrow pointer shows the position with respect to the effective depth d (i.e., in figure 4, the conventional design for the span 1.50m and effective depth 165mm is sufficient for a Z value of 3.5m/kg$^{1/3}$). For example, the obvious way to improve the blast resistant abilities of this conventional design in the impulsive regime is increasing d with $A_s/(bd)$ remaining unchanged and providing compressive reinforcement.

7. CONCLUSIONS

Any conventional design can withstand blast loads up to a certain magnitude. Knowing this limit is helpful in improving a conventional design and making it more blast resistant. In this research, the main aim was to develop design envelopes for reinforced concrete cantilever slabs in order to find out their position in a blast environment and then determine the improvements needed to make them blast resistant. The conclusions reached are as follows.

Conventional designs can be improved and made blast resistant. If the requirement is to protect the structure from collapse, the structure should at least satisfy the impulsive regime limits. The improvements needed to push a conventional design into the impulsive regime are minor and easily achievable. Quasi-static/dynamic regime gives the best protection from blast loading. However, pushing a conventional design towards quasi-static/dynamic regime needs greater improvements such as a larger quantity of tensile and shear reinforcement. Therefore, considering the cost impact, it can be recommended that conventional designs (structures with less blast risks) should be kept within the limits of the impulsive regime.

Increasing slab thickness (effective depth) is more effective in making structures blast resistant than increasing tensile and compressive reinforcement. There is a limit to the amount of tensile and compressive steel needed for an element to be in the impulsive regime and a requirement for sufficient mass (connected with effective depth). If steel is increased (without increasing the effective depth), the natural frequency and the time the element needs to reach its maximum deflection decrease due to which the element moves away from the impulsive regime (i.e. the element will enter into the quasi-static/dynamic regime where many modifications are necessary to resist blast loads). Therefore a design check for conventional designs should be introduced to see whether the element is within the impulsive regime.

Most practical sizes of conventional cantilever slab designs can be analyzed using the envelopes developed in this research. The authors are suggesting that, using this methodology, design envelopes can be developed for any other structural element to observe their behavior in a blast environment.

8. ACKNOWLEDGMENTS

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Cost Effective and Speedy Construction for High-Rise Buildings in Sri Lanka by Using Aluminium Panel System Formworks

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Abstract: *Formwork is one of the most important factors in determining the success of a construction project in terms of speed, quality cost and safety of work as it accounts about 40% of the total project cost of the structure. To minimize the costs the contractor needs to complete the project as soon as possible and the client wants the building to use the building as early as possible for the intended purpose. In high-rise building construction the most efficient way to speed up the work is by achieving a very short floor cycle. That directly depends on the selected form work type for the construction. This paper will present about the existing formwork types in Sri Lanka and the available new techniques in formwork erection. This paper will clearly present an analysis and comparison of costs and durations of projects when using different types of formworks. The main objective of this paper is to identify the least no. of typical storeys required in a high-rise building construction project, to use aluminium panel system formwork.*

Keywords: *Formwork system, duration, total project cost, preliminary running cost, typical storeys.*

1. INTRODUCTION

The development of formworks is parallel with the growth of concrete construction throughout past few decades. With the development and increasing of population people tend to construct high-rise buildings and construction of a tall building was not easy at the early days. With the development, the man made the tasks easy by inventing new machinery and new techniques. One such area related to high-rise construction is the type of the formwork used in the construction. At the early days people used conventional type formwork where the timber planks were supported on timber columns. With the advancement of the science man used plywood instead of timber planks and pipe supports with various kinds of jacks instead of timber supports.

Then the man invented small units of formworks when the same structure is repeating such as slab forms, flying forms for the walls…etc. finally the greatest invention came for the complete system. At the beginning the system was made out of steel and which was very heavy. Then the man paid his attention towards reducing the weight of the formwork system. Now the materials for formwork have extended to aluminium, plastic, fiber glass…etc.

But still the aluminium panel system formwork is not much used in Sri Lanka and most of the contractors do not like to shift to the latest technology as they have the doubt of facing losses in the project and they are very much familiar with the existing formwork type, the modern conventional type. At the same time contactors have a false belief that aluminium panel system formwork is only suitable for very tall buildings which are having thirty to forty storeys. Though the aluminium panel system formwork reduces the project duration and hence the total project cost, the formwork system is a bit expensive. So this research was carried out to analyze the cost for each formwork type and compare the values obtained for the total project cost when different types of formworks are used in the construction project and find the least no. of storeys sufficient to use the system formwork in the construction project.

1.1. Available Formwork Types in Sri Lanka

1.1.2. Conventional Type of Formwork

This is the most traditional type of formwork and this uses timber, bamboo, masonry and carpentry to complete construction. Low initial cost, low experience factor and low weight are some of the advantages while high floor cycle, poor finish, and high labor requirement are the disadvantages of this formwork type.

This formwork type is still in practice in two – three storey building construction projects.

1.1.3. Modern Conventional type of Formwork

Modern conventional type formwork is as much the same as the traditional slab formwork method and the only difference is that steel props and various types of jacks (U jacks, T jacks) are used as supports in the formwork instead of timber supports and ply wood sheets are used instead of timber planks on slab decks, beams and columns. The advantages of this type are low initial cost, low skilled labor requirement and can use in places where there are a lot of deviations in the structure. Poor finish, high labor requirement and high floor cycle are some of the disadvantages. This is the most commonly used formwork type in Sri Lanka even in the big buildings.

1.1.4. Semi System Formwork

This is a more advanced formwork type than the modern conventional type as there are pre-fabricated formwork items. For example there are pre-fabricated formworks for slab panels and supports. When using this type of formwork ply wood should be used additionally for slab deck, beams and columns for the surface. There are several types of semi system formwork such as table forms, flying forms…etc. DOCA is the most famous brand for this type of formwork and some people know about this type only as "DOCA formwork".

1.1.5. Aluminium Panel System Formwork

System formwork has prefabricated modular components with casting panels. The system formwork can suit the required shape of concrete structure. The speedy and quality construction is the biggest advantage in this type while high initial cost is the main disadvantage and hence this is not economical to use in low-rise buildings. But this is the most economical form of formwork type to be used in high-rise building construction when it is having few (more that 10) typical storeys.

2. RESEARCH STUDY

This research is the continuation of the MSc. Study done by Eng. Arjuna Gunatilaka. There he has done the study based on more than 25 storeys. The expectation of this study is to find the optimum typical storeys required (least no. of typical storeys) to use an aluminium panel system formwork in a building construction project.

There are many cost components in construction projects contributing to the total cost of the project. Preliminary running cost is a key cost component when comparing the total cost of a project and the duration of the project is the governing factor of the preliminary running cost.

Total Project Cost = Cost for the materials and labour + Preliminary running cost + Machinery and equipment cost + Waste material handling cost + Cost for safety in the site + Cost for finishes + Walking/working platform cost + … (1)

When a project is considered the material requirement is unique as it depends on the design. But the labor requirement and the duration of the project are totally depending on the technologies and the construction methods used in the project. One of the most time consuming activities in a high-rise building construction project is the construction of the main structure. At the same time it requires more labor. But when the latest technologies are used it can reduce both the time and labor requirement in constructing the main structure.

In a high-rise building projects formwork plays a major role as it directly affect the floor cycle and hence it will reduce the time taken to construct the main structure and because of that the total duration of the project will go down drastically. In this research cost comparisons of total project cost has done for several projects when each project is using all the three types of formwork types. (Some of the results are directly taken from the Eng. Arjuna Gunatilaka's MSc. Study and the same thing is done for some other projects which are having different no. of storeys. At the same time the lots of calculations are done to find an optimum no. of storeys (least no. of typical storeys) to use the aluminium panel system formwork in a high-rise building. The calculations are adjusted for different storey levels to find the least no. of typical storeys in a high-rise building. At the same time some relationships of cost components are derived to ease the calculations for future study.

3. THE RESULTS

3.1 The Obtained Results

The calculations are done for several selected high-rise building construction projects when different types of formworks are used in each project and the obtained results can be shown in a tabular form as follows.

Table 1 Summary of the analysis
(Data is taken from Sanken Construction (Pvt) Ltd.)

Project	No. of Storeys	Total project cost (LKR in millions)		
		Aluminium panel system formwork	Semi system formwork (DOCA)	Conventional formwork
Emperor Apartment Tower	35	2102.93	2203.68	2325.26
On three 20 building	38	1849.43	1926.56	2027.52
Green path Hotel complex	25	1864	1942.56	2032.38
Residential apartments at Rajagiriya	22	912.5	957.6	1012.32

3.2 Results for the Emperor Apartment Tower at Kolpity

In this project the used formwork type was semi system (DOCA) formwork and all the values are in LKR and in millions. The building is a 35 storey building. The main contractor for this building is Sanken Construction (Pvt) Ltd. (earlier Sanken Lanka (Pvt) Ltd.).

Table 2 Summary of the analysis (Emperor Apartment Tower)
(Extracted from COST COMPARISON OF FORWORK TYPES FOR ABOVE 25 STOREY BUILDINGS)

	Aluminium panel system formwork	Semi system formwork (DOCA)	Conventional formwork
Duration	14 months	21 months	31 months
Formwork cost	**48.63**	**43.81**	**36.8**
Preliminary cost	144.07	216.1	319
Finishes affected by the formwork	43.34	52.24	69.54
Waste disposal affected by the formwork	0	13.45	19.66
Machinery affected by the formwork	14.45	25.81	40.55
Total	**2102.93**	**2203.68**	**2325.26**

When the results are presented graphically it is easy to compare the results it can be shown graphically as follows.

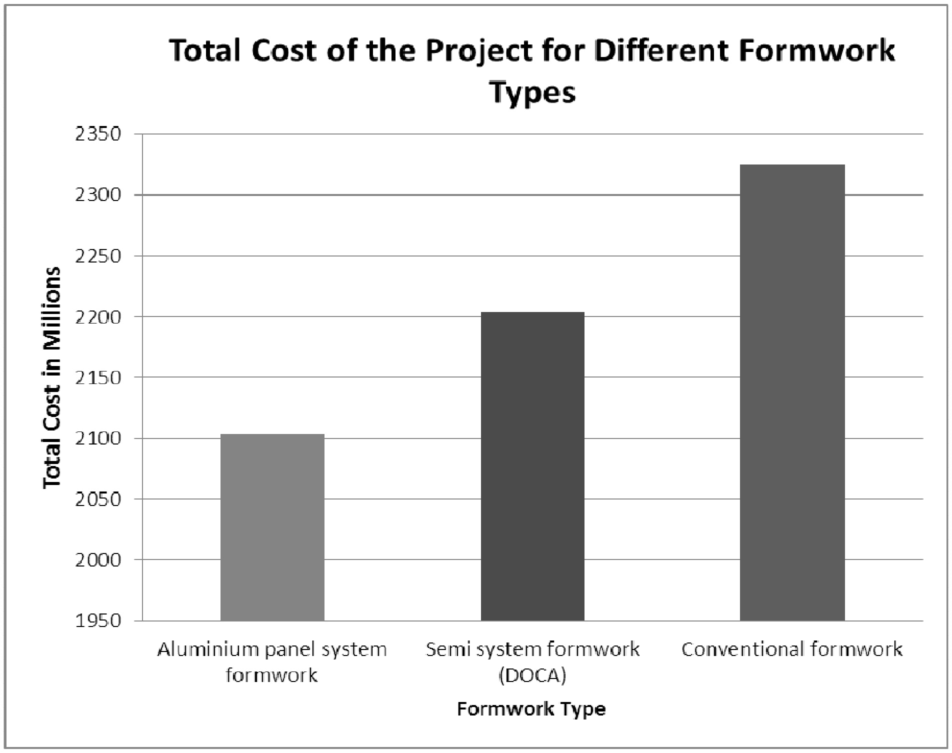

Figure 1: Cost of the total project when using different type of formworks (Emperor Apartment Tower)
(Extracted from COST COMPARISON OF FORWORK TYPES FOR ABOVE 25 STOREY BUILDINGS)

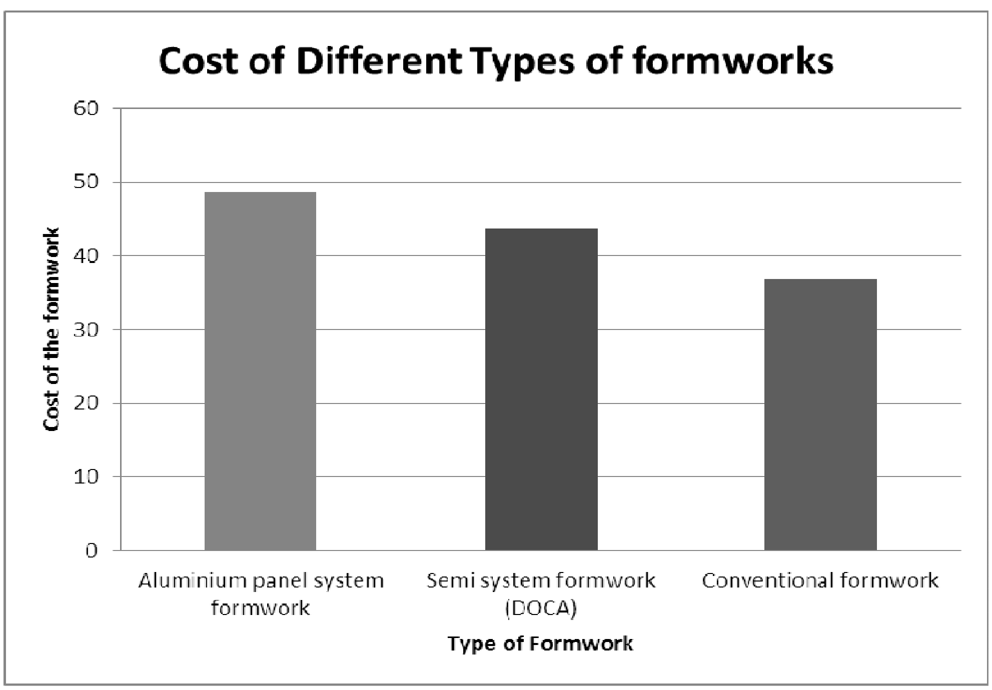

Figure 2: Cost of Different type of formworks (Emperor Apartment Tower)
(Extracted from COST COMPARISON OF FORWORK TYPES FOR ABOVE 25 STOREY BUILDINGS)

The calculations are done for each case assuming the necessary conditions to find the optimum no. of storeys (least no. of typical floors) to use the aluminium panel system formwork. The result obtained was using aluminium panel system formwork is economical for high-rise buildings of more than 10 storeys.

The results obtained for an assumed project of 10 storeys is as follows. Here all the other conditions are taken as same for the above mentioned Emperor Apartment Tower project. In this calculation all the values are in LKR and in millions.

Table 3: Summary of the analysis (Assumed 10 storey building)

	Aluminium panel system formwork	Semi system formwork (DOCA)	Conventional formwork
Duration	8 months	10 months	13 months
Formwork cost	**41.2**	**29.7**	**12.3**
Preliminary cost	82.33	102.90	133.77
Finishes affected by the formwork	12.38	14.93	19.87
Waste disposal affected by the formwork	0	3.84	5.62
Machinery affected by the formwork	14.64	17.8	22.54
Total	**144.11+X**	**166.17+X**	**192.86+X**

where X is the common cost for the building (which are not affecting by the formwork type)

In a construction project most of the other cost items are same. So the total cost of the above project is obtained by adding those costs to the costs which are varying due to the formwork type used in the project. In selecting the aluminium panel system formwork the most important thing to pay our attention is whether the cost saving due to shortening the project duration (preliminary running cost, machinery cost), in less finishing work and in less waste disposal is greater than the excess cost for the formwork system.

The aluminium panel system formwork can be used, so far the following inequality is satisfied for no. of typical storeys

$$(P-Q)\,NA \leq Y\,(D-14N) + M\,(D-14N) + W \qquad (2)$$

where P is the cost for aluminium panel system formwork per square meter, Q is the cost for modern conventional formwork per square meter, N is the no. of typical storeys, A is the floor area of a typical floor in square meters, D is the project duration in days, Y is the preliminary running cost per day, M is the machinery cost per day and W is the cost saving in waste disposal.

4. CONCLUSIONS

From the results it is clearly seen that the total project cost is vary as follows for the building which are having more than 10 typical storeys.

Modern Conventional Formwork > Semi System formwork > Aluminium Panel System Formwork (3)

And the most important thing obtained this study is that identifying of the optimum no of storeys for use aluminium panel system formwork in the high-rise building construction. That is if there are 10 or more typical storeys in the building the most economical formwork type to use is aluminium panel system formwork.
If the construction is properly controlled this is worth even in buildings which are having less than 10 typical storeys. But for the Sri Lankan context; with less experience and less technology it is recommend to use aluminium panel system formwork in buildings which are having 10 or more typical storeys.

Not only the seen benefits, there are many unseen benefits of using aluminium panel system formwork in the high-rise building construction project.

For example the system can be sold after using, it can be used in other slabs doing slight variations (there is no need to buy materials again), less waste disposal hence the cost for waste disposal is saved and can obtain a high quality concrete surface and due to that less finishing work will be there and hence the cost for finishes will be reduced.

5. ACKNOWLEDGMENTS

Apart from the efforts done by me, the success of this research paper depends largely on the encouragement and guidelines of many others. I take this opportunity to express my gratitude to the people who have been supportive hands in doing this research paper

I would like to show my greatest appreciation to Eng. Arjuna Gunatilaka for giving me required data from his MSc. Study “COST COMPARISON OF FORWORK TYPES FOR ABOVE 25 STOREY BUILDINGS” and for giving me the opportunity to collect data for this research paper from the construction sites of Sanken Construction (Pvt) Ltd. The guidance and support received from all the other people were vital for the success of the research paper. I am grateful for their constant support and help.

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Study on moisture loss and drying shrinkage behaviour of mortar with mineral admixture based on pore structure

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Abstract: *The moisture loss and drying shrinkage behavior of mortar with mineral admixture such as blast-furnace slag and fly ash are investigated focusing on the microstructure affected by curing temperature. The curing at high temperature greatly accelerates the hydration reaction of the mortar with mineral admixture to make pore structure denser and leads to smaller moisture loss than that of mortar without mineral admixture. The moisture loss is almost proportional to the accumulated volume of pores whose radii are larger than the radius at the liquid/vapour interface based on the Kelvin equation and BET theory of absorbed water. When the shrinkage is assumed to be induced only by the capillary tension, the estimated shrinkage shows different tendency from experimental one. It suggests that the other shrinkage driving forces that have been believed to be dominant under severe drying conditions should be taken into account even under normal drying condition (RH=60% at 20 degree).*

Keywords: *Granulated blast-furnace slag, Fly ash, Shrinkage, Moisture loss, Curing temperature*

1. INTRODUCTION

Mineral admixtures such as blast-furnace slag and fly ash have been widely used replacing with cement in Japan in order to reduce CO_2 emission arising from the cement production. Since they retard the hydration reaction and decrease the hydration heat, the application is mainly to the massive concrete for dam or underground structures. It has been reported, however, that numerous shrinkage cracks were found in the massive concrete structures using the slag cement. Although the slow strength development, small tensile creep, large autogenous shrinkage and others due to the slag replacement are considered as the reasons for the cracking, it has been not consistently understood since they are strongly dependent on mix proportion, curing, and boundary conditions. In the case of the concrete containing fly ash, such serious problems have not yet been reported but it is important to comprehend the possibility of the shrinkage cracking consistently in order to avoid the unexpected damage in the structures.

The hydration reaction of the concrete with mineral admixtures is dependent on the curing temperature. The internal part of the massive concrete structure and the precast concrete with admixture that has been recently used are exposed to high temperature at early ages and the microstructure at elevated temperature could be changed. In this paper, the authors study the influence of curing temperature on moisture loss and drying shrinkage of mortar with blast-furnace slag and fly ash focusing on the microstructure.

2. EXPERIMENTAL PROGRAM

2.1. Materials

The mortar with or without mineral admixture was cast. The mix proportions are given in Table 1. Water-binder ratio is 50 % and the volume ratio of the fine aggregate is 40 % in all specimens. The specimen without mineral admixture is named OPC, while the specimens containing blast-furnace slag and fly ash are referred as BS30 and FA20, respectively. The weight replacement ratios of slag and fly ash with cement are 30% and 20%, respectively.

Table 1 Mix proportions of mortar (kg/m^3)

Specimen	Water to Binder ratio	Water	**Cement** Ordinary Portland cement Specific gravity: 3.15 Blaine specific surface: 3260cm^2/g	**Slag** BF4000 slag Specific gravity: 2.89 Blaine specific surface: 4440cm^2/g	**Fly ash** II type of fly ash Specific gravity: 2.24 Blaine specific surface: 4130cm^2/g	**Fine aggregate** River sand Specific gravity: 2.52
OPC	50 %	367	734	—	—	1028
BS30	50 %	363	509	218	—	1028
FA20	50 %	356	569	—	142	1028

2.2. Moisture Loss and Drying Shrinkage Test

Prismatic mortar specimens with 40 x 40 mm cross section and 160 mm length were used for the test. The form was removed one day after casting and then all specimens were adequately cured in water (submerged) at 20 or 60 ºC. At seven days of age, each specimen was dried at a relative humidity of 60% and 20 ºC in the controlled chamber. Although the water curing at 60 ºC is not probable in reality, the extreme condition was provided in order to clearly comprehend the phenomena and the mechanism which affect the moisture loss and drying shrinkage. During drying, moisture loss and shrinkage were simultaneously measured using the same specimen. Moisture loss was obtained by dividing the reduction of the specimen weight after the curing by the volume. Longitudinal length change at 100 mm center part on the surface of the specimen was measured with the contact gauge having 0.001 mm resolution. All of results were obtained by averaging values of three specimens.

2.2. Mercury Intrusion Porosimetry (MIP) Test

The mercury intrusion porosimetry (MIP) test was curried out to examine the pore structure of each specimen. The 56 days dried specimens used for moisture loss and shrinkage test were stored under sealing condition at 20 ºC for about 2 months and then crushed into small pieces from 4 mm to 5 mm. The small pieces were exposed to D-dry (vacuum drying at –80 ºC) for 120 hours to measure the pore structure with mercury intrusion porosimeter. The surface tension of mercury and the contact angle between mercury and sample are set to 0.484 N/m and 130°, respectively. Assuming that the pore structure is assumed to be cylinder, the volume of each pore size from 3.3 nm to 360 μm was measured and obtained averaging the two measured values.

3. EXPERIMENTAL RESULTS AND DISCUSSION

3.1. Moisture Loss

Firstly, the influence of the curing temperature on moisture loss at drying is discussed. Figures 1 and 2 represent the moisture loss of mortar exposed to the drying condition (RH=60% at 20 °C) after 7 days curing in water of 20 and 60 °C, respectively. The 20 °C curing condition provides the smallest moisture loss of OPC mortar, while the moisture loss of OPC specimen is the largest in the case of 60 °C curing. The slower hydration reaction of specimen with mineral admixture under normal temperature makes microstructure coarser after the curing at 20 °C and results in larger moisture loss of mortars with slag and fly ash than that of OPC specimen. On the other hand, the reaction of BS30 and FA20 specimens due to the latent hydraulic property of slag and pozzolanic reaction with $Ca(OH)_2$ is promoted during 7 days curing in 60 °C water and denser pore structure leads to retard of moisture evaporation from the specimens. When the fly ash is contained in the mortar, especially, the reaction at the elevated temperature could be significantly accelerated and results in the considerable reduction of the moisture loss comparing to that after 20 °C curing. It indicates that the temperature sensitivity of hydration reaction of fly ash cement is more significant than that of slag cement.

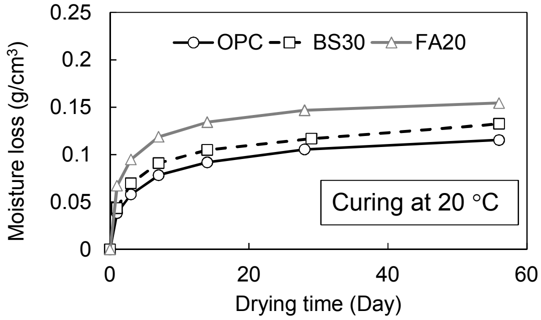

Figure 1 Moisture loss of each specimen (Curing at 20 °C)

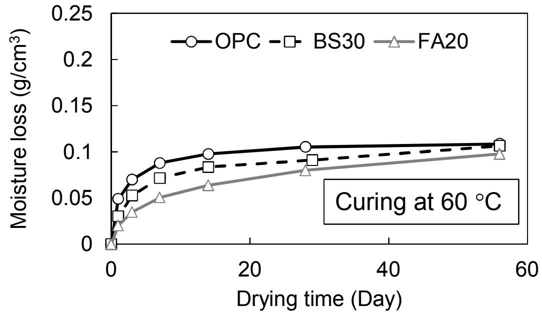

Figure 2 Moisture loss of each specimen (Curing at 60 °C)

Figures 3 and 4 show the pore size distribution obtained by MIP test. There is more volume of relatively large pores with more than 10 nm in mortars with slag and fly ash than that in OPC specimen. The curing at high temperature fills the relatively large pores with hydration products and increases the volume of fine pores with less than 10 nm size as shown in Figure 4. The fine pores are formed the most in the case of FA20 specimens. The result of the pore size distribution also indicates that the reaction of fly ash cement is the most sensitive to curing temperature and the promotion makes the pore structure dense.

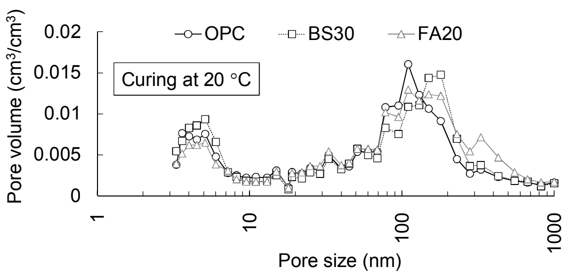

Figure 3 Pore size distribution of each specimen after drying (Curing at 20 °C)

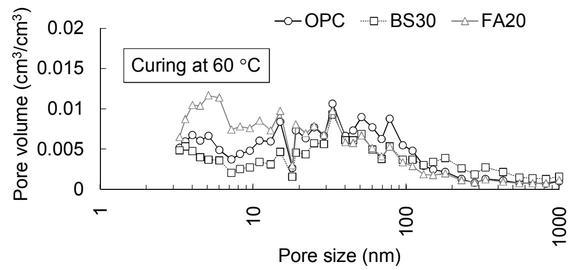

Figure 4 Pore size distribution of each specimen after drying (Curing at 60 °C)

Next, the moisture loss behavior under the drying of RH=60% at 20 °C is discussed based on the pore structure. According to Maekawa et al. (1999), the pore radius r_c, which is at the equilibrated interface between liquid and vapor, is identified as shown below.

$$r_c = Cr_s, where\ \ C = 2.15 \qquad 1)$$

$$r_s = \frac{2\gamma M_w}{\rho RT \ln h} \qquad 2)$$

where, γ: surface tension of water [N/m], ρ: density of liquid water [kg/m^3], R: the gas constant [J/mol/K], T: absolute temperature [K], M_w: molecular mass of the water [kg/m^3], and h: relative humidity, and the constant C has been obtained after numerous comparisons of the analytical predictions of the equilibrated interface radii, r_c, as obtained from the modified BET theory and r_s as given by Kelvin equation. Since the r_c under the drying condition (RH=60% at 20 °C) is calculated as about 4.5 nm according to Eq3. (1), the moisture existed in the pores larger than 9 nm could evaporate at equilibrium of the drying. Thus, the volume of pores larger than 9.5 nm obtained by the MIP test is accumulated and the results of all specimens are summarized as shown in Tables 2 and 3.

In the case of BS 30 and FA20 specimens, the accumulated volume of the pores larger than 9.5 nm is decreased when the water curing at 60 °C is provided, while the pores volume of OPC specimen is not so different in curing temperature. This result also suggests that the evaporable water which existed in the pores larger than 9.5 nm is reduced after curing at elevated temperature in the case of mortar with mineral admixture and results in the smaller moisture loss than that of mortar without admixture, as mentioned above.

The order of the accumulated volume of pores larger than 9.5 nm in each specimen cured at 20 °C is almost proportional to the order of moisture loss as shown in Figure 1. Although the pores volume larger than 9.5 nm in BS30 specimen is slightly smaller than that in OPC specimen with larger moisture loss, it could be attributed to the gradual pore structure formation of BS30 specimen during drying due to slower hydration reaction. The coarser pore structure of BS30 specimen at the beginning of drying after the curing could disperse more moisture than OPC specimen and lead to larger moisture loss even at 56 days of drying.

The accumulated volume of pores larger than 9.5 nm in BS30 specimen cured at 60 °C is much smaller than that of FA20 specimen but the moisture loss of BS30 specimen is larger than that of FA20 specimen. It could be ascribed to the moisture trapped in the inkbottle-shaped pores as shown in Figure 5. It is reported (Maekawa et al 1999) that the probability water entrapment in a pore of radius r larger than the pores of radius r_c would be dependent on the accumulated volume of the pores whose radius is smaller than r_c (This probability means to be dependent on the chance of intersection of the larger pores with the smaller completely filled pores). Table 2 indicates that the volume of fine pores whose radius is smaller than r_c in FA20 specimen is much more than that in BS30 specimen. Thus, such more fine pores of FA20 specimen trapped more moisture in the pores with the radius larger than r_c and leads to smaller moisture loss than that of BS30 specimen. It is concluded that the equilibrated moisture loss of each specimen with different mix proportions and curing temperature can be almost interpreted considering the accumulated volume of pores with radius larger or smaller than the equilibrated interface radius r_c.

Table 2 Accumulated volume of pores from 3.3 nm to 9.5nm and 9.5 nm to 360 μm (Curing at 20 °C)

Specimen	Pore size from 3.3 nm to 9.5nm (cm^3/cm^3)	Pore size from 9.5 nm to 360 μm (cm^3/cm^3)
OPC	0.0429	0.1654
BS30	0.0499	0.1647
FA20	0.0367	0.1841

Table 2 Accumulated volume of pores from 3.3 nm to 9.5nm and 9.5 nm to 360 μm (Curing at 60 °C)

Specimen	Pore size from 3.3 nm to 9.5nm (cm^3/cm^3)	Pore size from 9.5 nm to 360 μm (cm^3/cm^3)
OPC	0.0434	0.1673
BS30	0.0306	0.1419
FA20	0.0741	0.1548

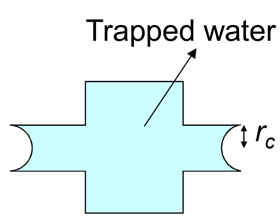

Figure 5 Trapped water in inkbottle-shaped pores

3.2. Drying Shrinkage

The results of drying shrinkage of each specimen are shown in Figures 6 and 7. The drying shrinkage is not so different in each specimen. Focusing on the curing temperature, when specimens are cured at 60ºC, the drying shrinkage is smaller than that of specimens that are cured at 20 °C in all cases. As mentioned above, the curing at elevated temperature can promote the hydration process and achieve fine pore structure. It is concluded that dense pore structure due to the progressive curing can cause larger elastic stiffness and smaller moisture loss and result in smaller shrinkage of specimens.

Here, the shrinkage behaviour of each specimen is discussed only focusing on the capillary tension theory (Powers, 1968) that has been believed to be the most dominant under normal drying condition (room temperature and moderate relative humidity). The capillary tension P_c at equilibrium is expressed using the parameters in Eq. (2) as below.

$$P_c = -\frac{2\gamma}{r_s} \tag{3}$$

The capillary force which acts in the pores is proportional to the surface area of pores where moisture exists. Thus, the shrinkage stress S_c arising from the capillary tension in saturated pores is idealized as,

$$S_c = P_c \frac{A_s}{A_t} = -\frac{2\gamma}{r_s}\frac{A_s}{A_t} \tag{4}$$

where, A_s is the surface area of pores where moisture exists at equilibrium and A_t is the total surface area of pores. Since the surface area of pores is dependent to the pores volume, in the paper, the surface area fraction A_s/A_t is assumed to simply be proportional to the pores volume fraction V_s/V_t (V_s: accumulated volume of pores larger than 9.5 nm(≈2 x r_c), V_t: total accumulated pores). Table 3 summarizes the V_s/V_t of each specimen.

Since P_c is the same at equilibrium in all cases, the shrinkage stress due to the capillary tension could be dependent on the V_s/V_t. The amount of the shrinkage at equilibrium also depends on the Young's modulus of the specimens. The Young's modulus after 56 days of drying was measured using ϕ50 x 100 mm cylinder specimens and the results are given in Figure 8. According to Table 3 and Figure 8, when curing temperature is 20 °C, BS30 with the largest V_s/V_t and relatively small Young's modulus should shrink the most but such tendency was not observed in reality. In the case of curing at 60 °C, the V_s/V_t of FA20 specimen is 1.5 times larger than that of other specimens and leads to about 1.5 times larger shrinkage based on the capillary theory, because the Young's modulus is not so different in each specimen. In the experiment, however, the shrinkage of FA20 specimen at equilibrium is slightly larger than that of other specimens. Consequently, it is difficult to explain the amount of shrinkage at equilibrium only from a viewpoint of capillary theory. Other shrinkage mechanisms such as disjoining pressure and surface energy of C-S-H gel (Powers, 1968) which have been believed to be dominant under severe drying condition should be taken into account for consistent discussion of shrinkage behavior depending on the mix proportion and curing temperature.

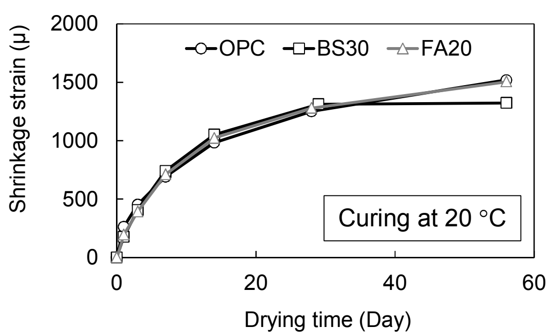

Figure 6 Drying shrinkage of each specimen (Curing at 20 °C)

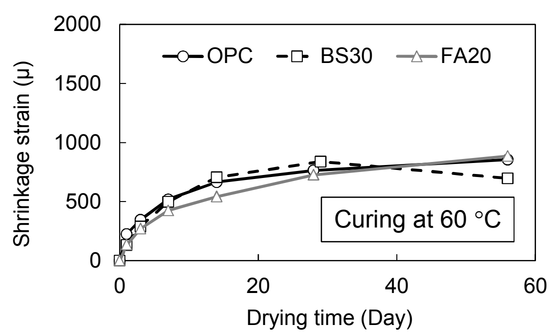

Figure 7 Drying shrinkage of each specimen (Curing at 60 °C)

Table 3 V_s/V_t of each specimen

Curing temperature	Specimen	V_s/V_t
20 °C	OPC	0.206
	BS30	0.233
	FA20	0.166
60 °C	OPC	0.206
	BS30	0.177
	FA20	0.324

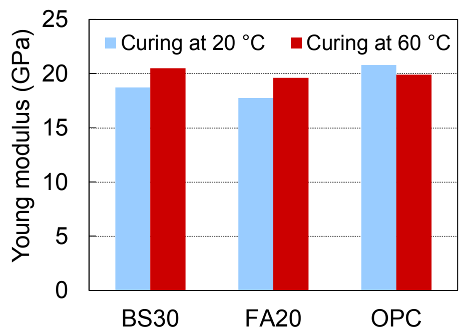

Figure 8 Young's modulus of each specimen after curing

4. CONCLUSION

In this paper, the moisture loss and drying shrinkage behavior of mortar with blast-furnace slag and fly ash are discussed from a viewpoint of the microstructure affected by curing temperature. Key findings are summarized below.

i) The curing at high temperature greatly accelerates the hydration reaction of the mortar with mineral admixture to make pore structure denser and leads to smaller moisture loss than that of mortar without mineral admixture.
ii) The equilibrated moisture loss of each specimen with different mix proportions and curing temperature can be almost estimated based on the accumulated volume of pores with radius larger or smaller than the equilibrated interface radius between liquid and vapor.
iii) When the shrinkage is assumed to be induced only by the capillary tension, the estimated shrinkage from the shrinkage force and Young's modulus shows different tendency among mix proportions from experimental one. It suggests that the other shrinkage driving forces that have been believed to be dominant under severe drying condition should be taken into account even under normal drying condition of RH=60% at 20 °C.

5. ACKNOWLEDGEMENT

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Influence of Fine Aggregate Types on the Performance Self Flowing Concrete

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Abstract: *Self-compacting concrete (SCC) was first developed to achieve durable concrete structures and help cast concrete into complex geometries without compromising the quality of the cast. This research is carried out to understand the influence of locally available fine aggregate types (river sand, quarry dust and offshore sand) on the properties of SCC concrete. As the fine aggregate proportion to coarse aggregate is considered important for the rheology, three different aggregate proportions from each of the fine aggregate type were investigated. In order to evaluate influence of aggregate proportions on the compressive strength, three w/c ratios was considered for each aggregate proportion, bringing the number of mixes for a fine aggregate type to nine and the total number of mixes in the investigation for the three different aggregate types to 27. As the particle size distributions of the different fine aggregate types are different to one another, a separate series with different aggregate types manipulated to confirm to a single particle size distribution was also carried out. This study was limited to single aggregate proportion and hence only nine additional mixes were resulted for the three aggregate types. The influence of fine aggregate type and proportion, on the harden properties of concrete is evaluated in terms of compressive strength and shrinkage of concrete. In addition, water requirement under constant doze of viscosity modifying agent is taken to evaluate the performance of mixes in fresh state. Results of the study indicated that quarry dust as fine aggregate has highest 28 days compressive strength for all the different water cement ratios. All fine aggregate types have recorded higher strength when the proportion of fine aggregate to total aggregate content is 60%. Offshore sand mixes recorded the lowest shrinkage and also lowest water content to achieve the conformity requirement of self-compacting concrete. Although quarry dust required less water compared to river sand, it had the highest shrinkage among the three aggregate types.*

Keywords: Self-compacting concrete (SCC), Concrete mix design, Offshore sand, River sand, Quarry dust.

1. BACKGROUND

Requirements to cast concrete into intricate shapes, complex geometries and sections of highly congested reinforcement arrangements are common demands in today's construction industry. Ensuring durability in complex casts is a major challenge for the engineers. Concrete mixes with self-compacting properties that allows concrete compact itself into complex geometries with minimum effort is one solution for such demanding conditions. In late 1980's and early 90's research lead by Prof. H. Okamura of the University of Tokyo pioneered in the development of such mixes of concrete with high fluidity which they coined as Self Compacting Concrete (SCC)(Okamura 2003, Oguchi M, et. al. 1996). SCC can be described as a high performance material which flows under its own weight by completely filling of formworks even when access is hindered by narrow gaps between reinforcement bars. SCC can also be used in situations where it is difficult or impossible to use mechanical compaction for fresh concrete, such as underwater concreting, cast in-situ pile foundations, machine bases and columns or walls with congested reinforcement. There are numerous applications of self- compacting concrete world over. Fig. 1, the anchorage block of the longest cable stays bridge, Akashi kaikyo, japan is one such well documented use of SCC. As mixing and casting for SCC has to be done in much control environment, which can be easily achieved in precast site, use of self-compacting concrete has become the preferred choice in the precast industry today (Brameshuber, W. et. al. (2002)). However, application of self-compacting concrete in Sri Lankan construction and precast industry are still very limited. The silting chamber of the Upper Kothmale Dam is one of limited examples of a recent application of SCC mixes in Sri Lanka. Due to lack of usage of SCC, influence of locally available aggregate types in making SCC and their influence on fresh and hardened properties of SCC has not been comprehensively studied and understood.

Figure 1 Anchorage block of Akashi Kaikyo bridge

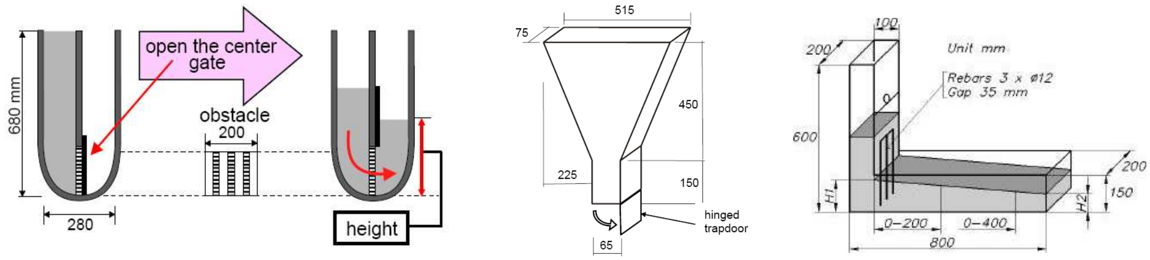
As self-compacting concrete essentially means flowing concrete, the usual devises that measure workability of normal concrete mixes like slump test, compaction factor test, and VB time test becomes unsuitable for SCC. Fig. 2 gives a glimpse of contrasting workability achieved by the normal and self-compacting concrete. To be able to measure and compare the performance of self-compacting concrete, researches have come up with different devices and measurements to explain the rheology of self-compacting concrete.

Normal concrete: Slump test	Self-compacting concrete Slump flow test	Self-Compacting concrete V Funnel test	Self-compacting concrete U-Box test

Figure 2 Measure of workability of the different concrete

U-Box test, funnel test and J-ring test are some of the other examples of test for determining the workability or fluidity of the self-compacting concrete. Each different test attempts to look into different aspects of the self-compacting concrete namely; filling capacity, passing ability and segregation resistance.

In the U box test difference in the height of concrete in the two legs of a standard U tube is taken as a measure of the workability of concrete. As it uses an obstacle in the form of equally spaced bars between the two legs, it essentially mimics the flow through congested reinforcement arrangement and measures the passing ability SCC (see Fig. 3a)). In the funnel test, time taken to empty the funnel is taken as a measurement for the workability of the concrete mix (see Fig. 3b)). Funnel test looked at the segregation resistance of the SCC mix. Similar to the U box test, L box test also employs an obstacle and measures the passing ability of SCC and uses the ratio of the concrete height in the horizontal leg as indication of the workability of the mix (see Fig. 3c)). Professional bodies like JSCE (JSCE 2005) have come forward to recognize such testing procedures and standardize the same so that they can be adopted universally to compare self-compacting concrete.



4 a) U box test 4 b)Funnel test 4 c)L box test

Figure 3 Workability measuring tests for self-compacting concrete

Table 2 standard specification

Method	Unit	Typical range of values	
		Min.	Max.
Slump Flow	mm	650	800
T_{50cm} slump flow	S	2	5
U-box (h_1-h_2)	mm	0	30
V-funnel	S	6	12
J-ring	mm	0	10

2. INTRODUCTION

Gneisses and Charnockite form crushed rocks is our main source of coarse aggregate and is available in abundance. River sand is the first choice fine aggregate type for structural concrete in Sri Lanka. However, due to scarcity and regulation in sand mining, quarry dust and offshore sand are now being increasingly used as an alternative to fine aggregate in concrete. River sand has long been considered as an unsustainable solution for the fine aggregate requirement of the country. Quarry dust which is a by-product of metal crushing amounts only between 20-30% of the total aggregate being crushed. Furthermore demand for lot of other uses like masonry block and paving block making, quarry dust doesn't provide a comprehensive solution for the fine aggregate requirement of the country. However, being an island nation with substantial offshore sand deposits, offshore sand has a great potential to cater the country's demand for fine aggregate. Fig. 5 shows the usual sources of fine aggregate (River sand) and coarse aggregate (Crushed stones) of concrete and alternative sources of fine aggregate types (Quarry dust and Offshore sand) for concrete.

Crushed stones (Coarse) River sand (Fine) Quarry dust Offshore sand

Figure 4 Aggregate for concrete

The three fine aggregate types used for concrete in this study largely differ from one another in terms of the particle size distribution, shape, surface texture and moreover by its geological formation. These aggregate characteristics as well as its proportions used in concrete, have a significant impact on filling ability, segregation resistance, and passing ability of self-compacting concrete. To find the influence of these aggregate parameters on flow characteristics and hardened properties of concrete is the objective of this research. In the experimental program usability of locally available fine aggregates (such as river sand, offshore sand, and quarry dust) in enhancing flowing characteristics are tested and compared with one another.

Fig. 6 shows the particle size distribution of the different fine aggregate types; river sand, offshore sand and quarry dust compared with upper and lower bounds of BS882 requirement of fine aggregate in concrete(British standard Institute 2002). Though it is clear that all the fine aggregate conform to the said requirement and falls between the upper and lower limits, they have different finesses to one another. Offshore sand being the finest of all the fine aggregate types came on top among the particle size distribution curves, followed by river sand. Quarry dust recorded the coarsest particle size distribution.

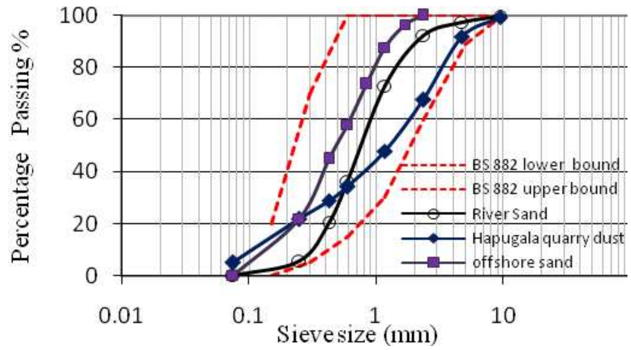


Figure 5 Particle size distribution of different fine aggregate types.

Other indicators of fineness like fineness fraction which is defined as 600 μm passing aggregate content as a percentage of the total fine aggregate content is found to be 60% for offshore sand while the value is only about 30% for the other two aggregate types. The British method of mix selection (Department of Environment (1975)), which employs the 600 μm passing measurement to find the fine aggregate proportion from the total aggregate content, suggest that higher the fineness fraction lesser the quantity of fine aggregate required to make good concrete.

3. SIGNIFICANCE OF THE STUDY

Influence of locally available fine aggregate types on the performance of self-compacting concrete has not been sufficiently studied. There is also a reluctance to use offshore sand and quarry dust as an alternative to river sand in concrete. High Cl- concentration of offshore sand seems not be the only reason not to use offshore sand for concrete. Users have no confidence in some of the important properties of concrete, like strength development and shrinkage characteristics, when alternative aggregate types are used.

Research on the influence of fine aggregate on the properties of normal concrete reveals that in term of strength both offshore sand and quarry dust perform better than river sand. It is considered that both aggregate packing and particle size distribution is the main explanation for the higher strength in the offshore sand mixes for normal concrete (Alluthwatta AGHAD et. al. 2011). Extending this study to self-compacted concrete is important as SCC uses more fine aggregate than normal concrete and therefore the influence of fine aggregate can be even more significant. From the results of normal concrete; offshore sand recorded the highest strength while quarry dust came second (Alluthwatta AGHAD et. al. 2011). However, in SCC the highest compressive strength was recorded in quarry dust mixes while offshore sand and river sand recorded almost similar compressive strengths lower than the quarry dust mixes. This is a strong suggestion that aggregate influence attributed to packing of aggregate has only secondary influences in SCC. In normal concrete quarry dust recorded the highest water demand for all workability ranges tested. However this was not to be case for SCC. In SCC river sand has recorded the highest water demand. However similar to normal concrete, quarry dust mixes recorded the highest shrinkage. Larger shrinkage under relatively lower water content suggests clear influence of quarry dust on shrinkage characteristics of concrete. Differences in the results in the normal concrete compared with SCC in many ways suggest that particle size distribution has only secondary influences on the properties of SCC. Almost similar results obtained between the mixes with natural particle distribution and particle size distribution manipulated to have identical particle size distribution provide further evidence for lesser influence of particle size distribution on the properties of SCC.

4. METHODOLOGY

Establishment of minimum fine aggregate proportion is the first step in the process of selecting aggregate proportions for the self-compacting concrete. To this end different fine aggregate proportions were tested under a constant doze of viscosity modifiers to find the minimum proportion of fine aggregate required to make SSC. U Box was used as the preferred mode to test the conformity of the self-compacting concrete. Once conformity is ensured by U box the mixes were checked for segregation resistance using the funnel test and filling capacity based on the slump flow test. With offshore sand as fine aggregate, it is found that 50% of offshore sand by total aggregate weight is the minimum fine aggregate percentage for making self-compacted concrete. This percentage is found to be 55% for river sand and 60% for the quarry dust. All these results were obtained for Glenium C320 doze of 1500 ml per 100kg of cement. With this initial results of minimum percentage of fine aggregate required under different aggregate types, common fine aggregate proportions to total aggregate proportions of 60%, 70% and 80% was considered to study the influence of fine aggregate type on the properties of self-compacting concrete. As self-compacting concrete requires only conformity to the workability requirement specified in the table 1, mixes with different workability ceases to become a variable for studying the SCC. This makes aggregate type, fine aggregate proportion to total aggregate, and the w/c ratio the only variables for the study. Fig.6 show the different mixes studied for determining the influence of fine aggregate types on the properties of SCC. To make mixes with different aggregate and mixes of same aggregate with different w/c ratio comparable to each other same doze of viscosity modifier, 1500 ml for 100kg of cement is used for all mixes. In order to achieve the desirable workability under the constant doze of viscosity modifier water content was adjusted. The results of different water quantities required to achieve the required fluidity are then used as an indicator for the performance of different aggregate types.

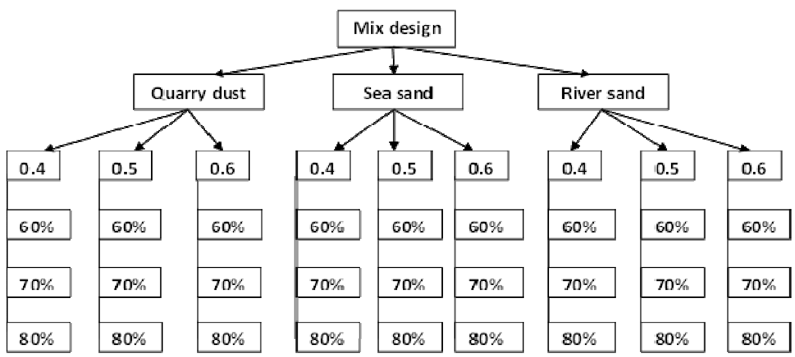


Figure 6 Different mixes considered in the study.

In addition to the main study to find the aggregate influence on the properties of concrete, a separate study to understand what might have caused it, or to understand the influence of particle size distribution on the properties of concrete, a separate mixer series with particle size distribution of the different fine aggregate manipulated to bring to a single size is also carried out. Single particle size distribution is achieved by first sieving the aggregate into different aggregate sizes and then remixing it to a predetermined aggregate distribution. In this study particle size distribution of quarry dust and river sand is manipulated to bring them to the same particles size distribution of offshore sand. Given the large quantities of fine aggregate types to be sieved in order to make them into a single particle size distribution, only 70% fine aggregate proportions to total aggregate content was considered in this series. The main comparison of the aggregate influence is done based on the compressive strength and water demand to make self compacting mixes under constant doze of viscosity modifier agent. In addition aggregate influence on shrinkage characteristics of concrete is also measured.

5. RESULTS AND DISCUSSION

5.1. Water demand of the mix

Table 2 show the different water content for all the different aggregate types. In this experimental investigation of SCC under constant dosage of viscosity modifying agent, river sand mixes recorded the highest water demand to achieve the required fluidity while offshore sand recorded the lowest. Water demand can be influenced by many factors. Fineness is one factor. Higher the fines essentially mean more surface area to volume and therefore require more water to wet the additional surface area. Texture or the roughness of the aggregate surfaces is another factor that influences the water demand. Rough surface textures provide more surface area to hold water around the aggregate and requires addition water to overcome the surface friction created by the roughness of the aggregate.

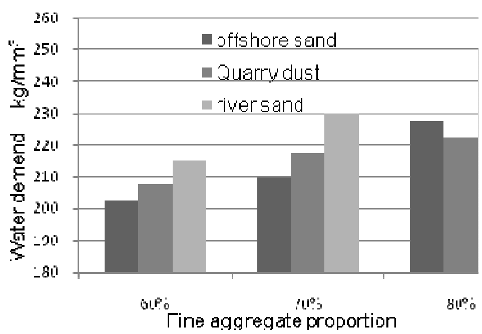


Figure 7 The increase in water demand with the increase in aggregate proportions (w/c =0.7)

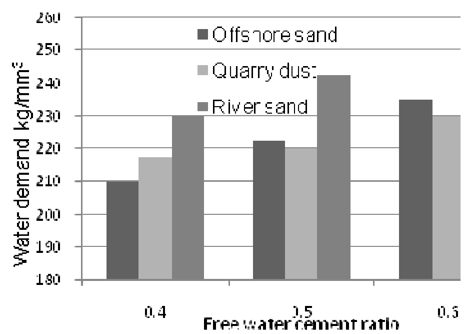


Figure 8 The increasing water demand with the increasing w/c ratio for 80% fine proportion.

Table 2. Results of the concrete mixes series

Mix name	W/C	Water demand kg/m³	28 days kN/mm²	increment % compared to River sand
OS/0.4/60%	0.4	202.5	74.63	7.25%
OS/0.4/70%	0.4	210	68.89	2.57%
OS/0.4/80%	0.4	227.5	66.88	3.69%
OS/0.5/60%	0.5	210	56.83	-7.18%
OS/0.5/70%	0.5	222.5	52.10	-7.51%
OS/0.5/80%	0.5	230	50.20	-3.50%
OS/0.6/60%	0.6	215	46.99	4.95%
OS/0.6/70%	0.6	235	43.67	2.63%
OS/0.6/80%	0.6	250	41.65	11.66%
RS/0.4/60%	0.4	215	69.58	0.00%
RS/0.4/70%	0.4	230	67.16	0.00%
RS/0.4/80%	0.4	250	64.50	0.00%
RS/0.5/60%	0.5	225	61.23	0.00%
RS/0.5/70%	0.5	242.5	56.32	0.00%
RS/0.5/80%	0.5	252.5	52.02	0.00%
RS/0.6/60%	0.6	235	44.77	0.00%
RS/0.6/70%	0.6	245	42.55	0.00%
RS/0.6/80%	0.6	255	37.30	0.00%
QD/0.4/60%	0.4	207.5	83.33	19.76%
QD/0.4/70%	0.4	217.5	79.94	19.01%
QD/0.4/80%	0.4	222.5	75.31	16.75%
QD/0.5/60%	0.5	215	65.50	6.97%
QD/0.5/70%	0.5	220	60.30	7.06%
QD/0.5/80%	0.5	227.5	56.07	7.78%
QD/0.6/60%	0.6	222.5	46.98	4.93%
QD/0.6/70%	0.6	230	42.57	0.05%
QD/0.6/80%	0.6	242.5	37.21	-2.40%

Fig. 7 shows the increase in water demand for the different fine aggregate types with the increase in aggregate proportions recorded at 0.6 w/c ratio. This trend is found to be true for all the w/c ratio tested. Fig. 7 also shows that the river sand requires higher water content for all the aggregate proportions. Fig. 8 shows the water demands for different w/c ratios recorded for 60%fine aggregate proportion to total aggregate content. With the increase in w/c ratio cement content and therefore the admixture dosage in the mix is expected to be dropped. As earlier mentioned, fluidity in SCC is effected by both water content & admixture dosage. Therefore this higher water demand for the higher w/c ratio is attributable to regain the effect of admixture dosage.

5.2 Compressive strength of concrete

Compressive strength of aggregate is one of the most important properties of concrete and accordingly more attention was given to determine the aggregate influence on the compressive strength of concrete. Along with the water demand Table 2 provides 28 days compressive strength of all the mixes conducted in this experimental investigation. For all SCC mixes cast at different aggregate proportions and water cement ratio, quarry dust mixes have recorded the highest strength. Offshore sand has come next while river sand has recorded the lowest 28 day compressive strength. Fig. 9 shows the strength vs. w/c ratio for quarry dust conducted for different fine aggregate proportions 60%, 70% and 80%. Results clearly indicate that lower fine aggregate proportion (60% of fine aggregate content to total aggregate content) have resulted higher strength. It is also found that same relationship is true for the other aggregate type. Fig. 10 shows the strength vs. w/c ratio of the mixes with 70% fine aggregate proportion, with natural particles size distribution and particles size distribution manipulated to match offshore sand distribution. Similar results obtained by the two particles size distribution of the single aggregate type indicate that particles size distribution has only secondary influence on strength of SCC.

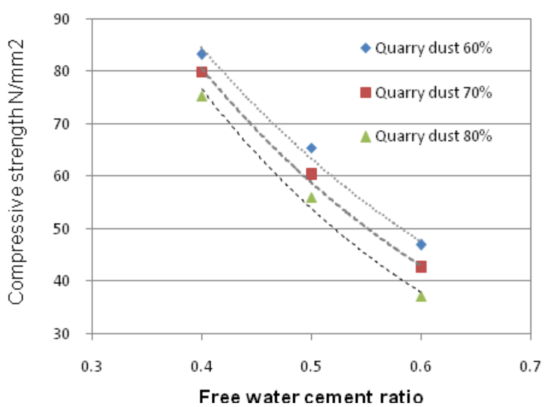


Figure 9 Strength vs. water cement ratio for different proportions of quarry dust

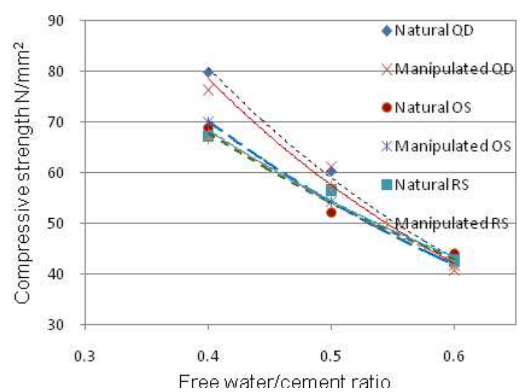


Figure 10 Strength vs. water cement ratio for different particle size distributions

Fig. 11 shows the average strength vs. water cement ratio for the different aggregate types, collated and averaged disregarding the proportion of fine aggregate to total aggregate content. From the average results of strength vs. water cement ratio for different aggregate type it is clear that quarry dust record higher strength compared to other aggregate types. Quarry dust recording higher strength than all other aggregate types is further confirmation of the less sensitivity of particle size distribution and packing on the properties of SCC. Fig. 12 shows strength vs. w/c ratio for the three aggregate types for normal concrete and self-compacting concrete (Alluthwatta AGHD et. al (2011)). Results indicate that SCC has always produced higher strength than normal concrete.

It is also seen that, unlike in the case of normal concrete where offshore sand produced higher strength, in SCC, quarry dust has produced the highest strength. It is also seen the margins of differences of the recorded strength between different aggregate types have narrowed down considerably for the SCC. For normal concrete, it is evident from the higher water demand that mixes with quarry dust as fine aggregate is most difficult to workwith (Rajapaksha RWCN et.al (2009). However, there is marked contrast of its performance as fine aggregate of SCC. Lower water demand for quarry dust compared with river sand mixes and the highest recorded compressive strength for quarry dust mixes are indications that viscosity modified self-compacting concrete have largely eliminated the influence of fine aggregate due to particle size distribution and aggregate packing.

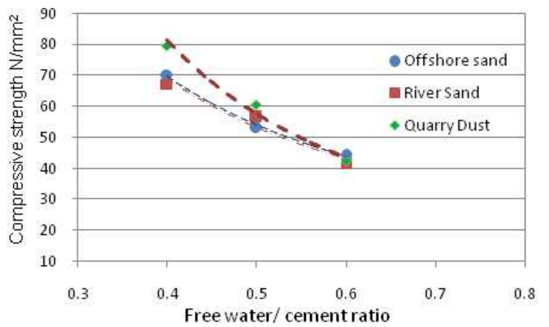


Figure 11 Average strength vs. w/c ratio for Self-compacting concrete.

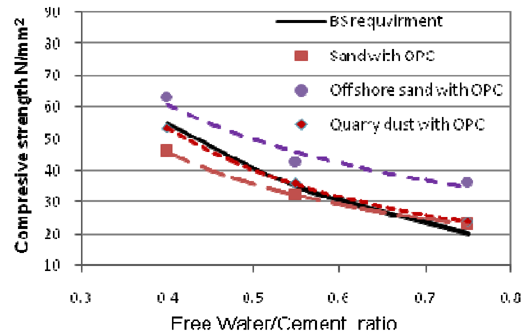


Figure 12 Ave. strength vs. w/c ratio for normal concrete (Alluthwatta AGHD et. al 2011)

6. CONCLUSIONS

Overall results of the experimental study indicate that both offshore sand and quarry dust is a viable alternative for river sand as fine aggregate of self-compacting concrete. Based on strength quarry dust is found to be the better fine aggregate type for self-compacting concrete. Reason for higher strength in quarry dust can be attributed to the possible better bond between the aggregate and cement paste. SCC mixes has recorded different trend for the water required under the constant doze of viscosity modifying agent and compressive strength compare to normal concrete mixes. Study with particle size distribution of the different aggregate manipulated to a single particle size distribution reveals no significant change in strength between the mixes with natural particle distribution and single particle distribution. This means that the particle size distribution has only secondary influences on the SCC.

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Solar Powered Automated Irrigation System

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Abstract: *The variation of spatial and temporal distribution of available water for irrigation makes significant demand on water conservation techniques. Hence solar powered Automated Irrigation System provides a sustainable solution to enhance water use efficiency in the agricultural fields using renewable energy. This system allows farmers to apply the right amount of water at the right time. This system can automatically irrigate the fields according to the pre-defined conditions. It allocates water according to the crop water requirement and availability of solar radiation. The results revealed that, system would be a best option for medium size agriculture field. And this automated system water wastage of the tested field could be reduced by 50% of normal irrigation wastage. Besides, human attention was reduced on irrigation significantly. This intelligent system can be improved by adding temperature and total dissolve solid sensors. As well as system can be program using 8051 40 pin microcontroller and it will reduce the power consumption. Apart from that, the system can be developed to control remotely by using internet and GSM technology and field condition can access by the farmer from anywhere in the world*

Keywords: *Solar powered, Automated irrigation system, Water use efficiency*

1. INTRODUCTION

Irrigation water has become a scare resource due to the special and temporal availability of irrigable water. Mainly, bimodal rainfall pattern of Sri Lanka is considered as main source of rain fed agriculture. Other than that, irrigated water management practices could be found in major irrigation schemes. Most of the irrigation schemes, allocate water in bulk form and presently the irrigation efficiency in farmer's field is about 30 to 40 percent. The water loss, not available to crops is about 60 to 70 percent (Lewis, n.d.). Thus, it is necessary to enhance Water Use Efficiency (WUE) in irrigated lands. Food and Agriculture Organization (FAO) has suggested many promising strategies for raising WUE. The strategic water management practices are soil-water conservation measures through crop residue incorporation, adequate land preparation for crop establishment, rainwater harvesting, and conservation tillage to increase water infiltration, reduce runoff and improve soil moisture storage. In addition, novel irrigation technologies such as supplementary irrigation (some irrigation inputs to supplement inadequate rainfall), deficit irrigation (eliminating irrigation at times that have little impact on yield) and drip irrigation (targeting irrigation water to plant in root zones (FAO, 2006).
Moreover, the variation of spatial and temporal distribution of available water for irrigation makes significant demand on water conservation techniques. Further, continuous extraction of water from ground water table, creates numerous current and future consequences for mankind. Finally, the water crisis could be lead to conflicts among different communities by creating numerous socio-economic issues. Hence, it is crucial to investigate sustainable solutions for the development of the nation.

Even though, the micro irrigation systems are used in different agricultural fields, most of them are regulated manually. It requires regular intervals to operate using man power and it reduces the efficiency of the large scale irrigation lands. In addition, this regular interval operation system based on man power, leads to excess water allocation than the crop water requirement and insufficient water allocation when plants requires more water based on diurnal variation and seasonal changes. Water deficiency can be detrimental to plants before visible wilting occurs. Retardation of the crop growth rate, late flowering, and reduction of the yield are some of the significant effects which can be caused due to water deficiency. Moreover, excess water in the root zone cause ill health of the root zone and vegetation, additional cost for farmer, wasting of water and time wastage. Salinity of the soil can be increase due to the continuous

rising of water table in long run with the excess application of water.
Hence solar powered Automated Irrigation System (AIS) provides a sustainable solution to enhance water use efficiency in the agricultural fields using renewable energy. It provides water for plants according to the crop water requirement and this system operates according to the soil moisture condition of the root zone of the plant. The system has designed to operate using solar energy; hence it could be used for the areas where the accessibility of National Electricity grid is not popular. Further, it does not affected by the energy crisis which can be altered the livelihood of the public. The proposed system helps to control irrigable water over the agricultural fields. Thus it reduces excessive pressure on farmers to pay additional water tariff on water. In addition pump water irrigation also save additional cost for water pumping, reduces the conveyance and distribution losses in the field level. Moreover, energy consumption on water pumps could be reduced by efficient water allocation based on the crop water requirement.
Further, automated irrigation system allows farmers to apply the right amount of water at the right time. This solar powered system does not require man power for operation. This intelligent system can detect the soil moisture availability and perform automatically based on pre-defined logical conditions. This system reduces run off from over watering saturated soils, avoid irrigating at the wrong time of the day, which will improve crop performance by ensuring adequate water and nutrient balancing. Further, it prevents Salinity of agricultural lands which cause for poor productivity and land degradation etc. In addition, this system helps in time saving, removal of human error in adjusting available soil moisture level and to maximize their net profit.

The best time of day to irrigate is the subject of some debate. One group suggests that early morning is the best time, while another group claims that afternoon is the best. Very few people consider night irrigation to be a viable alternative due to concerns of increased disease pressure. In all practicality, disease is not increased except when lawns are routinely over-irrigated (The Clemson University Cooperative Extension Service).
According to the Stuart et al. (Stuart *et al.,* 2004), they have concluded that, irrigation timing had a significant effect on plant growth, container temperature and water use efficiency.
Considering above facts on irrigation time there is a necessity to operate the irrigation system in night time as well and reduce the evopotranspiration.

The main objective of this study is to develop a solar powered automated irrigation system for efficient water allocation for fields based on crop water requirement using renewable energy.

2. METHODOLOGY

Experimental setups were arranged and experimented using relevant hardware components and software components. Sensor parts of the circuit was developed using op-amp IC LM324. Soil probes used to detect soil moisture levels. AT89C2051 microcontroller was used for decision making process. Pre-defined operational conditions were used to operate the system without any failure. Algorithms were developed logically and it was used for software development of the system. Assembly language was used to write the program in to the microcontroller.

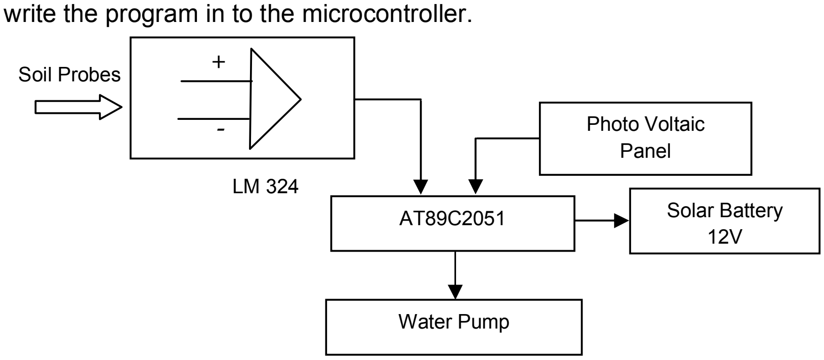

Figure 1: Diagram of Automated Irrigation System

2.1. Hardware Components Used for the System Development

AT89C2051 microcontroller, LM 324, PV module, solar battery, simple electronic circuits were used as the main hardware components for the development of solar powered AIS.

Table 1 Microcontroller Based Solar Charger Circuit

Device	**Types of the Device**
Semiconductors	AT89C2051, ADC0831, NTE 3041, 7805(5V), BC547(npn), BS170, IRF540, 6A4, 1N4007, 7.5V zener diode
Resistors	8.2kΩ, 1.2kΩ, 10kΩ, 20kΩ, 330Ω
Capacitors	100μF(63V), 100μF(16V), 0.1μF, 10μF(16V), 33pF, 0.01μF
Miscellaneous	On/off switch, Push-to-on switch12V, 1C/O relay, 12MHz crystal, 16x2b line display, Photo Voltaic(PV) module, 10A fuse, 10 pin connector, solar battery

Table 2 Automated Irrigation System

Device	**Types of the Device**
Semiconductors	AT89C2051, LM 324, BC547, 1N4007
Resistors	56kΩ, 100kΩ VR, 330Ω
Capacitors	1μF(16V), 33pF
Miscellaneous	12V Relay, 12MHz crystal

When consider the main hardware components, AT89C2051 microcontroller was used to regulate the operation of solar battery and the water pump. Meteorological parameters and electrical parameters were taken as inputs for the microcontroller. Solar intensity and soil moisture levels were considered as the meteorological inputs which were detected by different sensors. Photo Voltaic (PV) panel voltage and solar battery voltage was considered as the electrical inputs which were used to program the microcontroller.

The LM324 contains four independent high gain operational amplifiers with internal frequency compensation (LM324 data sheet, 2007).

10w, 16.8V PV module was used for the experiment and 12V, 9AMH lead acid battery was used as the solar battery.

2.2. Software

The source program for the microcontroller was written in assembly language and assembled using Keil software .which is freely available on the internet for download.

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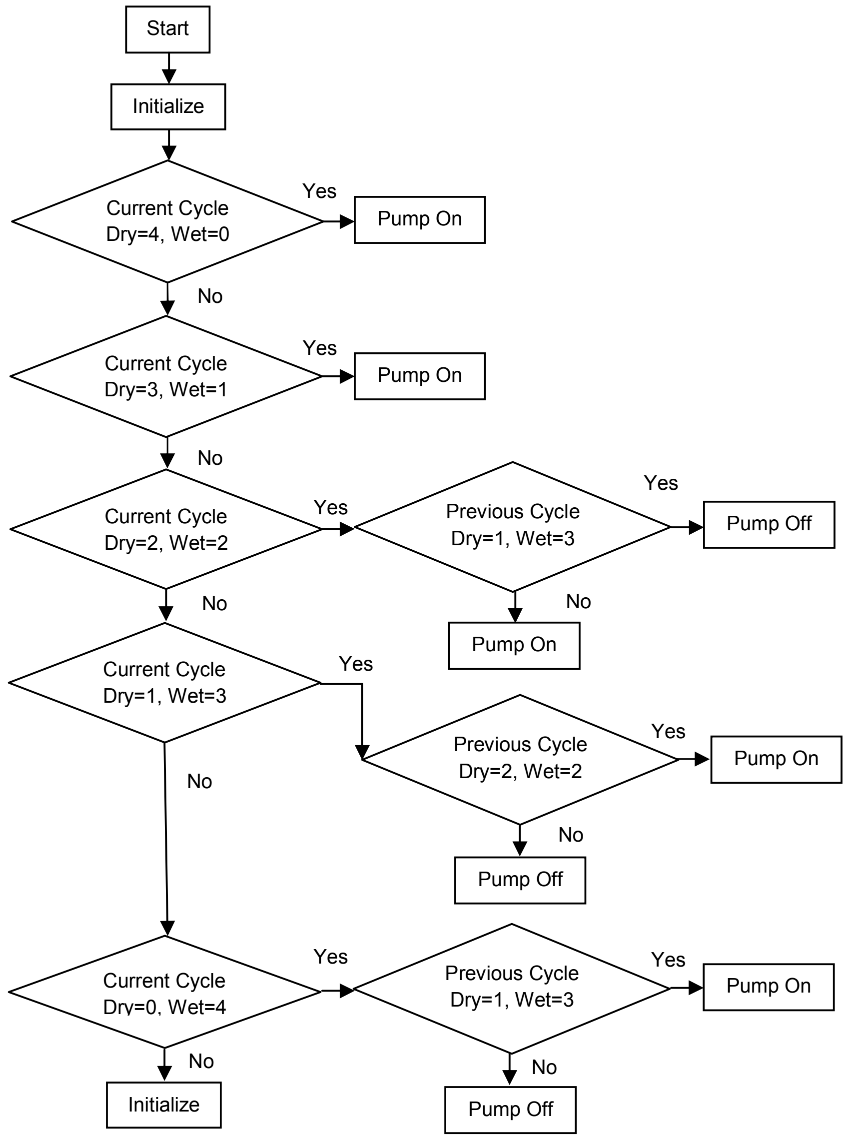
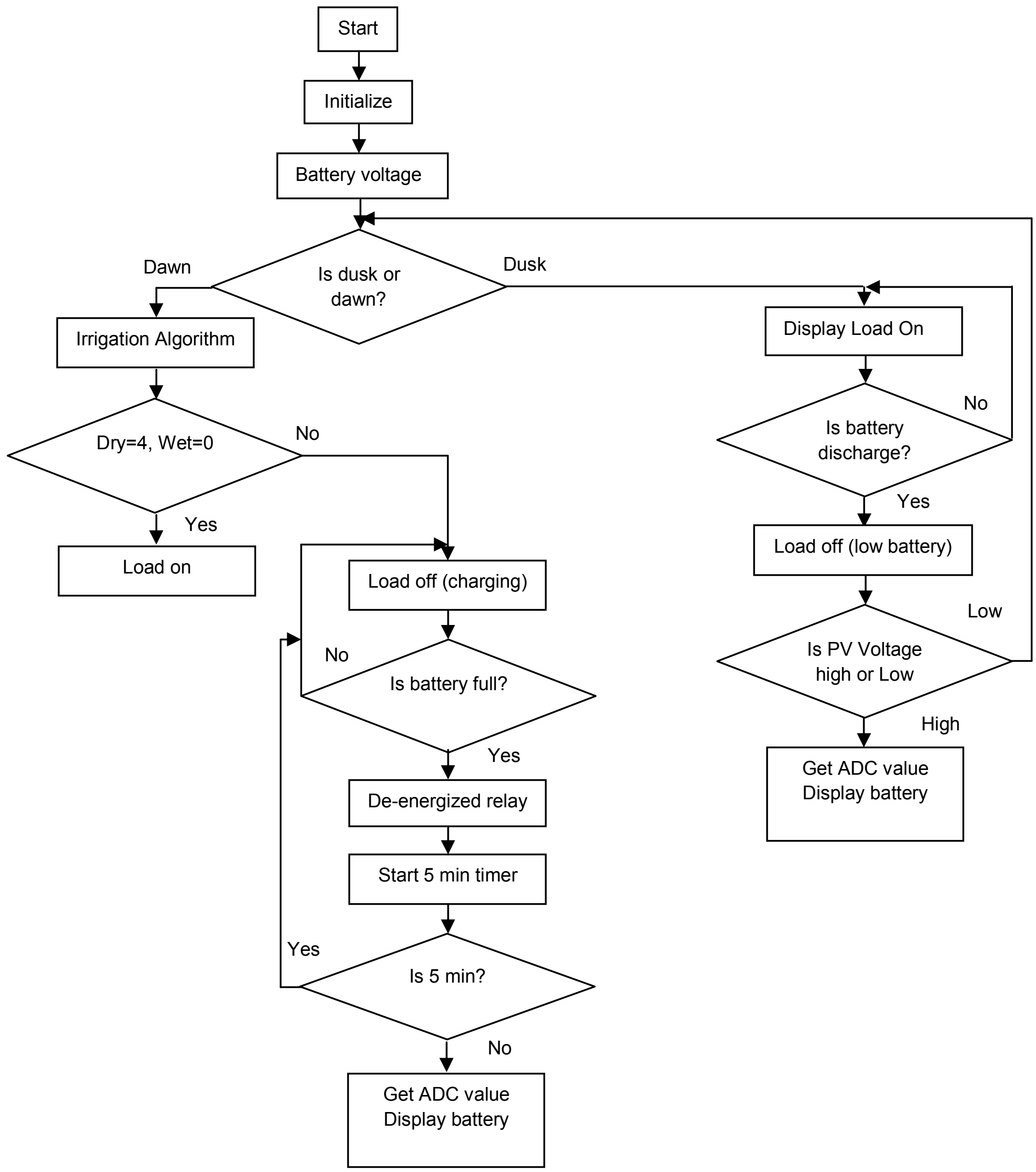
2.2.1. Control Algorithm of Automated Irrigation System



2.2.2. Control Algorithm of System Load



System load algorithm

Algorithm starts, initialize and it takes battery voltage as the initial input. At the first decision making point, system evaluates whether it is dusk or dawn. If it is dawn, operation jumps in to the irrigation algorithm. In the irrigation algorithm if it comes to emergency condition as Dry = 4 and Wet = 0, it is a critical stage for the crop and it is essential to irrigate water as soon as possible before crop comes to its permanent wilting point. In this situation system identifies its critically needs to "load on" which helps for pump operation. Otherwise, if there is no critical condition as Dry = 4, Wet = 0, system will off the load.

In a normal situation, if it is a dawn, load will off and solar battery charging will be started. In this cycle, in a particular point, it takes a decision whether battery is full or not, If battery is full, relay will be de-energized and at the same time 5min timer will be started. When timer comes to 5 min point, charging will be started again and load will be off. If timer value does not 5 min, it gets ADC value and displays battery capacity.

When it is dusk, LCD displays load on and continuously check whether battery is discharging or not. If it is yes load will make off and displays low battery. In the next decision making point it checks whether the PV cell voltage is low or not according to pre-defined voltage level in the system. If it is below the pre-defined voltage level, control jumps to check the dusk or dawn condition. If it is high, it continuously updates the voltage of the battery on LCD panel.

Irrigation algorithm

The logics of the algorithm help to identify whether water is filling to the field or whether water is evaporating from the field. Further, logics and decision making conditions help to maintain at least 25% soil moisture condition of the soil and it always maintain >25% of moisture in the field. In the algorithm, when starts a new clock cycle, it initialize and at the decision making point it evaluates whether current cycle Dry = 4, Wet = 0 and it indicates as the field in the critical stage and intelligent system give a signals to operate the pump. When it comes to a condition like “current cycle Dry = 3, Wet = 1, the field in dry condition and needs to operate water pump. In another cycle, if it comes to a condition like “current cycle Dry = 2, Wet = 2, it needs further information to make a decision. Thus, it evaluates the stored data of the previous cycle. If previous cycle is Dry = 1, Wet = 3, it is an indication of the irrigated water has being evaporating. Then pump will not be operating since it has 75% moisture level in field.

3. RESULTS AND DISCUSSION

The system was tested in the field conditions and results revealed that, system would be a best option for medium size agriculture field. Operational amplifier reference voltage could be changed according to the crop type and moisture availability in the field by using potentiometer. Excess power was stored in the solar battery and it was used when day light was low. Excess charging was prohibited by using pulse width modulation technique and it helped to reduce the temperature of the solar battery and increase the life time of the battery. When all probes were dry, water pump was switched on until all the probes were getting wet. As well as when all probes were wet, it allocated to dry till two probes and utilize the water and power sustainable manner. If further reduction of the moisture of the soil occurred, it leaded to switch on the pump till all probes getting wet. Because of this automated system water wastage of the tested field could be reduced by 50% of normal irrigation wastage and reduce evopotranpiration by induced dark time operation. Besides human attention was reduced on irrigation due to automation.

4. CONCLUSION AND RECOMMENDATIONS

This intelligent system can be improved by adding temperature and total dissolve solid sensors. As well as system can be developed by using 40 pin 8051 microcontroller and it will reduce the total power consumption and initial cost. Apart from that, the system can be developed to control remotely by using internet and GSM technology and field condition can be accessed by the farmer from anywhere in the world. Further studies should be undertaken to identify each crop water requirement and application of Solar Powered Automated Irrigation System according to the moisture retention capacity of the Field and environmental conditions. Since this is initial experimental stage, the performance evaluation criteria of the system should be thoroughly investigated.

When consider the cost of the solar powered AIS and operational cost, the initial establishment and fabrication costs are considerable. But it is more cost effective in long run. The cost of embedded system was 5000 rupees with imported duty charges of some components.

5. ACKNOWLEDGEMENTS

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Mitigation and Redevelopment as a Sustainable Development Strategy for Disaster Prone Human Settlements: With special reference to Landslide at Peradeniya Town

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Abstract: *Settlements cannot be relocated due to two reasons; cost of infrastructure and the socio-economic factors in that particular location. As such Peradeniya town center has been developed at a key road intersection. Peradeniya has been identified as landslide prone in 2004 through the Landslide Hazards Mapping Programme (LHMP). In 2009 NBRO initiated a project to mitigate the landslide at Peradeniya. With the implementation of the landslide mitigation activities successfully NBRO initiated another study to formulate town development plan for Peradeniya as a post landslide mitigation redevelopment strategy. The study conducted considering multidisciplinary aspects of land. Finally, town development plan was formulated under the vision of "Gateway to the Kandy city, University and Botanical Garden." This is to preserve the main character of the town while enhancing aesthetic and economic value of the settlement. Likewise, there are settlements vulnerable for natural hazards but it's possible to redevelop following proper geological and land use planning principles to preserve the character of that particular settlement to ensure the sustainable development without allowing valuable land resources to be abandoned.*

Key Words: *Mitigation and redevelopment, development plan, multidisciplinary aspects of land, socio-economic factors*

1. INTRODUCTION

Peradeniya town centre is located in Kandy District, Central Province, Sri Lanka (Figure 1). It is located 110 Km's from the capital city of Colombo and just 6 Km's away from the historic city of Kandy. Mainly, the town centre has been evolved from the history due to location at a key road intersection of A1 road and A5 road, where physical development is inevitable with high road connectivity (Munasinghe, 2010). Because of this, the town centre is functioning as a transition node which has lead to perform as a market centre providing opportunity to perform as a livelihood as well.

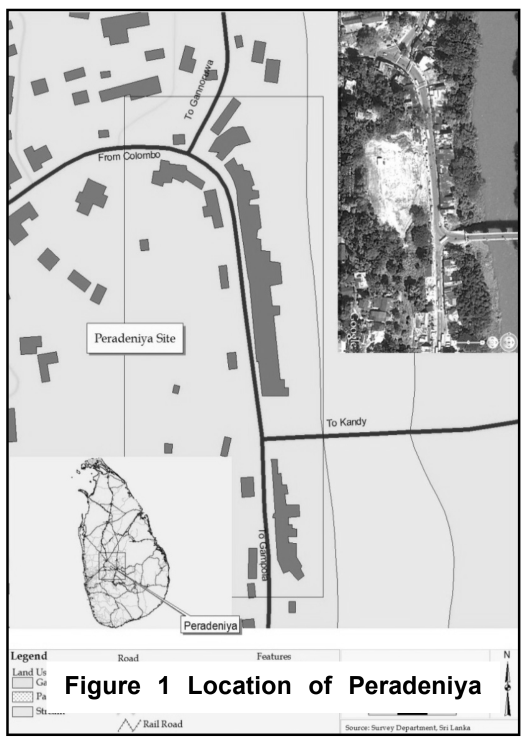

Figure 1 Location of Peradeniya

In regional context, Peradeniya town got its reputation with the location of nationally important places closed to the town. Most importantly, location of Peradeniy Botanical Garden and University of Peradeniya, which is functioning in national context, make significant influence on town development. At the same time, the town is operating as a gateway to the Kandy city which is the second largest city of Sri Lanka. Therefore, the Peradeniya is an important place even in regional context. But, the Peradeniya town center had been identified as vulnerable to landslide hazard by National Building Research Organization (NBRO) in 2004 through Landslide Hazards Mapping Programme (LHMP). Then, NBRO initiated its first and ever landslide mitigation

project at Peradeniya in 2009, with the increased capacity and skills to mitigate the landslide incidents. At the same time, another project has been formulated to relocate the Peradeniya town by another public organization without considering the socio-economic value of Peradeniya town at the present location or the landslide risk reduction measures. As Arambepola, 2008 describes that there are some instances especially where human settlements are threatened by landslide it will be difficult to relocate a complete settlement to another area as socio-economic factors may force the inhabitant communities in getting relocated into a new area.
Therefore, this research study was conducted to formulate an appropriate Peradeniya town development plan, as post landslide mitigation redevelopment strategy. The study was based on the multidisciplinary assessment of land; physical, social, economic and environmental aspects.

2. METHODOLOGY

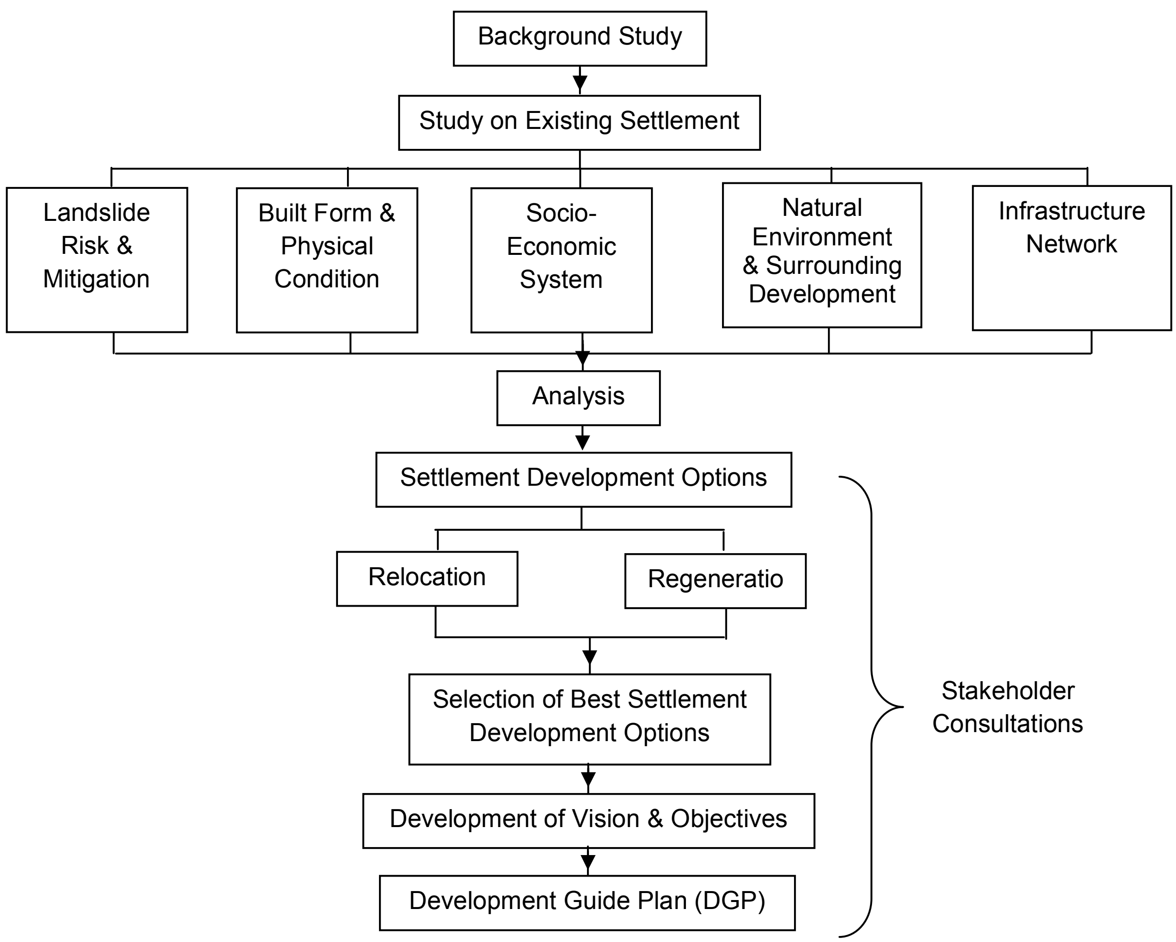

Figure 2 Study Process

Peradeniya town development plan formulation is involved with sequence of actions described by figure 02. Accordingly, extensive field investigations were carried out at the initial stage with the objective of identifying the characteristics of Peradeniya town. In this, study was carried out on five sections as landslide risk and mitigation activities, built form and physical condition, socio-economic system, natural environment and surrounding development and infrastructure network. Landslide risk and the mitigation activities being implemented by NBRO is key, since formulation of redevelopment strategy for the Peradeniya is heavily depend on the geological stabilization of landslide. As well as socio-economic studies were important in order to identify the functionality of the existing settlement which must be considered in the formulation of redevelopment plan. The data was collected on each of these sections through field surveys. Secondly, analysis was done to identify the dominant issues and potentials associated with the settlement. In this, SWOT analysis was done. Thirdly, decision was taken on appropriate Peradeniya town development scenario based on initial studies. In this, key stakeholders of the project were consulted extensively since the responsibility is vested with the stakeholders on Peradeniya development. In this, vision and objectives for Peradeniya town development was formulated which would guide the future development. Finally, Development Guide Plan (DGP) was formulated including structure plan, master plan and precinct plan.

2. LANDSLIDE LOCATIONS AND LANDSLIDE RISK MITIGATION PROJECT

Location of Landslide at Peradeniya town is illustrated by figure 3. Accordingly, shops located in front of the landslide locations are at high risk. At the same time vehicular and pedestrian traffics at main artery are the other categories at high risk. The landslide mitigation activities are implemented by NBRO in two phases (figure 3). The entire mitigation project will be completed by 2015 liberating the settlement from the landslide risk. The project included investigation, design and implementation of structural mitigation measures in the area. At present, phase 1 of the mitigation activities are implemented successfully and will be completed by 2012 (figure 4 and 5).

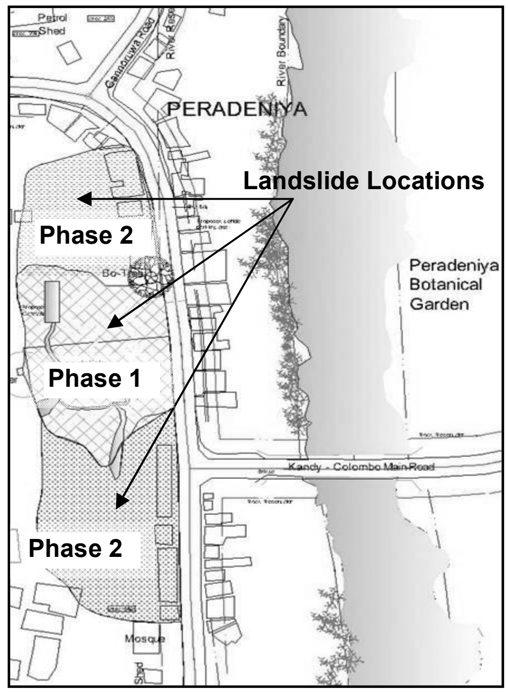

Figure 3 Landslide Location and phases of Mitigation project

Figure 4 Landslide mitigation programme in 2010

Figure 5 Current situation of Landslide mitigation by 2012

3. KEY FINDINGS

Key findings of the analysis are listed bellow;

- Majority of the shops are located on the Mahaweli River reservation, the buildings are obsolete and structurally not safe (Mahaweli Authority, 2008).

- Approximately 50% of the buildings are located on private lands and the balance on public lands (NBRO, 2011).

- The town experiences heavy vehicular and pedestrian traffic. Width of the carriage ways and pedestrian walkways are much below the national standards and no clearly defined pedestrian crossings are available. Due to non availability of parking facilities, on road parking takes place that further curtails the road width for vehicular traffic (NBRO, 2011).

- Retail outlets of day to day home needs (Groceries), Cafeterias, Textile shops, vegetable and fruits outlets are the main commercial activities. All together there are 148 shop units are located in the Peradeniya town centre (NBRO, 2011).

- Although the town is located in an environmentally pleasing natural environment such views are fully covered by the location of buildings on the Mahaweli River reservation (NBRO, 2011).

- Perception of the majority of the business community is that the business functions well due to the location of the town in a major intersection and being a transit point and location of national level institutions such as the University, Peradeniya Garden and Teacher Training Institution. Re-location of the town in a different location will jeopardise its existence. It is necessary to re-develop the town at the same location with good infrastructure. It is also necessary to provide spaces for some of the lacking commercial activities particularly banks (NBRO, 2011).

- Majority of the pedestrians were commuters as transit bus passengers. Only about 1/3rd of the pedestrians do retail shopping in Peradeniya town. Provision of vehicle parking facilities, sanitary facilities and pedestrian walks are the main requirements noted by the interviewed pedestrians (NBRO, 2011).

- New commercial spaces would be developed along the landslide mitigated area with the possibility to clear the land parcels through landslide mitigation (NBRO, 2011).

The study reveals that, performance of Peradeniya town is significantly important in both local and regional context with its strategic location at a key intersection. Accordingly, it is perform as a transition point, market centre and livelihood for the community dwelling at surrounding localities. Hence, the activities pertaining to those functions are inevitable from the town. At the same time, due to implementation of landslide mitigation activities successfully, the town would be developed following proper land use planning principles. In this, main issues are to be overcome while harnessing its potentials. Hence, a policy decision was taken "to re-develop the town at the same location as a model township."

4. PERADENIYA TOWN DEVELOPMENT PLAN

The vision for the Peradeniya town development is "**Gateway to the Kandy city, University and Botanical Garden**" that describes the Peradeniya as main gateway to such places. Realizing this vision, main objectives are; Ease traffic congestion and create a pedestrian friendly township, Integrate all the aesthetically pleasing natural and built environments into the new town design, Provide adequate public conveniences such as sanitary facilities and open spaces, Improve the business environment, Conserve the Mahaweli River and its reservation. Then, the structure plan for the Peradeniya town development has been formulated (figure 10).

Key Components of the proposed structure plan are;

- **Proposed Linear Park (figure 9);** main requirement for the establishment of a linear park along the Mahaweli River side is to ensure the comfort, convenient and safety of the pedestrians. Proposed linear park will be consisted with pedestrians walk ways, vehicular parking places, a cafeteria, public sanitary facilities and places for leisure and recreation. This linear park will be extended integrating Peradeniya railway station and botanical garden with the town.

- **New shopping complex (figure 8);** present development activities located along the Mahaweli river reservation will be removed and relocated at landslide mitigated area since, most of the present building units are located in river reservation and operating as a barrier to view the aesthetic view of the location. In this, 115 new shopping units will be constructed as per the relocation necessity. As well as, banking and related facilities will be provided which is demanded by town users.

- **View corridors (figure 7);** view corridor will be provided along the proposed linear park and top of the landslide mitigated land. Major elements create aesthetic views are botanical garden, Peradeniya bridge and Hanthana mountain range.

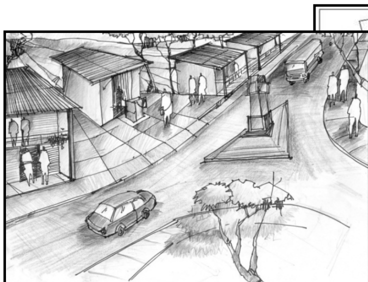

Figure 6 Sketch of proposed Landmark

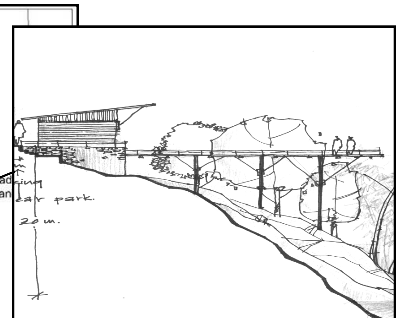

Figure 7 Sketch of proposed

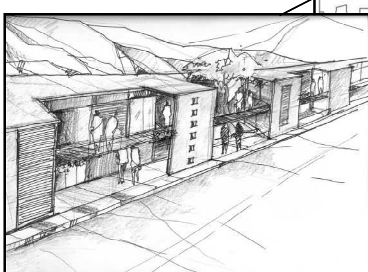

Figure 8 Sketch of proposed Commercial Development

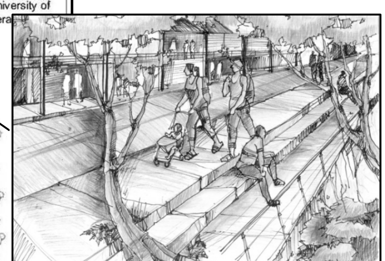

Figure 9 Sketch of proposed

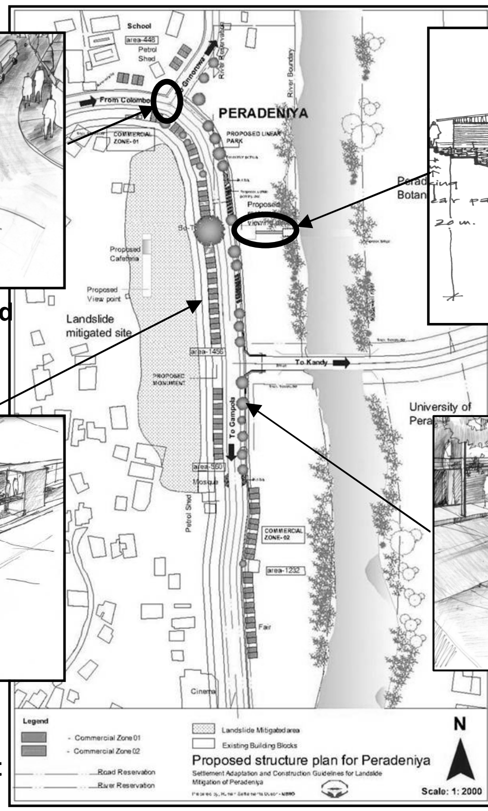

Figure 10 Proposed Structure plan for Peradeniya Town Development

5. CONCLUSION & RECOMMENDATIONS

This study was conducted with the objective to formulate Peradeniya town development plan as post landslide mitigation development strategy. In this, slop stabilization done through the geological investigation was a pre requirement. Then, human settlements investigation was carried out considering multidisciplinary aspects of land; physical, social, economic and environment. The study reveals that existence of Peradeniya town centre at present location is important to facilitate its functions; transition node, market centre, livelihood and gateway to historic city of Kandy, Peradeniya Botanical Garden and

University of Peradeniya mandated by its location. At the same time, common view of the town user's; traders and pedestrians, also to develop the Peradeniya town centre at present location minimizing issues associated with the settlement. Hence, considering research findings, a development plan was formulated under the vision of **"Gateway to the Kandy city, University and Botanical Garden."** This is mainly due to its prime function as a transition node at a major road intersection. Under this vision, key design proposals are linear park, new commercial zone and viewing corridors. In this, more spaces will be allocated for vehicle parking, pedestrian walk ways and public sanitary facilities which would solve most of the issues associated with present town centre such as vehicular traffic congestion, lack of pedestrian's safety, lack of public sanitary facilities...etc, availability of obsolete building structures for commercial activities and hiding the aesthetic view by present development pattern.

Finally, necessary alterations will be done for this development plan consulting different stakeholders. Thereafter, implementation mechanism will be formulated sharing the responsibilities between respective institutions and organization.

Likewise, there are settlements vulnerable for natural hazards and posses inherited values which would not be able to relocate simply. Therefore, scientific studies to be carried out to formulate disaster risk mitigation measures and to identify the highest and best use of land such as residential, commercial and agricultural or any other form of development which can be utilized in appropriate manner to ensure the sustainable development of the country without allowing valuable land resources to be abandoned.

6. ACKNOWLEDGEMENT

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Occupant Satisfaction on Indoor Comfort in a Green Building

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Abstract: *Sustainable Development was innovated, as a new concept to act against the repercussions of the climate change and Sustainable Built Environment performs a pivotal role with total commitment within this innovation. Many international organizations have focused their interest on this new phenomenon and as one of these organizations; USGBC (United States Green Building Council) established the LEED (Leadership in Energy and Environmental Design) certification program, for buildings to be performed as truly as green buildings. Therefore, it is important to find out; the occupants of these green buildings are truly satisfied with their workplace environment. As a result, Indoor Environmental Quality of LEED platinum rated green building was deeply evaluated, questioning the occupants, focusing on the aspect of thermal comfort as the person's psychological state of mind. Though, the building was within its extended comfort zone, from the analysis, it was found that, among all the variables, less number of occupants were satisfied with the thermal comfort and significant number of dissatisfied occupants were located in a specific area of the workplace and that justified, there should be some external factors which already exist, around the workplace, which are affecting the thermal comfort of the occupants. Therefore, this research emphasized that designers, need to think beyond the standards and guidelines, when ensuring the occupant satisfaction within new sustainable construction methodologies.*

Keywords: *Green Buildings, LEED, Post Occupancy Evaluation, Occupant's satisfaction, Thermal Comfort.*

1. INTRODUCTION

As Rachel (1962) stated, in her book "Silent Spring" the man has used his significant powers, to alter the nature to make his life more comfortable and as a result, today the world is facing to many global issues and climate change has become the most critical threat that the world is facing today. However, it is very important to understand that climate change is not just another issue in this complicated world of proliferating issues. Climate change is THE issue, which unchecked, will swamp all other issues (Gelbspan, 2005).Therefore, many entities have started to pay their attention towards the climate change and recognizing these risks, governments and other entities around the world are acting now to limit potential damage from climate change rather than waiting and having to take more costly, reactive measures in the future (Pew Center, 2009).

"Sustainable Development" was innovated to overcome the daunting challenges of climate change. According to, United Nations World Commission on Environment and Development (1987), "Sustainable development is 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 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 state of technology and social organization on the environment's ability to meet present and future needs. The adaption of principals of sustainable development will play an important role in this context especially, with respect to the creation of built environments and sustainable or green building plays a pivotal role in sustainable development. (Kiberi, 2008). Sustainable buildings are designed and constructed to high environmental standards and thereby; minimize energy requirements, reduce water consumption, use materials which are of low environmental impact; low embodied energy and resource efficient, reduce wastage, conserve / enhance the natural environment and safeguard human health and wellbeing. (Clark,2010) and (Bhamra & Lofthous, 2007).

According to Kiberi (2008), and U.S. Green Building Council (2010), LEED (Leadership in Energy and Environmental Design) is one of the building performance assessments to buildings to be performed as truly as green buildings, which was established by the USGBC; (United States Green Building Council) as one of the organizations that concern about the sustainability. According to the USGBC and its LEED program, green buildings reduce the negative impacts of buildings on occupants and the environment in five general categories: sustainable site planning, safeguarding water and water efficiency, energy efficiency and renewable energy, conservation of materials and resources, and indoor environmental quality(Yudelson, 2007).

However, it is important to find out, that after fulfilling such important and critical requirements stated in the LEED certification process, whether LEED certified building can truly satisfy the occupants of the building and POE (Post Occupancy Evaluation) has to be carried out to find out the occupant satisfaction in the building. “POE” is a process of systematically evaluating the performance of buildings after they have been built and occupied for some time (Federal Facilities Council, 2001). According to Kumar et al (2009), Shove (1975) and Egan (2004), the best aspect to be evaluated during the POE was identified as the “Thermal Comfort” and it was defined in British Standard BS EN ISO 7730 as: “That condition of mind which expresses, satisfaction with the thermal environment”. So the term “Thermal Comfort” describes a person’s psychological state of mind and is usually referred to in terms of whether someone is feeling too hot or too cold.

The most suitable LEED platinum rated green building was selected, after conducting careful analysis on the buildings which, were stated in the LEED rated green buildings in Sri Lanka and detail study of the selected building was carried out and captured it as a sustainable design. The selected building was housed for apparel manufacturing process and it was located in Thulhiriya and according to the Department of Meteorology, Sri Lanka (2010), the mean annual temperature is 28.0 ºC, neutrality temperature 26.3 ºC, altitude (h) above mean sea level (m) h < 100 in the selected location.

2. OBJECTIVES AND METHODOLOGY

The key objective of this research is to justify, whether the occupants in LEED rated green buildings are truly satisfied with respect to the thermal comfort in their workplace.

The following methodology was developed to achieve the objective of the research:

All relevant drawings and documents, which were submitted for LEED certification was carefully analyzed, observing and comparing with the complex and cross checked with the interview held with the maintenance engineer, which was structured according to the aspects that were specified in LEED for New Construction Version 2.2 handbook (2005). Further, a pilot survey was carried out with all the occupants of the entire complex including all three production floors. The first section of the questionnaire was covered the general information of the respondent such as age, gender, job title and nature of the work. The second section was designed to find out the occupant’s overall satisfaction on the building using the Lickert scale technique (Corbetta, 2003) and the variables that analyzed were; general satisfaction on the building, general satisfaction on workroom, overall ventilation, thermal comfort, level of artificial lighting, views, acoustic quality, office furniture, machinery, equipments, finishing material, clearness and maintenance, attention and concentration to work, awareness and communication, interactive behaviors and wellbeing. The sample questionnaire was stated in Appendix A[2].

Once, deciding on the appropriate floor, a simple and straightforward reference key was established to use during data recording and data analyzing, after careful study on the layout plan of the selected floor and analysis was concentrated on the orientation of the building, orientation of the interior, openings, production lines, cutting and administration area, walkways, positions of the staff, machinery placements, etc.

Detailed questionnaire survey, which was designed specifically, focused on the thermal comfort, was carried out with entire occupants of the selected floor. The questionnaire consisted with three main sections. The first section consisted with the general information of the respondent; the second section was focused on thermal comfort enclosing 4 sub sections based on temperature, humidity, air velocity and overall satisfaction. The final section was to comment on the indoor environmental quality of the work place. The sample questionnaire was stated in Appendix B[3].

[2] Appendix A is available with the corresponding author to be obtained.

[3] Appendix B is available with the corresponding author to be obtained.

3. DATA ANALYSIS AND FINDINGS

Previously recorded drawings, documents, data together with the interview held with the maintenance engineer, proved, that the complex was constructed according to original drawings concentrating to maximize the passive features and tried it's best to perform the complex efficiently using those passive features and the complex has fulfilled the requirements that needed for LEED platinum award.

Based on the pilot survey that was carried out in all three floors, when analyzed the sample according to the gender, majority represented the females and that was 53.1 % from the sample, as illustrated in Figure 1.

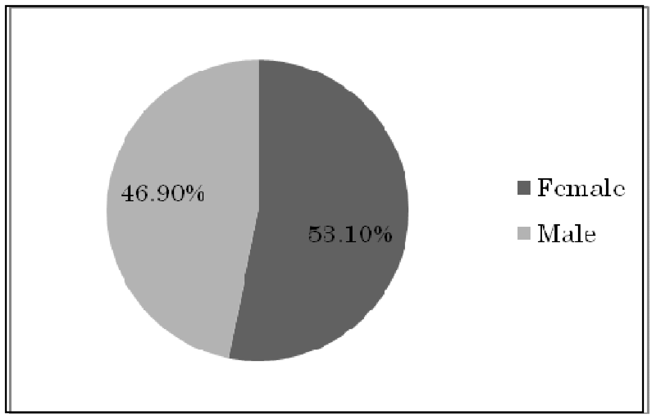

Figure 1: Composition of the sample with reference to the gender

With respect to the quantitative aspects; most common age group of the sample spanned from 19 to 34 years, as in Figure 2 and the majority of occupants in the sample has experience of 2 years, as in Figure 3 and it was represented on X axis in months.

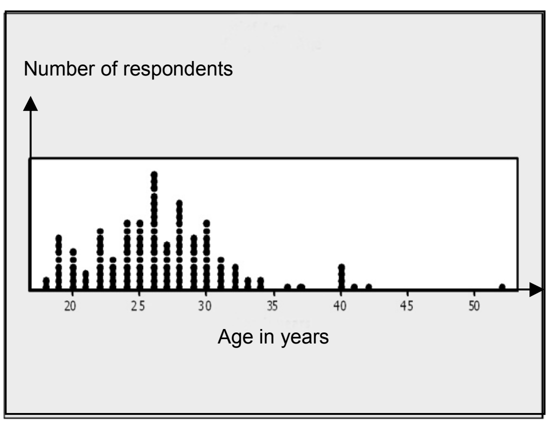

Figure 2: Data description with reference to age

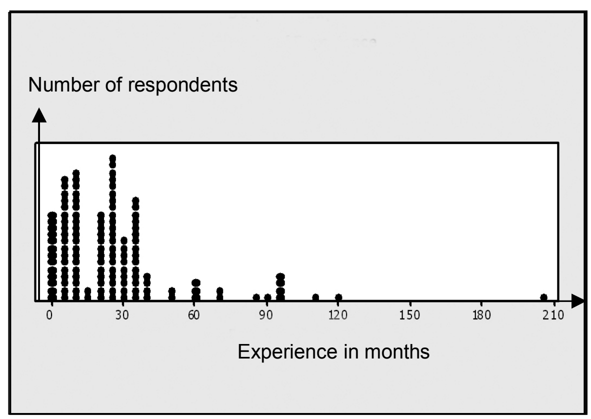

Figure 3: Data description with reference to experience

Based on the second section of the questionnaire, the findings can be illustrated as in Figure 4.

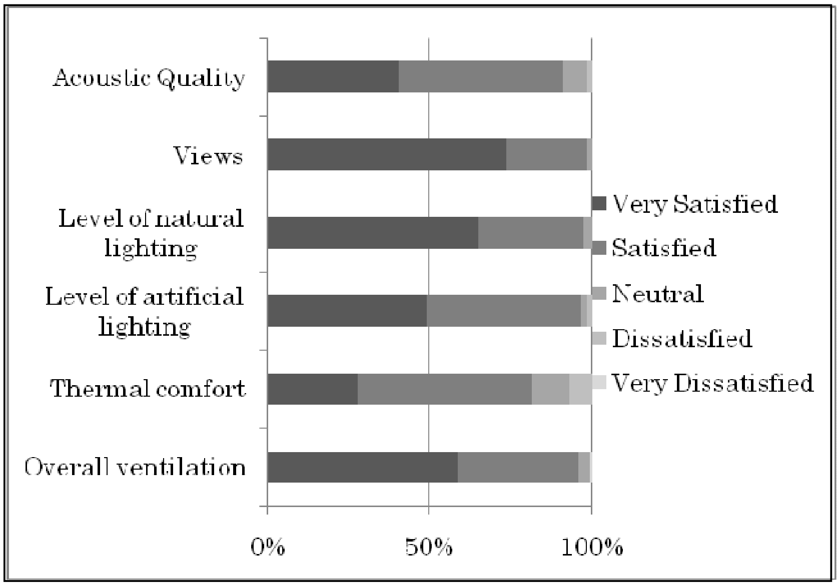

Figure 4: Occupant's satisfaction

Figure 4 represented the occupant's satisfaction in all three floors and according to that, when compared the acoustic quality, views, natural lighting, artificial lighting, thermal comfort and ventilation, majority was very satisfied with the view, which they were able to enjoy from their working place. Almost similar number of satisfied occupants can be seen on natural lighting and the ventilation, within the complex. Among all the variables, there were occupants who were dissatisfied with acoustic quality, artificial lighting and thermal comfort and similar number of dissatisfied occupants can be seen with respect to acoustic quality and the artificial lighting. Considerable number of occupants were dissatisfied with thermal comfort than acoustic quality and artificial lighting and that proved, even though the occupants were satisfied on all the variables, less number of occupants were satisfied with thermal comfort and at the same time there were more number of occupants who have neutral feeling on the thermal comfort when compared to other variables.

Therefore, after careful consideration, aspect of thermal comfort was further analyzed, to find out which floor has more number of occupants, who were not satisfied with the thermal comfort of their workplace. From the comparison carried out, considering all three production floors, according to Figure 5, "Nelum"; the only production floor located in an upper level was identified, that has a drastic variation with compared to other two floors.

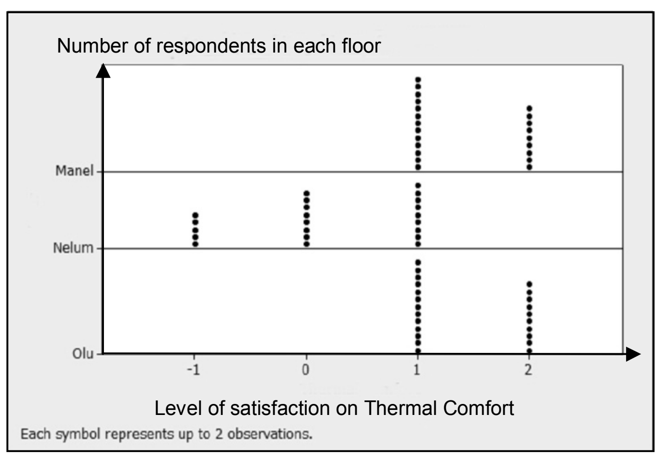

Figure 5 : Comparison of level of satisfaction with respect to thermal comfort in three production floors

Likewise, the most appropriate floor to conduct the questionnaire survey was determined as the only production floor, which was located in an upper level and drawings, documents and the interview, which was held with the maintenance engineer, was helped for the selection. It was perceived that, the selected production floor has more exposure to the sun and less trees surrounding it, which has more tendency of heating up. The longer facades of the building oriented to north-south axis and the shorter facades to east – west and to get the maximum cross ventilation to the floor, the openable glazed windows were placed on all four sides of the walls. There were 10 production lines with sewing machines and center of the floor housed the cutting and administration area with other relevant machinery. Periphery was demarcated by 4'-0' wide walkway and each production line was separated by 3'-0" wide walkways. The layout shows in Figure 6.

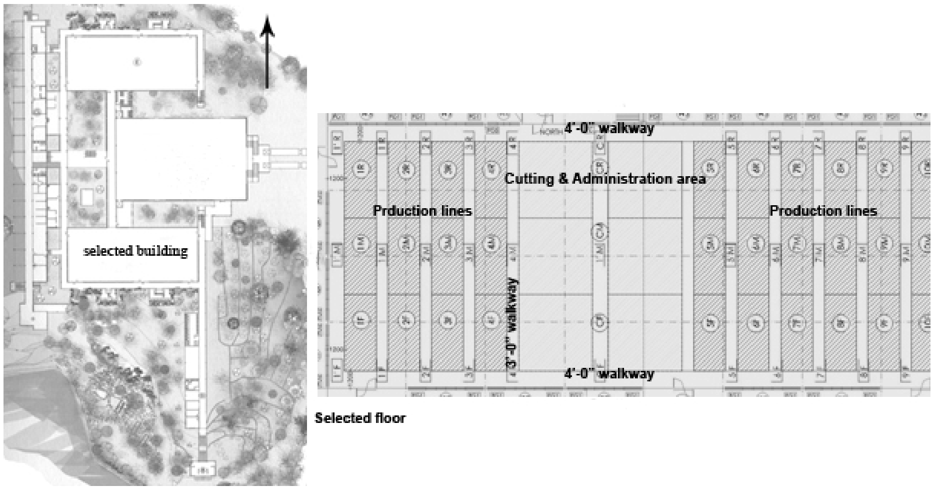

Figure 6: Layout of the complex and the layout plan of the selected floor

From the first section of the detailed questionnaire survey carried out in the selected production floor, there were only few male population and it was only 5.6% of the sample and 94.4% were female population.

The second section contained 4 main sub sections; Temperature, Humidity, Air velocity and overall satisfaction with respect to thermal comfort and each of these sub sections followed series of questions and during the analysis the responses of each subsection were analyzed separately concentrating on the responded population rate and the majority level of satisfaction for the each subsection. Finally, majority level of satisfaction with respect to the aspect of thermal comfort was summarized and illustrated in Figure 7.

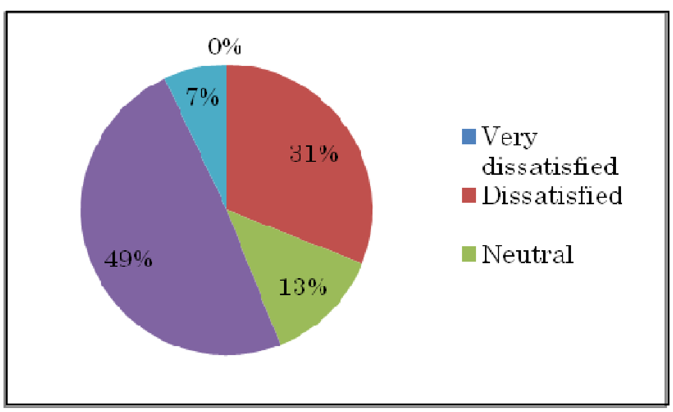

Figure 7: Level of satisfaction of the selected sample with respect to thermal comfort

Figure 7 shows, 13% from the sample population has neutral feeling about the workplace and 31% was not satisfied with the factors effecting to the thermal comfort within their workplace. However, 56% was satisfied with the thermal comfort of the workplace and of the satisfied population, 7% were very satisfied and there were not any, very dissatisfied occupants in the sample. Therefore, finally it can be stated that, though, the majority of 56% of the selected sample was clearly satisfied with the thermal comfort of their workplace, another 31% was not satisfied.

Open comments with respect to indoor environmental quality of the workplace were sated in the third section of the questionnaire and, it was found that from the respondents who have commented on that section, were complained about one common aspect; the uncomfortable situation they feel within the workplace as "too hot".

4. DISCUSSION

Further analysis was carried out to find out the locations of the dissatisfied occupants and when it was marked on the floor plan shown in Figure 8, it was identified, compared to the orientation of the floor, more dissatisfied occupants were located in the area facing towards the "west". This lead to floor divided into 2 zones as Zone 01 and Zone 02.

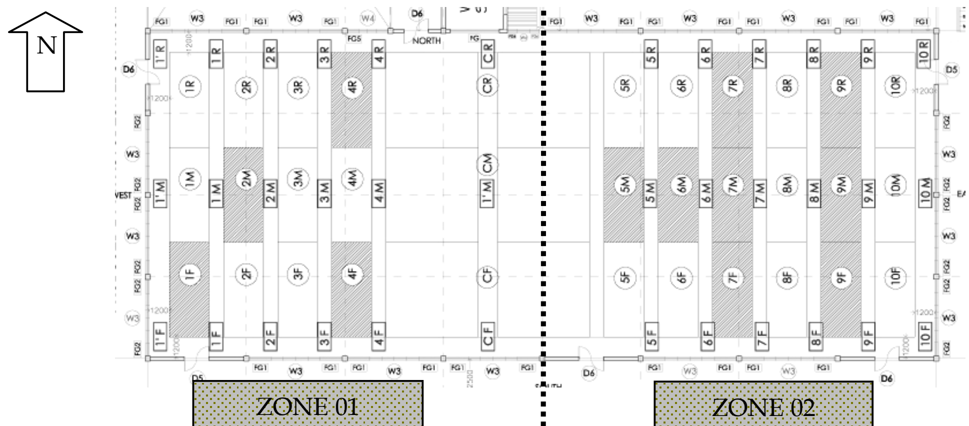

Figure 8: Locations of dissatisfied occupants with respect to Zone 01 and Zone 02

When compared the respondents of each zone, the findings can be illustrated in Figure 9.

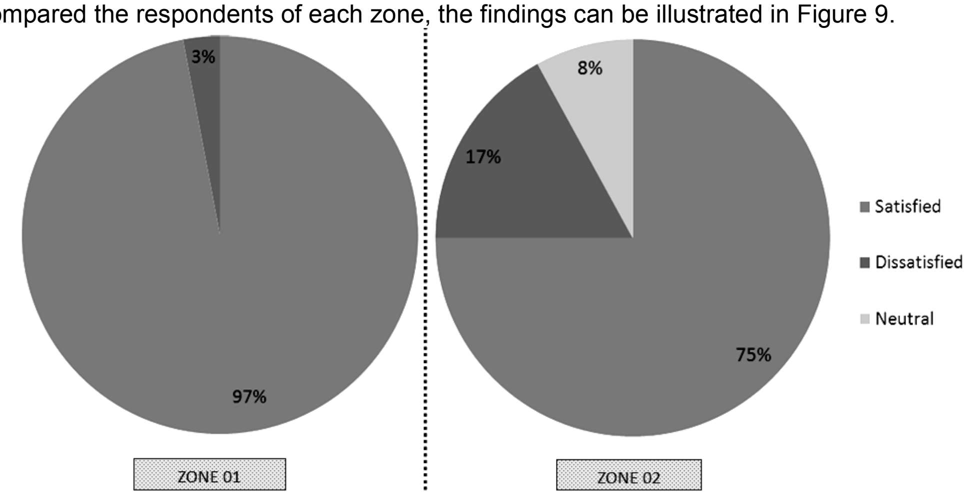

Figure 9: Comparison between Zone 01 & Zone 02

Conversely, according to, Holcim (Lanka) Ltd (2009), production spaces and offices are ventilated and cooled by evaporative cooling units. These units draw in fresh air, filter it, and add moisture to lower the dry-bulb temperature. The air is distributed through a balanced system of ducts and fed into the spaces, which remain under positive static pressure. Indoor air is not recirculated, but extracted by suitably sized exhaust fans to ensure effective moisture and heat removal. The air-exchange rate is about 40 air changes per hour. Indoor air movement is perceptible, about 0.8 meters per second at the minimum. ANSI/ASHRAE Standard 55-2004 allows the thermal comfort zone to be extended upward by about 2.7°C when indoor air velocity is 0.7 meters per second. The psychometric chart at Figure 10 shows, the extended comfort zone at higher indoor air speeds considering ASHRAE 55-2004 and research conducted in tropical climates.

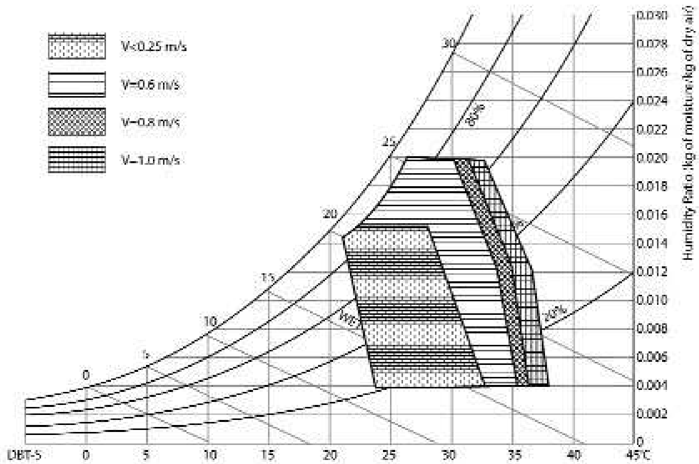

Figure 10: Psychometric Chart from the ASHRAE standard handbook showing extended comfort zone.

The cooling system at the complex is designed to make use of these zones and make use of Section 5.2.3 of ANSI/ ASHRAE Standard 55-2004. Year round, the indoor dry-bulb temperature is up to 3°C cooler than the outdoors, and the indoor relative humidity about ten percent higher than the outdoors. Humidistat in each cooling unit keeps the indoor relative humidity at or below 80 percent. Staff wears short-sleeve shirts, many workers go barefoot. The combination of dressing cool, activity at low metabolic rates, and air movement makes the plant a comfortable working environment.

Though, the work place was within its extended comfort zone, as LEED platinum rated green building, the questionnaire survey proved that, building was incapable to satisfy the occupants with respect to thermal comfort. When analyzed the Figure. 7, from the complete sample, 56% were satisfied and 31% were dissatisfied with the thermal comfort in the work place. According to Dear & Brager (2002), about 80% of the people would be thermally comfortable within a band of 7- 8 ºC about the neutrality temperature in free running buildings and width of psychometric chart can be considered as ±4 ºC from the neutrality temperature. In free running buildings, it is necessary to have some air movement. When the air velocities are significant as found close to fan, it can produce physiological cooling effect. The air velocities can be in the range of 0.25 m/s to 1.0 m/s. Such indoor air velocities can be easily achieved by using table fans or ceiling fans. When the physiological cooling effects are taken into account, the comfort zone can be extended to include a broader range (Jayasinghe & Jayasinghe, 2009). Therefore, it can be seen that, though 56% of majority satisfied with the thermal comfort in the work place, it has not reached the reasonable limit for the minimum percentage to decide the workplace thermally comfortable. This explained, even though the workplace technically satisfied the requirements of thermal comfort, physically the occupants were not satisfied.

Further, according to Figure 8 and 9, it was cleared that, there should be some external factors, which already exist, around the workplace, which are directly affect the thermal comfort of the occupants, which was displayed from the drastic difference between two zones with respect to number of dissatisfied occupants.

5. CONCLUSION

The significant performance of USGBC and the LEED certification program, encourage green buildings within the sustainable build environment, to buildings to be performed as truly as green buildings by following, high environmental standards and difficult procedures. Therefore, it is important to find out, whether LEED certified green buildings can truly satisfy the occupants. POE was conducted, focusing on aspects of thermal comfort to find out the occupant satisfaction on a LEED certified green building and it was mainly contained with a pilot survey and a detailed questionnaire survey based on the key variables of thermal comfort. Though, the building was within its extended comfort zone, from the questionnaire analysis, it was proved; that this selected workplace was not thermally comfortable for the occupants. Simultaneously, two zones were identified in the same floor and it was found that one zone has 22% dissatisfied occupants with respect to thermal comfort, than the other zone and that proved some external factors are effecting to the thermal comfort, which was not discovered in the standards or guidelines. From the research, it was evidenced that, buildings can technically achieve their goals by following established guidelines and standards, however, there can be significant and sensible design aspects beyond that guidelines and standards, which can be the most important aspects that makes the occupants satisfied.

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Socio Economic Impacts of the Guadiana jetty

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Abstract: *The importance of beaches to the tourism industry and the need to protect such resources is not only vital to the economy of nations but for many localities and regions. Coastal erosion is the major reason of disappearance of beaches. As a result, it requires application of a series of engineering techniques. Therefore, hard and soft engineering solutions are used for the shore protection and utilization through recreational purposes. The Guadiana shore protection jetty Protugal and Spain border represents a good example of such a problem. After the jetty was constructed, the west of the river mouth, experienced a continuous trend of accretion, while the down drift sectors were subjected to severe erosion. This jetty is a source of environmental problems and creates long term economic impacts in positive and negative ways to the surrounding coast-line. The present study showed that, the Portuguese side has gained more economic and social benefits, and conversely, the Spanish side has lost their economic income and land due to the jetty construction.*

Keywords: *Guadiana, jetty, erosion, economic impact.*

1. INTRODUCTION

Beaches are scarce resources stretching between the coast and the land. However, 95% of the world beaches are eroding due to human activities and moving dynamics (Pranzini & Rossi, 1995). Beach erosion is a common and expected event. Natural shoreline erosion is not a threat to the coast or to the beach. Erosion and longshore sediment transport are, in fact, an integral part of long term coastal evolution and an important aspect of the dynamic system. Coastal erosion does not necessarily mean the beach is disappearing; it is simply changing its location. The sandy beach exists in a balance between sand supply, beach shape, wave energy and sea-level rise. Most construction on or near the shoreline, changes this balance and reduces the natural flexibility of the beach.

The engineering structures are constructed to protect properties, increase accessibility to transport and improve the economy. These structures cause indirect long term impacts on coastline. A jetty is a coastal engineering structures extending from a shore into a body of water and is usually constructed for several purposes. One of the main reasons is to stabilize one or both sides of the inlet from shifting its position. Jetties also prevent large volumes of sediment from filling in the inlet by permitting its continued navigation (Davis and FitzGerald, 2004; Schwartz, 2005).

Downstream erosion of jetty usually results in need of extend structures to protect greater lengths of the shoreline (Phillips & Jones, 2006). Therefore, alternative soft engineering techniques are used conjunction with natural coastal processes. These included the construction of submerged jetties that reduced the effective offshore depth, and consequently reduced the wave power and erosion of the beach (Aminti et al., 2002). The Beach nourishment method is another soft engineering solution. The new, wider beach, serves shore protection from the impacts of storms and increases recreational benefits with new tourism related opportunities (Benassai et al., 2001).

In addition, the SPICOSA project is mainly focused on the support and implementation of a framework for sustainable development in the area of Coastal Zone Management within the European regions (SPICOSA, 2009). Further, SPICOSA considers the need for better integration of scientific knowledge in the implementation of local developmental policies, while balance of ecological, economic and social considerations at an appropriate level (SPICOSA, 2009). Therefore, under this project, different types of case studies were selected throughout the European directive. The Guadiana Estuary is one of the Study Site Applications (SSA) used to describe the System Approach Framework (SAF). To develop adaptable SAF within the Guadiana estuary, the policy issues were identified. River discharge reduction due to dams, coastal erosion on the Spanish beaches, sediment infilling of tidal creeks and channels on salt marshes and substitution of fine sediment stock with sand intrusion over tidal plans are some of the issues which need to be discussed under the SPICOSA project. Therefore, studies on the behaviour of sediment along the Guadiana coast and effects to social, economic and ecological aspects are of great importance in the development of SSA.

The economic impacts of the Guadiana jetty is related with erosion and land accretion beside the jetty along the beach. The behavior of the land area is well documented before and after jetty construction (e.g. Gonzalez et al., 2001a; 2001b). However, the area of sand accumulated after the construction of a jetty is poorly understood and assessment have been inconclusive. Thus studying the accumulated area of sand gives an idea of the amount of sand accreted within a year (yearly behavior) and can be identify the specific locations of sand accumulation. At the same time, the SPICOSA project aims to discuss the sediment starvation of the Guadiana estuary in theoretical and practical ways. Although the Spanish side of Isla Canela beach is highly eroded due to long shore sediment transport (Dias et al., 2004), the accreted sand area in Portuguese side is an asset to Portugal for economic development. However the long term economic impact of Guadiana jetty due to erosion in Spain Isla Canela beach and accreted sand in Portugal are poorly documented. Therefore, it is important to identify the relationships between the cost of erosion and the cost of accretion in the Guadiana jetty. Thus, primary objective of present research was to assess the long term economic impacts due to jetty construction.

Furthermore, this study provides a precise case study for the economic impacts of hard and soft engineering coastal structures. In the evaluation of the economic impacts of the Guadiana jetty, tourist and property values were considered as main criteria. In here, natural intervention of storms, sea level increases and coastal floods were not considered to describe the economic impacts. Understanding of spatial distribution of pressures that cause erosion along given coastline will useful to identify locations that need mitigating measures for erosion control. Moreover, it is important to understand the present situation, tourist attractions and land use types of the neighboring coastlines of Guadiana jetty for successful future development plans.

2. STUDY AREA

The Guadiana River is a trans-boundary river where the national border of Portugal and Spain runs the last 200 km of the river course. The Guadiana estuary mouth alters sand migration paths across the main estuarine channel and inhibits the obstruction of (international) shipping ways. Therefore, the jetty was built in 1972-1974 (Dias, 1988) in the Guadiana estuary to protect the channel entrance. This jetty belongs to Portugal which is 2040 m in length (Gonzalez et al., 2001). The geographical coordinates on the Portuguese margin of the jetty are 37º10' N and 7º24'E (SPICOSA, 2009). Further, there is a 300 m long groyne in Portuguese side, 1.7 km west (Figure 1) of the jetty to prevent a quick infilling of the adjacent western border of the jetty (Dias, 1988). The Portuguese side of the adjacent coastline comprises two administrative divisions, namely, Monte Gordo and Vila Real de Santo Antonio. The coastal stretch in the Spanish side is a part of Ayamonte municipality which is involves two administrative boundaries, called, Isla Canela and Isla del Moral. Figure 1 shows the satellite image of the area (Year 2000) and administrative boundaries (Municipal) of both countries. In addition, the administrative map shows the main urbanized areas belongs to both countries.

The Guadiana river mouth beach is sandy, which is typical of wave dominated environments. The mouth of the Guadiana estuary is a highly dynamic area, with considerable movement of sediments and associated morphological changes (Morales, 1997; Gonzalez et al., 2001: Lobo et al., 2004). The Guadiana estuary has a semidiurnal mesotidal system, with a mean tidal range of 2.0 m, ranging between maximum values of 3.8 m and a minimum of 0.5 m (Instituto Hidrogra´fico, 1998). The wave regime is characterized by waves of low to medium energy, including both Atlantic swell waves and local sea waves (Morales, 1997; Morales et al., 2006).

Figure 1 Boundary of the Study Area

The offshore coastal wave regime is primarily dominated from west and south west (approximately 50% of occurrences) and south east waves have a significant influence on the beach (with 25% of occurrences) (Costa, 1994; Gonzalez et al., 2001). Therefore, the net annual littoral drift is from west to east (Gonzalez et al., 2001; Morales et al., 2006). Furthermore, due to the jetty, there are modified sediment dynamics within the delta and as a result, there have been considerable changes in its morphology (Morales, 1997). The delta front is made up of several barrier islands such as Isla Canela and Isla del Moral. Also transverse growth of the Monte Gordo Beach spit has occurred on the Portuguese side. The Monte Gordo beach is located along the Portugal coast. However, at present the Guadiana River basin has been under growing pressure from anthropogenic activities such as tourism, agriculture, industry, dam construction and urban pressure (Morales et al., 2006). The Alqueva dam which built in 2002, along the Guadiana River is a risk to the downstream ecosystems, mainly to the estuary and adjacent coastal areas (Morais, 2008). Similarly, the Spanish coast of the Guadiana River mouth has anthropogenic influences due to the tourism industries such as the Isla Canela Tourist Resort and Isla del Moral Tourist Resort village. All of these activities are main considerations in addressing the long term economic impacts of Guadiana jetty.

3. METHODS AND DATA ANALYSIS

This study focused mainly to find economic impacts. Evolution of the area of the Guadiana Estuary mouth was studied mainly by using the available aerial photographs and field investigations. However, in order to apply the selected method, the study area was divided into two different parts: namely the Portuguese side and Spanish side. To describe the economic impacts, Portuguese and Spanish margins were used. The Guadiana jetty, western side of the study area was in between the Portuguese jetty and groyne structure, eastern side study of the area was in between Punta de la Espada to end of Isla del Moral beach.

The available projected aerial photographs (which were done for the Megasig project) were used for this research. All the aerial photographs were scanned and were geo-referenced by using a series of geographic tie points. The projection Datum was Intern_1924_Transverse_Mercator_Megasig. The available aerial photographs are: 1977, 1980, 1985, 1986, 1991, 1994, 1996, 1998, 2002 and 2005. The scale of the photographs varies between 1:8000, and 1:30000. Study area boundaries were demarcated by inspecting the coverage of available aerial photographs.

Areas were drawn using ArcVeiw GIS 3.3 software and ArcVeiw GIS 9.1 software. A series of geomorphologic elements were identified and mapped on all photographs. The criteria for distinction of the areas were drawn by shape, shade of grey (or colour when available) texture and context. The selected boundary of the study area was equal in all aerial photographs. Every map should represent the same geographical boundary when describing the morphological variation. According to this, areas were sketched to all the available aerial photographs. The areas that were drawn on the Portuguese and Spanish side were used to describe the land area change and to evaluate the economic impacts. Furthermore, the area was found in km^2 by using the ArcVeiw GIS 3.3 software.

Moreover, key morphologic and anthropogenic elements were identified during field visits. The aim of the field visit is to explain the present situation and to inspect the economic impacts due to the jetty construction along the Spanish and Portuguese study area. In Spain coast socio-economic changes and locations of beach erosion were inspected from Punta de la Espada to the end of Isla del Moral jetty. On the Portuguese coast field investigation was carried out from the jetty to the groyne structure to inspect sedimentation and the impact of tourists to the beach. Photographs were taken, while walking along the coastline to describe the important features of the coastline. Major economic developments that occurred during the past years in the study area were identified during the field investigations evaluated the economic impacts of these developments. Evaluation of economic impact was limited to the “physical carrying capacity,” which is the number of individuals a beach can physically accommodate. Also, to calculate the economic impacts, cost of the sand on each side was valued by using the cost of sand nourishment and volume of eroded and accreted sand. Furthermore, the impacts to the tourism’s income were calculated by valuing the land area and required land area per tourist.

4. RESULTS AND DISCUSSION

During the field visits, it was identified that the beach section between Punta de la Espada and Isla Canela was subjected to severe erosion (Figure 2). It was clearly found that severe erosion had occurred in the Guadiana river mouth.

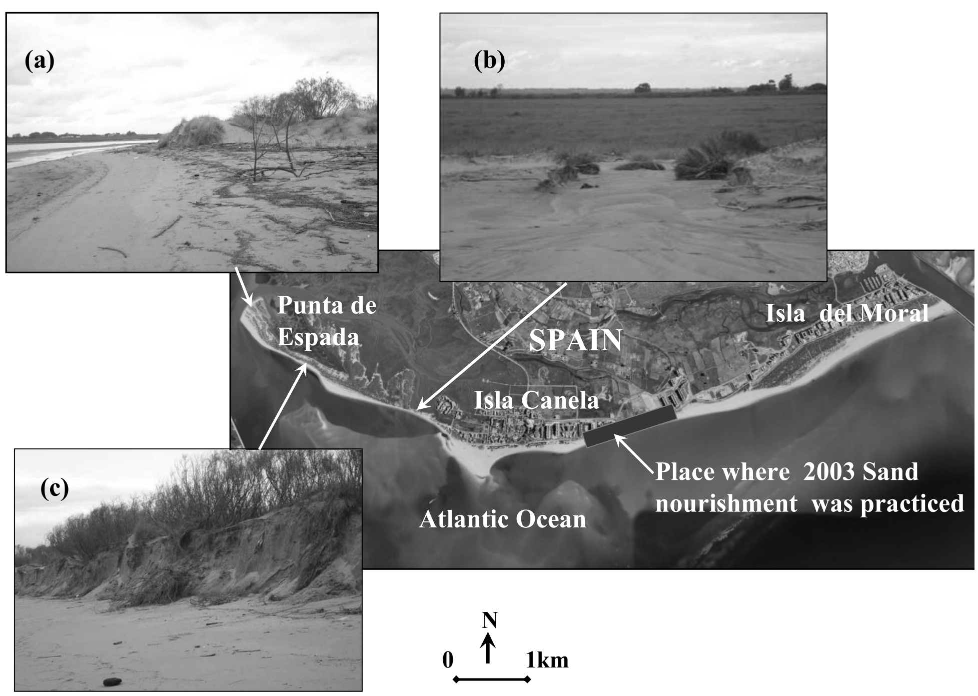

Figure 2 Observed eroded places in Spanish Margin during the field investigation

Also, at the end of the Isla Canela beach, this erosion is severe. Because the jetty is placed perpendicularly to the shoreline and that traps sand on the updrift side by extending out into the water and interrupting the littoral drift, this causes deposition of sand. However, after the water column loses its suspended sand load, its velocity increases. As a result, this causes it to trap around the jetty and pull more sand away from the down-drift side, resulting in downdrift beach erosion. Therefore, with high velocity, waves erode downdrift of the Spanish beach and carried more sediment away from the beach. The end of Isla Canela beach directly contact to the longshore current paths. Consequently, high velocity longshore currents bring more sand away from the Isla Canela beach and caused severe erosion.

In 2003, sand nourishment took place along the Spanish side which is shown in Figure 2. For the artificial nourishment 414 000 m^3 of sand was used with a cost of 2.5 Million Euros (Institute of Hydrographic, Spain). This took place at the end of Isla Canela tourist village. By analyzing the 2005 aerial photographs, it can be clearly identified that, the place, where sand nourishment was practiced is widen. Consequently, it can be said that five years after sand nourishment, beach become wider. On the other hand, the Western end of artificial sand nourished area now (year 2010) has a much wider beach section as found in field visits.

Further, from field investigations it was found that new sediment spits developed in front of the Spanish coast as shown in Figure 3. Also, inlets are sinks of sediment in which ebb and flood shoals are formed. These ebb and flood shoals are supplied predominately by longshore sediment transport. Thus, the construction of a jetty at an inlet will alter the tidal currents; disturb the longshore transport and cause formation of new ebb and flood shoals. Therefore, the jetty construction at an existing inlet will confine the ebb-tidal current and push the ebb shoal offshore from its original location. Also, part of the longshore littoral drifts pass to sea and deposit in the bottom. This ebb shoals migrates onshore and give the appearance of accretion by longshore transport on the down-drift side of the inlet. This will be continuing until the disappearance of the previous portion of the ebb shoal and removing the sand source. The down-drift and, possibly, up-drift beaches will then begin to erode. However, these sediment pathways for natural bypassing depend on wave conditions particularly between typical seas (Militello and Kraus, 2003). However, the Guadiana jetty is long relative to surf zone, therefore, pushing the ebb shoal to outside the active littoral zone.

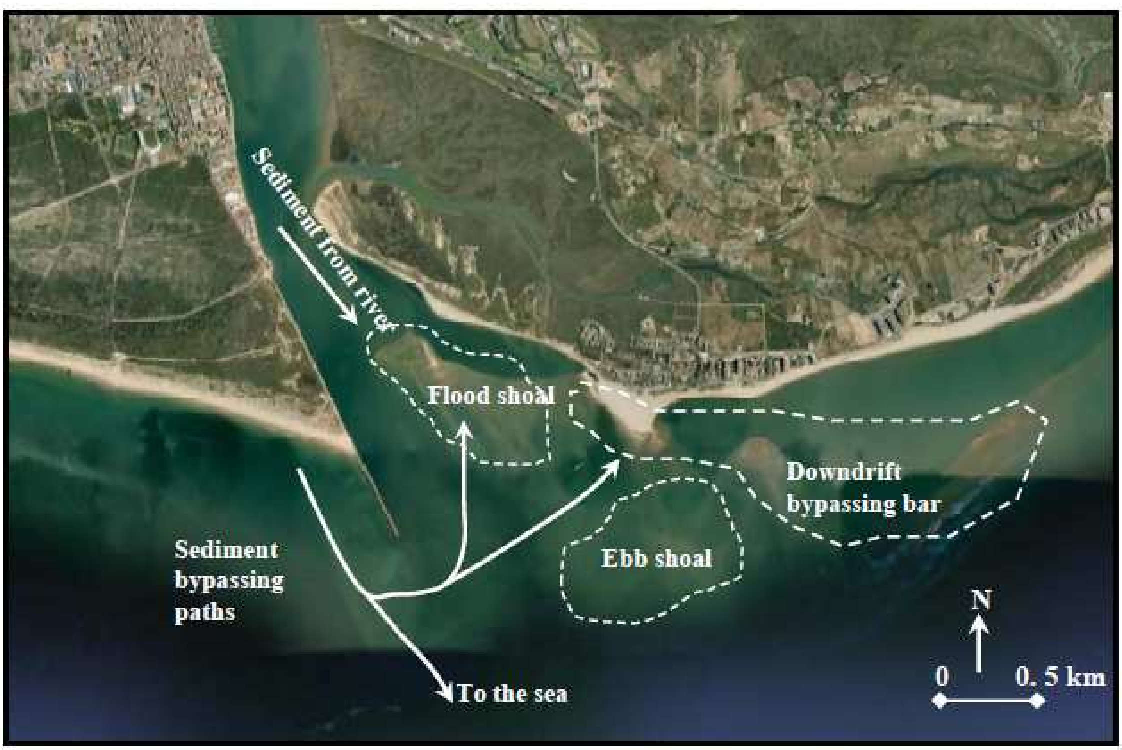

Figure 3 Sediment behaviour of Guadiana river mouth

Therefore, these new ebb shoal bars or O Brill bank is migrating to the Spanish downdrift side. Finally, at present it can be seen that the new sand spits occur in the eastern estuary margin. Further, ebb shoals and particularly flood shoals offer a good source of material for the beach nourishment (Militello and Kraus, 2001).

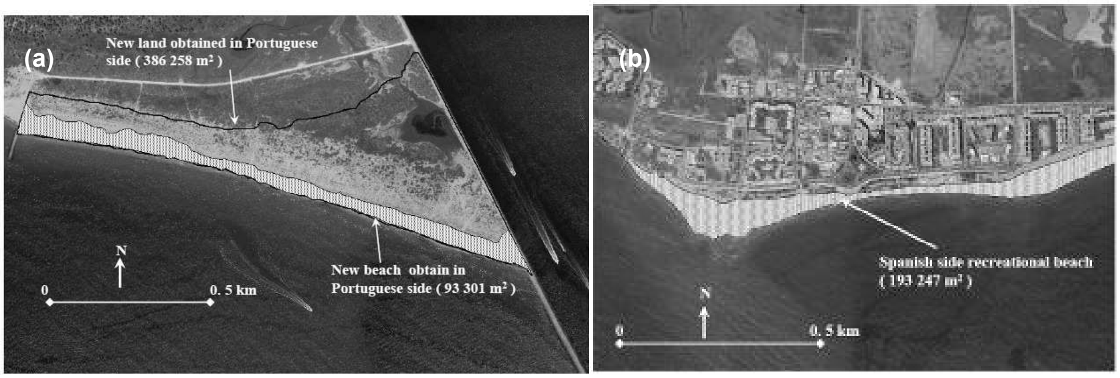

Figure 4 Area use for the beach recreational activities in year 2005
(a) Portuguese Beach, (b) Spanish Beach

After analysing the aerial photographs, it was identified that, the western estuary margin has an accreted sand area of 386 258 m^2 after the construction of the jetty between 1977 and 2005. Out of that new land area, a portion of 93 301 m^2 belongs to the Portuguese beach (Figure 4(a)). This new beach can be used for tourism purposes. At presen,t without proper facilities, this new beach is being used for the tourism industry. On the other hand, loss of land in the Spanish side due to the erosion from 1977 to 2005 (boundary is Punta de la Espada to End of Isla Canela beach) is 245 462 m^2. However, 193 247 m^2 area is used for beach recreational purposes in front of the Isla Canela beach in year 2005 (Figure 4(a)). For recreational purpose, one tourist needs 10 m^2 of surface area without overcrowding the beach (Da Silva et al., 2007). Therefore, as a result of accretion, 9 330 tourists can use the Portuguese beach and because of erosion, the Spanish beach loss 24 546 tourists.

Table 1 Tourist economic income gain in Spanish and Portuguese sectors

(Negative value represents the loss of income)

Study Site Location	Recreational beach Land Area (m^2)	Required land area per tourist (m^2/person)	Number of Tourists	Tourist Income (Euro/Person)	Total income (Euro in millions)
Portuguese	93 301	10	9 330	500	4.6
Spanish	193 247	10	19 325	500	- 9.6

Furthermore, it was estimated that approximately 500 Euros income can be gained from each person (Institute of Tourism Spain 2001 statistics). According to the Table 1, it can be seen that the Portuguese economy gains nearly 4.6 million Euros and the Spanish the economy loses 9.6 million Euros within a year. One can clearly identified that nearly double of the Portuguese income is loss by the Spanish government due to the beach erosion. Tourism income directly and indirectly impacts economic growth, jobs, and foreign exchange. The Portuguese's economy has ability to utilize a totally new beach area for tourism purposes. On the other hand, the Spanish economy lost 9.6 million of income including direct and indirect jobs, foreign exchange, hotels, property, services and goods.

5. CONCLUSION

Addressing economic and social impacts of Guadiana's river mouth is a vital issue at present, due to the long term impacts experienced by both bordering countries of Portugal and Spain. According to the present research, it was found that the Portuguese gained new income due to the jetty construction while at the same time the Spanish economy lost double that of the Portuguese side's income. Therefore, protecting the Spanish recreational beaches from further erosion is a highly important aspect to the Spanish economy. On the other hand, according to the above results, total area of Spanish beach increases after sand nourishment. Hence, as a solution for future erosion, sand nourishment can be applied on the Spanish eroded beach especially end of the Isla Canela beach. The Spanish side's newly obtained ebb shoal sand volume is a significant economic gain. Sediments in ebb shoal sand are same sizes that are present in Spanish beach. Thus, sand in ebb shoal can be used for the sand nourishment on the Spanish beach. This will reduce the cost of sand transport for the sand nourishment project and reduce the erosion. Even though, sand nourishment is a costly process, in the long term nourished land area contributes to a reduction in the effects of the Spanish economy due to beach erosion. Thus management of sediment deprivation and related coastal erosion problems along the Guadiana jetties should provide long term benefits to Spain and Portugal. Moreover, the SPICOSA project is currently seeking to identify the present situation and involve in finding a long term coastal zone management system and alternative implementation. Thus, present research is part of the SPICOSA project, which has been primarily focused to address the economic impacts and the processes of sedimentary behavior after the construction of Guadiana jetty.

6. ACKNOWLEDGMENTS

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A set of surrogate parameters to evaluate harvested roof runoff quality

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Abstract: *This paper presents the outcomes of a research project, which focused on developing a set of surrogate parameters to evaluate roof runoff quality using simulated rainfall. Use of surrogate parameters to evaluate roof runoff quality has the potential to enhance the rapid generation of harvested rainwater quality data based on on-site measurements and thereby reduce resource intensive laboratory analysis. Pollutant buildup and washoff samples were collected from a model roof surface placed in a residential suburb in Gold Coast, Queensland State, Australia. The collected samples were tested for a range of physio-chemical parameters which are key indicators of nutrients, solids and organic matter. The analysis revealed that [total dissolved solids (TDS)]; [electrical conductivity (EC),turbidity (TTU)] as appropriate surrogate parameters for dissolved total nitrogen (DTN) and total solids (TS) respectively. No surrogate parameters were identified for phosphorus.*

Keywords: *roof surface pollutants, stormwater pollution, rainwater harvesting, surrogate parameters*

1. INTRODUCTION

Roof surfaces have been identified as important contributor of pollutants into urban stormwater runoff in urban areas. Even during the small rain vents higher fraction of pollutants which are accumulated on urban roof surfaces are washed off and get into the stormwater runoff. Hence, deterioration of receiving water quality is a significant concern. However, there is a growing awarwenss on using harvested ruoff runoff wherer water scarcity is a significant concern. Furthermore, increase attention has been already paid for the potential of using the harvested rainwater for domestic purposes (Evans et al. 2006, Meera and Ahammed, 2006).

Rainwater harvesting is considered as a sustainable water management practice as it provides a feasible approach to reduce the pressure on natural water resources. Currently, harvested roof runoff is primarily used for non potable purposes. However, due to the continuing urbanisation and scarcity of natural water resources, techniques to use rainwater as a potable water supply are increasingly investigated. Zorn and Wheatley (2009) noted that harvested rainwater can be used for a number of domestic purposes such as toilet use and washing machine use without undergoing treatment.

Currently, there has been growing interest also in the use of harvested rainwater as an alternative source for drinking water (Meera and Ahammed, 2006; Zorn and Wheatley, 2009). In determining the end use and the potential success of such an option, the possible problems associated with water quality need to be analysed and the feasibility of using rainwater as a source of water for household use will need to be determined. As noted by several researchers (for example Meera and Ahammed, 2006), harvested rainwater can contain significant amounts of pollutants such as heavy metals, nutrients and pathogens. Evans et al. (2006) stated that the potential pollutants in rainwater harvesting systems are likely to arise from depositions by birds, small mammals, airborne micro-organisms and chemical contaminants. The decay of these pollutants within a rainwater tank can also contribute to pollution. There is no clear agreement on the physico-chemical and microbiological quality and health risk associated with roof harvested rainwater. Several researchers have suggested that the use of roof runoff for potable purposes can lead to a possible health risk (for example Lye, 2002; Evans et al. 2006). In this context, approaches have been made to protect the harvested rainwater quality by implementing control measures such as first flush devices and filters.

Development of effective control measures to safeguard the harvested rainwater quality requires in-depth knowledge on pollutant build-up and wash-off characteristics on roof surfaces. At present, data to generate the requisite knowledge is scarce, partial or sometimes contradictory (Gromaire et al. 1999).
The lack of knowledge on key water quality parameters and pollutant processes on roof surfaces are mainly attributed to difficulties in planning and conducting stormwater quality monitoring programs (Gromaire et al. 1999). Investigation of a large number of water quality parameters is time consuming and resource intensive (US FHWA 2001). Furthermore, dealing with a range of variables in stormwater runoff monitoring programs requires sophisticated knowledge of these variables related to the wash-off process (US FHWA 2001;Martinez 2005). On the other hand, cost effective and robust methods for the continuous measurement of pollutant concentrations are not yet fully developed (Grayson et al. 1996). Therefore, it is important to identify a suite of easy-to-measure surrogate parameters which can be correlated to water quality parameters of interest. The relationships between key water quality parameters and its surrogate parameters will provide a convenient approach to evaluate the quality of roof runoff directly, without carrying out resource intensive laboratory experiments. However, the utility of this approach depends on the quality of correlations between these different sets of parameters (Grayson et al. 1996).

2. MATERIALS AND METHODS

2.1. Sampling sites

A residential suburb, in Gold Coast, South East Queensland, Australia was selected for field investigations. The study sites were located within a typical urban residential suburb, Coomera, in Gold Coast. This residential suburb was selected due to its high rate of rainwater harvesting. Furthermore, as loads and types of pollutants on roof surfaces are significantly influenced by the land use, the understanding developed for residential roof surfaces would provide knowledge specific to rainwater harvesting (Egodawatta et al. 2009).

2.2. Research tools

The study was carried out using two model roof surfaces. Use of model roofs with a surface area of 3 m^2 eliminated the possible heterogeneity of the surface characteristics of actual roof surfaces. This in turn will help to enhance the transferability of the research outcomes. The characteristics of the model roofs closely replicated actual roof surfaces. The model roofs were made from two different cladding materials; corrugated steel and concrete tiles. These are the most widely used roofing materials in South East Queensland, where the study sites were located. The model roofs were mounted on a scissor lifting arrangement so that they can be lifted to the typical roofing height for pollutant accumulation and lowered to the ground level for sample collection. This arrangement was used to avoid the practical difficulties inherent in investigating pollutant build-up and wash-off on actual roofs.

Figure 1 Model roof surfaces used in the study

For the study of pollutant wash-off, rainfall simulation was employed in order to eliminate the dependency on natural rainfall. This approach provided greater flexibility and control of the fundamental rainfall parameters such as rainfall intensity and duration and its chemical quality. (Herngren et al. 2006; Egodawatta and Goonetilleke, 2008). The specially designed rainfall simulator (see Figure 2) consisted of an A-frame structure with three Veejet 80100 nozzles connected to a nozzle boom and standing at 2.5 m above the ground level. The nozzle boom can swing in either direction with controlled speed and delay. This enables the simulator to be calibrated for different intensities. A detailed description of the rainfall simulator can be found in Herngren (2005).

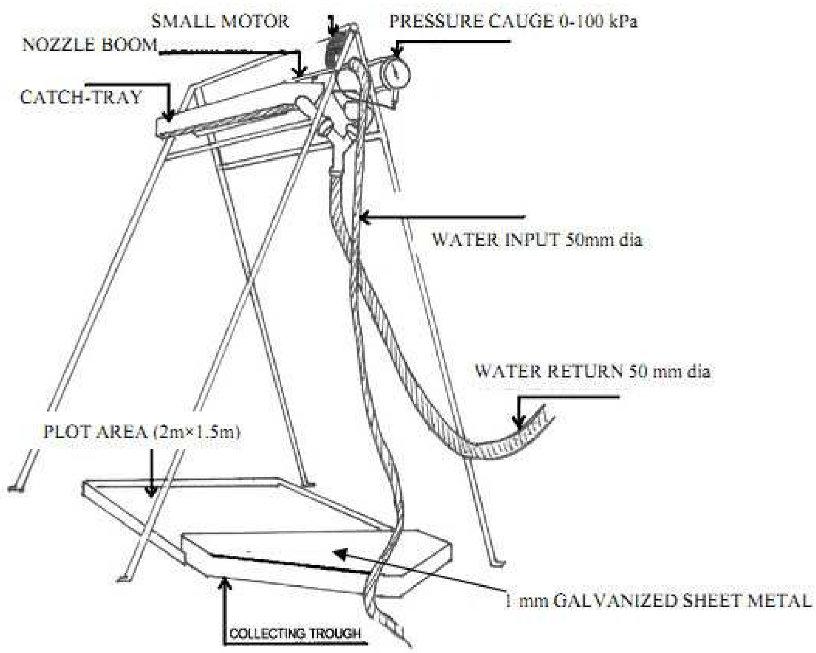


Figure 2 Schematic diagram of the rainfall simulator used for the study (Adapted from Herngren et al. 2005)

2.3. Sample collection and testing

Wash-off sample collection was carried out for six simulated rain events. Average rainfall intensities of 65 and 86 mm/hr intensities were simulated for the first sampling episode, 115 and 135 mm/hr intensities for the second sampling episode and 20 and 40 mm/hr intensities for the third sampling episode. The selection of these rainfall intensities and durations were based on the regional rainfall events in the Gold Coast area. The selected intensity range represents more than 90% of the regional rainfall events (Egodawatta 2007).

Half of the roof surface used for wash off sampling as the other half was used for pollutant build-up sampling (Miguntanna 2009) This was done by fixing the gutter at the other half of the roof surface (see Figure 3). Build-up samples were collected by washing the roof surface four times with approximately 7 L of deionised water. A soft brush was also used to brush the surface while washing. A roof gutter was placed to collect the sample and to direct it to a polyethylene container kept underneath the gutter opening (see Figure 3). The gutter was thoroughly washed before and after each sample collection. For the washoff sampling 20, 86 and 135 mm/hr intensities were simulated on the corrugated steel roof surface and 40, 65 and 115 mm/hr intensities were simulated on the concrete tile roof surface. For the simulations, the rainfall simulator was placed exactly above the lowered model roof. The simulator was raised to maintain 2.5 m average height from roof to nozzle boom of the simulator. Simulations were conducted until relatively clean runoff was observed. The wash-off was directed to the containers which were kept underneath the gutter as shown in Figure 3. Finally, the model roof was lifted to typical roofing height and left at the site until the next sampling episode.

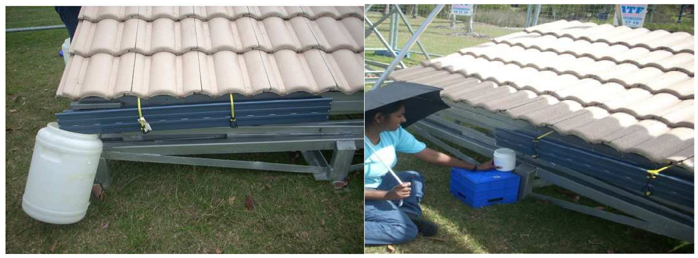


Figure 3 Collection of a) pollutant build-up samples; b) Wash-off samples

Finally, all the collected samples were tested for TS, TOC, NO2-, NO3-, TKN, TN, PO_4^{3-} and TP according to the methods specified in Standard Methods for the Examination of Water and Waste Water (2005).The details of the test methods used is given in Table 1.

Table 1 Details of the test methods used

Parameter	**Test Method**	**Comments**
pH	4500H (APHA, 2005)	Combined pH/EC meter was used
Electrical conductivity (EC)	2520B (APHA, 2005)	Combined pH/EC meter was used
Turbidity (TTU)	2130B (APHA 2005)	Turbidity meter was used
Total suspended solids(TSS) and Total dissolved solids (TDS) Total solids (TS)	2540D and 2540C (APHA, 2005) Addition of TSS and TDS taken as TS	Total sample filtered through 1 µm glass fibre filter paper and analysed for TSS. The filtrate tested for TDS.
Total organic carbon (TOC) Dissloved organic carbon (DOC)	Measured using according to the 5310C (APHA, 2005)	Shimadzu TOC-5000A Total Organic Carbon Analyzer was used
Nitrite nitrogen (NO_2^-)	4500 -NO_2^--B (APHA, 2005).	SmartChem 140 discreet analyser was used
Nitrate nitrogen (NO_3^-)	4500 –NO_3^--F (APHA, 2005).	SEAL discrete analyser was used
Total kjeldahl nitrogen (TKN)	351.2. (US EPA 1993)	
Total nitrogen (TN)	Addition of NO_2^-, NO_3^- and TKN	
Ortho-Phosphate (PO_4^{3-})	4500-P-F (APHA, 2005).	SEAL discrete analyser was used
Total phosphorus (TP)	365.4. (US EPA, 1983).	

2.4. Data analysis

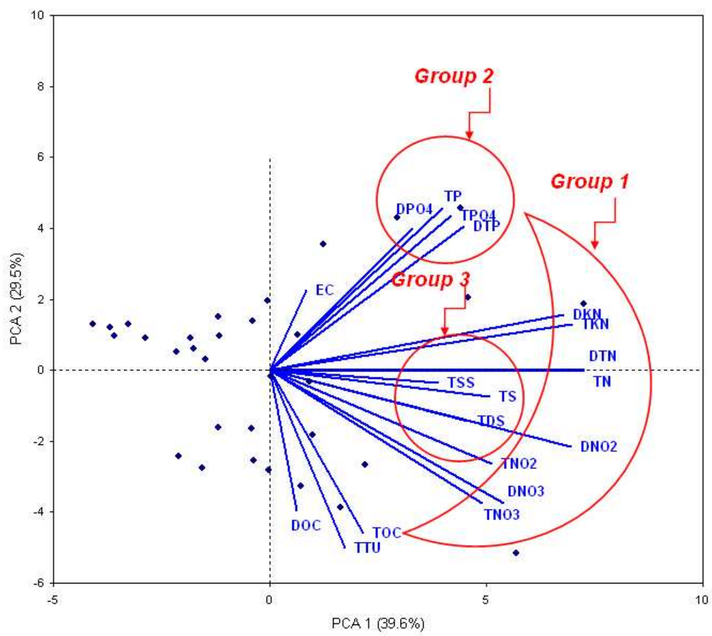
Data analysis was primarily carried out using two common multivariate data analysis techniques; Principal Component Analysis (PCA) and Partial Least Squares regression (PLS). PCA has been widely used as a pattern recognition technique in numerous water quality research studies (for example Settle et al., 2007). In the PCA biplot, vectors representing parameters which form an acute angle are considered as correlated parameters and those which are perpendicular are considered as uncorrelated. PLS is a well known factor analysis method which is principally applied for prediction. Detailed description of PCA and PLS and its applications can be found in research literature (for example Kim et al., 2007; Kokot and Phuong,1998).

2.4.1. Selection criteria for the identification of surrogate parameters

The physio-chemical monitoring of stormwater runoff quality is constrained by a number of factors associated with the measuring of key water quality parameters such as laboratory facilities, sophisticated operational techniques of instruments and extensive resources (Settle et al. 2001). Therefore, the identification of a set of easy to measure parameters as surrogates for key water quality parameters would enhance stormwater quality monitoring studies. Among the parameters given in Table 1, EC, TTU, TSS, TDS, TOC and DOC can be considered as relatively easy to measure. Therefore, special attention was given to finding correlations for nitrogen and phosphorus compounds with pH, EC, TTU, TSS, TDS, TOC and DOC.

3. RESULTS AND DISCUSSION

Figure 4 shows the PCA analysis biplot. The PC1 versus PC2 biplot accounts for almost 70% of data variance. The main purpose of the PCA was to identify the correlated parameters and group them depending on their correlations. Consequently, the potential surrogate parameters were identified for those groups separately. The correlation matrix which is resultant from PCA shows the degree of correlation among the parameters (Table 2). The correlation coefficient greater than 0.50 was considered as strongly correlated parameters and parameters with 0.35-0.50 were considered as having some correlation.



Note: TTU- Turbidity; EC- Electrical conductivity; TNO2- Total nitrite-nitrogen; DNO2- Dissolved nitrite-nitrogen TOC- Total organic carbon; DOC- Dissolved organic carbon; TNO3- Total nitrate-nitrogen; DNO3- Dissolved nitrate- nitrogen; TKN- Total kjeldahl nitrogen; DKN- Dissolved kjeldahl nitrogen; TN- Total nitrogen; DTN- Dissolved total nitrogen; TSS- Total suspended solids; TDS- Total dissolved solids; TS- Total solids; TPO4- Total Phosphates; DPO4- Dissolved Total Phosphates; TP- Total phosphorus; DTP- Dissolved total phosphorus.

Figure 4 PCA biplot for all the physico- chemical parameters for roof Surfaces

Table 2 Correlation matrix obtained from PCA

Correlation Matrix																			
	EC	TTU	TSS	TDS	TS	TOC	DOC	TNO2	DNO2	TNO3	DNO3	TKN	DKN	TN	DTN	TPO4	DPO4	TP	DTP
EC	1.000																		
TTU	-0.170	1.000																	
TSS	0.585	0.338	1.000																
TDS	0.245	0.504	0.755	1.000															
TS	0.413	0.463	0.914	0.956	1.000														
TOC	-0.400	0.724	0.105	0.194	0.167	1.000													
DOC	-0.496	0.551	-0.017	0.057	0.028	0.716	1.000												
TNO2	-0.477	0.444	0.037	0.234	0.161	0.584	0.475	1.000											
DNO2	-0.160	0.502	0.401	0.647	0.580	0.499	0.297	0.814	1.000										
TNO3	-0.059	0.669	0.181	0.190	0.199	0.641	0.403	0.685	0.598	1.000									
DNO3	0.002	0.697	0.246	0.267	0.275	0.664	0.408	0.649	0.601	0.978	1.000								
TKN	0.065	-0.073	0.338	0.457	0.434	0.104	-0.064	0.563	0.756	0.239	0.250	1.000							
DKN	0.053	-0.125	0.260	0.416	0.373	0.041	-0.112	0.559	0.732	0.198	0.215	0.974	1.000						
TN	-0.007	0.153	0.366	0.508	0.478	0.297	0.071	0.693	0.865	0.458	0.464	0.965	0.921	1.000					
DTN	0.001	0.147	0.317	0.503	0.453	0.270	0.043	0.704	0.866	0.505	0.578	0.947	0.954	0.972	1.000				
TPO4	0.229	-0.515	0.132	-0.019	0.047	-0.346	-0.335	0.109	0.175	-0.136	-0.132	0.651	0.674	0.511	0.522	1.000			
DPO4	0.148	-0.431	0.065	-0.125	-0.049	-0.288	-0.280	0.080	0.083	-0.107	-0.116	0.491	0.495	0.383	0.375	0.948	1.000		
TP	0.328	-0.541	0.187	0.014	0.092	-0.403	-0.389	0.027	0.122	-0.197	-0.182	0.636	0.660	0.480	0.493	0.980	0.896	1.000	
DTP	0.396	-0.438	0.300	0.192	0.253	-0.367	-0.342	0.101	0.223	-0.167	-0.128	0.656	0.682	0.511	0.535	0.877	0.777	0.894	1.000

Figure 4 and Table 2 leads to the following conclusions:

- TNO3, DNO3, TNO2, DNO2, TN, DTN, TKN and DKN are strongly correlated to each other (See group 1);
- TPO4 ,DPO4, TP and DTP are strongly correlated to each other (See group 2);
- and
- TSS, TDS and TS are strongly correlated to each other (See group 3).

Based on the conclusions derived from the PCA biplot and the correlation matrix, potential surrogate parameters were identified for nitrogen, phosphorus and solids separately.

I. Identification of potential surrogate parameters for nitrogen compounds

All nitrogen compounds show good correlation to each other (Figure 4 and Table 2). According to several research findings, TN in roof surface runoff is a significant stormwater pollutant (Huang et al. 2007). Exploring the raw data matrix as seen in Table 3, the dissolved fraction of TN is the dominant form of nitrogen in roof surface runoff which is around 80%. Consequently, DTN was selected as the most representative parameter for all nitrogen compounds. The surrogate parameters identified for DTN would be the surrogate parameters for all nitrogen compounds.

Table 3 Mean concentrations of nitrogen compounds

Site ID	nitrogen parameter (mg/L)				Percentage in the dissolved fraction (%)	
	TKN	DKN	TN	DTN	TKN	TN
Roofs	0.54	0.44	0.70	0.57	80	81

According to PCA, DTN was strongly correlated to TDS. Therefore, TDS was selected as the best indicator of dissolved total nitrogen. This selection was further supported by the correlation coefficient of TDS and TN which is 0.503 (Table 2). This correlation was evident in raw data matrix also by indicating DTN decreases with decreasing TDS concentrations for all the intensities. Hence, TDS can be considered as a potential surrogate parameter for DTN in roof surface runoff.

II. Identification of potential surrogate parameters for phosphorus compounds

As in Figure 4 and Table 2 all phosphorus compounds are strongly correlated to each other with correlation coefficients of greater than 0.750. TP is the sum of all forms of phosphorus. Exploring the raw data matrix, it was noted that around 65% of total phosphorus is in particulate form (See Table 4). Therefore, TP can be considered as the indicator parameter for phosphorus in urban roof surface.

Table 4 Mean concentrations of phosphorus compounds

Site ID	Phosphorus parameter (mg/L)				Particulate percentage (%)	
	$TPO4^{3-}$	DPO_4^{3-}	TP	DTP	PO_4^{3-}	TP
Roof surface	1.80	0.64	1.85	0.64	65%	66%

According to Figure 5 TTU and TOC show negative correlation with TP. According to Table 2, correlation coefficient of TP with TTU and TOC are-0.541 and -0.403 respectively. However, exploring the raw data matrix, this negative correlation was not evident for all the intensities. It indicates TP concentration decreases with decreasing TOC and TTU concentrations which suggests positive correlation among the parameters. This pattern of variation was contradictory to the observations noted in PCA analysis. Therefore, identification of surrogate parameters for phosphorus was not successful.

III. Identification of potential surrogate parameters for TSS, TDS and TS

The identification of surrogate parameters for solids is also important as it can provide a convenient method to measure TSS, TDS and TS which are the key indicators of solids in roof runoff, based on simple field measurements such as EC and TTU. According to PCA it was noted that TSS is strongly correlated to EC and TDS is strongly correlated to TTU with correlation coefficients of 0.585 and 0.504 respectively. However, the correlation of TSS with EC and TDS with TTU are contradictory to the findings of several researchers who noted that EC and TTU as potential surrogate indicators for TDS and TSS respectively (Gippel 1995; Zeng and Rasmussen 2005). Furthermore, the correlation of TSS with EC and

TDS with TTU were not clear in the raw data matrix also. Therefore, identification of potential surrogate parameters for TSS and TDS was not successful for roof surface wash-off. However, TS shows limited correlation to EC and TTU with correlation coefficients of 0.413 and 0.463 respectively. As evident in raw data matrix also TS decreases with decreasing EC and TTU for most of the intensities. Therefore, both EC and TTU were considered as potential surrogate indicators for TS in roof surface runoff.

Table 5 presents the summary of the potential surrogate parameters which were obtained for nitrogen compounds, phosphorus compounds and solids in the runoff from both road and roof surfaces.

Table 5 Potential surrogate water quality parameters for nitrogen, phosphorus and solids

Constituent	Key indicator	Potential surrogate parameter
Nitrogen	Dissolved total nitrogen(DTN)	Total dissolved solids (TDS)
Phosphorus	Total phosphorus (TP)	Not found
Solids	Total suspended solids(TSS) Total dissolved solids (TDS) Total solids (TS)	Not found Not found Electrical conductivity(EC) Turbidity (TTU)

4. CONCLUSIONS

It is evident that roof surfaces as a significant contributor to stormwater pollutant load in urban area. The study was focus on identification of a set of easy to measure parameters as surrogates for key water quality parameters which would enhance harvested rainwater monitoring. For this purpose, PCA was used as a principal data analysis tool for identification of surrogate water quality parameters. It was noted that Dissolved total nitrogen (DTN) can be used as a representative parameter for all nitrogen compounds in wash-off while Total phosphorus (TP) can be used as a representative parameter for phosphorus compounds. The study identified Total dissolved solids (TDS) as a potential surrogate parameter for nitrogen. In addition, it was found that both Turbidity and EC can be used as surrogate parameters for Total solids (TS). However, the study was failed to identify surrogate parameters for phosphorous. The surrogate parameters have the potential to enhance the harvested roof runoff quality investigations without resource intensive laboratory based analysis of key water quality parameters. This knowledge is essential for providing effective mitigation actions to safeguard the harvested roof runoff quality in urban land uses.

5. ACKNOWLEDGMENTS

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Relation between changes in the modal properties and structural changes in an existing steel truss bridge

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Abstract: *This paper describes an investigation of the applicability of changes in modal properties of a bridge to structural health monitoring. Vibration measurements were conducted at an existing Warren truss bridge for road traffic in-service for more than 40 years. Experimental modal analysis applied to the measured data identified global vibration mode of a bridge span and local vibration modes dominated by vibrations of diagonal members. There were a partial fracture, cracks and pitting corrosions in diagonal members of the truss bridge detected during recent visual inspections and some of those damages were repaired during the investigation described in this paper. The effects of those structural changes on the modal properties identified were discussed with the results of finite element analysis so as to investigate the applicability of modal properties changes in global and local vibration modes to the identification of structural changes in truss bridges.*

Keywords: *vibration-based structural health monitoring, natural frequency, modal damping, truss bridge, vibration measurement*

1. INTRODUCTION

Recently, structural health monitoring has drawn more attention from bridge engineers, particularly in developed countries where the design life of substantial number of bridges is approaching to its end. The tragic collapse of the I-35W Mississippi River Bridge in Minnesota in the United States in 2007 may have emphasized needs for appropriate structural health monitoring. In Japan, extensive inspections after the collapse of the bridge in Minnesota revealed cracks and fractures in members of steel truss bridges, some of which reached a complete loss of the cross section of a member.

In health monitoring of bridges, visual inspection has been the principal technique, although the reliability of this subjective technique heavily depends on the skills and experiences of the inspector. Objective techniques for structural monitoring should be useful to assist the visual inspection. A possible objective technique that has been investigated worldwide is vibration-based health monitoring (e.g., Doebiling *et al*, 1996; Balageas *et al*, 2006; Boller *et al*, 2009). Most techniques proposed for vibration-based monitoring are based on the identification of changes in the dynamic characteristics of structure from recordings of structural vibration induced by various natural and artificial sources (e.g., Siringoringo & Fujino, 2008). It is assumed that structural damages are associated with changes in the mass, stiffness and damping of the structure that yield changes in the dynamic characteristics, such as natural frequencies, mode shapes, and modal damping.

The objective of the present study was to investigate the relation between changes in the natural frequencies and modal damping and structural changes in an existing steel truss bridge. There were local damages found recently in the bridge used in this study. Sets of vibration measurements were conducted to obtain vibration data for different structural states. The contents of this paper include a part of outcomes from studies reported in Yoshioka *et al* (2008, 2009, 2010a, b) that was summarised in Matsumoto *et al* (2010).

2. FIELD MEASUREMENTS

2.1. Bridge used in this study

A bridge over a river for road traffic in-service from 1965 was used in this study. The bridge consisted of five separated spans, each of which was a Warren truss with a span length of 70.77 m and a width of 6.0 m (Figure 1). The tension diagonal members had a H-section, whereas the compression diagonal members had a box section, as shown in Figures 1 (c, d). There were eight or nine oval holes in the web of each tension diagonal members, except those at the ends of each span, for the reduction of the weight of steel.

During a visual inspection in July 2007, a partial fracture was found near the bottom end of a longest tension diagonal member (D5 in Figure 1), which resulted in loss of the half of its cross section. There were cracks in four other longest tension diagonal members also. In August 2007, those damages were repaired by fixing additional steel plates with the same thickness as that of the member (i.e., 8 mm) by high strength bolts to cover both sides of the flanges and web for a length of about 1.5 m from the end of the member. Additionally, there were pitting corrosions in several shortest diagonal members (D1 in Figure 1) detected in the following inspections in 2009.

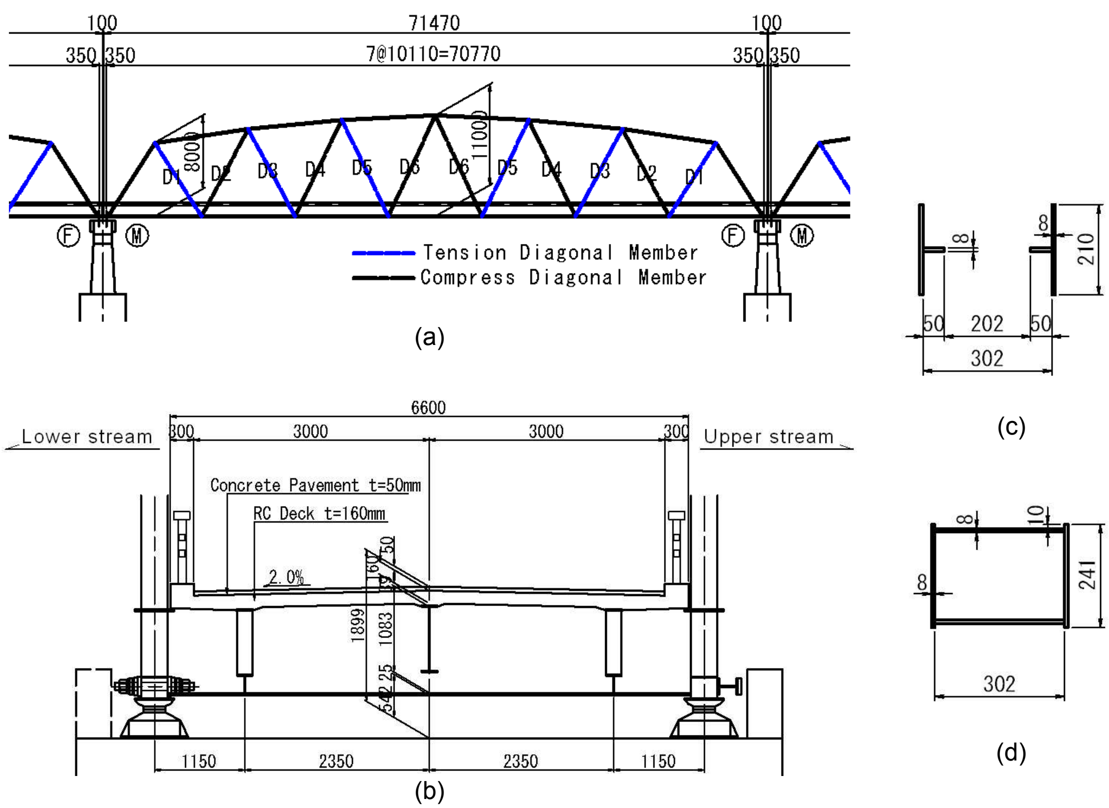


Figure 1 Steel truss bridge investigated: (a) side view of single span, (b) cross section of the bridge, (c) cross section of tension diagonal member, (d) cross section of compression diagonal member. Unit [mm].

2.2. Vibration measurements

Several sets of vibration measurements were made for different objectives. The objectives of those sets of measurements were to identify the effect of the partial fracture, cracks and pitting corrosions on the dynamic characteristics of the local dynamic characteristics of the diagonal members (Measurement 1) and the whole bridge (Measurement 2), and obtain better understanding of the dynamic characteristics of the bridge (Measurement 3), respectively.

Impact testing of the diagonal members was conducted in those with damage and without so as to identify the effect of damage on local vibration characteristics of the tension diagonal members (Measurement 1). Either a three-axis accelerometer unit consisting of three single-axis piezoelectric accelerometers or a set of four single-axis piezoelectric accelerometers was attached to the flange of tension diagonal members at the quarter point from the bottom. Impacts were applied by an impact hammer to the web of the diagonal member at the quarter point from the bottom. Additionally, ambient vibrations of a diagonal member were measured so as to understand the difference between the vibration modes of the diagonal member induced by impact testing and those induced by ambient vibration.

In order to identify the effect of damages on the dynamic characteristics of the bridge (Measurement 2), vibration of the bridge was measured at three locations: the lower chord members on both sides of the bridge and a longest tension diagonal member, D5 (Figure 2). The number of measurement locations was limited because the measurement needed to be completed during a short time period between the detection of the damages and the urgent reinforcement of the diagonal members. The measurement was conducted in the span with the local fracture found and, for comparison, in a span without damages. The acceleration of the lower chord member was measured in the vertical direction at the quarter point of the span. The accelerations in three orthogonal axes were measured in D5 at the quarter point from the bottom. Piezoresistive accelerometers were used in the measurement. Vibration of the bridge was induced by a dump truck with a total mass of about 200 kN running at different speeds between 20 and 40 km/h while the bridge was closed to other traffic.

Figure 3 shows the positions of transducers in Measurement 3 to obtain better understanding of the dynamic characteristics of the bridge. Four servo velocimeters were placed at different positions of a lower chord member and three piezoelectric accelerometers were attached at another lower chord member, as shown in Figure 3. The motion in the vertical direction was measured at all those locations. Additionally, the motions of five tension diagonal members were measured with piezoelectric accelerometers. At the quarter point from the bottom, two accelerometers were attached to the web to measure in-plane motion of the diagonal member and another accelerometer was attached to the flange to measure out-of-plane motion. A particular interest in this measurement was to understand more about the dynamic coupling between the diagonal members and the whole structure.

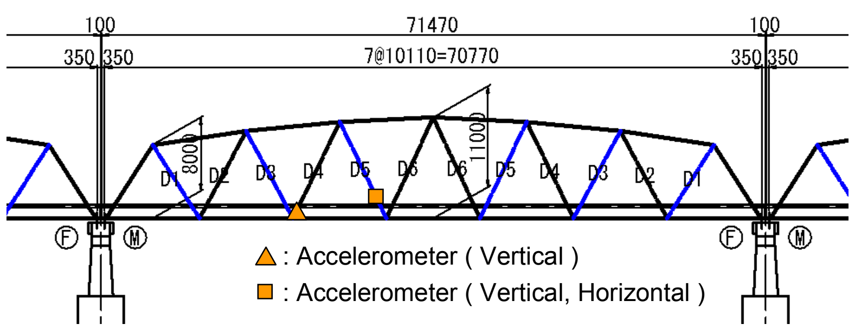


Figure 2 Positions of accelerometers in Measurement 2

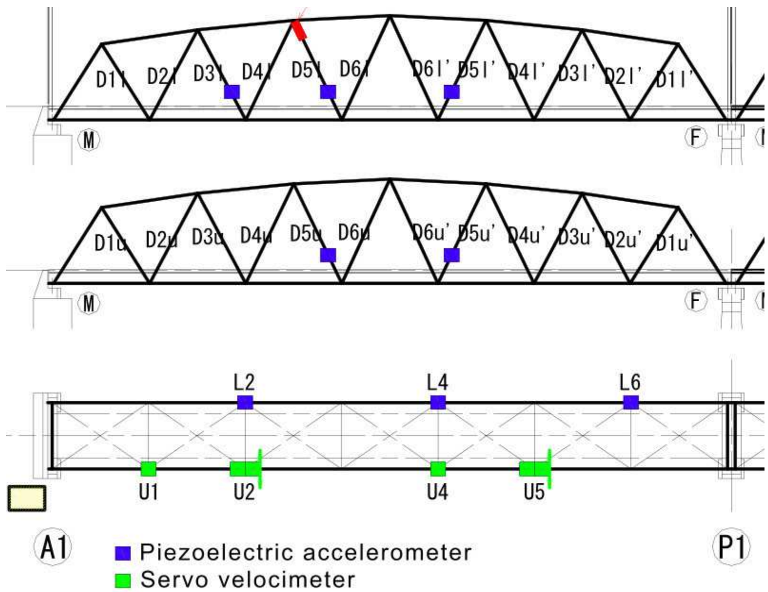


Figure 3 Positions of accelerometers in Measurement 3

3. NATURAL FREQUENCY CHANGE IN LOCAL MODES

Figure 4 shows an example of the results of impact testing that compares the Fourier spectra of the response to impact of a diagonal member D1 with pitting corrosions (referred to as corroded in the figure) and that of a D1 without corrosion (referred to as healthy). In the figure, the stabilization diagram obtained from the Eigensystem Realization Algorithm, ERA (Juang & Pappa, 1985) for the diagonal member without corrosion is compared with the corresponding Fourier spectrum. The Modal Amplitude Coherence, MAC, was used to identify reliable natural frequencies in the ERA. The comparison between the Fourier spectrum and the stabilization diagram from the ERA implies that the natural frequency identified from the measurement records in the impact testing were reliable.

It was observed that the natural frequencies observed at frequencies above 100 Hz were different between healthy and corroded diagonal members, while there were minor differences in the frequency range below 100 Hz (Figure 4). The natural frequencies of the diagonal member with pitting corrosions appeared to be lower than those of the diagonal member without corrosions. Similar trend were found with the comparison between this diagonal member with pitting corrosions and other healthy diagonal members with nominally the same dimensions. The decreases in the natural frequencies in the frequency range above 100 Hz for the corroded diagonal member may be associated with decreases in the modal stiffness of higher order local vibration modes that are attributed to the pitting corrosions.

Figure 5 compares the natural frequencies of the healthy diagonal member identified by impact testing and those identified from twenty different records of ambient vibration in the frequency range between 100 and 200 Hz where the effect of the pitting corrosions was observed. In the analysis of the ambient vibration records, the ERA was applied to free vibrations observed after vehicle pass-bys. The natural frequencies identified from ambient vibration records were a subset of the natural frequencies identified by impact testing. However, what natural frequencies were observed varied depending on the ambient records used in the analysis as observed in Figure 5. In practical applications, a continuous ambient vibration measurement combined with some statistical analysis can be used to identify all natural frequencies within a frequency range of interest.

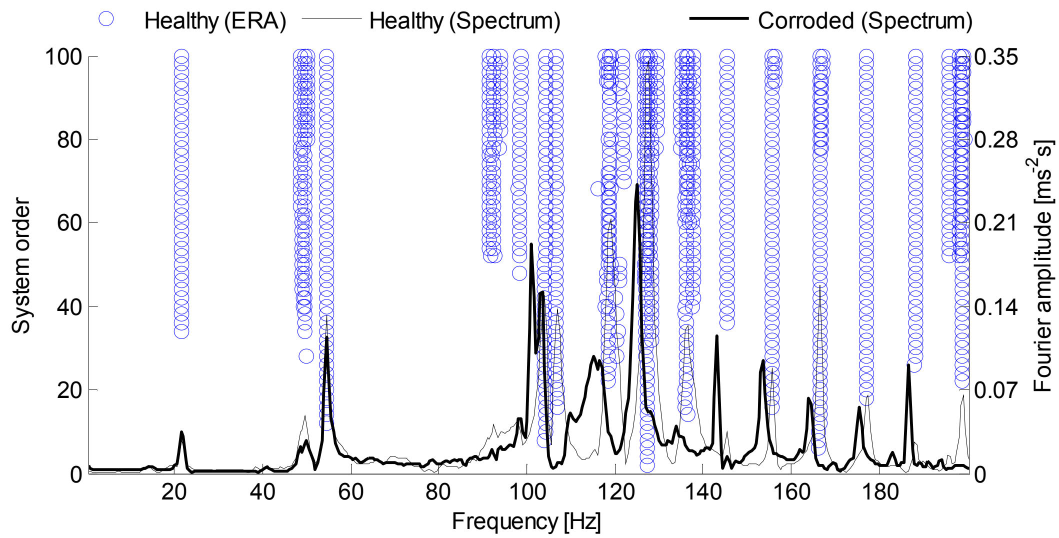


Figure 4 Fourier spectra of response of healthy and corroded diagonal members to impact. Stabilization diagram from ERA obtained for health diagonal member are also shown.

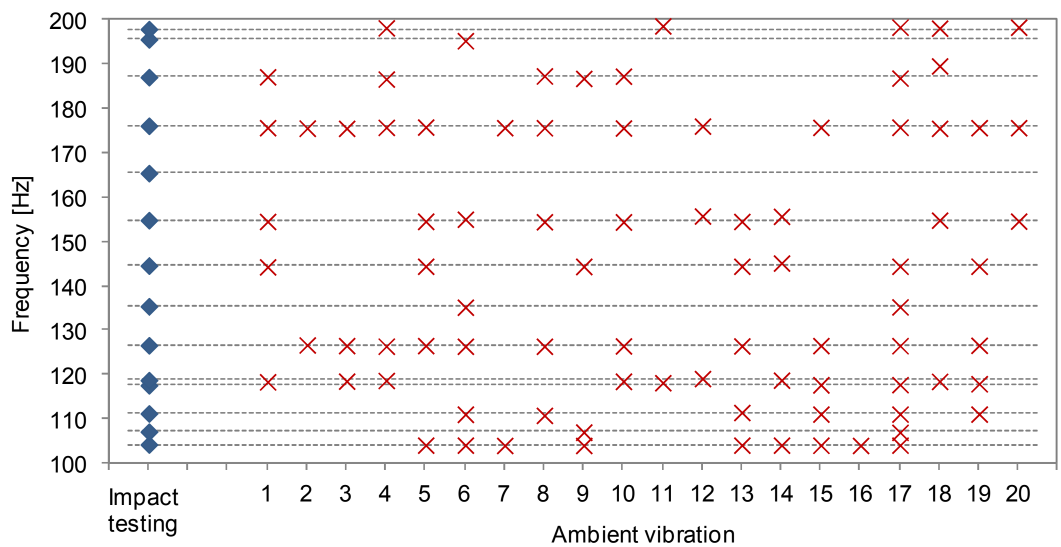


Figure 5 Natural frequencies identified by impact testing and 20 ambient vibration records

4. DAMPING CHANGE IN COUPLED MODE

It was understood that there were closely spaced vibration modes at frequencies around the natural frequencies of the diagonal members. This was partly because there were four diagonal members in a span that had nominally the same dimensions. Additionally, there were coupled vibration modes between the diagonal members and the whole structure. Figure 6 shows the mode shapes involving significant motion of the longest tension diagonals (D5). In the figure, the mode shapes identified from the field records measured in Measurement 3 by the ERA are compared with those obtained from theoretical modal analysis by finite element analysis. The figure shows a vibration mode dominated by the motion of the diagonal members (referred to as a local mode in this paper) and a mode involving the motions of the diagonal members and lower chord members (referred to as a coupled mode).
Figure 7 shows the changes in the modal frequencies and damping ratios before and after the reinforcement of the diagonal members identified in Measurement 2. The modal properties shown in the figure were obtained by the ERA. The figure shows that, in the local vibration mode dominated by the

motion of the diagonal member damaged and repaired, the modal frequency increased from 7.1 Hz to 9.8 Hz, approximately, and the modal damping ratio decreased from 0.0055 to 0.0039 after the reinforcement. In the global vibration modes involving the motion of the whole structure, there appeared to be changes in the modal damping ratio with minor changes in the modal frequency. It was noted that there was more variability in the identification of the modal damping ratio from the measurement records in the lowest order vibration modes, such as the mode at about 2.6 Hz in the figure, although the data are not presented in this paper. This variation in the damping was considered to be caused by the friction damping at the bearing supports that was dependent heavily on the displacement amplitude of vibration. In the global mode at about 7.3 Hz, however, there was less variability in the identification of modal damping ratio and the change in the modal damping shown in Figure 7 was more reliable than the changes in the lowest order vibration modes.

Figure 8 shows the relation between the changes in modal damping ratio found in the vibration mode at 7.26 Hz, as shown in Figure 7, and the dynamic coupling between the diagonal and lower chord members. The dynamic coupling between the diagonal and lower chord members are represented by the ratio of the modal amplitude of the diagonal member to the modal amplitude of the lower chord member in the figure. The modal amplitude of the diagonal member was obtained by subtracting the modal amplitude at the end of the diagonal member from that identified for the measurement location in the diagonal member.

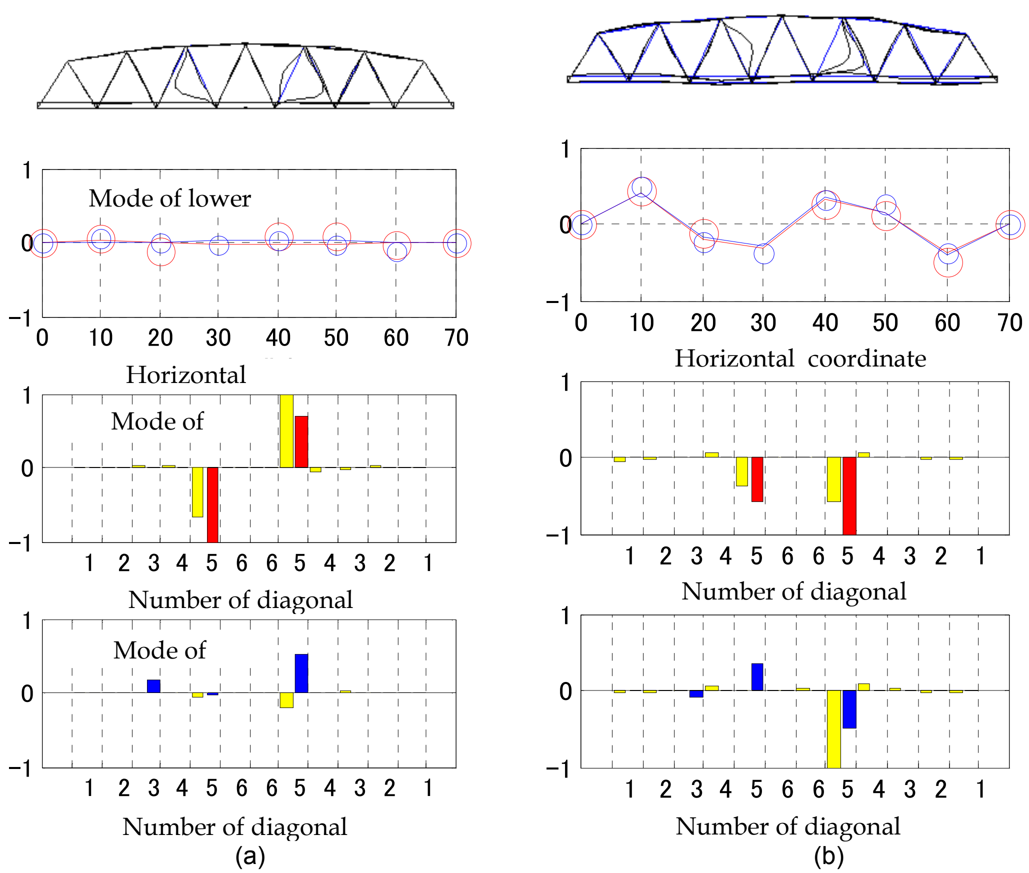


Figure 6 Modal amplitudes corresponding to (a) local mode at 9.264 Hz and (b) coupled mode at 9.325 Hz. The vi-bration modes identified experimentally are compared with those obtained theoretically.

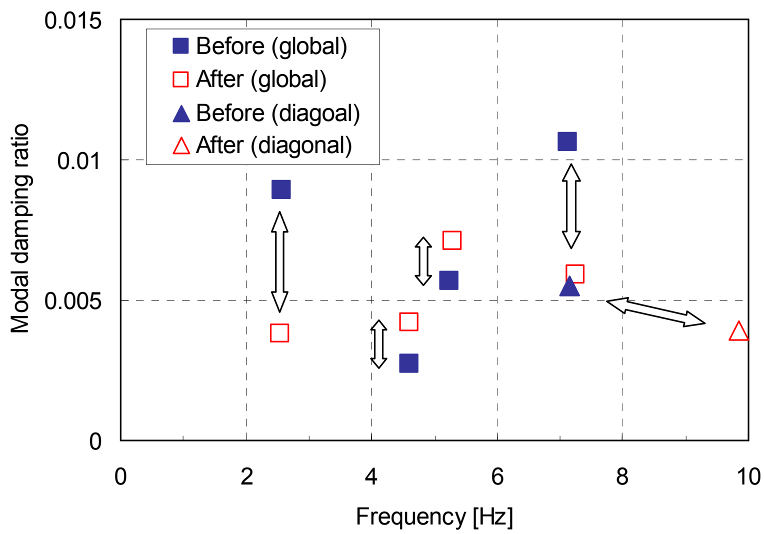
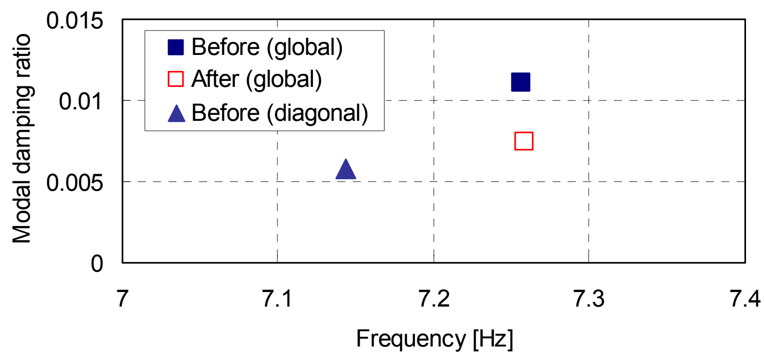
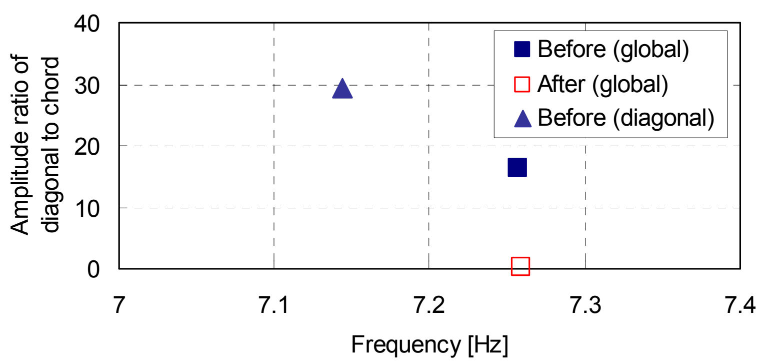


Figure 7 Examples of changes in modal frequencies and damping ratios before and after reinforcement of diagonal members.



(a)



(b)

Figure 8 Relation between changes in modal damping ratio and dynamic coupling between diagonal and lower chord members. (a) modal damping ratios and (b) amplitude ratios of diagonal member to lower chord member in local and global vibration modes before and after reinforcement

The vibration mode at 7.14 Hz shown in Figure 8 was the local mode dominated by the motion of the damaged tension diagonal member D5 as indicated by a high amplitude ratio of the diagonal member to the lower chord member (i.e., about 30). As shown in Figure 7, the modal frequency of the local mode increased from 7.14 Hz to 9.80 Hz after the reinforcement of the diagonal member: the modal properties of the mode after the reinforcement are not shown in Figure 8.

The decrease in the amplitude ratio of the diagonal member D5 to the lower chord member from about 16 to 0 in the global mode at 7.26 Hz, as shown in Figure 8, implies that there was no dynamic coupling between the diagonal member D5 and the lower chord member after the reinforcement due to the increase in the modal frequency of the local mode of D5. The decrease in the modal damping ratio of the global mode at 7.26 Hz may be associated with the loss of coupling between the diagonal and lower chord members that was caused by the change in the mechanical property of the diagonal member. This finding suggests that the modal damping ratios of global modes may be used as an indicator of local damages in diagonal members in steel truss bridges.

5. CONCLUSIONS

The results presented in this paper shows a possibility of the identification of local damages in steel truss bridges, such as damages in diagonal members, from changes in the modal properties of the structure obtained from vibration measurements. The natural frequencies of higher order local vibration modes of diagonal members can be a direct indicator of damages, although a practical implementation of the identification of natural frequencies of a number of diagonal members may need further development. A possible solution may be applying impact testing only on diagonal members that are identified as critical members in redundancy analysis. Ambient vibration may be able to be used instead of impact testing to identify natural frequencies of diagonal members. The identification of changes in the damping of global mode may be more practical in terms of the feasibility of measurement, although there is a need to improve the reliability of the identification of modal damping. The development of quantitative relation between changes in modal properties and damages requires further investigations.

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