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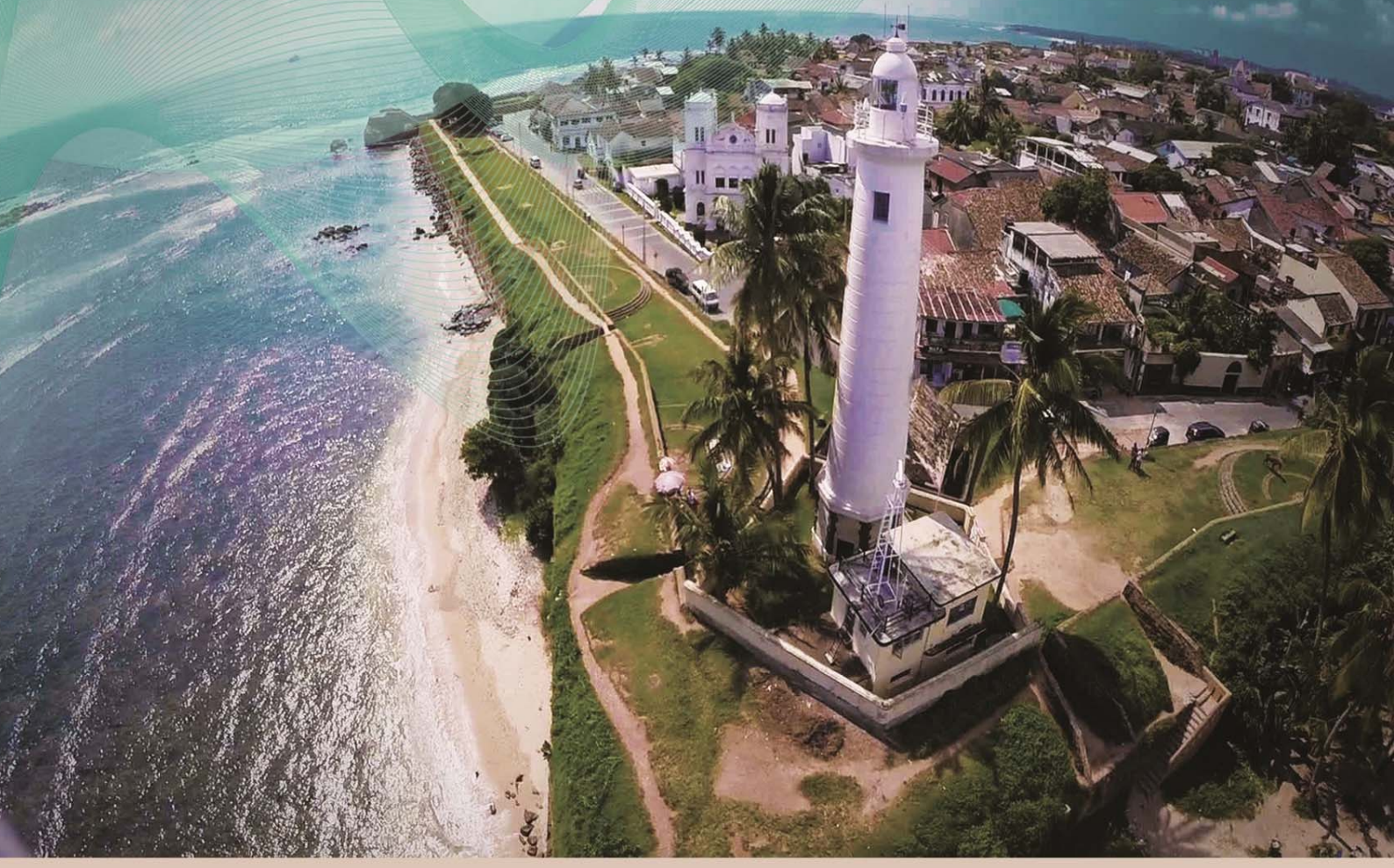

Faculty of Engineering,
University of Ruhuna,
Hapugala, Galle, Sri Lanka.
9th March 2015

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3rd International Symposium on Advances in Civil and Environmental Engineering Practices for Sustainable Development

ACEPS 2015

Department of Civil and Environmental Engineering
Faculty of Engineering
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Preface

Sustainable development has become core to the practice of Civil and Environmental Engineering and this needs to be reflected in any design methodology or practice related to the field. Until the recent past, the main task of civil engineers was to overcome the challenges imposed by the society, who is benefitted by the outcomes of design and construction methodologies related to the field of Civil and Environmental Engineering. Sustainable development is another challenge that civil engineers have an opportunity to address. The engineers' role is to see that the technological applications incorporate the principles of sustainable development. However it was only in the recent past that these principles were integrated with the design methodologies and practices of Civil and Environmental Engineering. Hence much disruption to the environment has been being caused by the outcomes of Civil and Environmental Engineering works that do not comply with the principles of sustainable development. However much to the delight of everyone, Civil and Environmental Engineers in the region have initiated integrating their practices with the principles of sustainable development, and have been contributing to the sustainable development since the recent past. At present there are many occasions in the region that the civil engineering is complying with the sustainable development.

Therefore it is imperative to create a venue to discuss such advanced sustainable engineering design methodologies and practices that are adopted in developing the region.With this objective, the Department of Civil and Environmental Engineering, Faculty of Engineering, University of Ruhuna, Galle, Sri Lanka is organizing an international symposium on Advances in Civil and Environmental Engineering Practices for Sustainable Development (ACEPS 2015) on 9th March 2015. The symposium aims to provide an excellent international forum for all the researchers, practitioners, students and academics in related fields to share their knowledge and results in theory, methodology and application on Civil and Environmental Engineering. ACEPS 2015 features a blend of unique topics of Civil and Environmental Engineering practices for sustainable development.The symposium theme represented by the fields mentioned below addresses the areas that demand advanced engineering developments complying with the principles of sustainable development.

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

The venue of the symposium is in Galle, a rapidly developing city, which houses the famous colonial fort that has received the world heritage status. This heritage houses many civil engineering outcomes leading to its own sustainability and the sustainability of the surrounding environment, from which the construction materials had been picked up. The Southern Province of Sri Lanka, which is well known for its ancient inheritance, has been going through a rapid development introducing many large scale infrastructure facilities. The knowledge on practices of sustainable development can contribute immensely to this developmental process paving a path to the symbiosis between the infrastructure facilities and environment.

ACEPS 2015 symposium consists of 47 papers divided into nine technical sessions and three plenary sessions. The symposium is graced by three precious keynote speeches delivered by internationally recognized experts namely, Dr.Joseph E. Odencrantz, Prof. S.A.S. Kulathilaka and Dr. S.Tachibana, during the plenary sessions. Dr.Joseph E. Odencrantz, with over twenty-five years of experience in the field of water and environmental engineering in the private sector, presents a keynote address titled "Property Line Contamination Issues and Associated Risks to Buildings plus Cross-Contamination Issues & Water Supply Protection". Professor S.A.S. Kulathilaka, based on his long time experience in the field of research, delivers a keynote speech titled "Experiences on Improvement of Soft Peaty Clays in Sri Lanka". Dr.S.Tachibana, based on his vast research experience, delivers a keynote address titled "Numerical and Analytical Approaches to the Problems of Unsaturated Ground".

ACEPS 2015 proceedings are composed of the most up-to-date, comprehensive, and worldwide state-of-the art knowledge on Civil and Environmental Engineering. All accepted articles have been peer-reviewed by two referees, and selected based on originality, significance and clarity for the purpose of the symposium. The articles, which address different themes of the symposium, include studies focused on solving real engineering problems, case studies on sustainable practices as well as original studies of academic interest.

The expert deliberations in the symposium would pave the way to work out a strategic framework of examining an effective action plan of incorporating the advances of Civil and Environmental Engineering into the design methodologies so as to mitigate the adverse environmental and social impacts. It would give the participants a broad overview of the latest research results, and provide them with a significant platform for academic connection and exchange. It would also create a forum to discuss advanced sustainable engineering practices that could be adopted in developing the region. The research work presented in the symposium would also initiate a dialogue on how the engineering developments should be taken place in the context of sustainable development. The organizers expect to sustain this dialogue by continuing the ACEPS symposium as an annual event.

Publication Committee – ACEPS 2015
9th March 2015

Message from the Vice Chancellor



It is my great pleasure to write this message to the proceedings of the 3rd International Symposium on Advances in Civil and Environmental Engineering Practices for Sustainable Development (ACEPS-2015). This is the third of the series of international symposium jointly organized by the Faculty of Engineering of University of Ruhuna and the Saitama University of Japan and hosted by the Department of Civil and Environmental Engineering.

The sustainable development is considered as one of the most important themes in today's context, since most of the development efforts taken by many countries all over the world are not sustainable. Therefore, at the United Nations Conference on Sustainable Development Rio+20 held in 2012, it was decided to formulate Sustainable Development Goals (SDGs) and include in the post 2015 development agenda. Therefore, this symposium provides an appropriate and well-timed platform for academics and researchers to present their findings and to exchange their ideas on topics such as sustainable built environment, climate change, hydroinformatics, and geotechnical engineering etc. which will definitely contribute to the Rio+20 SDGs.

I, as the Vice Chancellor, am very happy to note that the Faculty of Engineering, though it is a fairly young faculty, has shown their capacity and capability to contribute to the ocean of knowledge through organizing number of international and national symposia within a calendar year. They have realized the importance of vibrant research culture within the university to uplift the quality and relevance of university education. At this juncture, I would also like to mention the immense support extended by the Saitama University of Japan to our university especially with respect to human resources development and research.

Organizing an international symposium is always a big challenge and I am confident that the Faculty of Engineering would hold this symposium to the expectations of the international academic community. Therefore, once again, I would like to thank the Dean, Head of the Department of Civil and Environmental Engineering, and the organizing committee for their commitment in organizing this international symposium. I believe that all the participants will use this occasion to share their knowledge while obtaining necessary thrust in collaborative researches. I wish all the best for all paper presenters and participants.

Professor Gamini Senanayake
Vice Chancellor, University of Ruhuna

Message from the Deputy Vice Chancellor



It is with great pleasure that I send this message to the third International Symposium on Advances in Civil and Environmental Engineering Practices for Sustainable Development (ACEPS-2015) organized by the Department of Civil and Environmental Engineering, Faculty of Engineering, University of Ruhuna, Sri Lanka in collaboration with the Saitama University, Japan. I wish that the value of this international symposium will be enhanced by the participation of many scholars, researchers, academics, and practitioners from a wide range of well-reputed institutions in Sri Lanka and abroad.

It is further expected that ACEPS-2015 would continue to foster research in the fields of civil and environmental engineering and hence encourage innovative and sustainable solutions. I also believe that this international symposium will help scholars, fellow researchers, academics, practitioners, and undergraduate and postgraduate students in the fields of civil and environmental engineering to create researcher-networks for mutual benefit as well as for critical enquiry in their subject areas under review. Holding of this symposium at the University of Ruhuna will also facilitate to increase the awareness of research activities carried out at the University in above field.

I hope that ACEPS-2015 will generate new ideas, solutions, and concepts for advanced sustainable engineering practices to be adopted in the fast developing world. Making the ACEPS-2015 to be held on 09th March 2015 at the Faculty of Engineering, University of Ruhuna a reality should be a combined effort of all members of the symposium organizing committee and the Department of Civil and Environmental Engineering. I commend those who have been working tirelessly to organize this symposium and wish the ACEPS-2015 every success.

Dr. A.M.N. Alagiyawanna,
Deputy Vice Chancellor, University of Ruhuna

Message from the Dean



It is indeed with great pleasure I write this message to the 3rd International Symposium on Advances in Civil and Environmental Engineering Practices for Sustainable Development (ACEPS-2015) to be held in Faculty of Engineering, University of Ruhuna, and Galle, Sri Lanka on 09th March 2015.

Civil engineering is one of the engineering disciplines that is always in demand. Incorporation of sustainability concepts into a Civil Engineering has become an urgent and important issue in the present world. Engineers worldwide are being struggling to deal with the issues of sustainability. This symposium is a very useful opportunity for professionals, practitioners and researchers who would like to know and share their ideas and experiences on recent advances in the field of Civil Engineering while emphasizing on sustainability concepts. Moreover, this event will be a platform to meet the colleagues, to have social interaction and to establish collaborative work among the research groups.

ACEPS has become an annual international event in the Department of Civil and Environmental Engineering calendar. I thank all the members of the organizing committee for their continuing dedication in holding this symposium in the Faculty of Engineering for the third consecutive time. Further, I would like to congratulate the authors of the technical papers for their wonderful efforts and the achievements.

I wish ACEPS-2015 will be a very successful international event in the Faculty of Engineering, University of Ruhuna.

Dr. P. D. Chandana Perera
Dean, Faculty of Engineering, University of Ruhuna

Message from the Head, Department of Civil and Environmental Engineering / University of Ruhuna and Symposium Co-chair



It is our pleasure to welcome all the participants to the 3rd International Symposium on Advances in Civil and Environmental Engineering Practices for Sustainable Development (ACEPS-2015) and to beautiful ancient city Galle. It has been a real honor and a privilege to serve as the co-chair of this international symposium. The two Departments of Civil and Environmental Engineering in Saitama University, Japan and University of Ruhuna, Sri Lanka initiated collaborative research activities with the International Exchange Symposium on Sustainable Infrastructure Development in Asia in 2009, SATREPS Project in 2011, 1st ACEPS in 2012 and 2nd ACEPS in 2013. This is the third of the series of international symposia jointly organized by the Department of Civil and Environmental Engineering, Faculty of Engineering of University of Ruhuna, Sri Lanka and Saitama University, Japan.

The theme of the symposium, "Civil and Environmental Engineering Practices for Sustainable Development", is extremely relevant for today's world. The ACEPS international symposium is promoting innovative research for tomorrow's development and we are organizing this symposium as a meeting place for scientists with talents, knowledge, and dedication. Therefore, we trust that the symposium will produce enormous ideas from a variety of research and exchange the knowledge among experts, colleagues, and friends who are working for the world's sustainable development.

The Symposium focuses on six different sub topics: 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 and Environmental Engineering. Proceedings of the symposium contain the full papers of double-blind peer reviewed scientific contributions. Symposium's technical sessions are rich and varied with three keynote speeches and around fifty technical papers split between three parallel oral sessions on the symposium day.

We, the conference co-chair and organizing committee, express our sincere thanks to our chief guest, keynote speakers, authors, members of the international advisory committee, members of the editorial committee, financial sponsors, and many others who contributed their time and great efforts to make the symposium possible and to make it successful. Hope you will have a wonderful time.

Dr. Sudhira De Silva
Head, Department of Civil and Environmental Engineering, University of Ruhuna

Message from the Symposium Co-chair



We are very happy to hold the International Symposium on Advances in Civil and Environmental Engineering Practices for Sustainable Development-2015 (ACEPS-2015), organized by the Department of Civil and Environmental Engineering, University of Ruhuna.

Currently, Saitama University of Japan is promoting the JST-JICA Science and Technology Research Partnership for Sustainable Development (SATREPS) project together with partner institutes in Sri Lanka since April 2011. In the SATREPS project, University of Ruhuna, University of Peradeniya, Institute of Fundamental Studies (IFS), National Solid Waste Management Support Center (NSWMSC), and Central Environmental Authority (CEA) are participating from Sri Lanka, and The Center for Environmental Science in Saitama (CESS), National Institute of Advanced Industrial Science and Technology (AIST), and Waseda University are participating from Japan. The SATREPS project targets the development of pollution control and environmental restoration techniques of waste landfill sites and continues until March 2016. Under the support of SATREPS project, Saitama University is participating in the ACEPS since 2012.

In the SATREPS project, not only collaborative research activities but also staff and students exchanges among partner institutes, co-supervising undergraduate and graduate students have been conducted. We believe our SATREPS project would be a good example to promote further international collaborative research between Sri Lanka and other countries. We hope the conference will provide an excellent opportunity to have fruitful discussion on the advances in the fields of civil and environmental engineering for sustainable development of Sri Lanka.

We would like to thank Dr. Sudhira De Silva, the Head of the Department of Civil and Environmental Engineering, University of Ruhuna, and Dr. H.P. Sooriyaarachchi, Department of Civil and Environmental Engineering, University of Ruhuna, for their efforts in organizing this conference, and also thank to Dr. N.H. Priyankara, Dr. W.K.C. Neetha Dayanthi, Department of Civil and Environmental Engineering, University of Ruhuna, and Dr. A.M.N. Alagiyawanna, Deputy Vice Chancellor of University of Ruhuna for their dedicated supports to the SATREPS project.

Professor Ken Kawamoto
Graduate School of Science and Engineering, Saitama University, Japan
Principal Investigator of JST-JICA SATREPS project on ‘Development of pollution control and environmental restoration technologies of waste landfill sites taking into account geographical characteristics in Sri Lanka’

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Department of Civil and Environmental Engineering, Saitama University, Japan

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Acknowledgement

ACEPS-2015 is a result of the efforts of many people. Our sincere thanks are extended to Senior Professor Gamini Senanayaka, Vice Chancellor, University of Ruhuna for his contribution in organizing the event and gracing the event as the Chief Guest.

We would also like to acknowledge the support received from Dr. A.M.N. Alagiyawanna, Deputy Vice Chancellor, University of Ruhuna, for his valuable and constructive suggestions for planning the symposium, which motivated us to do our best. Our sincere gratitude is extended to Dr P.D.C. Perera, Dean, Faculty of Engineering, for facilitating the symposium to be held within the premises of Faculty of Engineering.

We received enormous help, which deserves a great appreciation, from the Symposium Co-chair, Prof. Ken Kawamoto, who led all the arrangements on behalf of Saitama University by contributing to the discussions, organizing sponsorships and encouraging article submissions from Saitama University including a keynote paper. Dr. Sudhira De Silva, being the Symposium Co-chair and Head of the Department of Civil and Environmental Engineering, gave his support to the fullest, which is quite appreciable.

Our deepest thanks go to Dr.Joseph E.Odencrantz, the keynote speaker in the opening ceremony and a member in the International Scientific Committee and Technical Committee of ACEPS 2015 symposium, for flying half way around the world just to grace the ACEPS 2015 symposium, bearing travel and accommodation expenses and spending his valuable time. We take this opportunity to thank Prof. S.A.S.Kulathilaka, the 2nd keynote speaker, for spending his precious time to add a useful piece of technical information to our knowledge. It is with great respect, we thank Dr. S. Tachibana, for traveling a long distance from Japan, to grace the event by delivering the 3rd keynote speech.

We would also like to acknowledge with much appreciation the crucial role of the multi-disciplinary Technical Committee of ACEPS 2015 symposium for their valuable pieces of advice toward the improvements of the symposium and the invaluable comments made to the authors during the reviewing process. Despite their busy schedules, they provided critical reviews, which helped to select the articles of high quality.

We are also grateful to the members of the International Scientific Committee, whose guidance ensured that the ACEPS 2015 symposium complies with the international standards.

We are highly obliged in taking the opportunity to sincerely thank all the session co-chairs and the members of the committee appointed for evaluating the oral presentations.

We are deeply grateful to the contribution of the sponsors, Ceylon Steel Corporation Limited, Holcim (Lanka) Ltd., Saitama University and JICA under SATREPS, for that this symposium would not have been a success without their financial support.

Moreover we owe thanks to the authors from across the world, who laid the foundation stones of the symposium by submitting their research findings.

Many thanks go to all the academic and non-academic staff of the Department of Civil and Environmental Engineering for their generous assistance in organizing the symposium. In addition the students of the Faculty of Engineering are appreciated for their contribution to different tasks of the symposium.

This task could not be completed and successful without the endeavor and co-operation from each member in the Organizing Committee led by the Symposium Secretary, Dr. N.H. Priyankara. All members worked hard to meet the deadlines spending their precious time. Last but not least, they are worth being acknowledged immensely.

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Keynote Addresses

Keynote Address

Property Line Contamination Issues and Associated Risks to Buildings plus Cross-Contamination Issues & Water Supply Protection

by

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Joe Odencrantz is a California-based environmental and water consultant with over twenty-five years of experience in the private sector. Dr. Odencrantz is a water and environmental expert with a unique set of qualifications and experience that spans from traditional civil and environmental engineering to current state-of-the-art investigation and remediation techniques. Dr. Odencrantz founded Tri-S Environmental (Tri-S) in 1994 and continues setting new standards of excellence in the practice of water and environmental management. He directs routine and complex analysis, investigation, vapor intrusion, water treatment and remediation projects on behalf of a variety of clients.

Property Line Contamination Issues and Associated Risks to Buildings plus Cross-Contamination Issues & Water Supply Protection

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Abstract: *Vapor phase transport of petroleum hydrocarbons and chlorinated solvents released into the subsurface is a challenging problem that requires special investigation and interpretation techniques. A case study is presented that illustrates the potential issues that can arise from historical release(s) of volatile organic compounds (VOCs-such as perchloroethylene [PCE]) to the subsurface and then into buildings (vapor intrusion). Site owner/developers are faced with deciphering fact from possible conjecture from professionals hired to conduct site assessments. Lateral migration of vapor contamination from one property to another can result in unnecessary burdens to properties adjacent to a property where a significant release occurred.*

Investigators faced with drilling through historical releases of chlorinated solvents in the subsurface should proceed with caution to avoid creating additional contamination issues (cross-contamination), i.e. prevent the situation from getting worse. A case study is presented that demonstrates the challenges that arise from drilling through known chlorinated solvents in soil into shallow groundwater.

Keywords: *Vapor intrusion, Chlorinated solvents, PCE, Cross-Contamination, Water supply, Property line.*

1. INTRODUCTION

Managing both petroleum hydrocarbon and chlorinated solvents releases to the subsurface is a large challenge for developers and new property owners faced with dealing with contamination emanating from adjacent or adjoining properties. Before rushing to judgment on the decision to move forward with subsurface investigation and remediation on a new property owner's site, a proper independent evaluation/study (due-diligence) of the subsurface contamination on adjacent properties should be conducted. A case study is presented that is based on a soil vapor extraction system that was put in operation after a consultant, who was engaged to perform an environmental assessment for an impending real estate transaction, seemingly took the assignment and then recommended soil remediation on the new property owner's site. It was demonstrated at a later date that the vapor contamination plume was originating at a property to the north, not the new owner's site.

A consultant hired by a power company was engaged to perform a site assessment on a piece of land xxxxxxx to the power company so that the terminus of a large, underground power trunk line could be constructed. The piece of land was investigated by the power company's consultant to determine if there were any subsurface concerns as part of the grading/shoring operation of the proposed structure. The consultant discovered a shallow PCE vapor plume in the shallow soils with concentrations as high as xx,xxx. micrograms per cubic meter (ug/m3) of PCE and proceeded to drill through this vapor cloud into the groundwater system in the following months without any special precautions. As a result, the groundwater concentrations were elevated for it appeared that the consultant dragged contamination into the groundwater system. After expert evaluation, additional monitoring wells were subsequently constructed using drilling practices designed to minimize the cross-contamination from the soil vapor into the groundwater system.

2. PROPERTY LINE CONTAMINATION AND PROPER ENVIRONMENTAL ASSESSMENT DEMONSTRATE REMEDIATION INADEQUACIES

As part of the environmental due-diligence of a commercial property transaction, the American Society for Testing and Materials (ASTM) Standard Practice for Environmental Site Assessments-Phase I Environmental Site Assessment Process (ASTM 1527-13) was utilized. ASTM 1527-13 states in Section 9.4.1.4 on page 18 "To the extent that indications of past uses of adjoining properties are visually and/or physically observed on the site visit, or are identified in the interviews or record review, they shall be noted and past uses so identified shall be described in the report if they are likely to indicate recognized environmental conditions in connection with the adjoining properties or the property." As such, an environmental investigator is responsible for examining the environmental records and current environmental remediation activities at adjoining sites/properties to the subject property for which a purchase is being contemplated. Figure 1 below is a cross-section approximately xx m to the north of the property line of the subject property that shows the extent of tetrachloroethylene (PCE) plume. The geology of the area is a comprised of alternating sand and clay/silt layers that vary in thickness from x to xx feet (~ x.x m) generally speaking. The fine-grained layers tend to trap or store chlorinated solvents and the vapors are slowly released over time (by molecular diffusion-much like perfume/cologne escaping from an opened bottle).

Redacted

Figure 1. Cross-section of PCE contamination with depth (ug/kg) approximately xx feet (xx m) north of the property line of the subject site. Bolded lines/shapes represent the xxx ug/kg contour lines.

In the year prior to the consultant performing the due-diligence for the prospective property owner, an investigation report was published (public domain and available at a local environmental regulatory agency) for the site to the north of the property line which contained Figure 1. The investigation report also discussed the site revitalization and decommissioning plan that was slated to take approximately one year. This site was a large xxxxxx manufacturing facility, in business for over xx years, midway through the process of decommissioning at the time the subject property owner's consultant was reporting on the subject properties due diligence investigation. The consultant did not report this information to the prospective property owner but focused his investigation on residual PCE from a former clarifier on the subject property that was investigated, remediated (by vapor extraction) and closed by the regulatory agency xxxxxx years earlier. The consultant proposed and performed an

active soil gas survey that showed low-levels of PCE vapor on the subject property. The consultant proposed to perform a vapor extraction remediation to reduce the vapor concentrations in the subsurface on the subject site to regulatory levels for indoor air. The consultant did not disclose to the client the possibility that PCE on the subject site could be coming from the large, former manufacturing facility just north of the property line. The consultant convinced the new subject property owner to re-open this previously closed site with the local regulatory Agency's voluntary site cleanup program.

Table 1. PCE concentrations in discrete soil samples on the property to the north and the vapor concentrations on the subject site. PL-Property Line.

REDACTED

REDACTED

Figure 2. PCE vapor plume depicted by each property owner's consultant prior to vapor extraction on the subject site owner's property. The red color indicates a maximum PCE concentration on the subject site of approximately XXX ug/L.

Table 1 contains the PCE concentrations in soil on the site to the north, as a function of depth and distance from the property line, and the resulting soil vapor concentrations on the subject site. Figure 2 shows the contours of PCE vapor on the property to the north at five feet below the surface prepared by their consultant and the pre-remediation vertically averaged PCE vapor contours on the subject site (x to xx feet below the surface) prepared by the consultant to the subject site owner. The full vertical extent of contamination on the property to the north is shown in Figure 1. The consultant for the subject site convinced the new property owner that the site would be cleaned up with relative ease and within a year after the property owner's purchase of the site. As you can see by the consultant's own depiction of the PCE vapor plume in Figure 2 in red, the PCE is clearly emanating from the property to the north of the property line. The former clarifier on the subject site is mapped near the bottom tip/green portion of the plume and clearly not the source of PCE vapors on the subject site.

The consultant to the subject site owner was unable to remediate the subject site to regulatory limits after XXXX years of remedial activities using vapor extraction because the source of PCE vapors was located on the other side of the property line. The consultant kept promising his client that cleanup would be completed with more time and money. That did not happen and after approximately four years, the property owner realized that something was amiss with the situation. The local regulatory agency will not close the site until the remediation is completed, which is now contingent upon indoor air concentration limits of PCE. In addition, the regulatory agency is requiring that other VOCs to be reduced below strict standards for human exposure. The project is now in a state of flux and the property owner is considering suing the property to the north for cost recovery and to assume the responsibility for site cleanup. The consultant for the subject site was released of his duties and the property owner hired a competent and ethical environmental professional to assume oversight of the cleanup strategy.

The PCE contamination from the source left in place at the property to the north creates a vapor intrusion risk to buildings in vicinity of the plume. The recent regulatory standards in effect at most states in the USA have a component of soil gas and indoor air quality assessment for remediation alternative evaluations. At this particular site, the site was closed by the local oversight Agency twenty years ago and would not have been re-opened by the State or the County based upon any residual vapors from the clarifier remediation. The previous consultant has created regulatory challenges and legal liability for the new subject property owner.

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3. CROSS-CONTAMINATION ISSUES & WATER SUPPLY PROTECTION

The consultant hired by a power company performed a soil gas survey at the property in which the construction of the terminus of a large subsurface power line (xxxx kilovolt [kV]) was to be installed. The consultant discovered as much as xx,xxx. ug/m3 in the shallow soil gas (x feet below grade) and then proposed to collect both discrete soil and groundwater (hydropunch) samples to further characterize the extent of contamination. The consultant discovered that as much as xxx ug/kg of PCE existed at the site approximately xx feet below grade underneath the previously determined soil gas hot spot. Figure 3 is a schematic diagram of the facts of the PCE regime prior to entering the groundwater system to collect groundwater samples at the site.

REDACTED

Figure 3. Potential Problems When Drilling Through PCE in Soil/Soil Gas.

The consultant's next step in the site characterization process was to collected groundwater hydropunch samples in the shallow groundwater using a Geoprobe® direct-push drilling technology. The consultant drilled through both the elevated PCE soil gas concentrations in the shallow soils and the PCE concentration of xxx ug/kg at xx feet below ground surface until they reached the shallow-most groundwater. First groundwater was encountered at approximately xx feet below ground surface. Upon reaching the groundwater regime, the water rose up the hole created by drilling to approximately xx feet below grade. The water was under pressure (confined/semi-confined) and rose thirteen feet in the borehole. There are multiple problems created by this investigatory method: a. dragging down of PCE vapor into groundwater, b. dragging down of soil cuttings that contain PCE into groundwater, c. the cross-contamination of groundwater that rose up within the borehole with PCE in the soil column (groundwater water may have not been impacted with PCE prior to the drilling activities. ASTM D6724-04 (2010) Standard Guide for Installation of Direct Push Groundwater Monitoring Wells provides excellent guidance and states "For direct push methods to provide accurate groundwater monitoring results, precautions must be taken to ensure that cross-contamination by "smearing" or "drag-down" (that is, driving shallow contamination to deeper levels) does not occur, and that hydraulic connections between otherwise isolated water bearing strata are not created."

Figure 4 shows the specifics of the drilling and well screen installation in the subsurface at this site in the vicinity of MW-X.

REDACTED

Figure 4. Well Screens/Sand Packs at MW-X Now Potentially Allow PCE In Soil/Soil Vapor To Enter The Ground Water System. The red circles are key depths near MW-X.

The consultant reported xxx ug/L of PCE in the groundwater at MW-X, which is a one-inch diameter well with a screen from xx to xx feet below ground surface. The installation of this monitoring well allows the PCE-impacted soil to continuously impact groundwater via dissolution from the soil matrix into the water phase. It is also now virtually impossible to determine whether the confined groundwater below xx feet below grade was previously impacted with PCE. The consultant reported this xxx ug/L to the local water regulatory agency and the agency required additional groundwater characterization as a result of this finding. The agency also required the consultant to install either two- or four-inch diameter monitoring wells and the consultant for the XXX strongly desired alternative drilling plus insisted on improved well construction techniques.

The future drilling at the site required a conductor casing (sometimes referred to a surface casing) be installed prior to the installation of groundwater wells in these shallow PCE source area zones to minimize the potential for cross-contamination. The groundwater monitoring wells are to be screened only in the groundwater bearing zone, in this instance xx to xx feet below ground surface, with short, targeted well screens that are isolated with a plug (seal) on top and bottom of the well screen (Note: Detailed lithology contained in the well/boring logs and not reflected in the general cross-section shown in Figure 4). The previously installed well screen for MW-X was ten feet long and provided a conduit for future recharge of the groundwater system with PCE. MW-X was abandoned by over-boring and then grouting prior to the installation of the future groundwater wells in the vicinity. In order to protect the water supply (the main groundwater aquifer below this site), preventive measures are a necessity to alleviate any future migration of PCE (and any breakdown products) into the perched, confined water-bearing zone and main aquifer.

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4. CONCLUSIONS

The two main topics in this paper are associated with the proper characterization and management of PCE in the subsurface. The first topic of the paper addressed the importance of investigating properties that are in the vicinity of a property that is potentially going to be purchased by an entity. If the due-diligence is flawed, the future property owner of the property which is the subject of due-diligence can be faced with unexpected monetary and regulatory liabilities. Legal liabilities are also a possibility for improper due-diligence may lead to lawsuit with the surrounding properties that could have impacted the subject property.

The second topic of this paper addressed the critical importance of minimizing cross-contamination when investigating chlorinated solvents in the subsurface. When an investigator knowingly drills through known PCE-impacted soil/soil vapor into confined groundwater, there are a host of problems that can result. When drilling through known contamination, preventative measures must be taken. If the investigator causes groundwater contamination as a result of hasty and improper investigation methods, large liabilities can result to both the site owner and the investigator. An ounce of prevention is worth a pound of cure when drilling through chlorinated solvent impacted sites.

5. ACKNOWLEDGMENTS

The author would like to thank the entities involved with each of the two projects that are the focus of this paper. Like most of our projects, both of these projects are confidential and great lengths have been taken to keep the location and the parties involved confidential. The author has chosen these two projects for they illustrate important topics for practitioners in the Built Environment to consider in the process of site development. The author extends his appreciation for the review of the draft manuscript by Mr. Joseph Nestor, P.E., P.G., owner of Nesco Environmental and who is based in the Charlotte, North Carolina, U.S.A. area.

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Keynote Address

Experiences on Improvement of Soft Peaty Clays in Sri Lanka

by

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Experiences on Improvement of Soft Peaty Clays in Sri Lanka

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Abstract: *Finding cost effective and rapid methods to improve the strength and stiffness characteristics of soft peaty clays is a major challenge faced by the Sri Lankan geotechnical engineers. Soft peaty clays with extremely high water contents were encountered in number of new highway projects. These peaty clays are extremely compressible and possess extremely low shear strengths. Laboratory studies showed that the primary and secondary consolidation characteristics of these peaty clays can be improved significantly by pre loading. In extremely soft peaty clays where any form of loading by application of a surcharge fill is not possible, initial improvements could be achieved by electro osmosis consolidation. Vacuum consolidation will also be effective in this context. Mixing of the peaty clays with up to around 20-25% cement will also result in comparable levels of improvements. Various industrial by products have been used with cement successfully. Use of reinforcing columns made of dense granular material to share a major component of the embankment loading is another technique that can be adopted in dealing with peaty soils.*

Keywords: preloading, electro-osmosis, vacuum consolidation mixing with cement.

1. INTRODUCTION

Due to the shortage of competent grounds, number of new infrastructure development projects such as; Katunayaka Expressway, Southern Highway and Outer Circular Highway in Sri Lanka are taken through areas underlain by very soft peaty clays. As the water table is very high in these sites the road elevation has to be increased by several meters to prevent flooding. Construction of high road embankments on these peaty clays in a cost effective manner within time restraints of the projects, to satisfy the necessary serviceability conditions, is a major challenge faced by the Sri Lankan geotechnical engineers

Extremely soft peaty clays encountered are with water contents of the order of 300-400% and of shear strength as low as 2 kN/m^2. Due to the relatively low organic contents in the range of 20%-40% these soils should be referred to as peaty clays rather than Peat. Peaty clay layers of thicknesses as high as 10-12m were encountered. These peaty clay layers will undergo very high primary and secondary consolidation settlements under the weight of an embankment fill. Quite often the embankments are to be constructed in stages or at an appropriately slow rate to prevent shear failures during construction. In extremely soft peaty clays of water like consistency encountered at some sites, the placed fill has sunk in to the peat mass and it was not be possible to place any fill. Removal of soft clay and replacement by a granular material had been tried at several occasions. However, finding dumping grounds for the removed soft peaty clay is another major problem.

As such, development of techniques for in situ improvement of the strength and stiffness properties of thick layers of soft peaty clays economically, within an acceptable time frame, is the approach to be adopted. In this context appropriate methods are to be developed to densify the peaty clay by the removal of as much water as possible or to solidify it by in situ mixing with a cementation agent. Alternatively, ground can be reinforced with columns of greater strength and stiffness made of either granular material or insitu cement mixing so that a major component of the load is transferred to these columns.
Ground improvement techniques applicable for soft peaty clays can be categorised as;

- Methods of consolidation of the ground – preloading, vacuum consolidation, electro osmosis
- Methods of solidifying the ground – Deep mixing with cement or any other material

- Use of reinforcing columns in an appropriate grid

2. IMPOVEMENT BY PRELODING

Preloading is one of the earliest methods used for the improvement of soft clays. In the construction of highways on soft clays, the highway embankment will have to be preloaded (Figure 1) with a surcharge equal or greater than the pavement and traffic load, to minimize the settlement of the road in service. The settlements of the peaty clay due the embankment load will be quite high and consequently the required fill thickness will be increased. With the need for introducing some over consolidation effect, the required fill thickness will be further increased. Due to extremely soft nature of the peaty clays, the required embankment load and preload will have to be applied in stages, or at a slow rate allowing sufficient time for the peaty clay to consolidate and gain in shear strength. The process can be accelerated by the use of vertical drains. (Figure 2 and Figure 3)

Preloading technique had been used very successfully in many highway projects such as Southern Expressway, Colombo Katunayaka Expressway and Outer Circular Highway. Preloading was done by placement of an extra height of fill to compensate for the pavement and traffic load with an additional surcharge as well. The fill had been placed at an appropriate slow rate to allow for some consolidation under the fill load so that the shear failures during construction are prevented. Monitoring of the settlement of the fill (Figure4 and Figure5) and the lateral displacement at the toe will provide useful information about potential failure. The rate of placement of fill has to be controlled based on the analysis of monitoring data. There are instances of failure when this was ignored. Preload fill should be maintained over a sufficiently long period to ensure that the peaty clay underneath has undergone the anticipated consolidation. The required preloading time could be estimated during the design but the decisions should be made based on the monitoring data. The removed load should be greater than the load imposed by the pavement and traffic and the peaty clay layer should be in an over-consolidated state during operation. With the monitoring data acquired during the preloading stage this aspect should be assessed and a sufficiently preload should be maintained.(Figure 6). The monitoring of settlements of the completed road should be monitored into a sufficiently long period.

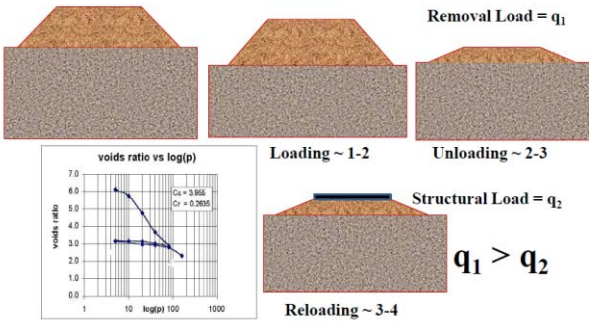

Figure 1 – Basic concept of preloading

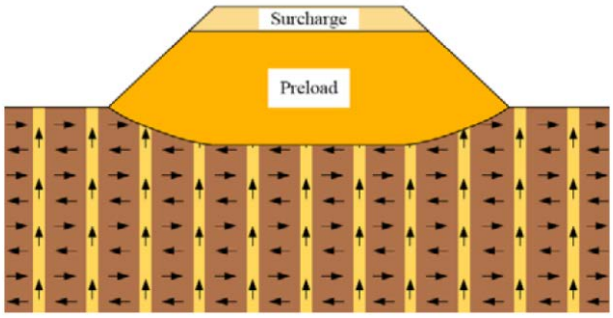

Figure 2- Preloading with vertical drains



Figure 3 – Installation of Prefabricated Vertical Drains

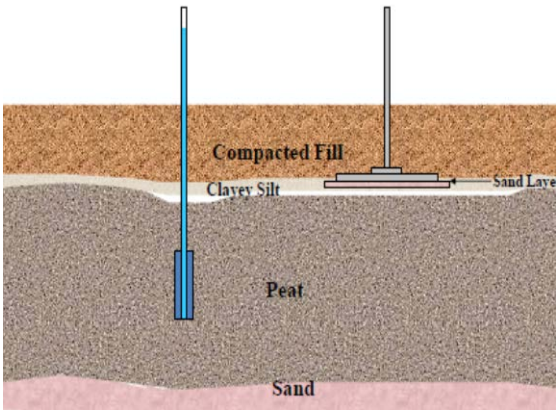

Figure 4 - Monitoring settlement and pwp

Improvements achievable by preloading in compressibility characteristics of peaty clay were studied under laboratory conditions by conducting Oedometer tests with loading, unloading and reloading increments. Peat clays exhibit very high secondary consolidation settlements. Hence the coefficient of secondary consolidation is a very important parameter. In field situations the achievable over consolidation ratio would be in the range of 1.1 to 1.2. As such, in some of the loading-unloading-reloading consolidation tests done in the laboratory the load increment ratios of the order of 1.05 were used instead of doubling in the conventional test.



Figure 5 – A settlement Plate

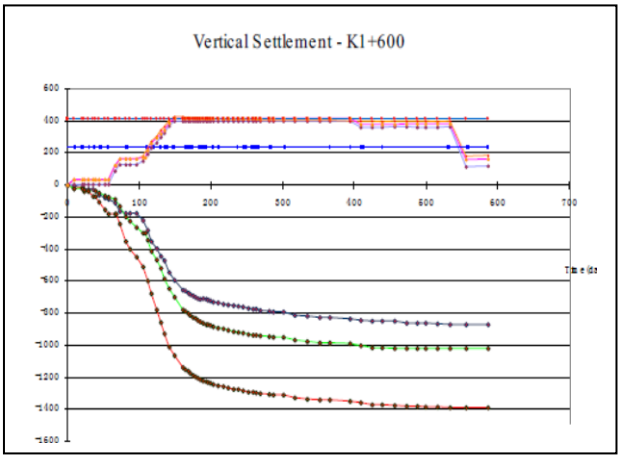

Figure 6 – Typical settlement monitoring data

3. IMPROVEMENT BY VACUUM CONSOLIDATION

When high embankments are constructed on extremely soft peat deposits of high layer thickness, it would not be possible to achieve the desired improvement with preloading only even with the use of vertical drains. Due to very soft nature of the existing soil it may not be possible to place an initial fill and maintain filling rates demanded by the construction schedules as shear failures that would occur. In such situations, construction of an embankment using vacuum consolidation and placement of the fill as the soil improves is the ideal solution. The vacuum consolidation technique had been widely used in the world and it was first applied in Sri Lanka in the construction of the Southern Expressway. In this process, the soft soil layer is sealed by a membrane and a vacuum was created underneath. The vacuum condition is transferred to the soil at a depth using vertical drains. Water pressure within the vertical drains were reduced by the vacuum while natural values prevail within the peaty clay. With the pressure gradient created water will move towards the vertical drains and get discharged. Initially a trial embankment was done in Southern expressway to assess the applicability of the technique for the peaty clay present in the location. The peaty clay layer at the location was with a thickness of around 6m with a water content around 400% and void ratio of the order of 6.0. The compression index is around 2.65 and undrained shear strength was very low at 5 kPa. A 9.6 m fill surcharge was applied in addition to the vacuum load and 1.5 m platform soil. The fill was applied in stages, for the stability considerations. The fill used was a lateritic soil with an average weight of 18 kN/m^3. The test embankment was instrumented to capture the settlements and pore water pressure. The vacuum consolidation was carried out using the “Compact Vacuum Consolidation” (CVC) patent by Maruyama Industry, Japan. The schematic construction is shown in Figure 7(Karunawardena and Nitiwana 2009).

In the construction of the trial embankment, a 1.5 m thick fill was constructed on the original ground surface to form a working platform for the band drain installation rig. Band drains were installed by a machine to a depth of around 6 m from the original ground surface in a square pattern with a spacing of 1 m. Thereafter, flexible horizontal drains (300 mm wide and 4 mm thick) (Figure 8) were laid on top of the fill with a horizontal spacing of 1 m and then connected to the vertical band drains in order to ensure adequate horizontal drainage capacity. Subsequently, the tank systems used in the patent vacuum consolidation method called CVC were installed and connected to the designed pipe systems. Small ditches were excavated perpendicular to the horizontal drains at 20 m intervals and filled with aggregates after placing perforated pipes.

Instrumentation such as settlement plates, displacement stakes, piezometers and differential settlement gauges were also installed at the designed depths. After installation of vertical, horizontal drains, pipes and separator tanks, the surface of the treatment area was covered by a protection sheet. Thereafter, an air tight sheet was laid on top and the periphery trench system was constructed to provide air tightness and the necessary anchorage at the boundary of the treatment area (Figure 10). Vacuum pressure was then applied using a patented vacuum pumping system by connecting the suction and water hoses to the vacuum pump. After confirming that there were no leaks through the air tight sheet, filling was commenced.

An extensive monitoring program was carried out. Number of settlement plates differential settlement gauges and piezometers were installed. A vacuum pressure monitoring unit was used to measure the vacuum pressure at the pump and under the sheet. In addition, a water discharge meter was used to measure the rate and the total discharged water flow due to the vacuum operation. Data were acquired through an automatic data acquisition system. A plot of observed data is presented in Figure 11. The process had been quite effective with the southern expressway.

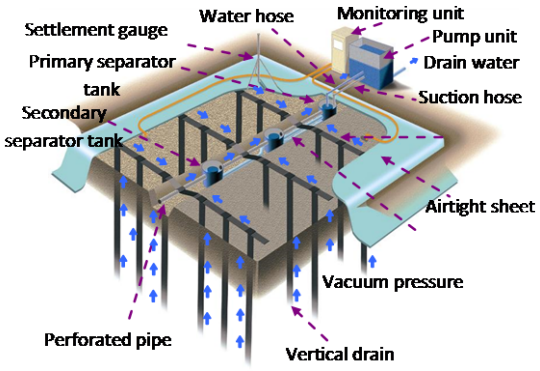

Figure 7 – Schematic of CVC



Figure 8 – Horizontal drains laid and Vacuum pump installed

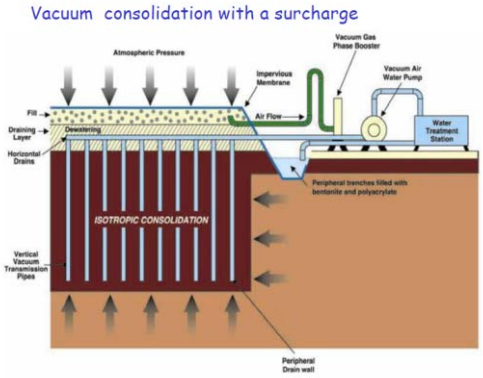

Figure 9 – Vacuum consolidation with a surcharge



Figure 10 – Membrane placed on surface

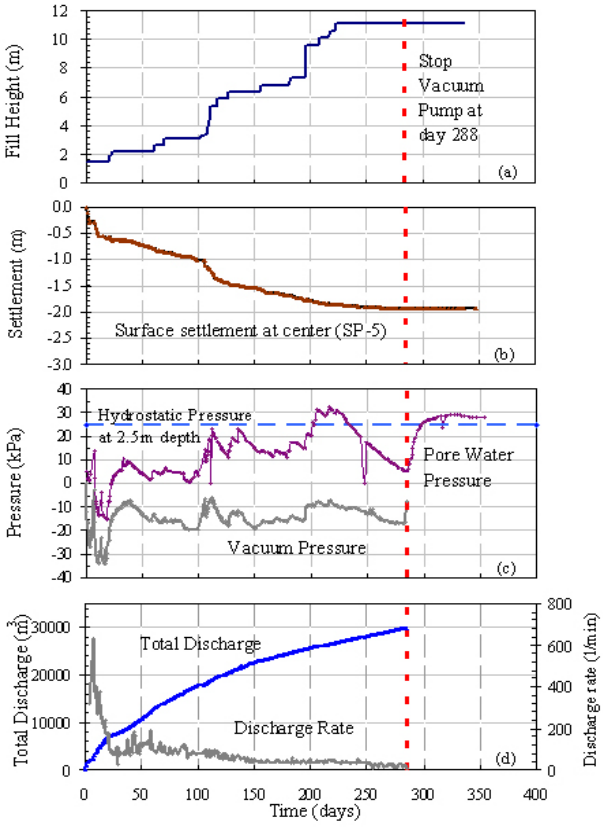

Figure 11 – Typical monitoring data

4. IMPROVEMENTS BY ELECTRO OSMOTIC CONSOLIDATION

Electro osmotic consolidation is another technique that can be adopted for the improvement of extremely soft peaty clay without application of a physical loading. Thus the shear failures are prevented.

When an electric field is applied across a fine grained soil mass, cations are attracted to the cathode and anions are attracted to the anode. As the ions migrate they carry their hydration water with them. In a clay water electrolyte system there are more cations than anions to neutralise the negatively charged clay particles. As such, there is a net flow of water towards the cathode. In addition to these movements, electrolysis of water takes place at the surface of electrodes, producing hydrogen ions at the anode and hydroxyl ions at the Cathode. Also, oxygen gas is produced at the anode and hydrogen gas is produced at the cathode. (Figure 13). As a result, the pH value decreases at the anode and increases at the cathode. If a series of vertical electrodes are placed in the clay layer making some of them anodes and others cathodes and a DC voltage difference is applied, water would move from anode to cathode. If hollow tubes are used as electrodes this water would come out at the cathodes and the Peaty clay would be subjected to consolidation. With polarity reversals at regular intervals a uniform improvement can be achieved over the treated area. When the polarity is reversed the voltage gradient applied can be increased to enhance the efficiency.

Pioneering studies on the improvement of peaty clay with electro osmotic consolidation were done under laboratory conditions at the University of Moratuwa. Electro kinetic properties of peaty clay were found to be within the range of values of other soils where the technique had been applied successfully (Kulathilaka 2006). Initial tests with electro osmotic consolidation under one dimensional conditions showed that compressibility and shear strength characteristics of Sri Lankan peaty clay could be significantly improved by the process (Kulathilaka 2003). Results of the subsequent tests where a field application of the process was modelled in a large box filled with peaty clay with perforated metal electrodes (Figure 14) also indicted that the process is viable (Kulathilaka and Sagarika 2006). However, the corrosion of metal electrodes (Figure 15) due to oxygen released and formation of cracks reduced the level of improvement achieved. With the use of vertical drains made of electro kinetic geosynthetics (EKG), the problem of corrosion was eliminated and the process could be continued for a long period with high voltage gradients.



Figure 12 – EO consolidation Site in Singapore



Figure 14 – EO with metal electrodes

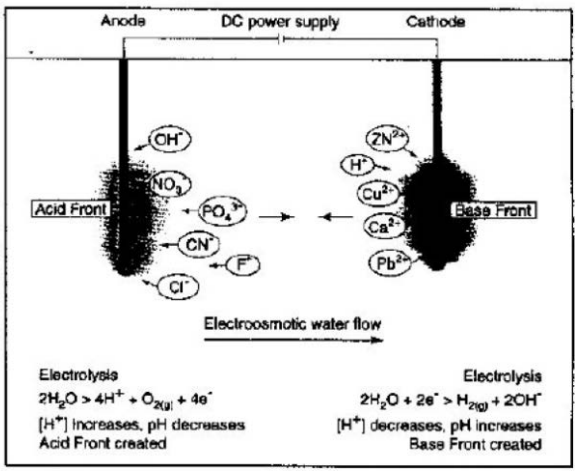

Figure 13 – Electro Osmosis Process

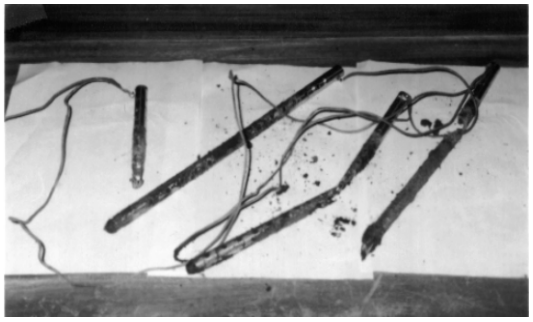

Figure 15 – Metal electrodes corroded

Several series of tests were done in the laboratory with electrokinetic geosythetics (EKG) (Figure 16) showed that considerable improvements could be achieved in strength and stiffness. The cost of electricity was found to be comparable with other techniques (Kulathilaka 2006) A sample of Peaty clay from southern expressway with moisture content 300% with polarity reversal over 12 days with voltage gradients from 37.5 V/m to 120 V/m(Figure 17) consumed 4.7 kWhr/m^3. The voltage gradient applied, the variation of PH and voltage variation across the sample are illustrated in Figure 18 and 19 respectively). This technique can be used at the early stages in a project to improve the shear strength to an adequate level so that a significant fill can be placed without causing shear failure. Thereafter power can be switched off and normal preloading process can continue. Fully instrumented field study need to be carried out.

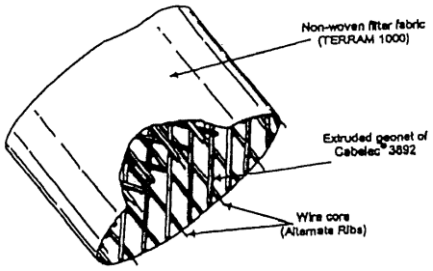

Figure 16 – Electrokinetic Geosynthetics(EKG)

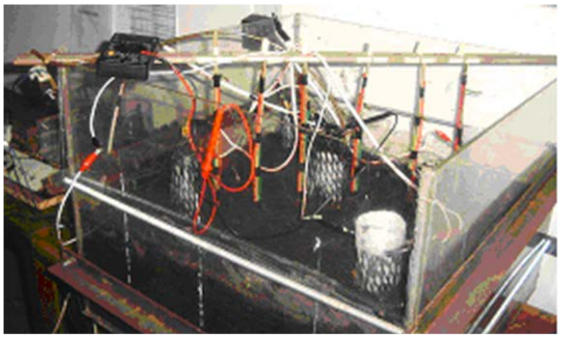

Figure 17 – EO of peaty clay with EKG

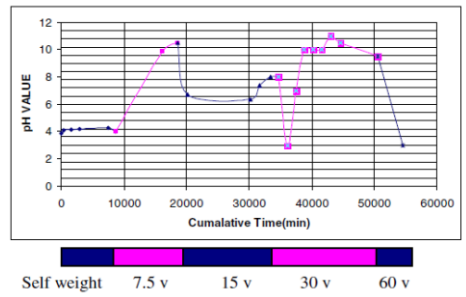

Figure 9. pH Variation with time – Test 3

Figure 18 – Voltage differences and Variation of PH

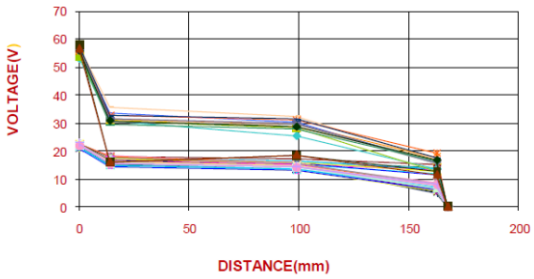

Figure 19 – Voltage distribution across electrodes

5. IMPROVEMENTS BY DEEP MIXING

The techniques of consolidation with; preloading, vacuum consolidation or electro osmosis will take a considerable time even with the use of vertical drains. Deep mixing with cement or lime or with a mix of some other industrial binders in situ is another technique that can be adopted to improve the engineering properties of a soft peaty clay. The binder can be injected into the soil over the layer thickness in the form of a grout (Figure 20). With Peaty clays that are with very high water contents, dry cement powder is used in the mixing process (Figure 21). Peaty soils were improved by mixing with cement, lime and blast furnace slag in number of instances in Sweden and Finland within last decade. The main advantage in the deep mixing process is that the pozzolonic reaction which causes the improvements is completed in around 28 days. Simple machinery were developed in Sweden (Jelisic 2002) (Figure 22) by modifying a crawler mounted crane and mass stabilization of thick peat layers were carried out successfully for the construction of highways and railways. Machinery of this type can be developed locally and the process of deep mixing would be practically viable. As such, mixing with cement or lime could be a plausible method for improvement of soft peaty clay layers encountered in Sri Lankan road projects.

When the peaty clay is mixed in-situ with an appropriate binder such as cement, the hydration reactions and the subsequent pozzolonic reactions will cause a major change in the microstructure

(Figure 24 and Figure 25) and considerable gain in strength and stiffness would be achievable within a relatively shorter period of time such as 28 days. When pore water in the soil reacts with the cement, hydration occurs rapidly and primary cementitious products such as Calcium Silicates, Calcium Aluminates and Hydrated Lime are formed. These particles bind together to form hardened skeleton matrices which enclose unaltered soil particles. Hydration of cement causes a rise of pH value of pore water and Soil Silica and Alumina will be dissolved in this strong bases. These products react with Calcium ions liberated in the process to form insoluble secondary cementitious products. This secondary reaction is known as the pozzolonic reaction. Since some of the calcium ions liberated will be used to satisfy the high exchange capacity of the organic matter, a greater amount of cement is required to stabilize an organic soil. (Maclean and Sherwood 1962).

In order to simulate the deep mixing procedure in the site, peaty clay was mixed with different percentages of a binder with a hand mixer in the laboratory of University of Moratuwa. In order ensure a uniform level of mixing in all cases, similar mixing speeds and mixing durations were used. The mix was placed in buckets and kept submerged for a period of 4 weeks for the pozzolonic action to be completed. These studies have shown that a cement content of the order of 20-25% is required to achieve a desired level of improvement.(Munasinghe 2001, Saputhantiri and Kulathilaka 2011). This is comparable with the cement weights of 200 - 250 kg per cubic meter of treated soils reported in literature (Lahtinen et al 1999). These laboratory studies should be verified with full scale field model studies.

These binders either in dry powder form or wet grout form are mixed with peaty clay in-situ using appropriate machinery. Several researchers in recent times have successfully used mixes of cement and various industrial by products such as blast furnaces slag to stabilize both inorganic and organic clays (Jegandan et al 2010). In the absence of such by products in Sri Lanka, Paddy husk an agricultural waste obtained from milling of rice and used to burnt bricks and tiles is tried out. Paddy Husk Ash (PHA) is a pozzolanic material with silica content of 82 – 87 %. The high percentage of siliceous materials in the PHA makes it an excellent material for stabilization. Recent laboratory studies done at University of Moratuwa indicated that PHA can be very effectively used with cement for this purpose. Primary and secondary consolidation characteristics as well as undrained shear strength were significantly improved.

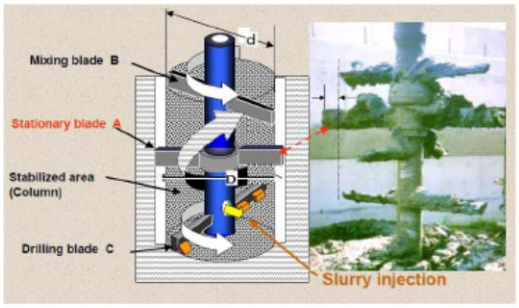

Figure 20 – Wet mixing method

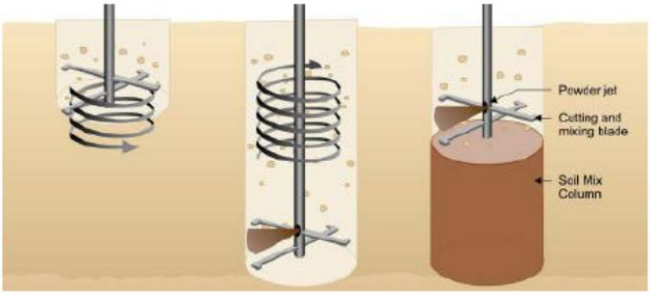

Figure 21 – Dry mixing method



Figure 22 – Mass Stabilization

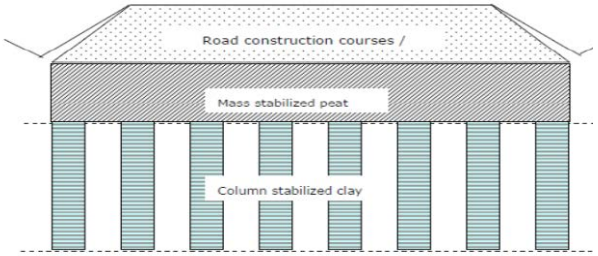

Figure 23 – Combined mass and column stabilization

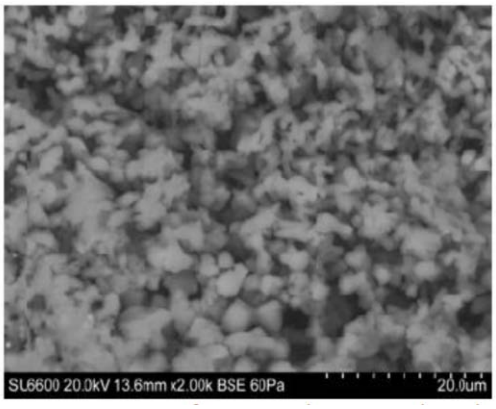

Figure 24 – Microstructure of mixed with 20% cement after 28 days

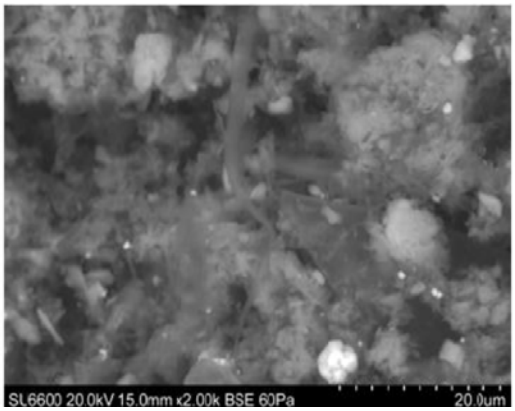

Figure 25 – Microstructure of natural peaty clay

6. ASSESSMENT OF IMPROVEMENTS IN COMPRESSIBILITY CHARACTERISTICS IN THE LABOARATORY

Improvements that could be achieved with the said ground improvement techniques were assessed in a number of research projects over the years through laboratory testing at University of Moratuwa. The preloading process was simulated by conducting Oedometer tests with loading, unloading and reloading increments. The specimen in the reloading increments resembles a preloaded soil. The coefficient of volume compressibility (m_v) and the coefficient of secondary consolidation (C_α) were evaluated in all the loading and reloading increments. The values of compression index (C_c) and recompression index (C_r) were also evaluated using the e Vs log σ plot. Considering the dominance of secondary consolidation in peaty clays loading increments of 3 days duration were used.

With the mixing of cement or PHA the samples were kept under submerged conditions for 28 days in buckets for the pozzolonic reaction to be completed. Thereafter, undisturbed samples were obtained to conduct the laboratory tests.

The comparison of C_c values and C_r values of a natural peat reveals the improvements achievable in primary consolidation characteristics due to preloading. Improvements achieved with other techniques such as electro osmosis and mixing with cement/PHA could be assessed by comparing the C_c values of natural peaty clay and treated samples. (Figure 26, Figure 28, Figure 30 and Figure 31).

The improvements achieved in secondary consolidation characteristics (C_α) due to preloading was assessed by comparing the C_α values in the same stress levels in the loading and reloading increments. The improvements with other treatment methods could be assessed by comparing the C_α values at similar stress levels in natural peaty clay and treated peaty clay. (Figure 27, Figure 29 and Figure 32).

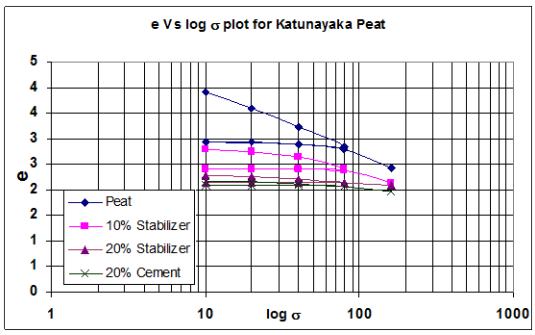

Figure 26 - e Vs log (σ) plots for treated peat

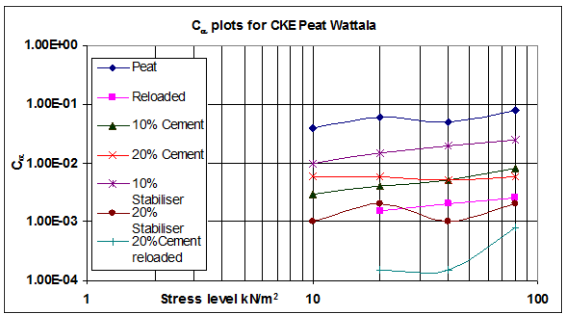

Figure 27 – Effect of treatment on C_α

Improvements achieved in secondary consolidation were presented in Figure 33. The term C_α is used for peaty clay undergoing virgin consolidation and $C_\alpha{}'$ is used for peaty clay in reloading increments,

Thus the ratio $C_\alpha^{/}/C_\alpha$ presents the reduction and it is plotted against the corresponding over consolidation ratio.

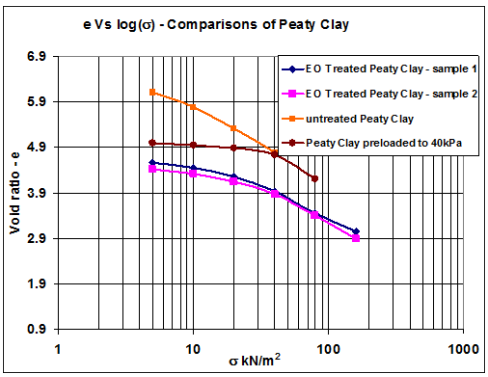

Figure 28 – Improvement of Cc by EO

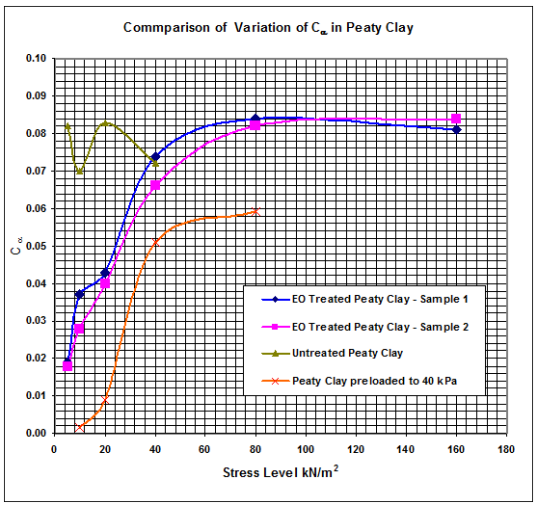

Figure 29 Improvement of C_α by Electro Osmosis

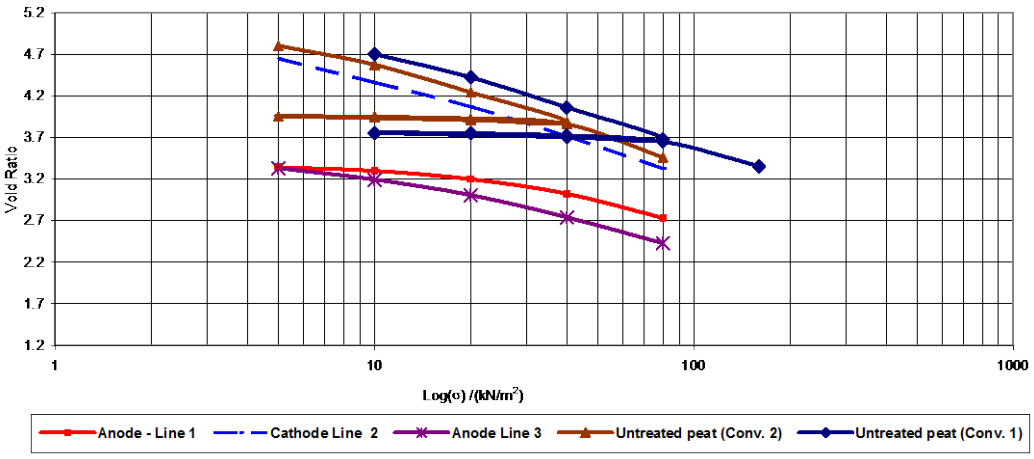

Figure 30 – Comparison of e Vs log (σ) plots for EO treated peaty clay

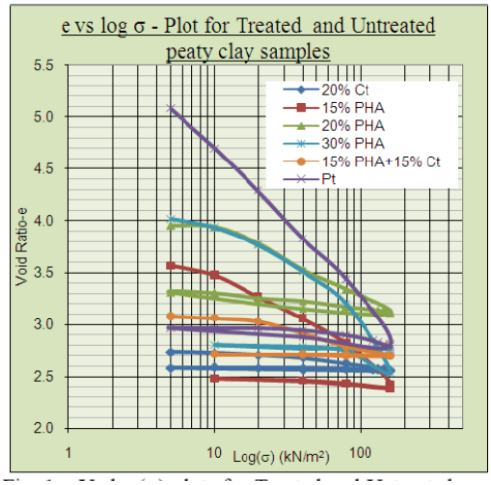

Figure 31- Improvement of primary consolidation characteristics by mixing with cement, PHA

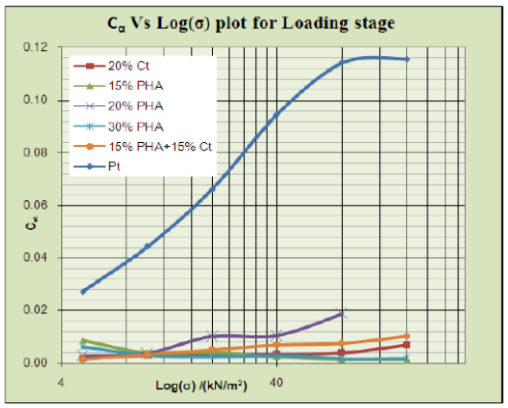

Figure 32 – Improvement of C_α by mixing with Ct/PHA

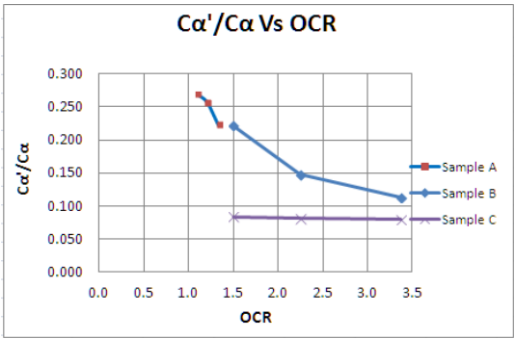

Figure 33 – Effect of OCR on C_α

7. ASSESSMENT IN IMPROVEEMENTS IN SHEAR STRENGTH

In the preloading process it is necessary to assess the improvements achieved in the undrained shear strength due to consolidation under the weight of fill placed already. Increase in the undrained strength of a peaty clay specimen due to the increase of the consolidation pressure, was evaluated by conducting consolidated undrained triaxial tests. It is admitted that this will produce isotropic consolidation although the consolidation under the embankment load is anisotropic. The ratio of increase in undrained strength (ΔC_u) to the increase of consolidation pressure ($\Delta\sigma_3$) was expressed as parameter α ($\alpha=\Delta C_u/\Delta\sigma_3$). The value of α is found to be 0.2 conservatively.

The shear strength of the cement or lime mixed peaty clay was evaluated by conducting unconsolidated undrained triaxial tests. Shear strength of very soft natural peaty clay specimen could not be determined by the triaxial tests as the soil specimen failed under its own weight when placed in the apparatus. Laboratory Vane Shear apparatus was used in such situations. Improvements achieved in electrically treated peaty clay were also evaluated in the same manner. Improvements in undrained shear strength achieved by different treatment techniques under laboratory conditions in a series of research projects are summarised in Table 1, Table 2 and Table 3.

Table 1 – Some records of improvement of C_u

Peat Type	Method of improvement	C_u kN/m^2
Amorpous Peat - Wattala	Natural	0.95 (vane)
	Cement 10%	10
	Cement 15%	12
	Cement 20%	20
Amorpous Peat – CKE Project Wattala	Natural	4.5
	Cement 10%	34.0
	Cement 20%	39.0
	10% Stabilizer	100.0
	20% Stabilizer	250.0
Amorpous Peat from Southern Expressway	Natural	3.6 (vane)
	Cement 5%	13.35
	Cement 10%	20.45
	15% Cement	29.50
	20% Cement	37.10
Amorpous Peat from Southern Expressway	Natural	1
	Electro osmosis	5.1 to 20.4.

Table 2–Improvement of Cu by mixing with cement/PHA

Sample	Specimen	Moisture Content	Cell Pressure kN/m^2	Strain at Failure %	Cu kN/m^2
20% Cement mixed Peat Samples	C1	138	50	13.44	30
	C2	132	75	8.96	
	C3	187	125	14.94	
15% PHA mixed Peat Samples	R1	240	50	45.72	7
	R2	242	75	17.78	
20% PHA mixed Peat Samples	P1	184	50	8.96	6
	P2	188	75	10.45	
30% PHA mixed Peat Samples	R1	242	50	25.40	9
	R2	228	75	20.32	
	R3	239	125	13.78	
15% PHA +15% Cement mixed Peat	C1	154	50	20.32	22
	C2	145	75	25.40	
	C3	169	125	33.02	

Cu of peaty clay = 2 kN/m^2

Table 3 – Improvement of Cu by EO

LOCATION	C_u(kN/m^2)
Near Cathode	19.16
10 cm from Cathode	29.49
Near Anode	38.04

C_u of peaty clay = 2 kN/m^2

8. CONCLUSIONS

Different ground improvement techniques that could be applied to improve the engineering characteristics of peat clay were discussed. Preloading with or without vertical drains would be the simplest technique causing least cost. However, with thick layers of soft peaty clay and/or with high embankments the technique may not be practically feasible due to possible shear failures and long time durations taken. Vacuum consolidation or electro osmotic consolidation would be very helpful in providing a significant increase in undrained shear strength initially and could be used along with preloading in later stages of the treatment.

Deep mixing of the soft peaty clay insitu with cement would provide a very significant increase in shear strength within a short time period of the order of one month so that the construction can be done at a faster rate. However with the high cation exchange capacity it is necessary to use a very high quantity of cement (25 % or 200 kg/m^3) and the cost would be quite high. But the advantage of completion of the project early must also be considered. Industrial by products such as Blast Furnace Slag had been used together with cement very effectively in Scandinavian countries and Europe. Initial research done in Sri Lanka indicated that Paddy Husk Ash a by product from agriculture and brick industry can be used together with a reduced percentage of cement to improve the engineering characteristics of peaty clay.

The levels of improvements achievable by different treatment techniques in compressibility characteristics and undrained shear strength were evaluated in the laboratory in a series of research projects over last two decades. The parameters compared in the assessment process are compression index and coefficient of volume compressibility for primary consolidation and the coefficient of secondary consolidation for secondary consolidation. The results showed that with all the improvement techniques used; preloading, electro osmosis and mixing with cement both the primary and secondary consolidation characteristics were significantly improved.

The laboratory test results also revealed that significant improvements could be achieved in the undrained shear strength by these improvement techniques. However it is essential to confirm the findings on deep mixing and electro osmotic consolidation treatments by instrumented field studies.

The other construction technique that was not discussed in detail is the use of reinforcing columns in a grid for the construction of high road or rail embankments. These columns could be made of compacted granular material (Stone columns) or deep mixing. Stone Columns (Crushed stone columns) were used very successfully in the construction of high embankments over very thick layers of soft peaty clay in the Colombo – Katunayaka expressway. The columns of diameter around 0.5 m - 0.7m are constructed in a grid of 1.2m – 1.5m. A greater share of the embankment load will be taken by these stiffer and stronger columns minimising the possibility of shear failure and reducing the settlements.

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Keynote Address

Numerical and Analytical Approaches to the Problems of Unsaturated Ground

by

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Dr. S. Tachibana is an assistant professor of Geotechnical Engineering, Saitama University. He received his doctoral degree in 2006 from the Tokyo Institute of Technology. After working at Kobe University as a post-doctoral researcher, he moved to Saitama University in 2008. His research has focused on soil mechanics, computational mechanics, deformation /stability analysis of soil/water coupled materials, unsaturated soil mechanics, strain localization of soils, earth-reinforcement mechanisms, and geo-environmental problems.

Numerical and Analytical approaches to the problems of unsaturated ground

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Abstract: *A large number of constitutive models for unsaturated soils have been developed since Bishop (1960) first proposed effective stress for unsaturated soils. This paper presents one of the constitutive models with single stress variable for unsaturated soils, called Se-hardening model (Ohno et al., 2006). The conceptual basis and mathematical formulations are described.*

Keywords: *unsaturated soil, constitutive model, water retention*

1. INTRODUCTION

There have been increasing demands of society on sustainable and resilient maintenance of earthwork. The long-term stability of earth structure such as fill dams and embankments relies on the initial quality at construction and changes in mechanical and hydraulic conditions. Therefore, in dealing with geohazards, key issues are estimation and modelling of soil behaviour over conceivable ranges of stress and pore pressure values and physical quantities such as void ratio and degree of saturation. To assess and predict the future performance not only of earthwork but natural slope, much effort have been made towards generalization of soil mechanics principles to unsaturated soils and application of these principles to numerical modelling.

Soils of which pore is partially saturated with water are defined as unsaturated soils. The mechanical behaviour of unsaturated soils takes on various pictures and depends upon not only stress changes but also saturation or suction changes. For instance, whether soil expands or shrinks due to wetting depends on the stress history and the current state. Such volume changes sometimes result in severe damage to soil structure or natural ground. On the other hand, it is well known that desaturation causes the increases of stiffness and strength of soil. Hence, it is required that constitutive model for unsaturated soil expresses such complex behaviours as mentioned here. The water retention which provides the relationship between the suction and the degree of saturation is also an essential characteristic in constitutive modelling. Since there exists a hydraulic hysteresis in this relationship, the degree of saturation is not determined uniquely even when the suction is specified. These irreversibility or history-dependency of both mechanical and hydraulical properties yield the material non-linearity and diversity.

As a starting point in constitutive modelling for unsaturated soils, the choice of independent stress variables determines how to analyse the laboratory data and how to establish the constitutive framework. The ways of this choice can be broadly classified into two categories; (1) net stress and suction approach and (2) single effective stress approach. The former approach was firstly applied by Coleman (1962) and Bishop and Blight (1963) and was supplemented with the argument by Fredlund and Morgenstern (1977) that any two stress variables among net stress $\sigma - p_a$, matric suction $p_a - p_w$ and $\sigma - p_w$ would be sufficient to describe fully the stress state of unsaturated soils. An elasto-plastic critical state model based on this approach was pioneered by Alonso et al. (1990) and it has been followed in many models (e.g., Wheeler and Sivakumar, 1995, Cui and Delage, 1996, and Thu et al., 2007). A distinctive feature of this approach is that the volume change due to net stress change is separated from that due to suction change. Accordingly, the compressibility due to net stress and suction changes are estimated and modelled separately. The latter approach deals with the

single effective stress into which the net stress and the matric suction are combined. This approach was subjected of criticism in the early works of constitutive modelling. Jennings and Burland (1962) and many subsequent researchers pointed out that the volume change behaviour could not be controlled by the single stress variable such as an effective stress proposed by Bishop (1959). In particular, it was a critical shortcoming that the model was unable to represent the swelling and collapse due to complex swelling transition. Therefore, it is common in recent fashion of modelling based on this approach to use not only the single effective stress but also the degree of saturation or its function as an internal variable or a hardening parameter by incorporating the hydraulic hysteresis to describe the appropriate behaviour of unsaturated soils (e.g., Ohno et al., 2007, and Kyokawa et al., 2009). There are state-of-the-art reviews providing the detailed explanation for the constitutive models besides mentioned here and their advantages and disadvantages, e.g, Wheeler and Karube (1996), Kohgo (2003), Gens (2010) and Sheng (2011). These papers may be of great use for studying this topic.

This paper presents one of the constitutive models with single stress variable for unsaturated soils, called Se-hardening model (Ohno et al., 2006). The conceptual basis and mathematical formulations are described.

2. CONSTITUTIVE MODELLING

Se-hardening model deals with not only the single effective stress but also the effective degree of saturation as a hardening parameter by incorporating the hysteresis curve in the soil-water retention characteristics (SWRC).

2.1. Stress and State Variables

The choice of stress variables is central in constitutive modelling since it affects on how to analyse the laboratory data and may determine the mathematical formulation. A general form of the effective stress for unsaturated soils is

$$\sigma'_{ij} = \overline{\sigma}_{ij} + f(s)\delta_{ij} \qquad (1)$$

where σ'_{ij} is the effective stress tensor, $\overline{\sigma}_{ij}$ the net stress, $f(s)$ the function of matric suction s, and δ_{ij} the Kronecker delta. The net stress is the total stress σ_{ij} in excess of the pore air pressure p_a, whereas the suction is the pore air pressure p_a in excess of the pore water pressure p_w. In Eq. (1), the function $f(s)$ is a scalar so that the effect of suction works on the effective stress isotropically. Bishop (1959) proposed the effective stress such that $f(s) = \chi s$ where χ is a state parameter which varies from 0 for dry to 1 for saturated states. In Se hardening model, the effective degree of saturation S_e is applied in Eq. (1) instead of a parameter χ as

$$f(s) = S_e s \text{ and } S_e = \frac{S_r - S_{r0}}{1 - S_{r0}}. \qquad (2), (3)$$

where S_r is the current degree of saturation, and S_{r0} the residual value of degree of saturation when $s \to \infty$. This equation will not only guarantee the continuity between saturated and unsaturated states, but also the continuity between unsaturated and mechanically driest states ($S_e = 0$ or $S_r = S_{r0}$). Karube et al. (1996) named the term $S_e s$ suction stress (not suction itself) and proposed further decomposition into two parts: meniscus stress and bulk stress. The former stress arises from the existence of adsorbed water films and the latter from the bulk water occupying a part of space among soil skeletons. This idea is of interest to consider the components of pore water and their contributions on soil behaviour. However, such decomposition requires another constitutive relation to determine either stress in addition to soil water retention curve, which causes awkward constitutive modelling. To avoid the complexity, therefore, Eq. (2) is used as it stands to define the effective stress in Se

hardening model. It is noted that the function $f(s)$ in Eq. (2) is not determined uniquely even when the suction s is specified due to debt of soil water retention hysteresis.

Experiment data proves the advantages in usage of the effective stress in Eq. (1) together with the suction stress $S_e s$: the linear critical state line without an intercept is uniquely determined in the mean effective stress p' and the stress deviator q plane regardless of the degree of saturation S_r (e.g., Geiser et al., 2000, Kawai et al., 2002). Moreover, Kawai et al. (2002) showed that the stress-dilatancy relation of unsaturated state coincides with that of saturated state by using the stress ratio q/p'. These experiment evidences suggest the applicability of the critical state elasto-plastic concept to unsaturated soil behaviour and the similarity of the shape of yield surface regardless of desaturation, which is one of reasons to use the effective stress combined with the suction stress $S_e s$ in the constitutive modelling.

2.2. Hardening and Yield Characteristics

Another advantage in usage of effective stress is the parallelism of normally compression lines and critical state lines for various effective degree of saturation S_e in the void ratio e and the logarithm of effective mean stress $\ln p'$ plane. The normally compression line for an arbitrary effective degree of saturation cannot be indeed obtained from a single test because it may be impossible to control the degree of saturation in laboratory test, but one can represent isoplethic lines of the effective degree of saturation in $e-\ln p'$ plane by organising the various isotropic loading test results of suction-controlled or net stress-controlled tests. Ohno et al. (2006) arranged the test results on clay obtained by Kato (1998), and showed that the normally compression lines of unsaturated dryer state run almost parallel and locate at higher stress level to that of saturated state in $e-\ln p'$ plane. In other words, this implies that an unsaturated dryer soil retains a looser state under same effective stress level or has a higher yield stress than a fully saturated soil. Similar trend can be seen in the critical state in $e-\ln p'$ plane (e.g., Sivakumar, 1993, Cui and Delage, 1996).
Figure 1 shows the conceptual diagram of normally compression lines of Se hardening model in $e-S_e-p'$ space. The compression line for a certain effective degree of saturation can be formulated as

$$e = e_0 - \lambda \ln \frac{p'}{\zeta p'_0} \tag{4}$$

where λ is the compression index, p'_0 the initial yield stress for saturated state, e_0 the void ratio at initial yielding. The parameter ζ is a function of the effective degree of saturation S_e to describe the increase of apparent initial yield stress caused by desaturation and can be modelled as

$$\zeta = \zeta(S_e) = \exp\left[(1-S_e)^n \ln a\right] \tag{5}$$

where a and n are material parameters, the former provides the location of compression line for driest unsaturated state ($S_e = 0$ or $S_r = S_{r0}$) and the latter determines that for an arbitrary unsaturated state. One can formulate the volumetric strain by using Eq. (4) as

$$\varepsilon_v = \frac{e_0 - e}{1+e_0} = \frac{\lambda}{1+e_0} \ln \frac{p'}{\zeta p'_0}. \tag{6}$$

The swelling line for a certain effective degree of saturation can be expressed as

$$e = e_0 - \kappa \ln \frac{p'}{\zeta p'_0} \tag{7}$$

where κ is the swelling index. Eq. (7) also assumes the parallelism of swelling lines for various effective degree of saturation S_e. The elastic volumetric strain can be obtained by using Eq. (7) in a similar way to Eq. (6) as

$$\varepsilon_v^e = \frac{e_0 - e}{1+e_0} = \frac{\kappa}{1+e_0} \ln \frac{p'}{\zeta p_0'} . \tag{8}$$

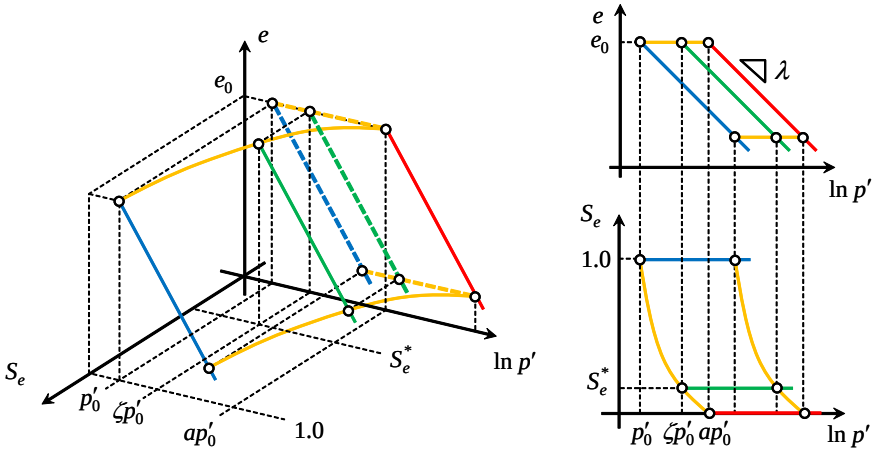

Figure 1 Normally compression lines of Se hardening model

Under the assumption of additive decomposition of total strain into elastic and plastic parts, the plastic volumetric strain is given by Eqs. (6) and (8) as

$$\varepsilon_v^p = \varepsilon_v - \varepsilon_v^e = \frac{\lambda - \kappa}{1+e_0} \ln \frac{p'}{\zeta p_0'} . \tag{9}$$

Thus, if one assumes the isotropic hardening, setting $p' = p_c'$ yields the following expression of the growth of pre-consolidation pressure p_c':

$$p_c' = \zeta p_0' \exp \frac{\varepsilon_v^p}{(\lambda - \kappa)/(1+e_0)} . \tag{10}$$

Eq. (10) represents the hardening law of this model and the pre-consolidation pressure p_c' as a stress-like hardening parameter can now be expressed as a function not only of the plastic volumetric strain ε_v^p but also the effective degree of saturation S_e via Eq. (5).

As mentioned in previous section, we can assume the existence of a single critical state line and the similarity of the shape of yield surface regardless of desaturation. Accordingly, the yield function for the unsaturated soil can be expressed as

$$f\left(\sigma_{ij}', \varepsilon_v^p, S_e\right) = \frac{\lambda - \kappa}{1+e_0} \ln \frac{p'}{\zeta p_0'} + \frac{\lambda - \kappa}{1+e_0} \frac{1}{M} \frac{q}{p'} - \varepsilon_v^p . \tag{11}$$

where M is the critical state parameter. In Eq. (11), the similar yield surface as the original Cam-clay model (Roscoe et al., 1963) is applied. Thus, this model reduces to the original Cam-clay model naturally and continuously when the soil transits to a saturated state ($S_e = 1$). One can apply the different shape of yield function such as a modified Cam-clay model (Roscoe and Burland, 1968) depending on the soil dilatancy. In any case, a key essence of Se hardening model is that the yield surface expands and/or shrinks with not only the plastic volumetric strain but also the effective degree of saturation as shown in Figure 2. Hence, this model enables to express higher stiffness and strength at dryer unsaturated state and diverse volume change behaviour due to wetting or drying.

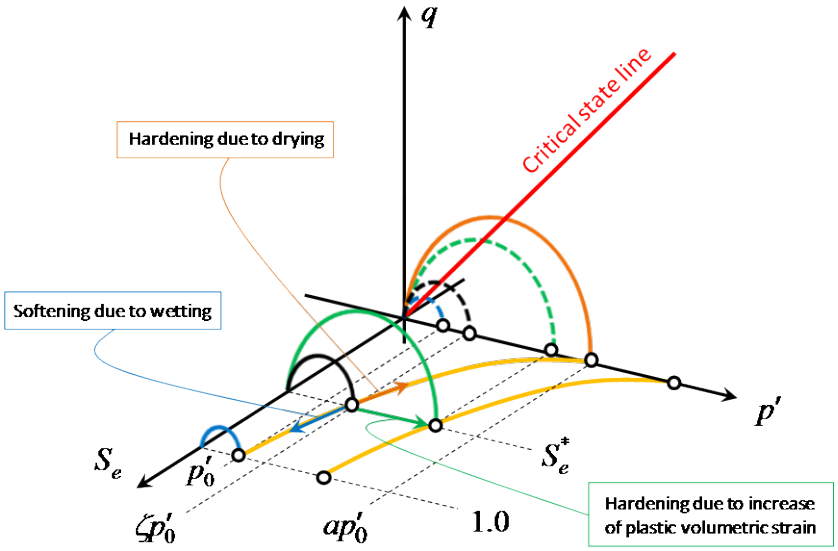

Figure 2 Yield surface of the Se hardening model

2.3. Stress-strain-Saturation Relation

The stress-strain-saturation relation of Se hardening model is derived in a similar way of classical plasticity. The isotropic part of elastic relation of Se hardening model can be obtained in the incremental form of Eq. (8) as

$$dp' = Kd\varepsilon_v^e + K_e dS_e . \tag{12}$$

where K and K_e are the elastic bulk modules, but the former is related to stress-strain relation whereas the latter stress-saturation relation. Both moduli can be expressed as

$$K = \frac{1+e_0}{\kappa} p' \text{ and } K_e = -p'n\left[(1-S_e)^{n-1}\ln a\right]. \tag{13}$$

By incorporating with Eq. (12), the elastic stress-strain-saturation relation is given by

$$d\sigma_{ij}' = D_{ijkl}^e d\varepsilon_{kl}^e + K_e dS_e \delta_{ij} . \tag{14}$$

where D_{ijkl}^e is the forth order elastic stiffness tensor in the form of

$$D_{ijkl}^e = \left(K - \frac{2}{3}G\right)\delta_{ij}\delta_{kl} + G\left(\delta_{ik}\delta_{jl} + \delta_{il}\delta_{jk}\right). \tag{15}$$

The elastic shear modulus G may be determined by applying the generalized Hooke's law as

$$G = \frac{3(1-2\nu')}{2(1+\nu')}K . \tag{16}$$

where ν' is the Poisson's ratio.
In order to derive the incremental elasto-plastic stress-strain-saturation relation, we apply the consistency condition of Eq. (11) as

$$df = \frac{\partial f}{\partial \sigma'_{ij}} d\sigma'_{ij} + \frac{\partial f}{\partial \varepsilon_v^p} d\varepsilon_v^p + \frac{\partial f}{\partial S_e} dS_e = 0 \,. \tag{17}$$

By substituting Eq. (14) of elastic relation with associated flow rule:

$$d\varepsilon_{ij}^p = \gamma \frac{\partial f}{\partial \sigma'_{ij}} \,. \tag{18}$$

one can derive the incremental stress-strain-saturation relation as

$$d\sigma'_{ij} = D_{ijkl}^{ep} d\varepsilon_{kl}^e + c_{ij} dS_e \,. \tag{19}$$

where,

$$D_{ijkl}^{ep} = D_{ijkl}^e - \frac{1}{H} D_{ijAB}^e \frac{\partial f}{\sigma'_{AB}} \frac{\partial f}{\sigma'_{CD}} D_{CDkl}^e \,. \tag{20}$$

$$c_{ij} = -\frac{1}{H} c_{ijAB}^e \frac{\partial f}{\sigma'_{AB}} \left(\frac{\partial f}{\partial p'} K_e + \frac{\partial f}{\partial S_e} \right) - K_e \delta_{ij} \,. \tag{21}$$

$$H = \frac{\partial f}{\sigma'_{AB}} c_{ABCD}^e \frac{\partial f}{\sigma'_{CD}} - \frac{\partial f}{\partial \varepsilon_v^p} \frac{\partial f}{\partial p'} \,. \tag{22}$$

3. CONCLUSION

One of the ultimate goals of constitutive modelling is to implement the model in a numerical method to solve the boundary value problem. To approach the problem of the unsaturated soil ground, the water retention curve with hysteresis characteristic should be imposed as another constitutive model in addition to Eq. (19). Constitutive models are not judged based on the stress variables they used, rather on their quantitative predictions on observed soil behaviour.

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Challenges and Solutions in Geotechnical Engineering

Optimization of Landfill Industrial Wastes in Japan by Mixing Various Waste Samples, Effects on Their Dry Bulk Densities and Consolidation

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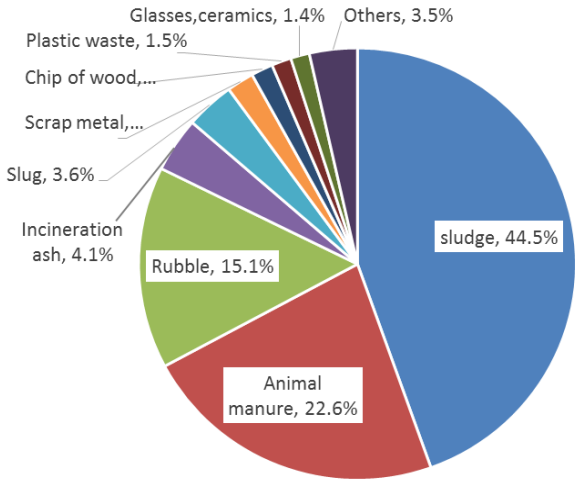
Abstract: *The purpose of study is to find out effects of waste compositions and mixing proportions on the consolidation and to optimize the dry bulk density of the waste samples. In this study, totally 6 different waste materials, un-burnable domestic waste, un-burnable industrial waste, incineration ash, crushed concrete, organic sludge and inorganic sludge, were used as tested materials. Standard compaction/proctor test results showed that maximum dry bulk densities of the incineration ash (1.53-1.74 g/cm^3) and crushed concrete (1.45g/ cm^3) were higher than the inorganic sludge (0.76 g/ cm^3) and organic sludge (0.90 g/ cm^3) respectively. The maximum dry bulk densities for mixed sample of inorganic sludge, concrete and incineration ash were greater than inorganic sludge. In especial, the maximum dry bulk density for the mixed sample with ratio 2:6:2 and 2:2:6 (1.65g/ cm^3) dry mass basis were 2.17 times greater than that of inorganic sludge. Consolidation tests were carried out for selected pre-compacted waste samples with degree of compaction higher than 90 % after the compaction tests. For the consolidation tests, oedometer test apparatus which dimension of 10 cm diameter and 10 cm height was used for the waste materials with particle size larger than 2mm. Results of each independent sample showed that the coefficient of consolidation (C_v) for crushed concrete and incineration ash was higher than organic and inorganic sludge wastes while compressibility of un-burnable industrial waste was higher than the other materials due to presence of compressible materials. As the mixing ratio of crushed concrete in the mixed samples increased, the compression index (C_c) decreased.*

Keywords: consolidation, compaction, wastes.

1. INTRODUCTION

Construction of new landfill facilities, especially for final waste disposal sites, are becoming increasingly difficult due to the scarcity of suitable sites and public resistance in most countries all over the world. Thus, optimization of landfilling space such as reduction in waste volume and prevention of landfilled ground settlement is significant for extending service life of existing facilities and for utilizing waste landfilled lands after closure.

The compaction of the solids waste by mixing of the various proportions can be considered as one of the important and primary methods for the optimization of landfilling space. The main factor that controls the optimization is the landfilling density of wastes. Maximizing the landfilling density allows reduction in landfill space requirement or to prolong the life of a facility (Ham et al., 1978). The landfilling density influences the stability of landfill, as high densities associates with the shear strength. Overall, moisture-density characteristics of solid waste are critical for both operations of landfill and engineering properties (Hanson et al., 2010). While waste compaction gives more optimistic result on the landfill capacity and long term stability, and it may drastically be affected on physical and chemical properties of waste.



※ Ministry of the Environment: Survey report of industrial waste emissions and disposing status in 2009

Figure 1. Waste production status, Japan

2. MATERIAL AND METHODOLOGY

2.1. Testing Materials

Six different types of wastes were being used to find out the compaction and consolidation properties of the each individual samples and different types of their mixed proportions. Figure 2 showing the waste materials used in research.



Unburnable/Domestic (Glasses, ceramic etc) Ogawa town (Saitama Prefecture)



Unburnable/industrial (Plastic, rubber etc) Intermediate treatment plant Saitama)



Incineration Ash (Intermediate treatment plan Saitama)



Crushed concrete (Intermediate treatment plan Saitama)



Organic Sludge (Soya sauce Factory Chiba Prefecture)



Inorganic Sludge (Water treatment plan Chiba Prefecture)

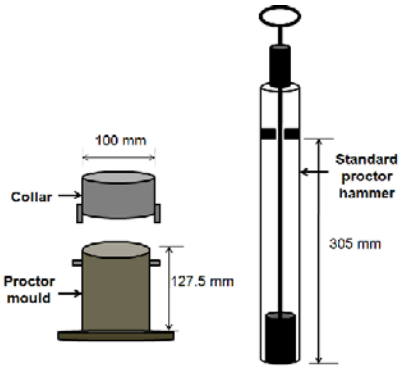
Figure 2. Waste Materials used in research

2.2. Methodology:

Compaction of samples were achieved by using standard compaction test and further modified oedometer was used for consolidation after getting more than 90%degree of compaction.

Standard compaction test (JIS A 1210 & ASTM D-4253)

Sample was prepared by using standard mould with inner diameter 10cm and length 12.75cm ($1000cm^3$). The sample was packed into the mould in three equal layers to make the homogenous compaction throughout the sample. The number of blows 25/layer and drop height of the hammer having weight of 2.5 kg was 30.5 cm. Finally bulk density of each sample was calculated. The samples for moisture contents were collected from the top and bottom to determine the average dry density of the sample. Finally, graph b/w the dry densities and moisture contents were drawn to calculate the maximum bulk dry density and optimum moisture contents. Schematic diagram of the proctor apparatus is shown in figure 3.



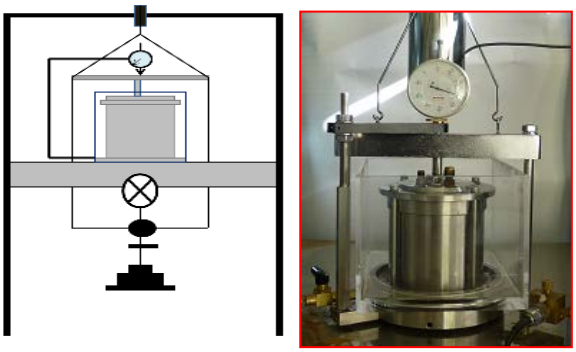


Mould dimensions = Diameter-10 cm
= Height- 12.75 cm
Weight of rammer = 2.5kg
Free drop height = 305mm
No. of free drops = 25/layer

Figure 3. Standard compaction test apparatus

One dimensional consolidation test (JISA 1217 & ASTM2435-90)

Consolidation test was conducted using a specially designed oedometer with 10-cm diameter and 10-cm height because of some sample having size (2-mm to 9-mm) more than 1/10th of the mould diameter and height. Samples were prepared with the standard proctor method having degree of compaction more than 90%. After the sample was placed in the consolidation apparatus, it was allowed to saturate by filling the water in the device and applied some initial pressure of 3.75 kPa to avoid the swelling of the sample. After saturation further load was applied and prolonged for one day to complete the primary consolidation. Additionally, from $\sqrt{t}$ method time for 90% and 100% consolidation were calculated. Loading was applied in eight steps (7.05 to 905 kPa) excluding the saturation loading of 3.75 kPa. Schematic diagram of the apparatus used in shown in figure 4.



Mould Dimensions;

Diameter (d) =10 cm
Height (h) =10 cm
Loading/ (kPa) =3.75-905

Figure 4. Oedometer test apparatus

3. RESULTS & DISCUSSION

Basics properties of the waste materials were determined including the sieve analysis, specific densities, loss on ignition, Atterberg limits, pH values and electrical conductivity etc.

Specific gravity of the incineration ash was comparatively greater than the other waste samples as shown in the table 1, while least one was observed in case of unburnable industrial waste sample.

Table 1 Waste samples properties

Waste type	D(mm)	w(%)	w_L(%)	w_p(%)	I_p	Gs	LOI(%)	pH	EC(mS/cm)	D_{50}(mm)
Unburnable/Domestic (glasses,ceramic,ete)	<2mm	1.63	-	-	-	2.50	1.2	7.2	0.24	5.50
	<9.5mm					2.46				
Unburnable/Industrial (plastic,rubber,ete)	<9.5mm	1.60	-	-	-	1.38	81.0	7.0	0.37	15.98
Incineration ash	<2mm	9.57	-	-	-	2.69	1.6	11.1	1.99	0.99
	<9.5mm					2.64				
Crushed concrete	2-9.5mm	0.00	-	-	-	2.34	9.5	11.0	0.47	16.06
Organic sludge	<2mm	46.38	70.22	64.56	4.82	2.22	28.3	4.9	8.60	0.040
Inorganic sludge	<2mm	65.76	291.42	130.75	160.67	2.20	38.2	6.6	0.29	0.011

*w = Moisture contents, w_L =Liquid limit, w_p =Plastic limit, I_p = Plastic index, G_s =Specific gravity, LOI = Loss of ignition, EC = Electrical conductivity, D_{50} = 50% finer by weight

3.1. Compaction Test Results

Standard proctor tests were conducted on each individual sample to check the maximum bulk dry densities of the samples.

In figure 5, experimental results showed that domestic wastes and incineration ash were independent of the initial moisture contents while crushed concrete having a slight effect of water absorption on its bulk dry density. Organic and inorganic sludge samples dry densities were relying on the addition of moistures. Unburnable industrial waste cannot be packed and compacted because of having rubber and plastics, so its density was also not effecting by initial moisture contents.

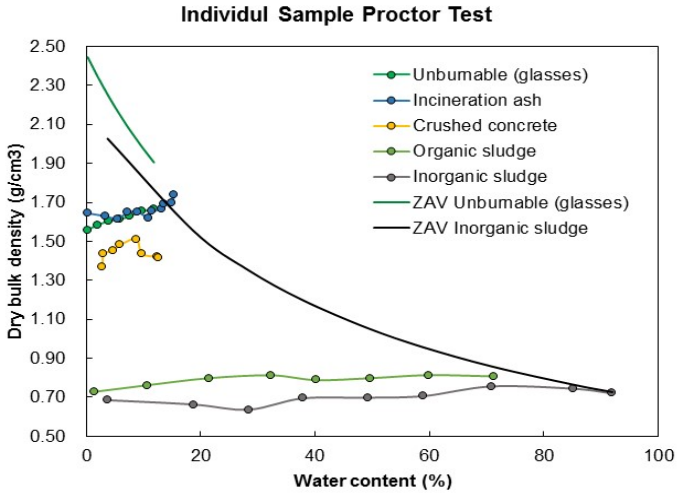


Figure 5. Compaction curves for individual samples

Table 2 Individual sample maximum bulk dry densities

Sample	Unburnable Domestic Waste	Incineration ash	Crushed concrete	Inorganic sludge	Organic Sludge
Max dry density of Soil (g/cm3)	1.78 - 1.83	1.74	1.45	0.76	0.90
Wopt (%)	---	---	8.0	70.9	57

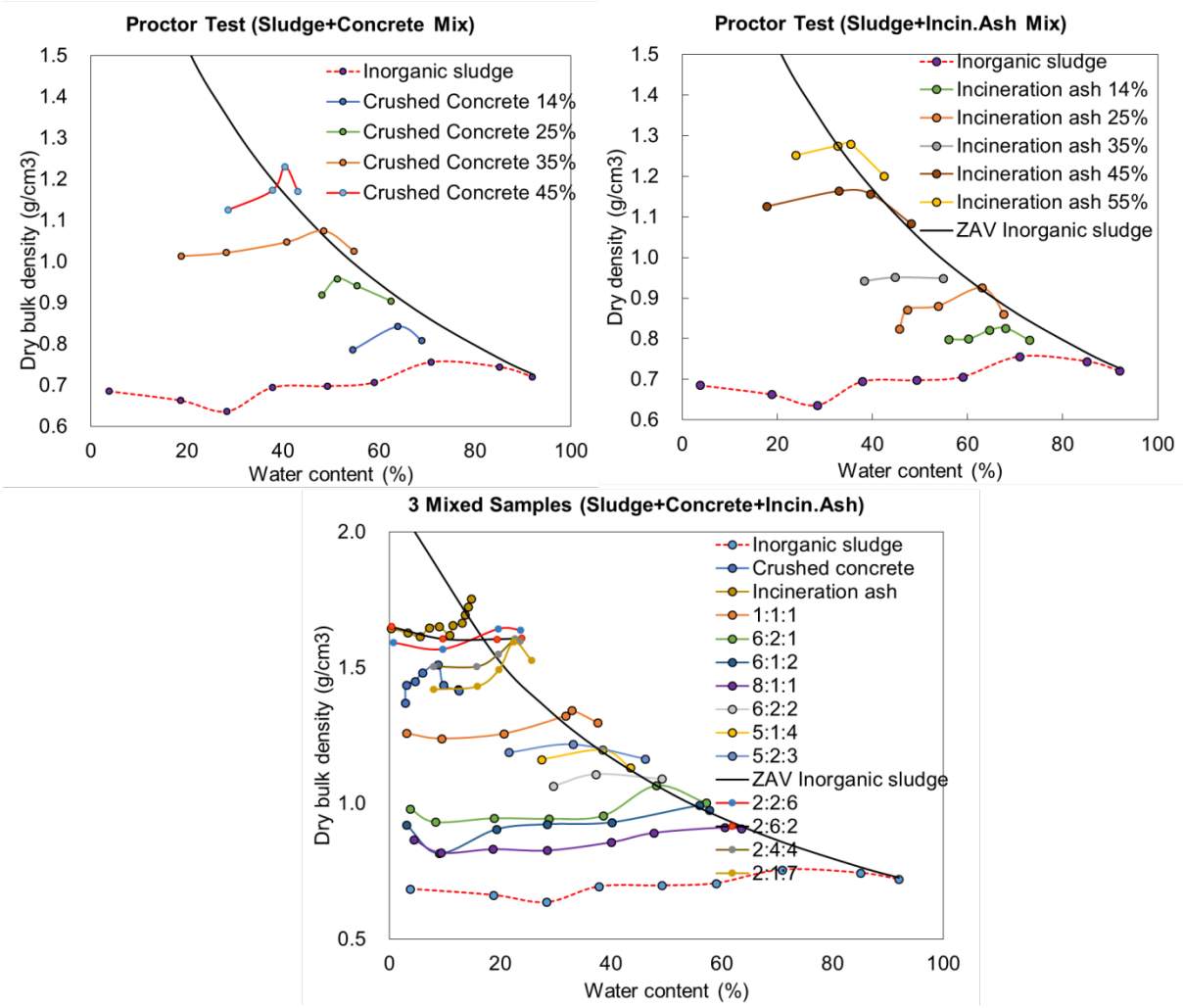


Figure 6. Proctor test on the mixed samples

It is obvious from the waste production status in Japan shown in figure 1, that major portion of the waste consists of the sludge (inorganic and organic sludge) while the maximum bulk dry density of the sludge was least amongst all the waste samples. Therefore, mixed sample were prepared to achieve the maximum dry bulk densities of the sample, so more material can be placed in less space and further it could be used for the engineering purposes.

As with the two mixed samples, inorganic sludge and crushed concrete, figure 6(a), bulk dry density is increased with the increase of the crushed concrete proportion. Similarly with the increase of incineration ash proportion to the inorganic sludge, increases the bulk dry density of the mixed sample and shows the maximum value in 55% ash proportion in the observed testing data, figure 6 (b). In three mixed samples, densities of samples further improved as shown in figure 6(c).

Inorganic sludge showed the least value of the dry bulk density while the highest value was observed in the incineration ash as shown in figure 5. Therefore, for making the optimum densities of the

sludge, it was mixed with other waste samples to improve its densities.

Table 3 Moisture and dry bulk densities relationship of mixed samples

	Label	Inorganic Sludge (%)	Crushed Concrete (%)	Incineration Ash (%)	Maximum ρs (g/cm3)	Wopt (%)
Two Mixed Sapmles	Crushed Concrete 14%	86	14	0	0.84	65.0
	Crushed Concrete 25%	75	24	0	0.96	60.0
	Crushed Concrete 35%	65	35	0	1.00	52.3
	Incineration Ash 14%	86	0	14	0.83	68.0
	Incineration Ash 25%	75	0	25	0.88	55.7
	Incineration Ash 35%	65	0	35	0.95	46.5
	Incineration Ash 45%	55	0	45	1.17	36.5
	30% Concrete + 70% Ash	0	30	70	1.64	---
	50% Concrete + 50% Ash	0	50	50	1.68	---
Three mixed Samples	8:1:1	80	10	10	0.91	62.0
	6:1:2	66.66	11.11	22.22	1.00	56.0
	6:2:1	66.66	22.22	11.11	1.01	50.5
	1:1:1	33.33	33.33	33.33	1.38	33.7
	6:2:2	60	20	20	1.11	38.4
	5:1:4	50	10	40	1.20	38.5
	5:2:3	50	20	30	1.22	32.0
	2:6:2	20	60	20	1.65	---
	2:2:6	20	20	60	1.65	21.0
	2:4:4	20	40	40	1.61	22.8
	2:1:7	20	10	70	1.59	22.5

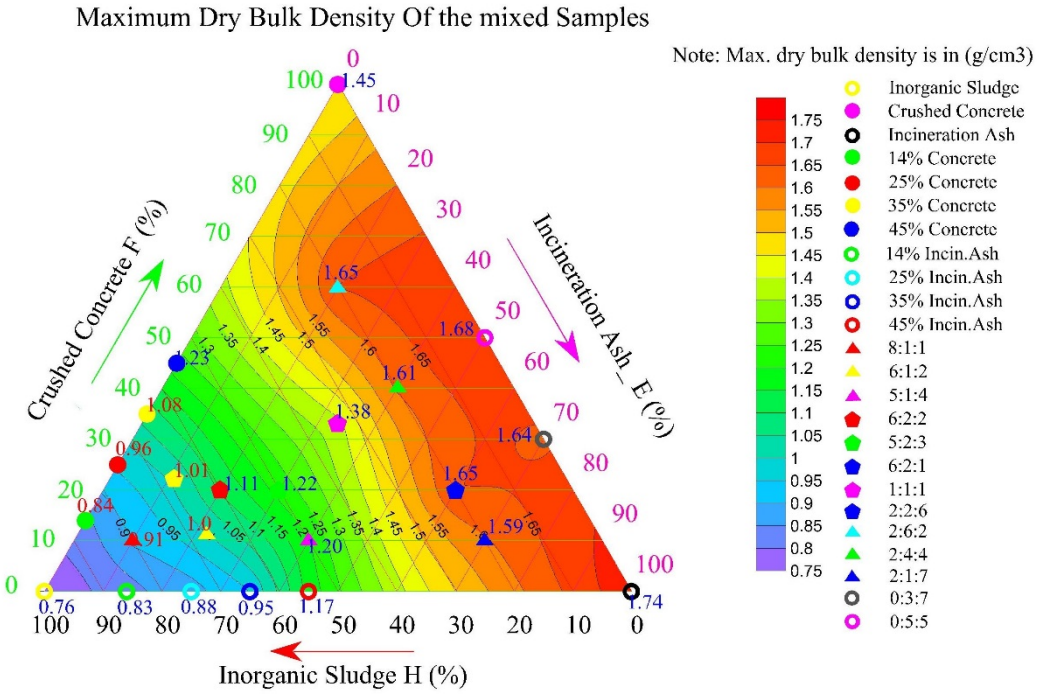


Figure 7. Contour plots for maximum dry densities for tested samples

3.2. Consolidation Test Results:

After doing the compaction tests on the waste samples, consolidation tests were conducted to evaluate the consolidation parameter like permeability (k), co-efficient of volume compressibility (m_v), co-efficient of consolidation (C_v) and compression index (C_c) of the waste samples.

Contours were drawn for mixed and individual sample's compression index shown in figure 9 and results showed that least compression index is observed in the 5:2:1, 5:2:2 and 1:1:1 while maximum was observed in the 25% incineration ash mixed sample which is 0.27. Experimental results also showed that with the addition of crushed concrete, compression index decreases as concrete is incompressible material.

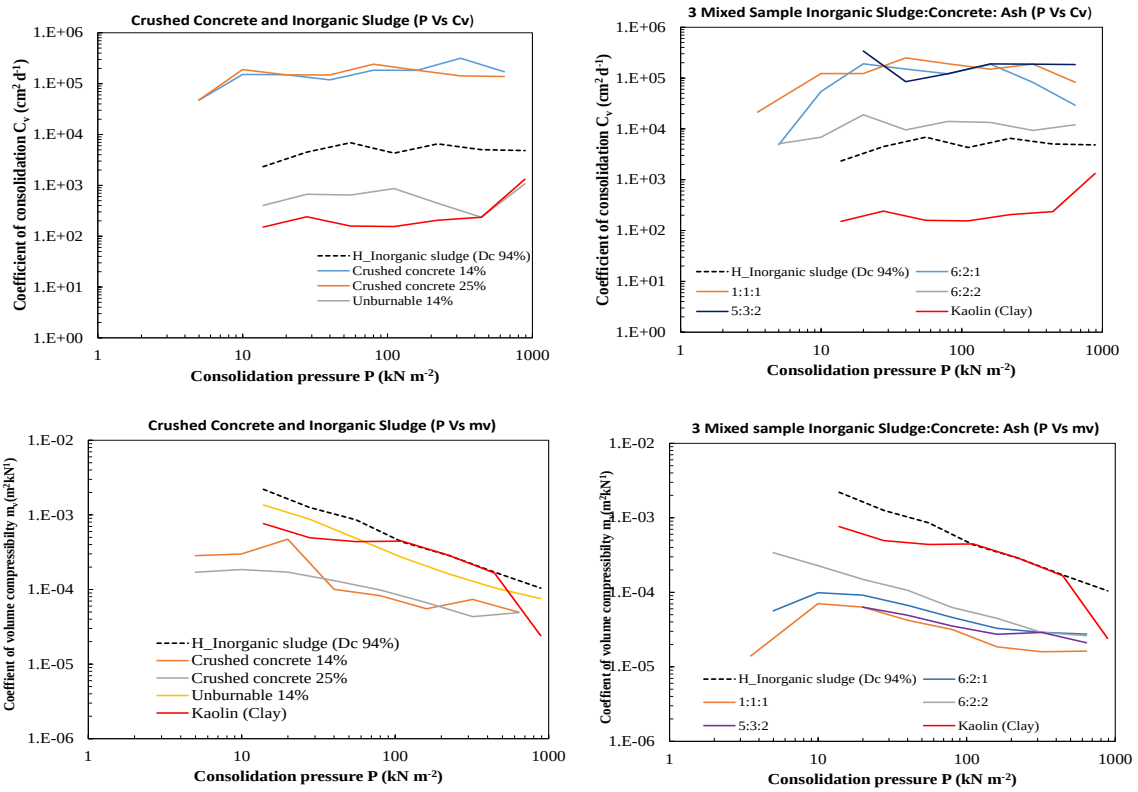


Figure 8. Coefficient of consolidation and volume compressibility

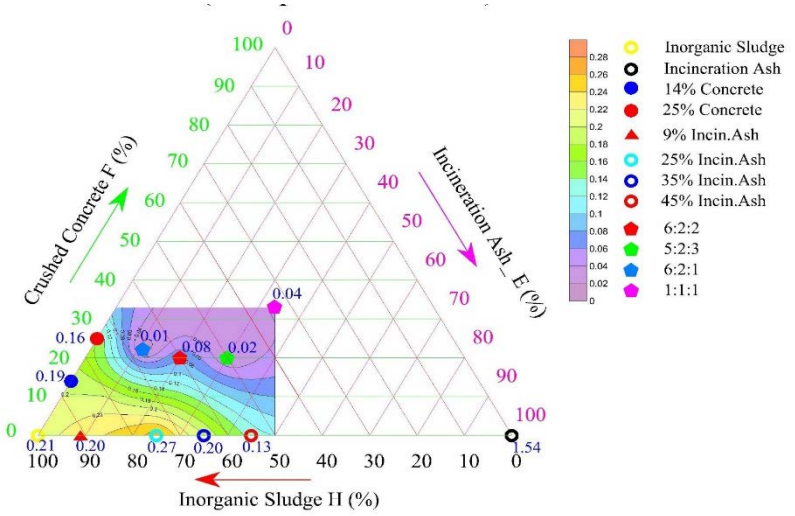


Figure 9. Contours plots for compression index, C_c, for tested samples

4. CONCLUSIONS

Compaction properties of organic and inorganic sludge improved by adding the other waste materials like concrete and incineration ash.

Regression analysis of the mixed samples showed that maximum dry bulk density having equal contribution from the crushed concrete and incineration ash as per their proportion.

Co-efficient of consolidation of inorganic sludge increases with increase of crushed concrete and ash proportions and coefficient of volume compressibility vice versa.

Compressibility of the inorganic sludge reduces significantly with the addition of crushed concrete as compared to incineration Ash.

Various mixed sample consolidation properties will be further investigated.

5. ACKNOWLEDGEMENT

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Measurements of pore Structure Parameters by Using Microfocus X-ray Computed Tomography for Variably Compacted Landfill Final Soil Cover

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Abstract: *Landfill site which accept unburnable organic waste is a significant emission source of greenhouse gases. Generally, to prevent rain/surface water infiltration into the waste layer, the final cover of landfill site is highly compacted. The compaction of final cover restricts the oxygen supply into the waste layer through the cover, causing methane generation under the anaerobic condition of waste layer. Numerous studies have been done to investigate the control of water infiltration through the final cover soil; however, there are few number of studies on gas transport through the final soil cover. Especially, studies on visualization and quantification of soil pore network for highly compacted soil like the final soil cover is highly limited. In this study, soil samples from a waste landfill site were compacted with different degree of compaction (i.e., different dry bulk densities) and were used for characterizing soil pore networking and structure using a microfocus X-ray Computed tomography (CT) system. The results showed that with increasing compaction, pore volume, effective pore radii, and tortuosity becomes smaller and coordination number becomes larger.*

Keywords: *Microfocus X- ray Computed Tomography (CT), Gas transport, Tortuosity, Soil pore network and structure*

1. INTRODUCTION

The final soil cover on a solid waste landfill consists of many layers of materials and is highly compacted. It is used to prevent rain/surface water infiltration in to the waste layer. Generally, final cover material consists of geo-membrane, less pervious soil or other relatively impervious materials. On the other hand, the landfill site has a significant emission source of greenhouse gases. Gas and mass transport in soils occur through the soil pore network, which is highly affected by soil physical properties including compaction, particle size, moisture content and total porosity. Several researchers including Lindquist et al. (1996, 2000), Luo et al. (2010) and Wildenschild and Sheppard (2012) used microfocus X-ray computed tomography (CT) for soils to characterize soil pore network characteristics with its functional structure. The application of X-ray CT to pore structure parameters for highly compacted soils like final soil cover has not yet been thoroughly studied. Recently, X-ray CT has emerged as a noninvasive technique for three dimensional examination of soil pore network. In recent years, different properties of soil pore and pore network have been investigated by using x-ray CT such as pore volume distribution (Perret et al., 1999), pore radii distribution (Lindquist et al.,2000), and tortuosity (Lindquist et al.,1996).

2. MATERIALS AND METHODS

In this study, soil samples were collected from Yorii landfill site in Saitama prefecture, Japan. Around 2.5 m thick soil layer above the waste layer as a final soil cover. Disturbed soil samples to depth -50cm were taken from the landfill final soil cover, and were sieved through a 2 mm mesh to separate gravel. The basic physical properties of landfill final cover soil are as shown in Table 1. Air-dried soil samples of particle size $d \leq$ 2mm were packed with different dry bulk densities such as 1.40, 1.55, and 1.65 $\mathrm{g/cm^3}$ by hand compaction and 1.68 $\mathrm{g/cm^3}$ by standard proctor compaction ($550\mathrm{KJ/m^3}$). The samples

with dry bulk densities of 1.40, 1.55, and 1.65 $\mathrm{g/cm^3}$ were directly packed in an acrylic core with dimensions of 5.61 cm diameter and 4.06 cm height. The sample with dry bulk density of 1.68 $\mathrm{g/cm^3}$ was taken from the proctor mould using the acrylic core after the proctor compaction.

Scanned images of soil pore structure were taken by a microfocus X-ray CT system (inspeXio SMX-90CT, Shimadzu Corporation, Japan). Then, by using software VGStudio MAX (Volume Graphics, Germany), they were reconstructed in 3-D images. Finally, using software of EXFact analysis (Nihon Visual Science, Inc.), they were analyzed to obtain pore structure parameters such as pore volume (*PV*), coordination number (*N*), effective pore radius (r_{eff}), and tortuosity (*T*). The ExFact analysis enable to analyze pore structure networks based on the 3-dimensional medial axis (3DMA) method (Lindquist et al. 1996). Figure 1 shows the schematic representation of pore unit in white color and grains in pink color. The coordination number is the number of pore throats that are connected to a pore body. The pore network has different pore coordination number. Niasar et al. (2010) explain that many pore network models have a coordination number of six, with pore throat oriented along the three principal directions of the lattice. He also suggests that geometry of the network is represented by shape and size of the bodies and pore throats. Tortuosity is a key parameter for predict the transport properties such as gas and mass transport. According to the classical definition of pore tortuosity is the ratio of average capillary tube length (L_e) to the length of porous media (*L*). In this sense, the tortuosity always greater than or equal to one. This definition may be valid only some geometrical assumption of the medium such as pores of porous media as uniform tortuous and non restricted tubes of similar diameter. Therefore, supplementary microscopic analysis X-ray CT is a valuable research tool. For determining suitable scanned images for pore structure and network, several scanning conditions for the microfocus X-ray CT system (in case of dry density 1.55 $\mathrm{gcm^{-3}}$) have been tested such as different combination of voxel size (10, 30 and 50 µm/pixel), scan number, view number, field of view (FOV), and region of interest (ROI). Voxel size is the resolution of CT image. It is the combination of volume and pixel. Average number specifies the average number of X-ray images acquired. Due to increasing the number of views will increase the time required for imaging but the CT image will have better resolution. FOV is the field of view. FOV (XY) and FOV (Z) is the field of view in XY cross section and vertical direction respectively. ROI is the range of analysis and taken from CT image. For analysis of maximum interior pore different region of interest were tested. Based on test results, voxel size of 30 µm/voxel with ROI of 150×150×500 voxels were used as the analysis condition with the maximum interior pore of 13.04 %. Other scanning conditions are summarized in Table 2. Scanned images for differently compacted samples are shown in Figure 3.

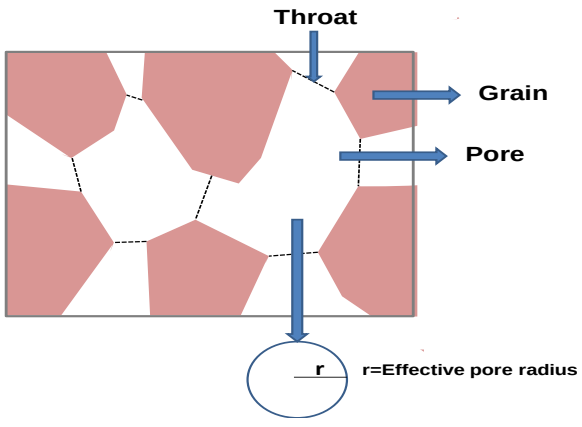


Figure 1 Schematic representation of pore unit in white color and grains in pink color.

Table1 Basic physical properties of landfill final cover soils (<2mm fraction).

Property	Sand (4.75-0.075mm)	Silt (0.075-0.005mm)	Clay (<0.005mm)	Particle density (gcm^{-3})
Value	42%	13%	9%	2.66

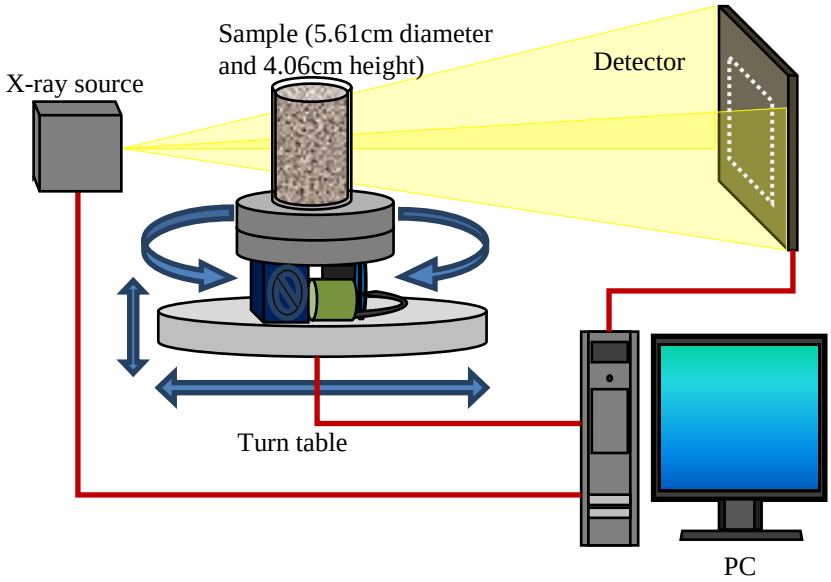
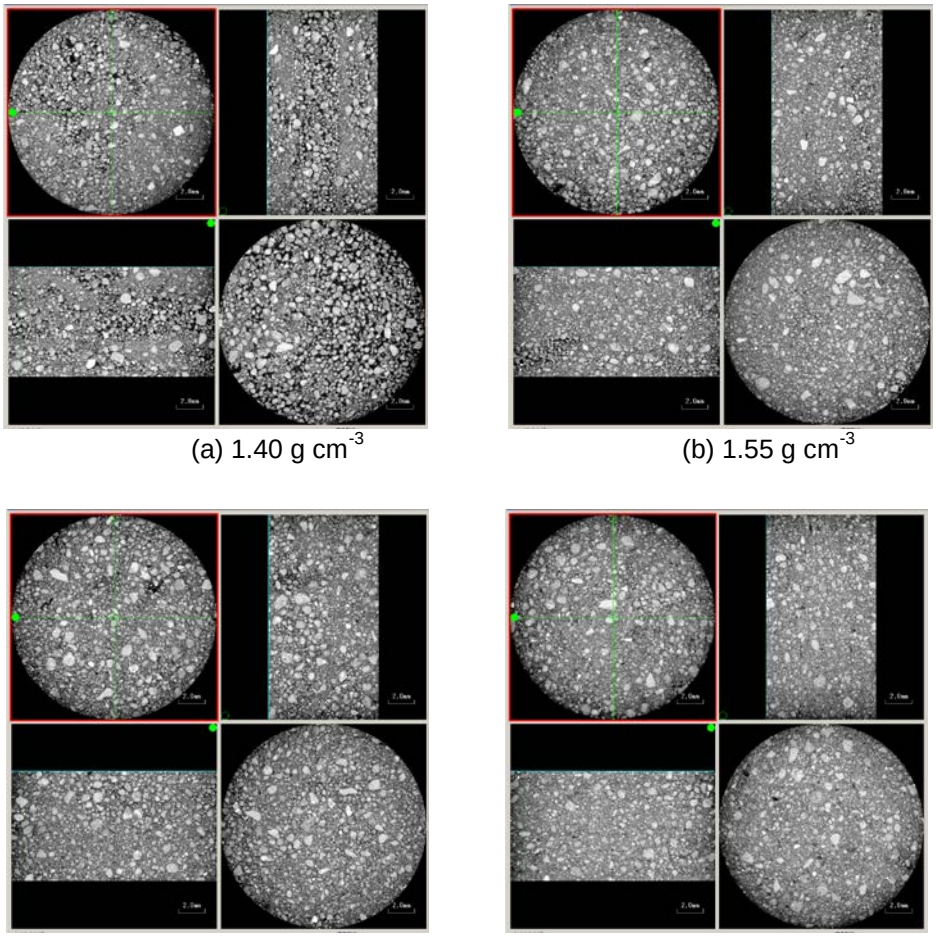


Figure 2 Schematic diagram of microfocus X-ray CT system

Table 2 Scanning and analysis conditions for the microfocus X-ray CT system.

SN	Item	Setting	Unit
1	Voxel size	30	μm/voxel
2	Average Number	4	
3	Scan Number	1	
4	View Number	3600	
5	FOV(Z)	16.4	mm
6	FOV(XY)	30.7	mm
7	Scaling Factor	100	
8	Image size	1024×1024	voxel
9	Number of Slices	548	
10	Region of interest(ROI)		
	X	150	voxel
	Y	150	voxel
	Z	500	voxel



(a) 1.40 g cm^{-3} (b) 1.55 g cm^{-3}

(c) 1.65 g cm^{-3} (d) 1.68 g cm^{-3}

Figure 3 Scanned images for differently-compacted samples

3. RESULTS AND DISCUSSION

Measured distributions for pore structure parameters with dry bulk density of 1.40 gcm^{-3} are exemplified in Figure 4. For each parameter, distributions from 3 ROIs were plotted and fitted under either normal or log scales. Pore volume (*PV*) and effective pore radius (r_{eff}) were fitted well with the log-normal distribution. Coordination number (*N*) and tortuosity in each direction (T_{xx}, T_{yy}, T_{zz}) were fitted well with normal distribution. Mean values (μ) and standard deviations (σ) for each dry bulk density condition were summarized in Table 3.

Table 3 Mean values (μ) and standard deviations (σ) for each dry bulk density condition.

Dry bulk density (g cm^{-3})	PV (mm^3)		r_{eff} (μm)		N		T_{xx}	
	μ	σ	μ	σ	μ	σ	μ	σ
1.40	6.05×10^{-3}	3.65	116.02	48.80	3.44	1.00	1.64	0.08
1.55	5.67×10^{-3}	3.43	110.15	44.65	3.60	1.13	1.60	0.06
1.65	5.03×10^{-3}	3.36	107.86	42.60	3.73	1.24	1.60	0.06
1.68	4.96×10^{-3}	3.38	106.71	41.78	3.73	1.24	1.58	0.06

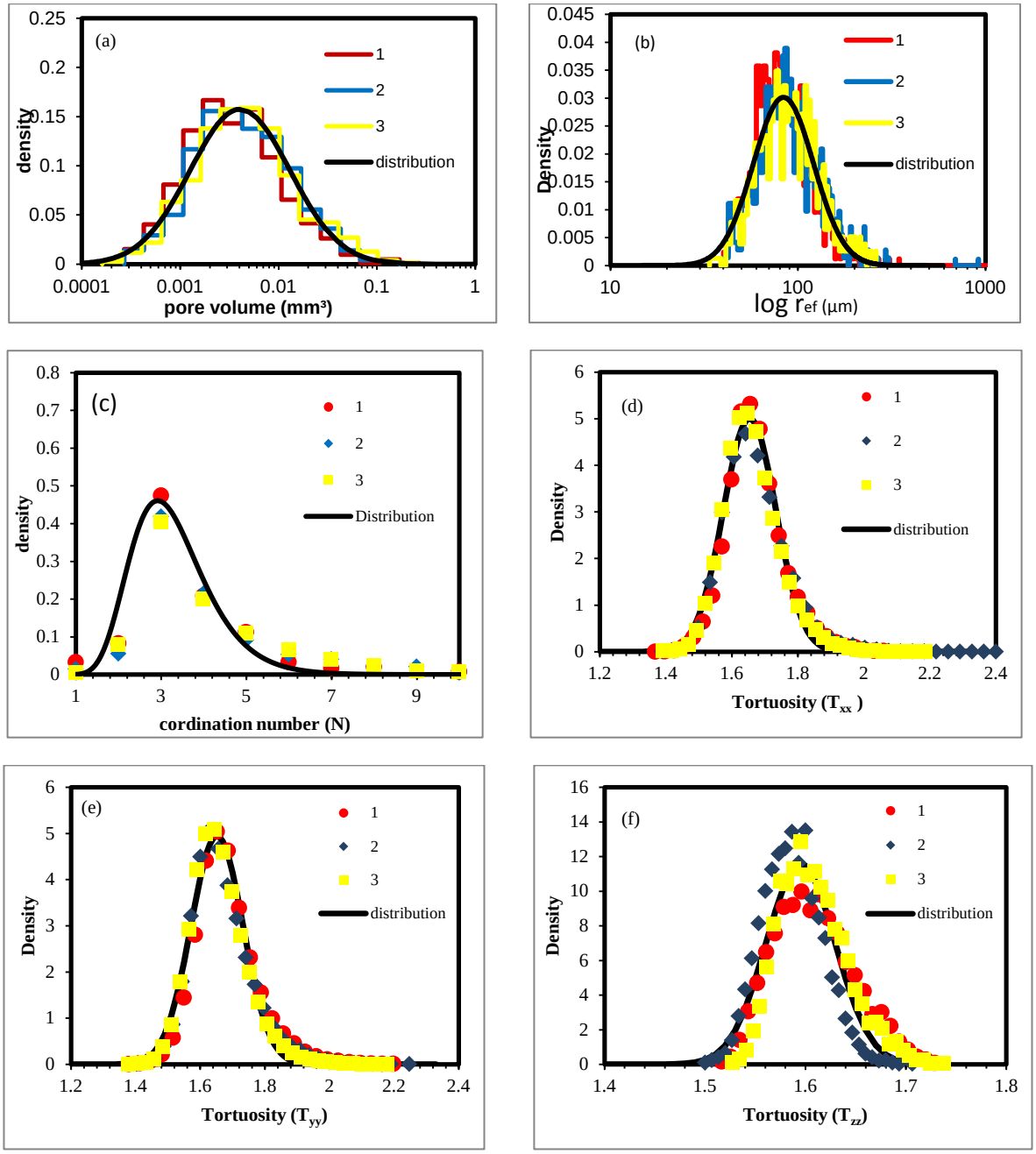


Figure 4 Distributions of pore structure parameters with dry bulk density of 1.40 gcm^{-3}. (a) Pore volume (PV), (b) Effective pore radius (r_{eff}), (c) Coordination number (N), (d) Tortuosity in XX direction (T_{xx}), (e) Tortuosity in YY direction (T_{yy}), and (f) Tortuosity in ZZ direction (T_{zz}).

Mean values for pore structure parameter from normal or log-normal distribution curves were plotted against the dry bulk density in Figure 5. With increasing in dry bulk density, pore volume, effective pore radius, and tortuosity in each direction decreased. On the other hand, the coordination number increased with increasing in dry bulk density. This is caused by compaction reduces pore size, with high bulk density throat is easy to be created because the distance between the soil particles become close and as a result coordination number increase and tortuosity decreased. According to the classical definition of pore tortuosity is the ratio of average capillary tube length (L_e) to the length of porous media (L). This is not necessarity property of the porous medium, but parameters may be valid only some geometrical assumption of the medium such as that of uni-dimensional capillary model. Therefore, supplementary microscopic analysis X-ray CT is a valuable research tool. To have quantitative understanding of gas transport process in the final cover soil, further studies will be

carried out to correlate the pore structure parameters to gas transport parameters such as soil-gas diffusion coefficient (D_p) and air permeability (k_a).

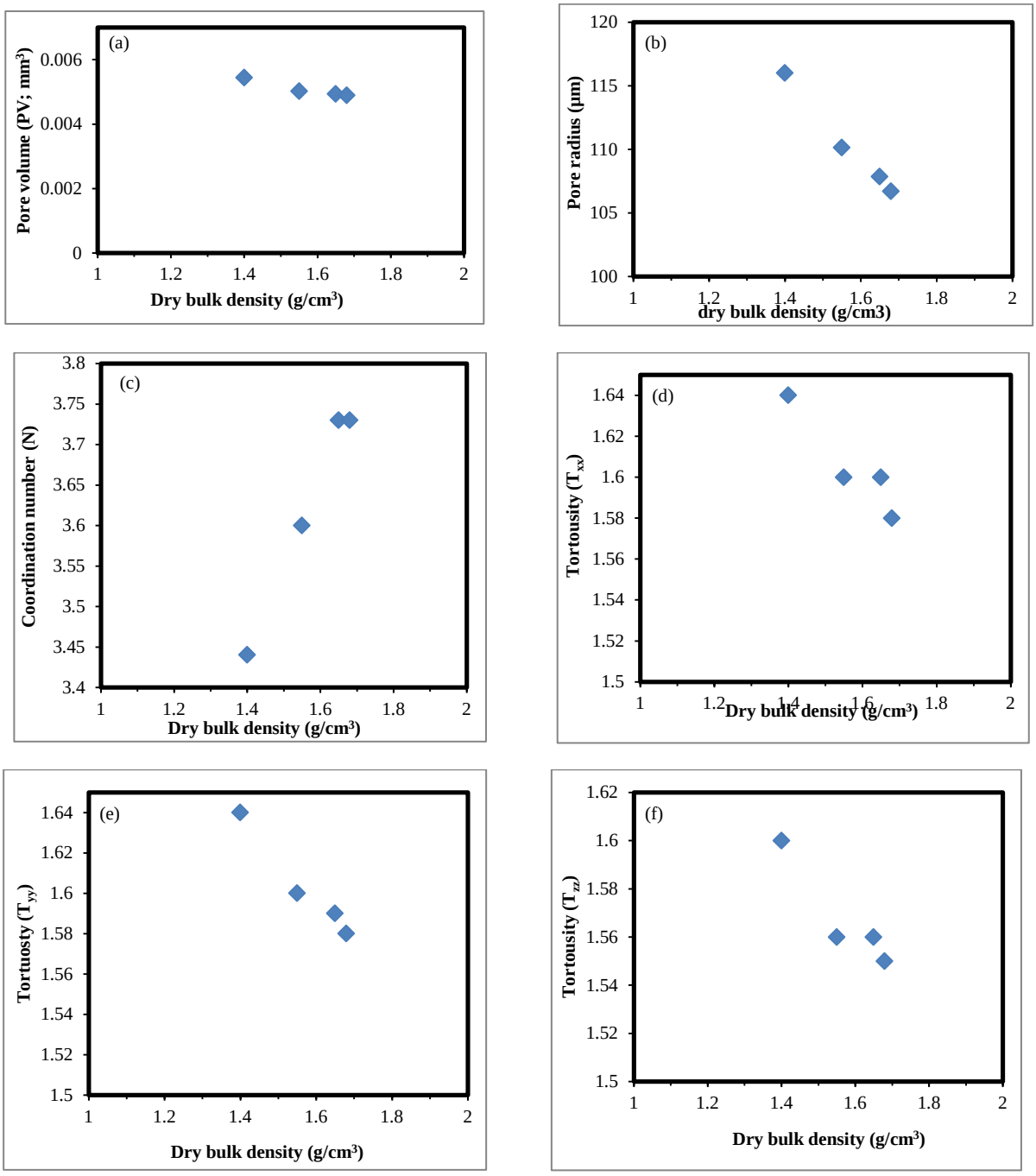


Figure 5 Pore structure parameters as a function of dry bulk density. (a) Pore volume (PV), (b) Coordination number (N), (c) Effective pore radius (r_{eff}), (d) Tortuosity in XX direction (T_{xx}). (e) Tortuosity in YY direction (T_{yy}), and (f) Tortuosity in ZZ direction (T_{zz}).

4. CONCLUSION

By using a microfocus X-ray CT system, the effects of compaction on pore structure parameters for landfill soil cover soil were examined. In this study pore structure parameters are highly dependent on compaction. Results showed that pore structure parameters such as pore volume, effective pore

radius, and tortuosity decreased and coordination number increased with increasing compaction. The calculation of pore structure parameters were calculated by really following the pore by using X-ray CT tool. However, in reality, the pore structure is complicated. Further studies are needed, but, these changes will affect gas transport parameters in the final soil cover.

5. ACKNOWLEDGEMENTS

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Barrier Performance of Geosynthetic Clay Liners under Elevated Thermo- Chemical Exposure

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Abstract: *This study investigated the combined effect of temperature and chemical exposure on the bentonite component of geosynthetic clay liner. Swell Index, liquid limit and hydraulic conductivity tests were carried out on bentonite using thirteen permeating liquids including DI water and salt solutions of NaCl, KCl, $CaCl_2$ and $MgCl_2$ with concentrations ranging from 0.01M to 1M. The tests were conducted at three different temperatures ranging from 20 ^oC to 60 ^oC. It was observed that concentration and type of cation have significant effects on the barrier performance of bentonite. Temperature was found to have little effect on the swell index and liquid limit of bentonite. However, hydraulic conductivity was found to increase significantly with increase in temperature. Hydraulic conductivity was correlated to swell and plasticity properties of bentonite at elevated temperatures and results are presented in the paper.*

Keywords: *Bentonite, Thermo-Chemical Exposure, Swell Index, Liquid Limit, Hydraulic Conductivity.*

1. INTRODUCTION

Geosynthetic clay liners (GCLs) are being used increasingly as barrier materials in bottom liner system of municipal solid waste landfills. GCLs consist of a thin layer of bentonite sandwiched between two geotextiles or glued to a geomemberane (Jo et. al 2001). The expected barrier performance of GCLs mainly relies on the performance of its bentonite component. Bentonite has the ability to swell on contact with water resulting in much lower values of hydraulic conductivity. However, when waste is placed in the landfill, its biological degradation produces chemicals that are dissolved with waste moisture and percolating rain water, generating landfill leachate. Further, biodegradation of waste also generates significant amount of heat. A rise in temperature up to 60 ^oC inside landfills near the bottom liner is reported by a number of researchers (Collins, 1993; Koerner and Koerner, 2006; Rowe, 2005; Yessiller et al., 2005; Yoshida et al., 1996). Therefore, it is important to understand and document the barrier performance of liner materials under the combined effects of leachate and temperature. However, most of the present studies focus on the effect of chemicals on geotechnical and hydraulic properties of bentonite at room temperature without considering the effect of temperature rise inside the landfill (di Maco, 1996; Ruhl and Daniel, 1997; Schackelford et al., 2000; Jo et al., 2001, Shan and Lai, 2002; Kolsad et al., 2004; Bouazza et al., 2006; Katsumi et al., 2007; Kaufhold and Dohrmann, 2009). A very limited number of studies have investigated the effect of chemicals on barrier performance of bentonite at elevated temperature conditions. Ishimori and Katsumi (2012) have investigated the effects of temperatures (20 ^oC and 60 ^oC) on the swelling capacity and barrier performance of bentonite using deionized water (DI water) and different concentrations of NaCl solutions. They found that swell and intrinsic permeability increased with an increase in temperature. Further, they found that the existing relationships obtained from free swell and hydraulic conductivity tests at room temperature are not applicable at elevated temperatures. Other studies focused on hydraulic conductivity (Cho and Lee, 1999) and hydro-mechanical behaviour (Villar and Lolret, 2003). These studies used only water as permeating liquid for specific applications in high level nuclear

waste repertories. Hence, further investigation is needed to understand the behaviour of GCLs under elevated thermo-chemical environment during the required long term performance of landfills.

This study employed single-species salt solutions to investigate the combined effects of elevated temperatures and chemicals on swell index, liquid limit and hydraulic conductivity properties of bentonite. Salt solutions of four major exchangeable cations in bentonite (Na^+, K^+, Ca^{2+}, Mg^{2+}), each at three different concentrations (0.01M, 0.1 M, 1M)) were used as hydrating liquids in addition to deionized water (DI water). The swell index, liquid limit and hydraulic conductivity tests were carried out at three different temperatures (20 ^oC, 40 ^oC and 60 ^oC). Swell Index and liquid limit tests were carried out using all the thirteen hydrating liquids. The hydraulic conductivity was evaluated using DI water and intermediate concentrations (0.1M) of all the four cation solutions.

2. THERMO-CHEMICAL DEPENDENT BEHAVIOUR OF BENTONITE

Bentonite is widely known as barrier material due its unique swelling and hydraulic properties. Both of these properties are seemed to be highly dependent on the chemistry and temperature of hydrating liquid.

The swelling of bentonite is classified as a two phase process (Likos and Bowders, 2010). The first phase, known as “crystalline swelling” occurs at small scale and a maximum of three to four layers of water molecules can be adsorbed in the interlayer around the interlayer cations which corresponds to less than 25% by mass. This phase is then followed by “osmotic swelling” where numerous layers of water molecules enter the interlayer. The osmotic phase can occur only when the exchange sites are occupied by monovalent cations (Norish and Quirk, 1954; Jellander and Mareelja, 1988) having Δradius (hydrated radius - ionic radius) to valance ratio greater than 300 and more than three hydration shells (Likos and Bowders, 2010). The osmotic phase is mainly associated with the thickness of diffused double layer (DDL) of bentonite particles given by the following relationship (Mitchell, 1993).

$$\frac{1}{K} = \sqrt{\frac{\varepsilon_o D k T}{2 n_o e^2 v^2}} \qquad (1)$$

Where, 1/K is the thickness of diffused double layer, ε_o is the vaccume permittivity (8.86×10^{-12} $C^2J^{-1}m^{-1}$), k is boltzmaz constant (1.38×10^{-23}), D is dielectric constant of pore fluid (D_{H2O}), T is absolute temperature, n_o is far-field ion concentration (ions/m^3), e is electric charge and v is the valance. The above equation implies that change is the thickness of diffused double layer is directly affected by the chemistry of pore fluid. An increase in ion concentration will tend to decrease the thickness of DDL and hence swelling will decrease, and vice versa. A decrease in the thickness of DDL is mainly attributed to the movement of bound water molecules that allow more space for water flow through the interlayer region causing an increase in hydraulic conductivity. The effect of temperature on the DDL thickness is slight (Mitchell, 1993) but how this slight change affects the swelling of bentonite is not clarified yet (Ishimori and Katsumi, 2012).

3. MATERIALS

3.1. Bentonite

The bentonite used in the study was sodium bentonite extracted in which powdered form from a commercially available GCL (Bentofix®) where it is sandwiched between a polypropylene woven geotextile and a polypropylene non woven geotextile. Bentonite was passed through US sieve no. 200 (75 µm) and then oven dried at 105 ^oC for subsequent testing. The particle density of bentonite for this typical GCL is 2.61 g/cm^3 (Rowe and Lake, 2000). The liquid limit is 559% and the specific surface area using BET technique is 21.081 m^2/g. Table 1 summarizes the physical properties of bentonite.

3.2. Permeating Liquids

The permeating liquids used for the swell index and liquid limit tests were DI water and the aqueous solutions of NaCl, KCl, $CaCl_2$ and $MgCl_2$. Concentrations of solutions varied from 0.01 to 1 M. Hydraulic conductivity tests were carried out using DI water, 0.01M NaCl and intermediate concentrations (0.1M) of all the four cation solutions. The tests were carried out at three different temperatures (20 ^oC, 40 ^oC and 60 ^oC). Chemicals prepared by Wako Pure Chemical Industries, Ltd. were purchased in powdered form and their formula weights were mixed with DI water to prepare the desired chemical solutions.

Table 1 Fundamental Characteristics of Bentonite

Property	Unit	Value
Natural water content	%	11.78
Liquid Limit	%	559
Plastic Limit	%	51
Plasticity Index	%	508
Swell Index	mL/2g	30
Hydraulic Conductivity	m/sec	1.95×10^{-10}
Particle density	g/cm^3	2.61
Specific Surface Area	m^2/g	21.081

3.3. Permeating Liquids

The permeating liquids used for the swell index and liquid limit tests were DI water and the aqueous solutions of NaCl, KCl, $CaCl_2$ and $MgCl_2$. Concentrations of solutions varied from 0.01 to 1 M. Hydraulic conductivity tests were carried out using DI water, 0.01M NaCl and intermediate concentrations (0.1M) of all the four cation solutions. The tests were carried out at three different temperatures (20 ^oC, 40 ^oC and 60 ^oC). Chemicals prepared by Wako Pure Chemical Industries, Ltd. were purchased in powdered form and their formula weights were mixed with DI water to prepare the desired chemical solutions.

4. TEST EQUIPMENTS AND EXPERIMENTAL PROCEDURE

4.1. Swell Index Tests

The swell index tests were carried out in accordance with ASTM D5890. Initially, a 100 mL graduated cylinder was filled up to 90 mL mark with the hydrating solution. Then, 2g of oven dried bentonite was dusted in approximately 0.1 g increments. After the dusting of bentonite was completed, the cylinder was filled up to the 100 mL mark with the hydrating solution washing the particles adhering to the walls of the cylinder. The cylinder was then covered with plastic wrap to prevent evaporation and carefully placed under the desired temperature environment. The cylinders were kept undisturbed for 24 h prior to taking the readings. The measurements were repeated in triplicates and the averages were used for subsequent data analysis.

4.2. Liquid Limit Tests

The liquid limit tests were carried out using the multipoint test method in accordance with ASTM D4318-10. Firstly, samples were prepared by thoroughly mixing the bentonite with hydrating liquid at different moisture contents. The samples were then allowed to hydrate for at least 24 hours in double wrapped sealed plastic bags in the specified temperature environment. To avoid evaporation during the hydration process, wet tissue papers were kept between both wrappings. The samples were remixed before starting the experiment. A minimum of five trials were performed for each test and the liquid limit was determined by using the equation of the flow curve.

4.3. Hydraulic Conductivity Tests

The hydraulic conductivity tests were conducted using a constant head rigid wall permeameter as shown in Figure 1. The apparatus consisted of a lower ring, a middle ring and an upper ring placed between an upper and lower cap all made of acrylic. First, the bentonite samples were prepared with the permeating solution with a moisture content equal to its liquid limit for that particular solution, and allowed to hydrate for at least 24 h in sealed plastic bags. After 24 h, the prehydrated bentonite samples were placed in the circular middle ring with a porous filter at the bottom placed on the top of lower ring. Typically the inner diameter of the middle ring was 6 cm with a height of 1 cm, which represents the dimensions of the sample.

The lower ring was first filled with permeating solution, then sample was placed in the middle ring by slight tamping, making due care to prevent entrapment of air bubbles inside the sample. A porous filter was then placed on top of the sample in such a way that the sample was sandwiched between the lower and upper porous filters to prevent washing off of the soil particles during the tests. After placing the sample and porous filters, the upper ring was placed on top of the middle ring and the cell was clamped after placing the top cap. To avoid leakage of water at the contact points, silicon grease was used.

The permeant solution was then supplied from a Mariotte tank to the sample through an inlet valve of the lower cell allowing air bubbles to escape. Once the permeating solution started coming out of the sample ensuring that all air bubbles have escaped, the upper ring was then filled with the permeating solution up to a height equal to that of the lower ring and the whole cell was made airtight. After that, the whole setup was placed in the climate controlled room with the temperature maintained at 20 ^{o}C or in an incubator with a pre-set temperature of 40 ^{o}C or 60 ^{o}C. Permeant solution was then supplied to the sample from the Mariotte tank through an inlet valve of the lower ring while maintaining a constant head of 30 cm across the sample which correspond the hydraulic gradient. The effluent was collected from an outlet valve of the upper ring in a collecting jar at specified intervals of time. To measure the evaporation, a reference jar with an opening equal to the colleting jar was placed near the apparatus in the same environment and the evaporated volume was added to the measured volume to minimize the effect of evaporation during the test. The tests were continued for several days and hydraulic conductivity was calculated using Darcy's law.

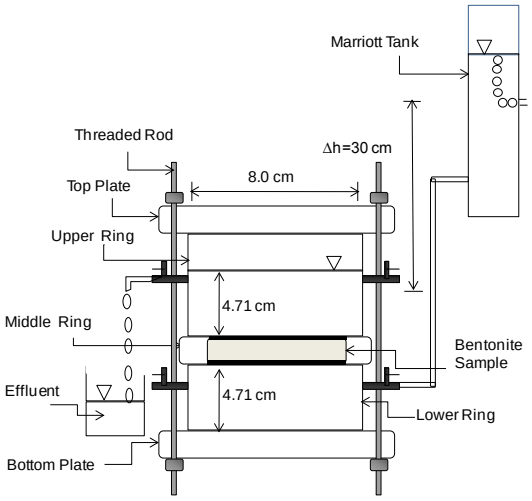

Figure 1 Schematic of hydraulic conductivity test apparatus

5. RESULTS AND DISCUSSIONS

5.1. Swell Index Tests

The swell index tests were carried out using all thirteen hydrating liquids. It was observed that the swell index decreased with an increase in concentration as shown in Figure 2. The swell index was

found to be highly controlled by the concentration of the solution in bulk pore water. This finding is consistent with the studies by Jo et al. 2000 who explained the phenomenon as a result of the release of interlayer water due to a gradient in free energy caused by the elevated concentrations in the bulk pore water. The highest value of the swell index was observed for the bentonite samples hydrated with NaCl solution at low concentration (0.01M) followed by the samples hydrated with DI water. As the bentonite is hydrated with NaCl solution, all exchange sites are occupied by Na^+ cation which take part in the osmotic swelling phase resulting an increase in swell volume. All other cations were found to give similar values of swell index at all concentrations with the lowest value of the swell index obtained for the bentonite samples hydrated with high concentration of KCl solution.

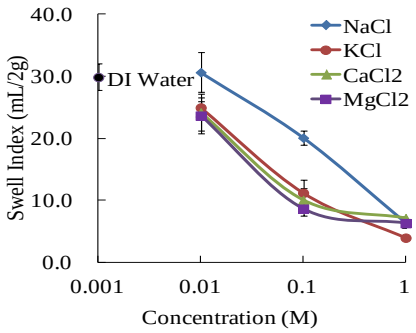

Figure 2 Effect of hydrating liquid on swell index of bentonite

Temperature of the hydrating liquid was found to have significant effect on bentonite samples hydrated with lower concentrations of monovalent cation solutions as shown in Figure 3 (a) and Figure 3 (b). At intermediate and high concentrations, however, the effect was insignificant. It was observed that for bentonite samples hydrated with DI water and low concentrations of monovalent cations, the swell index increased with an increase in temperature. Contrary to this, there was a tendency of a decrease in the swell index with an increase in temperature for divalent cations (Figure 3 (c) and Figure 3 (d)).

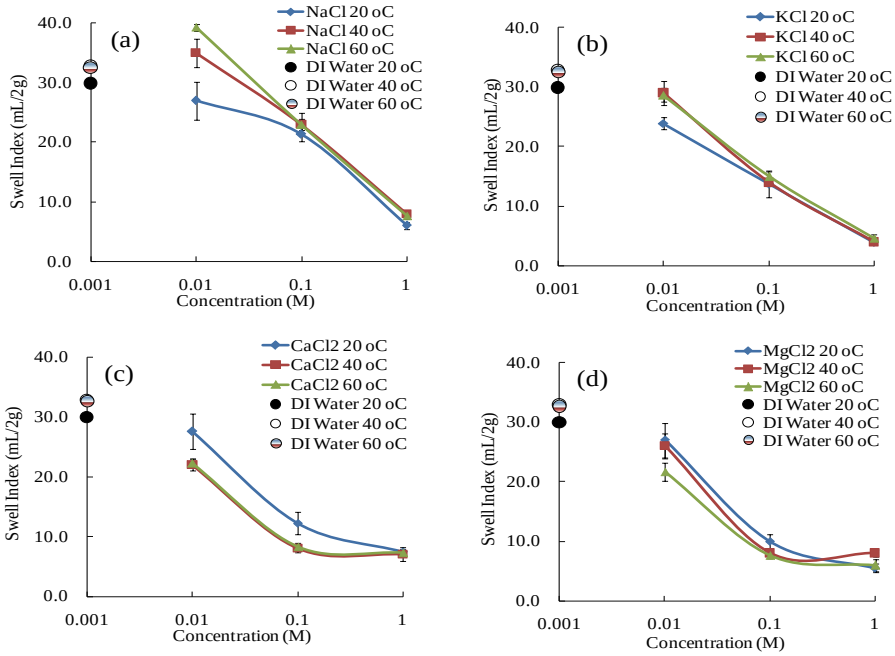

Figure 3 Effects of temperature on bentonite swell index

5.2. Liquid Limit Tests

Figure 4 shows the effect of liquid chemistry on the liquid limit of bentonite. It was observed that the liquid limit decreased with an increase in salt concentration irrespectively of the type of cation. The liquid limit was highest (559 %) with DI water.

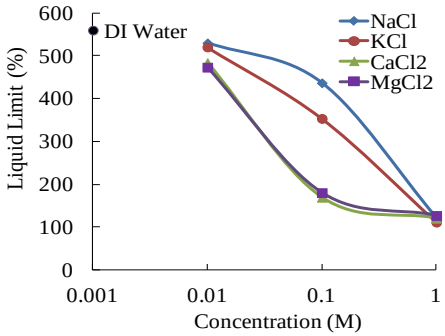

Figure 4 Effect of hydrating liquid on liquid limit of bentonite

At very low (0.01M) and very high (1M) concentrations, the liquid limit was found to be similar irrespective of the nature of the hydrating liquid. However, at the intermediate concentration (0.1M), both monovalent cations show higher than divalent cations. Both divalent cations showed similar values of the liquid limit at all concentrations.

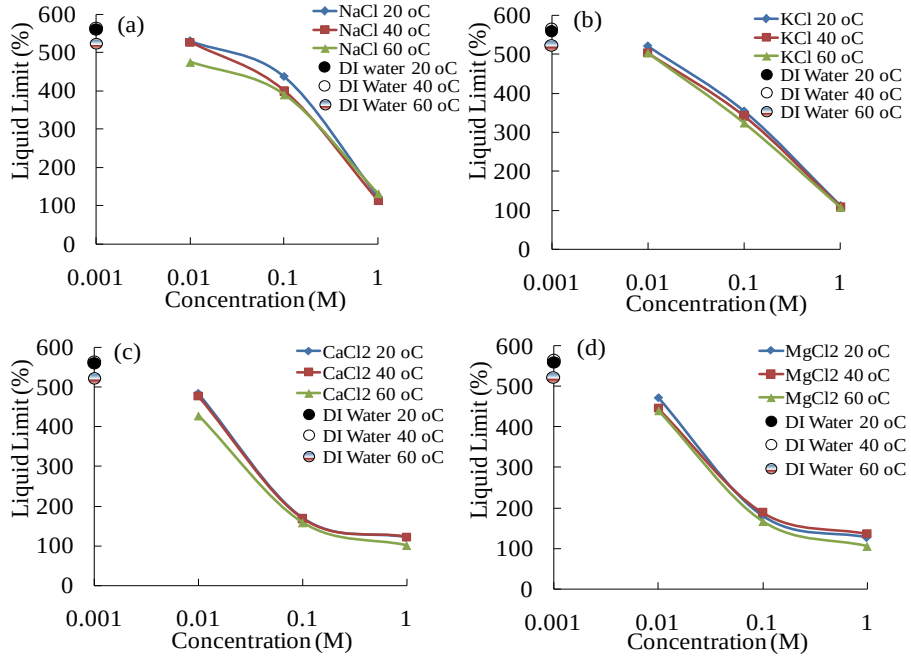

Figure 5 Effect of temperature on liquid limit of bentonite

The effect of temperature on the liquid limit of bentonite is shown in Figure 5. In general, the liquid limit was found to decrease slightly with an increase in temperature. DI water showed the highest value of liquid limit (559%) when hydrated at 20 ^{o}C and lowest at 60 ^{o}C (524%). A similar trend of decrease in liquid limit with increase in temperature was found for bentonite hydrated with NaCl solution at low and intermediate concentrations. However, at high concentrations, the effect of temperature was not significant. Temperature was found to have insignificant effect on the liquid limit of bentonite hydrated with KCl and divalent cation solutions ($CaCl_2$ and $MgCl_2$).

5.3. Correlation between Swell Index and Liquid Limit

A similar behaviour of swell index and liquid limit to hydrating liquid suggested that there exist a correlation between both properties. Significant and high correlations were found to exist between swell index and the liquid limit at all temperatures (Figure 6 (a) – Figure 6 (c)).

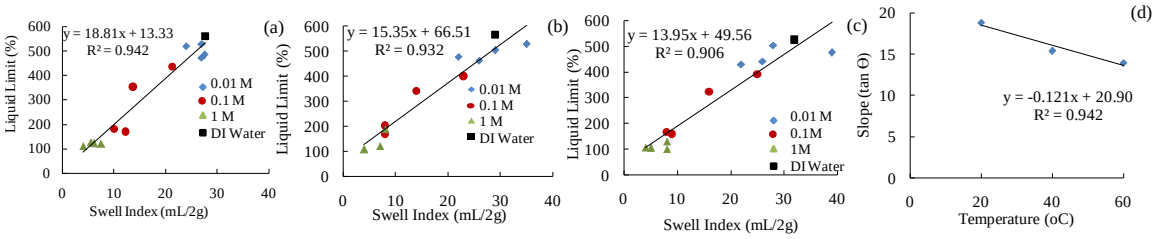

Figure 6 Correlation between swell index and liquid limit

Figure 6 (d) shows the relationship between slopes of the regression lines with temperature. It is seen that the slope decreases with increase in temperature which confirms a temperature dependence on both geotechnical properties.

6. HYDRAULIC CONDUCTIVITY

Hydraulic conductivity tests were carried out using DI water and intermediate concentrations of all four cations at the three temperatures. The tests were conducted on prehydrated and non-consolidated bentonite samples. The duration of each test (t), total pore volumes of flow (PVF), total porosity (ϕ) void ratio (e) and average hydraulic conductivity (k) of each permeated sample are shown in Table 2. It was observed that for all type of cations hydraulic conductivity do not change significantly with time as shown in Figure 7. In general, hydraulic conductivity was found to increase with increase in temperature.

Table 2 Summary of hydraulic conductivity tests

Permeant Liquid	Conc. (M)	20 °C					40 °C					60 °C				
		t (days)	PVF	ϕ	e	k (cm/sec)	t (days)	PVF	ϕ	e	k (m/sec)	t (days)	PVF	ϕ	e	k (m/sec)
NaCl	0.1	109	8.9	0.91	10.17	3.33×10^{-10}	113	25.67	0.92	11.15	1.05×10^{-09}	65	27.8	0.86	6.26	1.82×10^{-09}
KCl	0.1	78	41	0.90	9.93	2.12×10^{-09}	108	64.5	0.90	9.25	2.91×10^{-09}	58	13.01	0.86	6.04	8.74×10^{-10}
$CaCl_2$	0.1	111	71.8	0.81	4.42	2.48×10^{-09}	116	126.68	0.81	4.18	4.81×10^{-09}	61	67.19	0.90	9.47	4.96×10^{-09}
$MgCl_2$	0.1	89	29	0.82	4.79	1.06×10^{-09}	90	167.05	0.81	4.39	8.53×10^{-09}	61	80.02	0.81	4.39	5.66×10^{-09}
DI water	-	108	4.8	0.94	15.44	1.94×10^{-09}	122	11.24	0.93	13.82	3.95×10^{-09}	68	15.68	0.94	15.66	1.06×10^{-09}

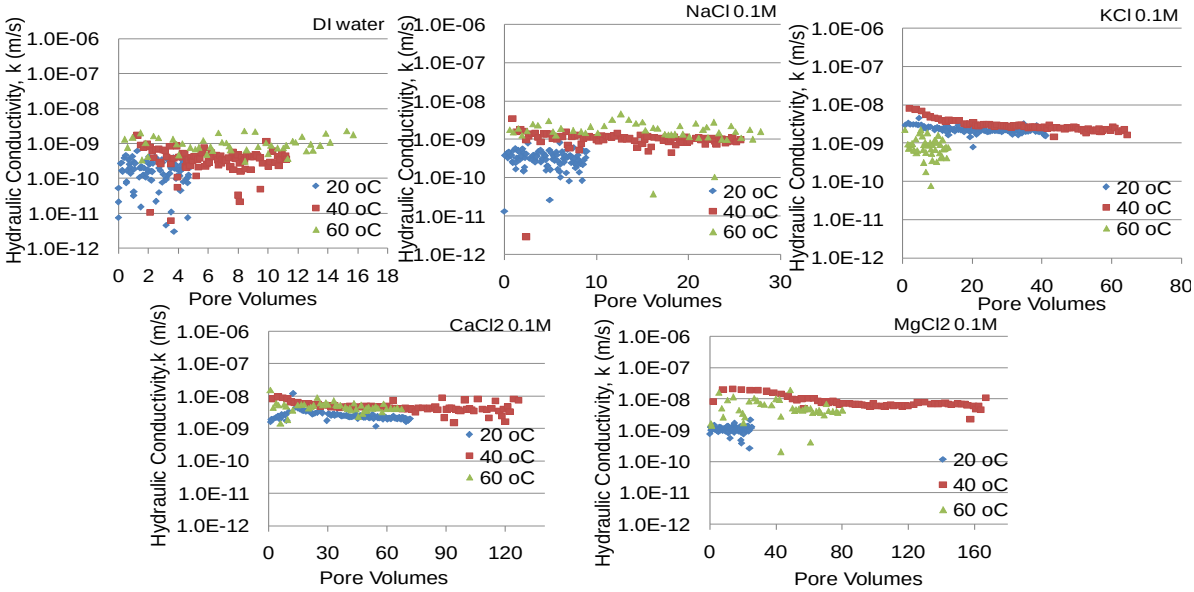

Figure 7 Effect of temperature on hydraulic conductivity of bentonite

Figure 8 shows the typical curve for accumulated flow volume for hydraulic conductivity tests. It is observed that flow volume is highly dependent on temperature. For the given permeation duration, a

higher temperature yields more flow volume and hence more pore volumes of flow PVFs are obtained for samples hydrated at elevated temperatures than at room temperature as shown in Table 2.

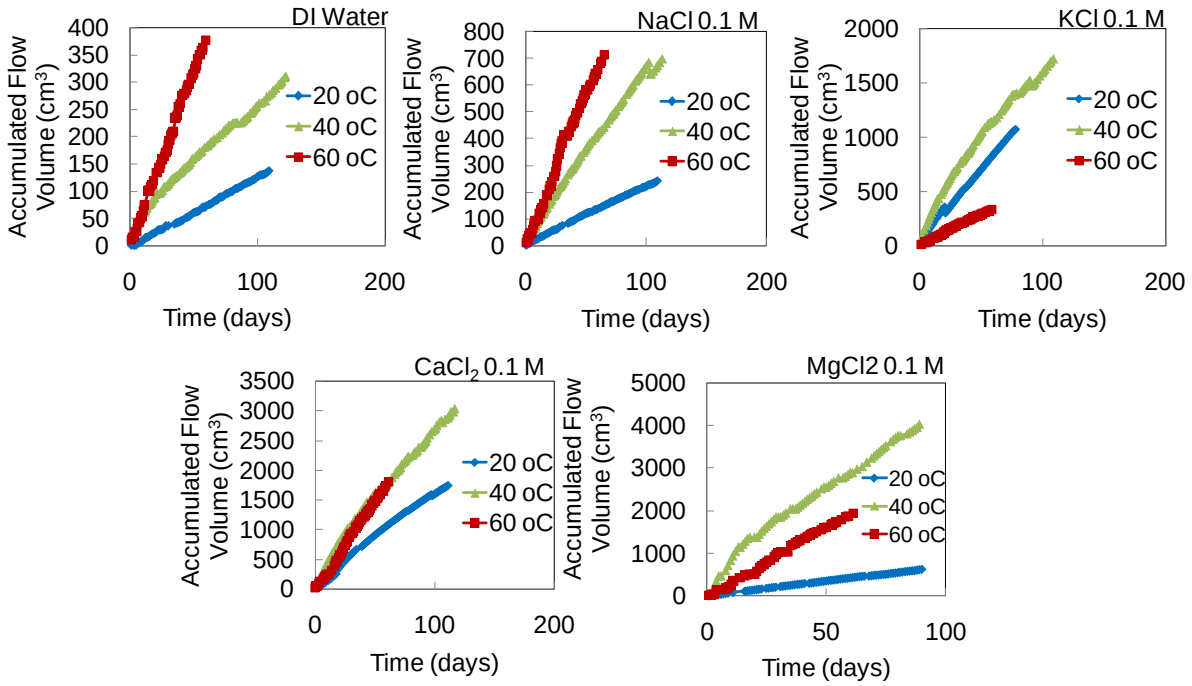

Figure 8 Effect of temperature on accumulated flow volume

6.1.1. Correlation between Swell Index and Hydraulic Conductivity

Figure 9 shows the correlation between swell index and normalized hydraulic conductivity of bentonite at all the given temperatures. Hydraulic conductivity, k is normalized by dividing it with hydraulic conductivity of bentonite permeated with DI water $K_{DI.}$

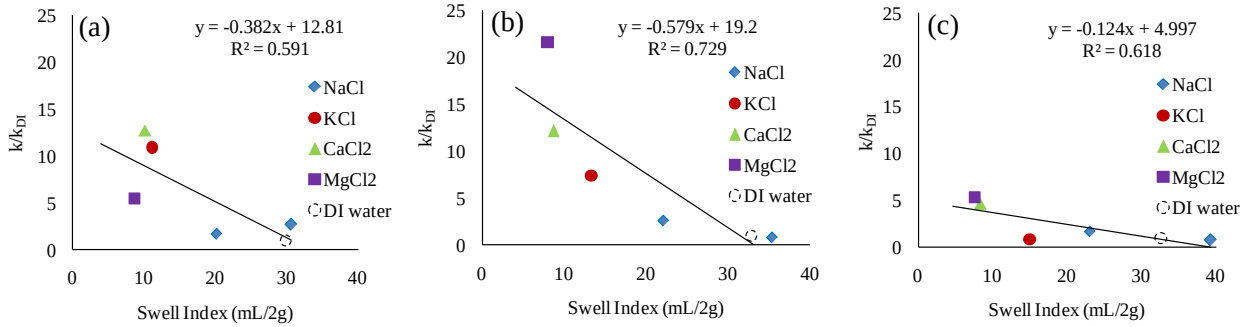

Figure 9 Correlation between hydraulic conductivity and swell index of bentonite at temperatures (a) 20 °C, (b) 40 °C and (c) 60 °C

7. CONCLUSION

This study investigated the combined effect of temperature and liquid chemistry on swell index, liquid limit and hydraulic conductivity of bentonite. Chemistry of hydrating liquid was found to have significant effect on swell index and liquid limit of bentonite at all the temperatures. Liquid limit was found to decrease whereas swell index and hydraulic conductivity were found to increase with increasing temperature irrespective of the nature of hydrating liquid. Temperature was found to have significant effect on hydraulic conductivity of bentonite. Significant and high correlations were found between all the three properties of bentonite at all the given temperatures.

8. ACKNOWLEDGEMENTS

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Long Term Settlement Behaviour of Dump Wastes in Sri Lanka; Case Study on Bloemendhal Dump Site

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Abstract: *Open dumping has been widely used to dump wastes in developing countries. In Sri Lanka, most of the dumpsites are open dumpsites. Due to high demand for lands, previously abandoned lands are now utilized for construction activities. However, due to heterogeneity and variability, construction or performance on these type of lands requires more studies. Therefore, settlement characteristics of dump waste are needed to be studied. This research paper discusses about long-term settlement behaviour of dump waste. Secondary compression is due to creep of the waste skeleton and biological decay. In general, secondary compression accounts a significant part of total settlement of dump area and it may occur over many years. This research was carried out to find settlement parameters of Bloemendhal dumpsite. According to the Unified Classification System, this sample was classified as well graded sand with silt and gravel (SM-SM). Standard Proctor compaction test reveals that, the sample has maximum dry density of 1418 kg/m^3, which is lower value compared to a traditional lateritic soil in Sri Lanka. Optimum moisture content is 24.5%. Primary consolidation parameters were evaluated through laboratory experiments. Sample has the coefficient of consolidation in the range of 1 - 4 m^2/year and coefficient of compressibility in the range of 0.05 – 0.1. The sample has lower secondary consolidation creep index, which is in the order of 10^{-3}. However, it is strongly suggested to study the long term biodegradation studies on fresh dumpsite samples.*

Keywords: *long-term settlement, secondary compression index, dump waste, creep of dump waste*

1. INTRODUCTION

Sri Lanka is a rapidly developing country with limited resource of land. It requires utilizing the lands, which were abandoned previously or lower lands in city areas. In city of Colombo, the wastage has been dumped in sites like Bloemendhal, Madhampitiya, Kolanawa etc.

Dumped waste decays with time and it becomes similar to a soil. It may also be used as a fill material. Construction on such lands may have issues due to its nature. Settlement, slope instability, shear failures and adverse chemical reactions are some potential issues, which are needed to be studied.

In this research, long-term settlement behaviour of dump waste is studied. There are three main stages of settlement in waste soil such as initial consolidation, primary consolidation and secondary settlement. Initial settlement occurs immediately when an external load is applied. It is generally associated with the immediate compaction of void space and particles due to a superimposed load. Primary consolidation is due to dissipation of pore water pressure and air from the void spaces. Secondary compression is due to creep of the waste skeleton and biological decay. In general, secondary compression accounts significant part of total settlement of dump area and occurs over many years (El-Fadel and Khoury, 2000). In this study, primary consolidation parameters and secondary compression creep parameters of Bloemendhal dumpsite are experimentally evaluated.

2. SETTLEMENT MODELS

Soil mechanics-based model, empirical models, rheological model and settlement model that incorporate biodegradation are four types of model to analyse the settlement (NurIrfah, 2010). Under the above categories, number of models was developed. However, most of them are usually defined for municipal solid waste or landfill which has proper leachate and gas circulation.

In Sri Lanka, most of them are open dumps, which do not have proper engineering applications to minimize adverse impacts of dumps. Therefore, following models, which could be used for both landfill and open dump waste, are considered (NurIrfah, 2010). Basic soil mechanics model of consolidation to estimate dump waste is Sowers model (Sowers, 1973), where the long-term consolidation associated with creep and biodegradation is expressed in terms of the secondary consolidation index. Bjarngard and Edgers model (Bjarngard and Edgers, 1990) subdivides the secondary compression into two sub phases, through the adjustment of two straight lines. The power creep model (Edil et al, 1990) is a simple relation for time dependent deformation under constant stress. Hyperbolic equation (Ling et al, 1998) is to compute settlement at a given time for ultimate settlement of the landfill.

3. EXPERIMENTAL WORK

To meet the objectives of this research laboratory experiments are required. A series of experiments were conducted according to appropriate guidelines such as BS 1377 and ASTM volume 04.08. Classification of sample and studying the compressibility characteristics are two identified objectives.

Classification of the sample was done according to Unified Soil Classification System (USCS). Chemical test such as pH value, chloride content and sulphite content were found to identify the impacts of the soil to structures and people who they would be working with it.

Proctor compaction test was done to find out the relationship between moisture content and dry unit weight of sample. Then consolidation tests were done to simulate the compaction that would be done or required when using the sample as a fill material. The secondary consolidation parameters were found from oedometer. Both primary and secondary consolidation parameters were derived from these consolidation tests.

3.1. Classification of Sample

It is reasonable to expect different particles such as organic components, metal particles, glass particles etc in the sample. Since biodegradation of organic particles have a significant impact on long-term settlement of dump waste, it is necessary to study about availability of organic components in sample.

It was also decided to perform both air and oven drying in the classification process. In oven drying process, some particles such as organic particles could be burnt Particle size distributions were performed for both type of samples. In addition, the particle size distribution was done in both wet and dry sieve methods. Therefore, altogether four different types of procedures were adopted in particles size distribution and Atterberg limit tests. Particle size distribution of four cases is shown in Fig.1.

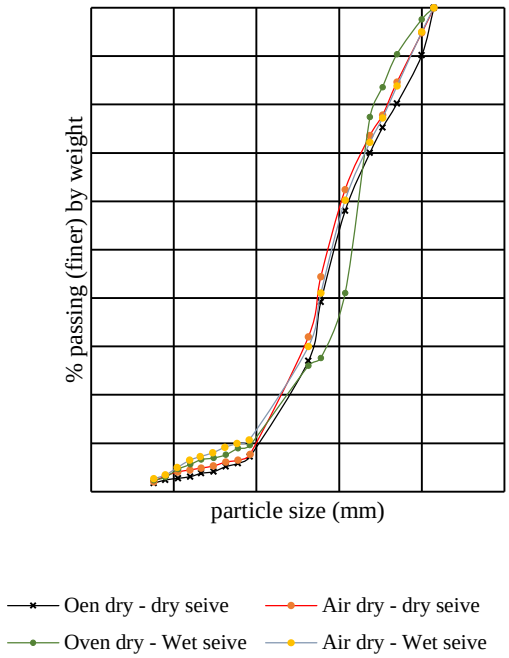

Figure 1. Particle size distribution

Liquid limits shall be found from both Casagrande apparatus and penetration method. Initially attempts with Cassagrande apparatus were not successful because of lower plasticity of sample and hence reliable results could not be achieved. Therefore, liquid limit was found using the penetration method. Table 1 shows the Atterberg limit values of sample.

Table 1 Plastic and Liquid limits

Type	Liquid limit	Plastic limit	Plastic index
Air dried and dry sieved	45	26	18
Air dried and wet sieved	35	28	7
Oven dried and dry sieved	38	27	11
Oven dried and wet sieved	30	27	3

3.2. Chemical tests

Chemical parameters of sample such as pH, chloride content and sulphite content were evaluated according to BS 1377: part 3.

Table 2 Chemical test results

Parameters	Values
P_H	7
Chloride content	0.02 %
Sulfite content	0.086 %

3.3. Compaction and consolidation

Since, standard Proctor compaction simulates the site procedures, tests were done on samples, which were air-dried and dry sieved. From particle size distribution curve, percentage of particles retained on sieve number 4 (4.25 mm) was found to be less than 20%. Therefore, method A (ASTM standard D698) was selected to perform. The Maximum dry unit weight is 1418 kg/m^3 obtained at an optimum moisture content of 24.5%.

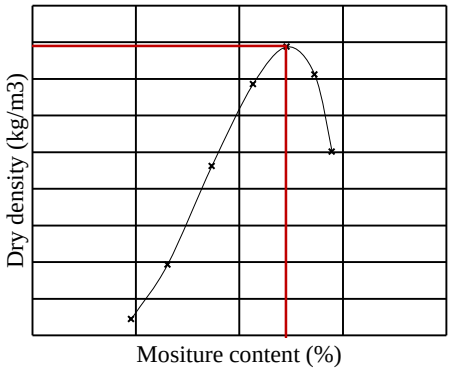

Figure 2. Moisture content vs Dry density

Specific gravity value of sample was evaluated according to BS 1377 : part 1, Pyknometer method. Specific gravity of sample was 1.96. Consolidation tests were done on the samples, which were taken from the standard Proctor compaction test. Therefore, it can simulate the field effect of compaction when sample is used as a fill material. This test was done for standard compaction of sample with different initial moisture contents. This test was done according to ASTM 2435, test method B.

4. CALCULATIONS

According to the Unified Soil Classification System, all considered four types of samples could be classified as SW – SM, which is well-graded sand with silt and gravel. Table 3 shows the primary and secondary creep consolidation parameters of such samples.

Table 3 Primary and Secondary consolidation parameters

ω (%)	Stress (kPa)	C_v (m^2/yr) log time method	C_v (m^2/yr) root time method	Cr	C_c	C_α (x 10^{-3})
15	50	1.78	5.01	0.008	0.17	4.17
	100	0.4	3.03			3.39
	200	0.31	1.26			3.12
24.5	50	0.33	4.27	0.01	0.08	3.2
	100	0.2	1.52			3.17
	200	0.25	1.6			2.47

5. DISCUSSION

Sample has been classified in four different conditions, but all of them has the same classification of well-graded sand with silt. Samples had considerably lower plastic index values, within the considered categories air-dried, dry sieve condition had a higher plastic index because of higher liquid limit. However, it can be noticed that, plastic limit is increased when sample is wet sieved, that indicates the availability of smaller amount of fine clay particles, which could be bonded with coarser particles. It leads to the sample near to the A-line in plastic index vs liquid limit curve. Table 4 (Robinson and Allam, 1996), shows the comparison of Atterberg limits and C_c of soils.

Chemical tests reveal that, considered sample is almost neutral with pH 7 and does not have a significant chloride or sulphite content. pH value indicates the unavailability of significant amount of other adverse chemical compounds. Further, it was found that the organic content was in the range of 11%.

Primary consolidation parameters such as coefficient of consolidation, primary compression index, recompression index and coefficient of volume compressibility describes that these parameters have lower value in optimum moisture content or maximum dry unit weight. Primary consolidation is lower in optimum moisture content. Secondary compression index has very lower value and lowest value is at the optimum moisture content.

Table 4 Comparison of Atterberg limits and C_c

Soil type	**LL**	**PL**	**C_c**
Boston clay	41	20	0.35
Chicago clay	60	20	0.4
New Orleans clay	80	25	0.3
Bloemendhal (Air dry – Dry sieve)	45	26	0.17

Table 5 Comparison of C_v

Soil	Pressure (kPa)	C_v (x 10^{-4} cm^2/ sec)	
		Logarithm of time method	Square root of time method
Brown soil	50 – 100	3.02	3.77
	100 - 200	2.86	3.4
Illite	50 – 100	1.34	3.13
	100 - 200	2.2	3.18
Chicago clay	50 – 100	1.3.7	17.4
	100 - 200	3.18	4.71
Bloemendhal Dumpsite ($\omega = 15\%$)	50 – 100	1.27	9.6
	100 - 200	0.98	3.99
Bloemendhal Dumpsite ω(24.5%)	50 – 100	0.634	4.82
	100 - 200	0.79	5.26

Table 5 shows comparison of coefficient of consolidation of different types of soils (Robinson and Allam, 1996), and Bloemendhal sample. C_v first decreases with increment of pressure then decreases.

6. CONCLUSIONS

It is generally mentioned that the long-term settlement of dump waste includes creep of soil and biodegradation. Secondary compression index of creep for different conditions are considerably lower in this particular sample. It is essential to consider both biodegradation and secondary compression index for long-term settlement of dump waste. Biodegradation occurs over the period of almost 3500 days (10 years) (Hossain and Gabr, 2005). However, Bloemendhal dumpsite is almost 30 years old and bottom part is almost decayed to form a soil (Nawagamuwa et al, 2013). Therefore, secondary consolidation due to biodegradation would not be significant in Bloemendhal sample. If bottom part of Bloemendhal dumpsite waste were used as fill material primary consolidation and secondary consolidation creep parameters should have to be considered. However further studies are required regarding biodegradation when dealing with relatively fresh samples.

7. ACKNOWLEDGMENTS

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Spatial Variation of Shear Strength and Consolidation Characteristics of a Municipal Landfill and its Implications on the Stability of the Fill- A Case Study

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Abstract: : *Municipal landfills which operate as open dumps in Sri Lanka do not satisfy any environment protection guidelines aimed at limiting the contamination of water resources and the damage caused to the surrounding environment. In addition, consideration of the stability of the landfill slopes is also a very important aspect of a landfill design. Slope failure of a landfill can occur during the construction of the landfill or after the closure of the landfill. In order to evaluate the stability of slopes it is imperative that representative shear strength properties of the slope materials which consist mostly of waste are determined. However, the highly heterogeneity nature of waste could pose uncertainty of the reliability of the stability analysis, if the influence of heterogeneity of shear strength properties on the stability of the slope is significant. This paper investigates the influence of spatial variation of shear strength properties on the slope stability of Udapalatha PS municipal landfill at Udapalatha in the Central Province of Sri Lanka. Direct shear and Oedometer tests were carried out to determine the shear strength and consolidation parameters respectively. These tests were done on recompacted waste materials obtained using percussion drilling and box sampling at two locations along the slope section. Factors of safety of the slopes of the landfill were evaluated using SLOPE/W and PLAXIS 2D software, assuming homogeneity and heterogeneity of properties. The above analysis yielded critical factors of safety ranging between 1.18 and 1.72 in Slope/W and 1.14 and 1.43 in Plaxis when heterogeneous properties were used in separate analyses assuming homogeneity. However, by considering spatial variation the factor of safety was found to be 1.62 and 1.32 in Slope/W and Plaxis analyses respectively emphasising the importance of considering spatial variation of shear strength properties in stability analyses.*

Keywords: *Landfill, Slope stability, Heterogeneity, PLAXIS 2D, SLOPE/W.*

1. INTRODUCTION

With the development of technology and rapid urbanisation, waste generated has also increased manyfold over the last few decades. Implementation of a proper waste disposal method is necessary to reduce harmful effects due to pollution of groundwater and the environment in general. An engineered landfill is a facility environmentally acceptable for disposal of waste as it is capable of reducing impacts due to landfilling.

In Sri Lanka, municipal solid waste is dumped in an unprotected environment, causing the pollution of groundwater sources and increasing the vulnerability of slope failures. Therefore, it is important to ensure adequate stability of these open dumps against slope failures (Gustavo and Catapreta, 2013). In order to evaluate the stability of slopes of open dumps, it is imperative that representative shear strength properties of the slope materials which consist mostly of waste are determined. However, the highly heterogeneous nature of waste due to different waste composition, particle size and degree of

degradation could result in less reliability of the stability analysis as discussed by Zhan et al. (2008). Therefore, although it is one of the most difficult tasks (Francisco et.al., 2013), determination of shear strength parameters clearly representing the heterogeneous nature of solid waste is a key aspect of carrying out a reliable slope stability analysis.

Dixon and Jones (2005) used laboratory tests, in-situ tests and back analysis to find out shear strength parameters of waste material. They concluded that knowledge of shear strength is required in order to assess waste slope stability. In situ measurement of shear strength of waste materials is not reliable due to presence of non-degradable materials such as polythene, glass, rubber etc., within the local mass of waste.

The consolidation parameters were found out from odometer test. However, the main purpose of this study was to analyze the stability of the landfill. In this study, the slope stability was analysed using PLAXIS and SLOPE/W. Francisco, J. et.al., (2013) and, Ravikumar and Andrews (2012) also showed the applicability of these software for slopes consisting of waste material.

This paper investigates the influence of spatial variation of shear strength properties on the slope stability of Udapalatha PS municipal landfill at Udapalatha in the Central Province of Sri Lanka located near the Mahaweli River. The Udapalatha landfill was used to dump municipal solid waste from Gampola municipal area which now remains abandoned. The slope stability analyses were carried out based on both limit equilibrium as well as finite element techniques using Slope/W and Plaxis software respectively.

2. MATERIALS AND METHODS

Udapalatha landfill, by way of its sequence of filling can be considered to consist of two regions where in one of which the solid waste had been dumped more recently (New site) compared to that in the other region (Old site). This study focuses on the slope stability of new site with reference to its spatial variation of shear strength parameters.

2.1 SAMPLE COLLECTION

The plan view of the Udapalatha landfill and the regions of Old and New sites with the arrangement of boreholes are shown in Figure 1. In order to evaluate shear strength parameters of waste materials at different locations representing spatial variation, disturbed and undisturbed samples of waste were obtained by carrying out percussion boring (PBH01 and PBH02) and undisturbed box sampling in trial pits near PBH02 and BH3 locations.

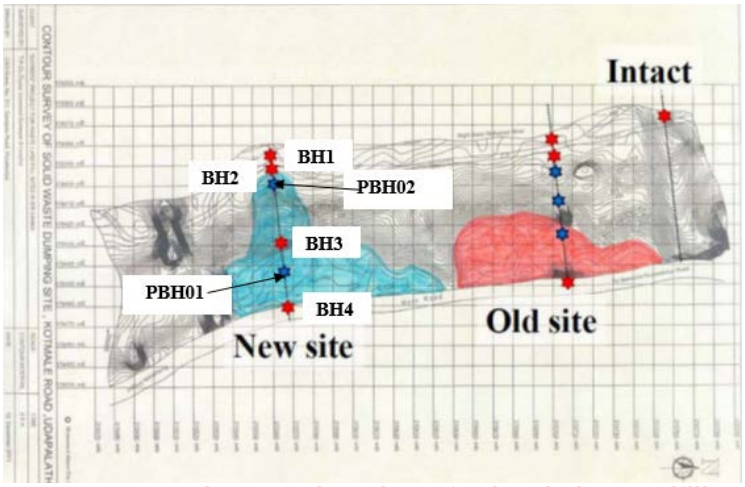

Figure 1 Plan View of Udapalatha Landfill

The cross section of the landfill at the new site of the landfill and its stratigraphy was developed using the contour plan and borehole log data as depicted in Figure 2.

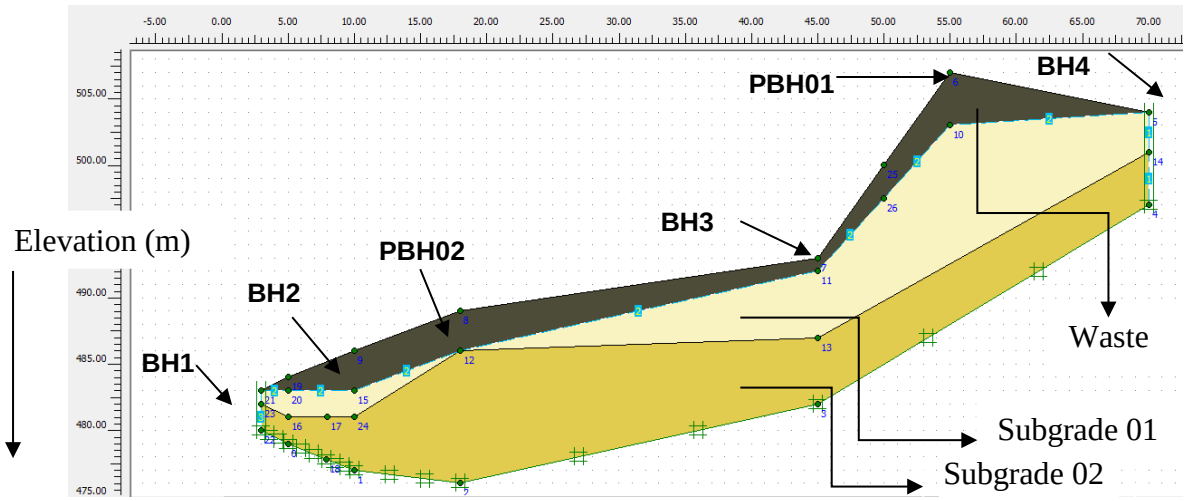

Figure 2 Modelling of Cross section of new site and locations of boreholes

2.2 Laboratory testing

The waste and subgrade material parameters required for the stability analyses carried out using Plaxis and Slope/W software were evaluated using specific gravity, direct shear and consolidation tests considering the most appropriate model that can be selected to simulate the load-deformation behaviours. The parameters for Plaxis analysis are effective cohesion (c'), effective friction angle (ϕ'), modified compression index (λ*), modified swelling index (κ*), modified creep index (μ*) and unit weight (γ). Here, λ* and κ* are the gradients of the normal consolidation line and the unloading line respectively in the volumetric strain–ln(stress) (i.e. ε_v-ln p') space. The parameter μ* is described in section 3.1.2. below. The input parameters for slope/w software are effective cohesion (c'), effective friction angle (ϕ') and unit weight (γ).

2.3 Stability analysis

The stability analysis of the critical slope was analysed using Slope/w and Plaxis software which uses limit equilibrium and finite element techniques respectively to arrive at the critical slip surface and the corresponding factor of safety.

3. RESULTS AND DISCUSSION

3.1 Laboratory test results

3.1.1 Direct shear test

Direct shear test was carried out on compacted samples obtained from BH02, PBH01 and PBH02 conforming to BS1377 (1990) under normal pressures of 50 kPa, 100 kPa and 150 kPa. Table 1 gives shear strength parameters obtained at these three locations.

Table 1 Shear strength parameters

Location	C'(kPa)	Φ'(˚)
BH 02	46.7	19.8
PBH02	50.2	14.0
PBH01	15.4	31.8

3.1.2 Consolidation test

Consolidation test was done on the undisturbed samples obtained in box samples located close to the borehole locations BH02, PBH01 and PBH02, conforming to BS1377 (1990). The undisturbed sample taken from steel box and the disturbed samples were compacted to the density of undisturbed sample. Disturbed sample was compacted into the consolidation ring using three layers. Table 2 gives Modified-compression index (λ*), Modified-swelling index (K*) obtained at these three locations.

Table 2 Consolidation parameters

Location	K*	λ*
BH 02	0.0020	0.0616
PBH02	0.0021	0.0707
PBH01	0.0056	0.0790

Determination of secondary consolidation parameter was carried out on an undisturbed sample of waste obtained from the borehole PBH02, consolidated up to a pressure of 625.9 kPa. In order to determine the value of coefficient of secondary consolidation (Cα), the test was continued for a period of about three weeks. In this study, it was assumed that the value of Cα does not show significant degree of spatial variation and therefore, only a single specimen was used to evaluate Cα. Figure 3 shows the variation of void ratio with log time of for the particular pressure. Using Figure 3, the Cα was found to be 0.0286 and the parameter μ* defined as μ*=Cα/2.3*(1+e) which is used as an input material parameter in the Plaxis analysis was calculated to be 5.8×10^{-3}.

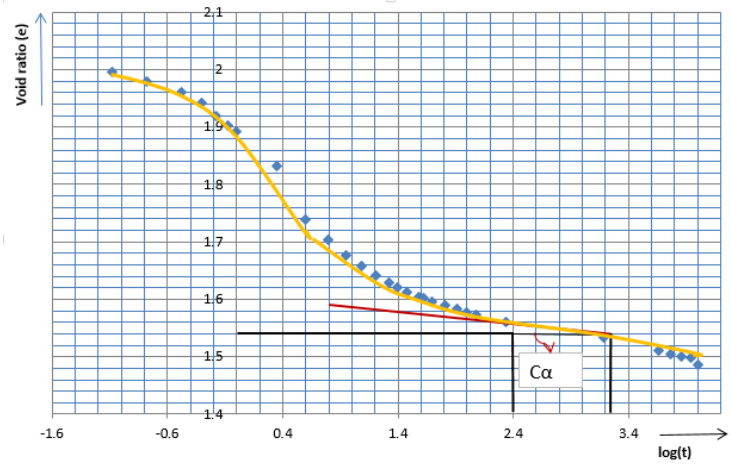

Figure 3 Variation void ratio with log (time)

3.2 Numerical analysis

As described earlier, the stability analyses were carried out using finite element and limit equilibrium methods using Plaxis and Slope/w software respectively. Figure 4 shows the cross sections of the landfill interpreted using the boreholes executed along the section as shown in Figure 2 which is consisting of the topmost waste layer underlain by two sub-grade layers. The input parameters used in the above analyses are summarized in Table 3.

3.2.1 Finite element analysis (PLAXIS 2D)

The landfill contained waste layers consisting of organic materials. Thus, in Plaxis analysis the soft soil creep model that could accommodate the effects of secondary consolidation was selected. The stability analysis of the landfill cross section shown in Figure 2 was analyzed using 15 noded constant strain triangular elements under plane strain conditions.

3.2.2 Limit equilibrium analysis (SLOPE/W)

The stability analysis of the above slope, based on limit equilibrium method was also carried out using Slope/w software. The input shear strength parameters were based on direct shear tests.

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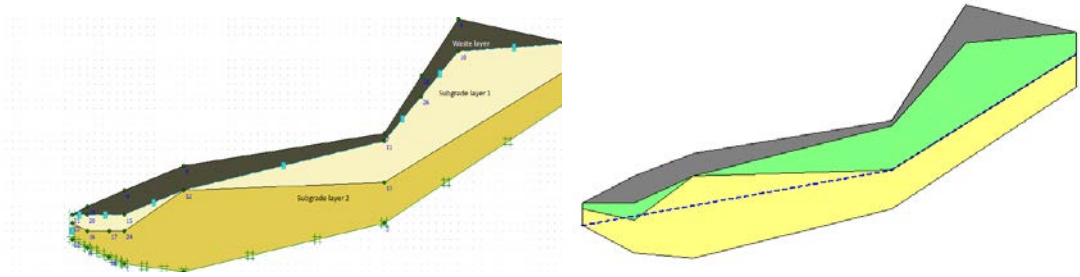


(a) (b)

Figure 4 Cross section of the landfill modelled by (a) PLAXIS 2D and (b) SLOPE/W

Table 3 Input parameters used in Plaxis and Slope/w analyses

Name	**Waste BH02**	**Waste PBH01**	**Waste PBH02**	**Subgrade 01***	**Subgrade 02***	**Unit**
Material model	Soft soil creep	Soft soil creep	Soft soil creep	Soft soil	Soft soil	-
Type	Drained	Drained	Drained	Drained	Drained	-
γ(unsaturated)	10.86	12.539	9.355	10.80	11.14	kN/m^3
γ(saturated)	16.48	16.26	13.629	16.31	16.51	kN/m^3
C	46.7	15.4	50.2	6.0	6.0	kN/m^2
φ	19.8	31.8	14.0	37.5	37.5	°
ψ	0.00	0.00	0.00	0.00	0.00	°
λ*	0.0616	0.0790	0.0707	0.0590	0.0590	-
k*	$2.0x10^{-3}$	$5.6x10^{-3}$	$2.1x10^{-3}$	$4.20x10^{-3}$	$3.4x10^{-3}$	-
μ*	$5.8x10^{-3}$	$5.8x10^{-3}$	$5.8x10^{-3}$	-	-	-

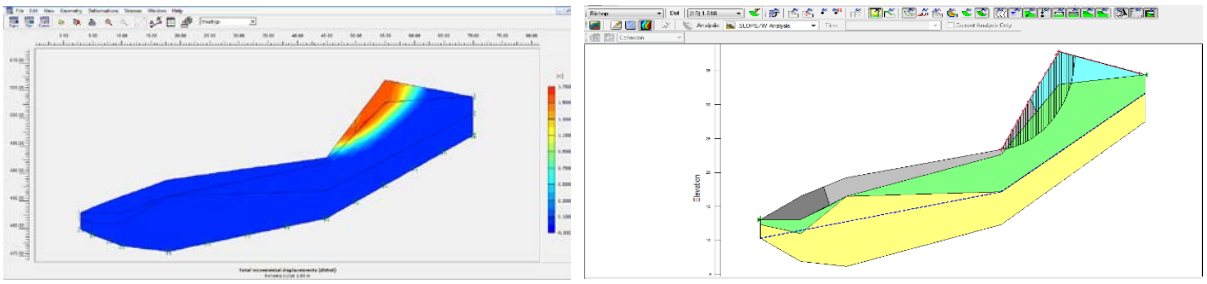
*-after Senthooran et.al., (2014).

Table 4 gives the summary of factor of safety values obtained from the above analyses.

Table 4 Summary of factor of safety values

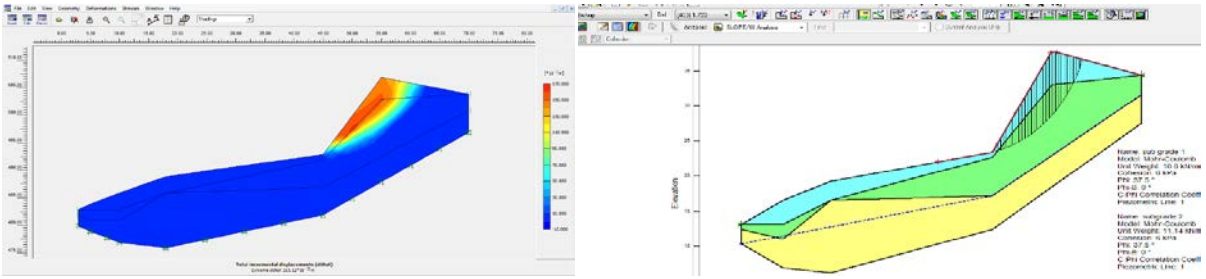
Analysis method		Cohesion (kPa)	Friction angle(°)	Factor of Safety	
				SLOPE/W	PLAXIS 2D
Spatial variation		-	-	1.618	1.319
Uniform	PBH01	15.4	31.8	1.184	1.139
	BH02	46.7	19.8	1.594	1.382
	PBH02	50.2	14	1.722	1.428

Figures 5 to 7 show critical failure surfaces of the landfill slope obtained from PLAXIS 2D and SLOPE/W analyses which showed similar critical failure surfaces.



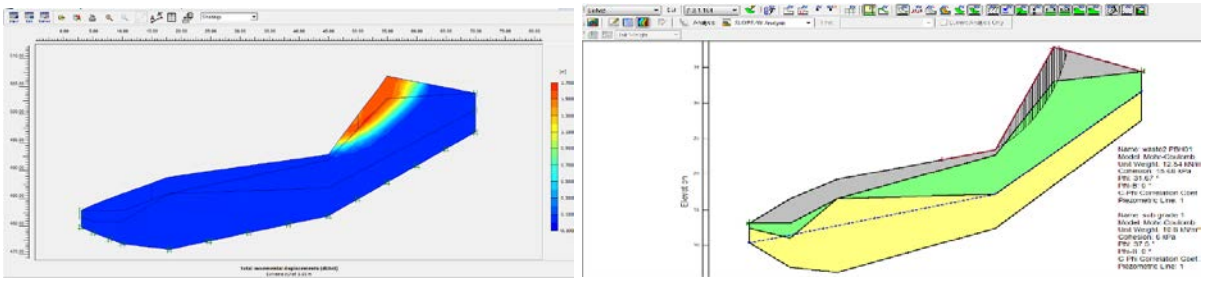
(a) (b)

Figure 5 Critical failure surfaces obtained from (a) PLAXIS 2D and (b) SLOPE/W analyses using spatial variation of shear strength parameters



(a) (b)

Figure 6 Critical failure surfaces obtained from (a) PLAXIS 2D and (b) SLOPE/W analyses using higher value of cohesion obtained from PBH02



(a) **(b)**

Figure 7 Critical failure surfaces obtained from (a) PLAXIS 2D and (b) SLOPE/W analyses using lower value of cohesion obtained from PBH01

The consolidation and direct shear test results for three different locations showed significant variation of parameters and also friction angle and cohesion of the landfill have higher amount of variation. The analyses were carried out for three different scenarios, including the spatial variation and analyses assigning parameters corresponding to PBH01 and PBH02 samples. The results of PBH01 sample gave a lower cohesion value and higher friction angle while the results of PBH02 samples showed higher cohesion value and lower friction angle. Based on the results of stability analyses using both equilibrium and finite element methods, the use of parameters corresponding to a particular location throughout the landfill has resulted in wide range of values of factor of safety. The same method was successfully used by Ravi kumar B. and Andrews O. (2012).

4. CONCLUSIONS

The effect of spatial variation of shear strength and consolidation parameters on the stability analysis of an abandoned open dump site was investigated. Based on the analyses carried out considering uniform as well as spatial variations of properties, the following conclusions are made.

(1) Shear strength and consolidation parameters showed considerable variation within the landfill.

(2) The range of values of factor of safety obtained corresponding to the uniform variation of lowest and highest values of effective cohesion was significant. Therefore, incorporation of spatial variation of properties would yield a reliable value of factor of safety.
(3) Using the spatial variation of parameters, the existing landfill is found to be stable considering the spatial variation of shear strength properties.

5 ACKNOWLEDGMENTS

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Insufficient Protecting Cover for Reinforcement of Piles in Weralagoda Bridge-Matara-Katharagama Railway Extension Project

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Abstract: *Pile foundations are commonly using worldwide for support high-rise buildings and Bridges which have heavy vertical loads. Cast in place bored Piles were used for construction of several bridges in Matara-Katharagama Railway extension Project. In this study it's going to mainly consider about Weralagoda Bridge pile construction witch used Percussion method. Corrosion of steel reinforcement in concrete reduces the service life of a structure, as well as causes a reduction in its structural capacity. In Weralagoda Bridge it was identified that nine out of thirty five piles have the issue of insufficient protecting cover for reinforcement due to insufficient Temporary Casing length, Shape of the cover block and Technique of installing the cover block.*

Keywords: *Percussion Bored pile, Concrete, Cover, Reinforcement.*

1. INTRODUCTION

Pile foundations are commonly using worldwide for support high-rise buildings and Bridges which have heavy vertical load. Piles are long and slender members which transfer the load to deeper soil or rock of high bearing capacity avoiding shallow soil of low bearing capacity. The main types of materials used for piles are Wood, steel and concrete. Piles made from these materials are driven, drilled or jacked into the ground and connected to pile caps. Depending upon type of soil, pile material and load transmitting characteristic piles are classified accordingly. The pile foundations should be carefully designed in accordance with the soil and load conditions, and the cost.

There are two ways to install a concrete pile. One method involves drilling and casting on site. This is also known as a "cast in situ" pile, because it is created at the construction site. The other method is precast concrete piles that are driven into the ground. With cast in situ piles, the hole is bored into the ground, a reinforcement cage installed and the hole gradually filled with concrete. Cast in place bored Piles were used for construction of several bridges in Matara-Katharagama Railway extension Project.

Cast in situ bored piles can be constructed by using Rotatory method and Percussion Method. In this study it's going to consider about Weralagoda Bridge pile construction which used Percussion method. Percussion drilling is a drilling technique in which a drill hammer attached to rope or cable is repeatedly raised and lowered, impacting soil and rock, and making a hole deeper.

The success of pile foundation work is influenced by the technique and quality of pile foundation work to a great extent. If the quality and structural integrity of these piles are in doubt, the safety of bridge will be jeopardised. Bad workmanship may easily cause defects in the pile itself and during construction the continuity and thoroughness over the entire pile foundation can be influenced very easily by improper speed of operating tremie pipe or other human and machine equipment factors during quick sand or pouring of concrete. This may cause possible defects in the pile and these defects can possibly be honeycomb, crack, necking, and insufficient protecting cover for reinforcement. (Yan-Chyuan & Shu-Jen 2006)

Weralagoda Bridge is located at change DK19+325 of Matara –Katharagama Railway extension Project. This Bridge was designed with seven piers and two abutments. According to the geological condition it was designed to use pile foundations for Abutment 01, Pier 01, Pier 02, Pier 03, Pier 04, Pier 05 and Pier 6.

In Weralagoda Bridge it was identified that nine out of thirty five piles have the issue of insufficient

protecting cover for reinforcement.

2. PROTECTING COVER FOR REINFORCEMENT

Corrosion of steel reinforcement in concrete reduces the service life of a structure, as well as causes a reduction in its structural capacity. The factors that affect the rate of corrosion of reinforcement in concrete include the following:

(i) Presence of harmful chemicals and the concentration of these chemicals.
(ii) Availability of moisture and oxygen.
(iii) Concrete density and permeability.
(iv) Thickness of concrete cover.

While the rate of corrosion of reinforcing bars can be reduced by providing proper concrete cover, additional measures are needed to ensure adequate and efficient protection in harsh environments. British Standards were used for the designs in this project. According to the BS 8500 and BS EN 2006 the nominal cover to the reinforcement can be assessed as follows:

$$Cover = c_{min} + \Delta c_{dev} \quad (1)$$

Where C_{min} should be set to satisfy Safe transmission of bond forces, Durability and Fire resistance and ΔC_{dev} should be taken as 10 mm. By considering those design criteria's it was design to have minimum 75mm cover for the piles in Matara-Katharagama extension Railway project.

2.1. Insufficient Protecting Cover for Reinforcement

According to the method of statements for pile constructions, contractor agreed for a jointly inspection with Engineer and contractor representative to identify the protecting cover to the reinforcement around the cut off level. This was done after excavate 0.5-1m below the cut off level by using cover meter. Contractor agreed to measure the Protecting cover after cleaning the contaminated concrete layer occurred around the pile due concrete mix with soul or bentonite slurry. Following piles were identified as the piles with Insufficient protecting cover for reinforcement at the cut off level.

Table 1 Minimum protecting cover for reinforcement at Weralagoda Bridge

Pile No	Approximate cover
Abutment 01-Pile 03	40mm
Abutment 01-Pile 04	30mm
Abutment 01-Pile 06	20mm
Pier 03-Pile 04	40 mm
Pier 04-Pile 01	50mm
Pier 05-Pile 02	50mm
Pier 05-Pile 04	20mm
Pier 06-Pile 01	50mm
Pier 06-Pile 05	40mm

After the identification of this issue contractor requested for further excavation until they can identify and verify there is adequate protecting cover for reinforcement. Contractor was able to identify that above mentioned piles achieved sufficient protecting cover for reinforcement at different levels below the cut off level as shown in Table 2.



Figure 1: Insufficient protecting cover for reinforcement of piles in Weralagoda Bridge

Table 2 Depth to sufficient protecting cover for reinforcement at Weralagoda Bridge

Pile No	Approximate Depth(From Cut-off Level)
Abutment 01-Pile 03	6m
Abutment 01-Pile 04	6m
Abutment 01-Pile 06	7m
Pier 03-Pile 04	1m
Pier 04-Pile 01	2m
Pier 05-Pile 02	2m
Pier 05-Pile 04	2m
Pier 06-Pile 01	2m
Pier 06-Pile 05	3m

3. POSSIBLE CAUSES FOR THIS ISSUE

After analyzing the methods and technique used in pile construction in Weralagoda Bridge, several possible causes' for insufficient protecting cover for reinforcement were the identified.

(i) Insufficient Temporary Casing length
(ii) Shape of the cover block
(iii) Technique of installing the cover block

3.1. Insufficient Temporary Casing length

Piling Contractor used temporary steel casings for bore-hole support. Such steel casings were installed into the ground by oscillating and jacking motion exerted by a hydraulic casing oscillator. Since most of the top soil layers of piling area in Weralagoda Bridge were identified as highly compacted laterite soil this casing installation was very difficult and time consuming. As a result, contractor used to insert temporary casing with the length of 3 m at Weralagoda pile construction site. According to the design drawings of Weralagoda Bridge and topography of the area it was identified that pile cap cut-off levels were 2-3m below the ground level.

Normally the reinforcement cage would bend due to the self-load. If there were sufficient cover blocks inside the temporary casing this bending effect would be minimized. Since the temporary casing was 3m in length and cut-off levels were 2-3m below the ground level, a few no of cover blocks were accommodated the temporary casing as shown in Figure 2.

Since the average pile length was 12m in Weralagoda Bridge possibility of bending maximizes in the

range of 4 -8 m below the ground level. In most defected piles it was identified that the protecting cover for reinforcement was insufficient in the range 4-6 m below the ground level. Further it was identified that most of these defected piles have 100-150mm cover in one side while on the opposite side of the pile have Insufficient protecting cover.

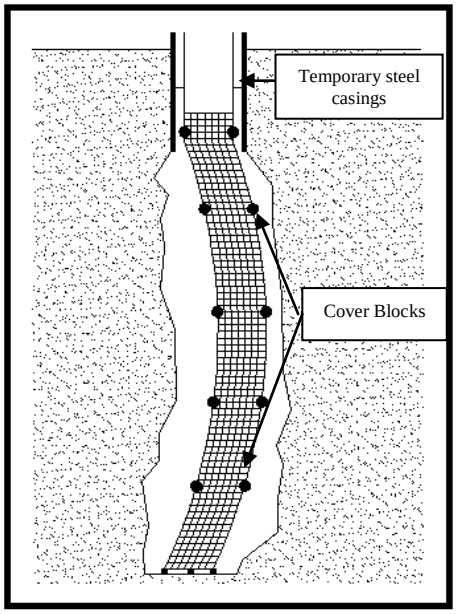

Figure 2 Insufficient Temporary Casing length

3.2. Cover Blocks

Shape of the cover block was also identified as a cause for this issue. As shown in figure 3 contactor used a round shape cover block. Pressure on a single cover block was very large because large force was appalled on a small area. Contractor used cover block with the spacing of 2 m in same plane. This also increased the load on single cover block. When the cover blocks with large load were located in week soil layers most of the cover block were forced in to the soil as shown in Figure 4.



Figure 3 Cover Block



Figure 4 Cover Block forced in to the soil

4. CORRECTIVE & PREVENTIVE ACTION

4.1. Corrective

Contractor agrees to hack defected piles on Pier 03, Pier 04, Pier 05 & Pier 06 up to the level they identified as there was sufficient cover and rebuild by using Grade 30 concrete. For the piles on

Abutment which contactor had to excavate nearly 8m to identify sufficient cover contractor agreed to use following steps for jacketing without breaking the original concrete cover.

(i) Cleaned the Pile shaft to expose sound concrete.
(ii) Applied a corrosion protection system on exposed reinforcement.
(iii) Applied a bonding agent on pile shaft.
(iv) Each lift height of the jacketing concrete was limited to 1.0 m

4.2. Preventive Action

After analysing the potential causes for this issue following preventive action were recommended for future constructions.

(i) Use the temporary casing to have at least 3m of reinforcement cage to be inside the temporary casing with adequate cover blocks.
(ii) Weld the reinforcement cage to the temporary casing after lifting two or three inches above the tip level to reduce the bending effect.
(iii) Change the Cover block shape to have large effective area as shown in Figure 5.
(iv) Use cover block in small interval with different planes.
(v) Temporary hang four steel bars to maintain the cover



Figure 5 Modified Cover Block

5. CONCLUSION

Sufficient protecting cover to the reinforcement is important for the durability of the pile foundation. Poor and inexperience workmanship is the main caused for the insufficient protecting cover to the reinforcement. Repairing those defected piles will consumed more money, material and time which will effect to the construction project directly, so it is important to pay a special attention on the length of the temporary casing and cover blokes.

6. ACKNOWLEDGEMENT

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Shrinkage Behavior of a Liner Material with Expansive Properties for use in an Engineered Landfill

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Abstract: *The liner material in an engineered landfill acts as a barrier for leachate migration and prevents contamination of nearby water resources. Compared to geosynthetic clay liners (GCL) and high density Polyethylene (HDPE) liners, compacted clay liners (CCL) are less expensive, if the materials are locally available. According to USEPA guidelines the recommended thickness of a CCL having a maximum hydraulic conductivity of 1 x 10^{-7} cm/s is 1 m. However, it is possible to reduce the thickness by using a less permeable soil such as that obtained from Moragahakanda, Central Province of Sri Lanka. This Moragahakanda soil is found to be an expansive soil and its hydraulic properties could be further improved by amending with Bentonite. However, in addition to the hydraulic conductivity criteria, it is necessary to examine its shrinkage behaviour during drying to investigate the sustainability of its use as a liner. In this study, Moragahakanda expansive soil, and that amended by 5% and 10% bentonite were used as candidate materials from which experimental liners were prepared to have the same void ratio as that obtained by consolidating the same liner materials to 46.2 kPa and 92.4 kPa having a diameter of 150 mm and thicknesses of 5 mm, 10 mm and 20 mm. The disks were allowed to air dry and the variations of moisture content, area shrinkage and possible crack pattern with time were evaluated. An image processing method using ArcGIS 10.2 software was used to determine area reduction and cracked area.*

Keywords: *Expansive soil, Landfill liner, Shrinkage, Image processing*

1. INTRODUCTION

In Sri Lanka, engineered landfills have not been established yet for disposal of municipal solid waste. However, the need for engineered landfills cannot be ignored with the increasing quantity of solid waste generation due to rapid urbanization. Generation of leachate which is a liquid produced when water or other liquids come into contact with waste or due to release of moisture present within itself creates one of the major challenges in designing and maintaining an engineered landfill. Therefore, an essential component of an engineered landfill is the liner system which acts as a barrier for leakage of leachate and prevents the transportation of contaminants to the nearby water resources. Synthetic liners consisting of geomembranes or geosynthetic clay liners (GCL) are widely in use as liner materials of engineered landfills constructed in developed and developing countries. However, as geosynthetic materials are not manufactured in Sri Lanka, Compacted Clay Liners (CCL) is an alternative to be considered as a material for a liner (Abeyrathne, 2012). However, in addition to meeting hydraulic conductivity criteria of a liner, one major challenge of using CCL as a liner material is the possibility of developing shrinkage cracks during a dry season. In this regard, use of expansive soils in CCL is advantageous as they exhibit a very high plasticity index (PI) which prolong the initiation of cracking upon drying and have the characteristic of self-healing upon wetting. Kurukulasuriya et.al. (2013) investigated expansive soil obtained from Moragahakanda in the Central Province of Sri Lanka and that amended by 5% and 10% bentonite (Bentofix®, NAUE GmbH & Co. KG, Germany) by weight as candidate materials for a liner and found that they satisfy short and long term hydraulic criteria. In this study, the shrinkage behavior of the same candidate materials obtained from Moragahakanda is investigated.

2. MATERIALS AND METHODS

The expansive soil for the study was obtained from the same location (07˚ 35'32.8″ N 80˚49'59.9″E) from which soil was obtained for an earlier study as reported by Wanigaratne et.al. (2012) in which the hydraulic conductivity was determined on specimens prepared by consolidating one dimensionally to pressures of 46 kPa, 92 kPa and 184 kPa. Initially, soil classification tests such as liquid limit, plastic limit, particle size distribution and specific gravity tests were carried out to identify the basic physical soil properties. These tests were carried out conforming to methods specified by British Standards. To identify the expansiveness of the soil obtained from Moragahakanda swell pressure test was carried out.

In this study, the shrinkage characteristics were investigated on the specimens having the same voids ratio as those used in the study of Wanigaratne et.al. (2012). However, the specimens were prepared by compaction to achieve the same voids ratio. This was possible by carrying out Proctor compaction test and evaluating the voids ratio corresponding to each compacted sample represented in the compaction curve. Therefore, a relationship between the dry density and voids ration can be developed from which the specimens for this study corresponding to the same voids ratio of that obtained by consolidation could be obtained by compacting to the required dry density.

Specimens of diameter 150 mm and thicknesses of 5 mm, 10 mm and 20 mm were prepared (as has been done by Atique and Sanchez, 2011), using Moragahakanda expansive soil and that amended by 5% and 10% bentonite compacted to voids ratios corresponding to specimens consolidated to 46 kPa and 92 kPa. Trimming of compacted specimens to above thicknesses was made possible by using templates of ring shape of required heights (Fig.1) into which the compacted samples were inserted during extraction from the compaction mould. Altogether, 18 specimens were prepared and placed on glass plates and left for air drying (Fig.2).

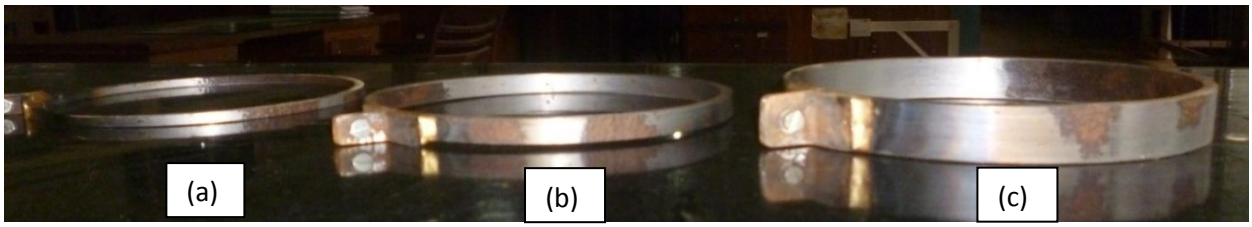


Figure 1. Circular shaped templates to prepare specimens of thickness (a) 5 mm, (b) 10 mm and (c) 20 mm



Figure 2. Compacted specimens of candidate materials left for air drying

The change in area of flat surface exposed to the atmosphere and the development of cracks were monitored daily during the drying process. For this purpose, digital photographs were taken using a 14 megapixel camera from a fixed height under almost constant light and ambient temperature conditions. This was facilitated by using a frame as shown in Fig.3. The moisture content too was computed at the time of taking the photograph using the weight of the specimen obtained immediately after taking the photograph.



Figure 3. The wooden frame used for photographing the compacted samples during air drying

These images were fed into GIS (Arcmap version 10.2) software and the “Iso Cluster Unsupervised Classification” tool was used to multivariate the image. Then, the “Extract by Attributes” tool was used to select the sample from the image and the area was calculated by converting the raster dataset to polygon features using the “Raster to Polygon” tool.

However, the output result of the GIS software will be valid for an image taken at a particular height. Therefore, it is necessary to calibrate the output result given by the software with reference to the height at which the photograph was taken. This was done by calculating the areas of a square and rectangular shapes placed within a larger circle and photographing at the same height the 5 mm, 10 mm and 20 mm thick specimens had been photographed (Fig.4).

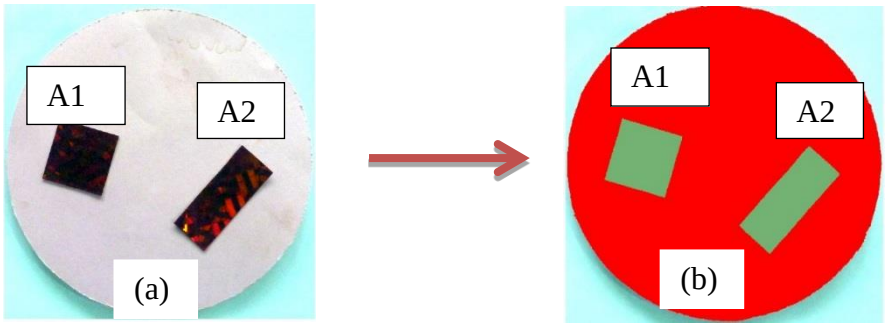


Figure 4. Image used for calibration of the GIS software computation (a) Object of known area (b) Classification image

3. RESULTS

3.1 Physical and Compaction Properties

The results of Atterberg Limits, Specific Gravity and Swelling pressure tests are given in Table 1 for Moragahakanda expansive soil (Soil M) and that amended by 5% (Soil M+5%) and 10% (Soil M+10%) Bentonite. The Moragahakanda soil was classified as Sandy Clay of Intermediate Plasticity (CIS).

The preparation of compacted candidate soil samples was carried out so that the void ratio of the compacted soil sample corresponds with that consolidated under 46 kPa and 92 kPa pressure. Table 2 gives the void ratio of the consolidated soil samples (Wanigaratne et.al.2012) and the corresponding values of dry density that should be achieved on the compacted soil samples used in this study.

Table 1 Physical and Compaction Properties of Candidate Soils

	Bentonite	Soil M	Soil M+5%	Soil M+10%
Liquid limit (%)	600 *	66	49 *	70 *
Plastic limit (%)	55*	29	21 *	25 *
Plasticity index (%)	545*	37	28 *	50 *
Specific Gravity	2.68*	2.59	2.57	2.55
Swell pressure (kPa)	-	126.1*	267.7 *	357.0 *
Maximum Dry Density (Mg/m^3)	-	1.65	1.48	1.71
Optimum Moisture Content (%)	-	18.0	26.8	14.0

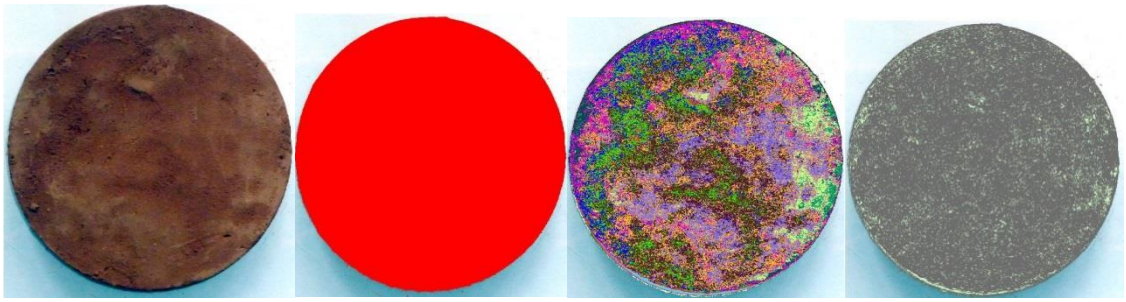
*- after Wanigaratne et.al., (2012)

Table 2. Target Dry Density of Compacted Candidate Soil Samples

	Soil M		Soil M+5%		Soil M+10%	
Consolidation Pressure (kPa)	46.2	92.4	46.2	92.4	46.2	92.4
Void Ratio	0.740	0.612	0.780	0.729	0.576	0.502
Dry density (Mg/m3)	1.53	1.65	1.44	1.48	1.62	1.69

3.2 Shrinkage Characteristics

An example of the transformation of digital images taken during the drying process by the GIS software to compute the surface and crack area (where applicable) is shown in Fig.5. The computed value given by the GIS software was multiplied by the calibration coefficient as given in Table 3 which was obtained by the procedure described in the earlier section. As expected, the greatest calibration coefficient was obtained when the object was furthest away from the camera lens.



(a) (b) (c) (d)

Figure 5. Steps of image processing in ArcGIS Software, (a) Sample, (b) Classification image, (c) Extracted image and (d) Rastered image

Table 3. Calibration coefficients used to compute surface areas of compacted specimens

Thickness of specimen (mm)	Calibration Coefficient
20	1.12
10	1.16
5	1.20

Fig.6 shows the rate of drying for specimens prepared to correspond with an initial voids ratio of samples consolidated to a pressure of 46.2 kPa. Fig.7 shows the same for specimens prepared to correspond with an initial voids ratio of samples consolidated to a pressure of 92.4 kPa. Figs. 6 and 7 show that the reduction of moisture content is rapid in thin specimens as well as in specimens whose initial voids ratio is high as those correspond with samples consolidated to 46.2 kPa. This is understandable as thin specimens and those specimens that correspond with low consolidation

pressure contained low amount of moisture and all the specimens had more or less same area of surface exposed to the atmosphere through which the exchange of water molecules and air could take place during the drying process.

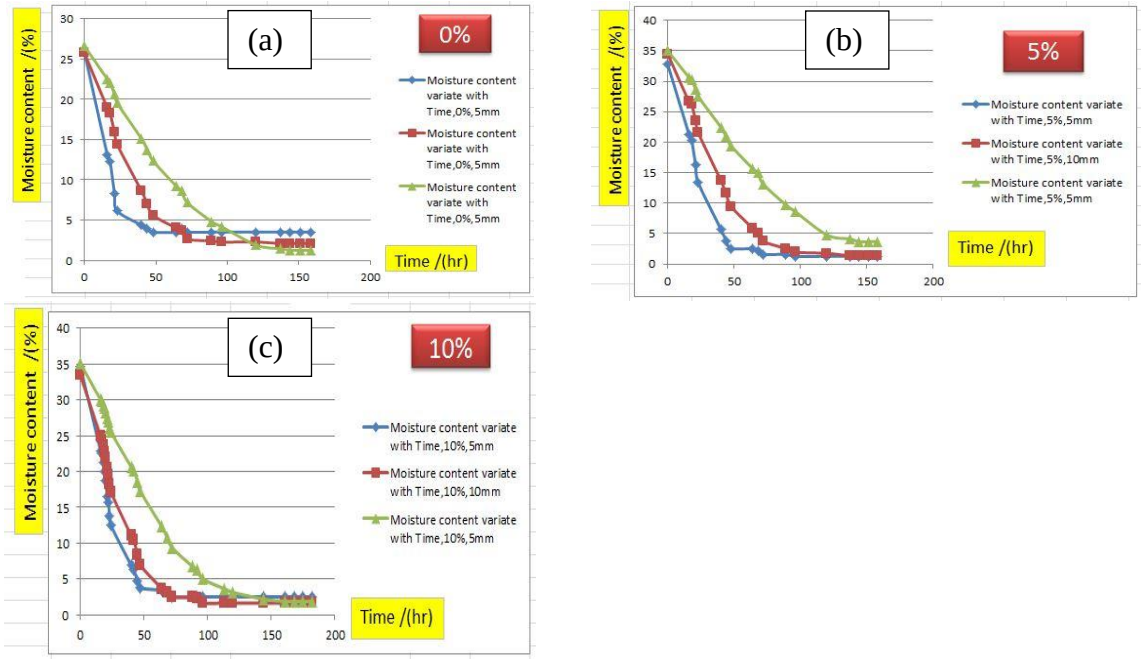


Fig.6 Rate of drying of specimens prepared to correspond with an initial voids ratio of samples consolidated to a pressure of 46.2 kPa (a) Moragahakanda expansive soil, (b) 5% Bentonite added, (c) 10% Bentonite added.

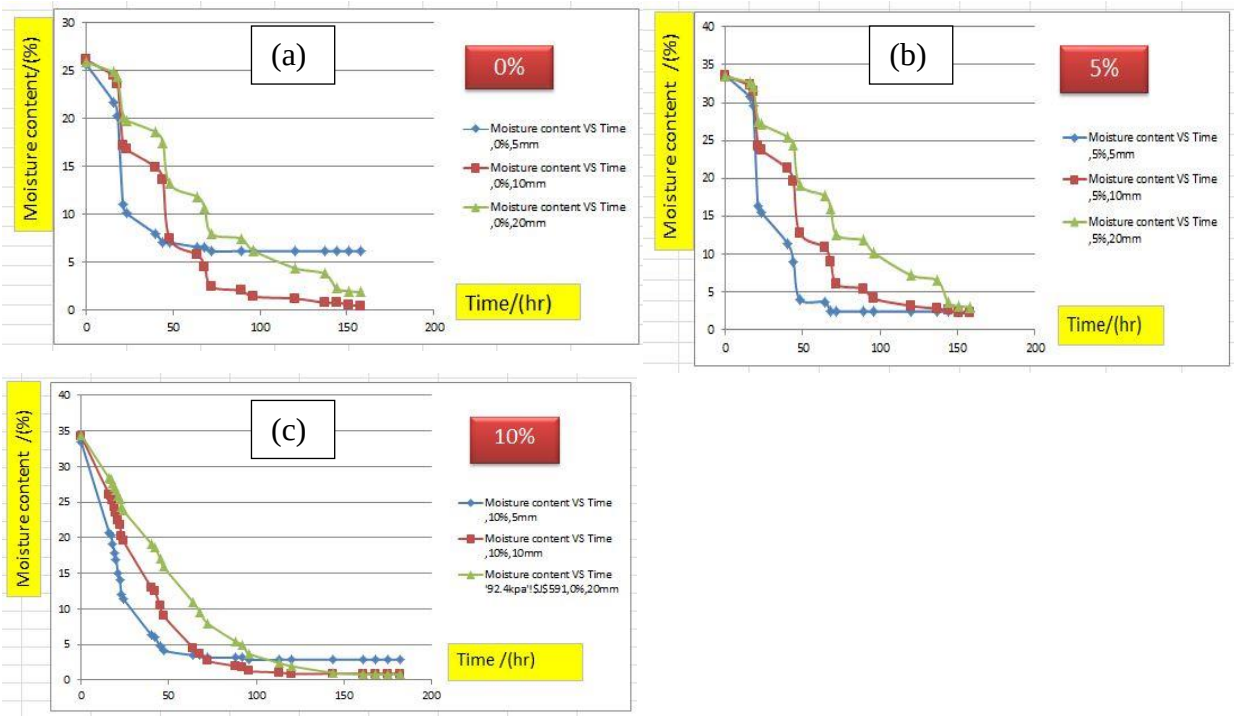


Figure 7. Rate of drying of specimens prepared to correspond with an initial voids ratio of samples consolidated to a pressure of 92.4 kPa (a) Moragahakanda expansive soil, (b) 5% Bentonite added, (c) 10% Bentonite added.

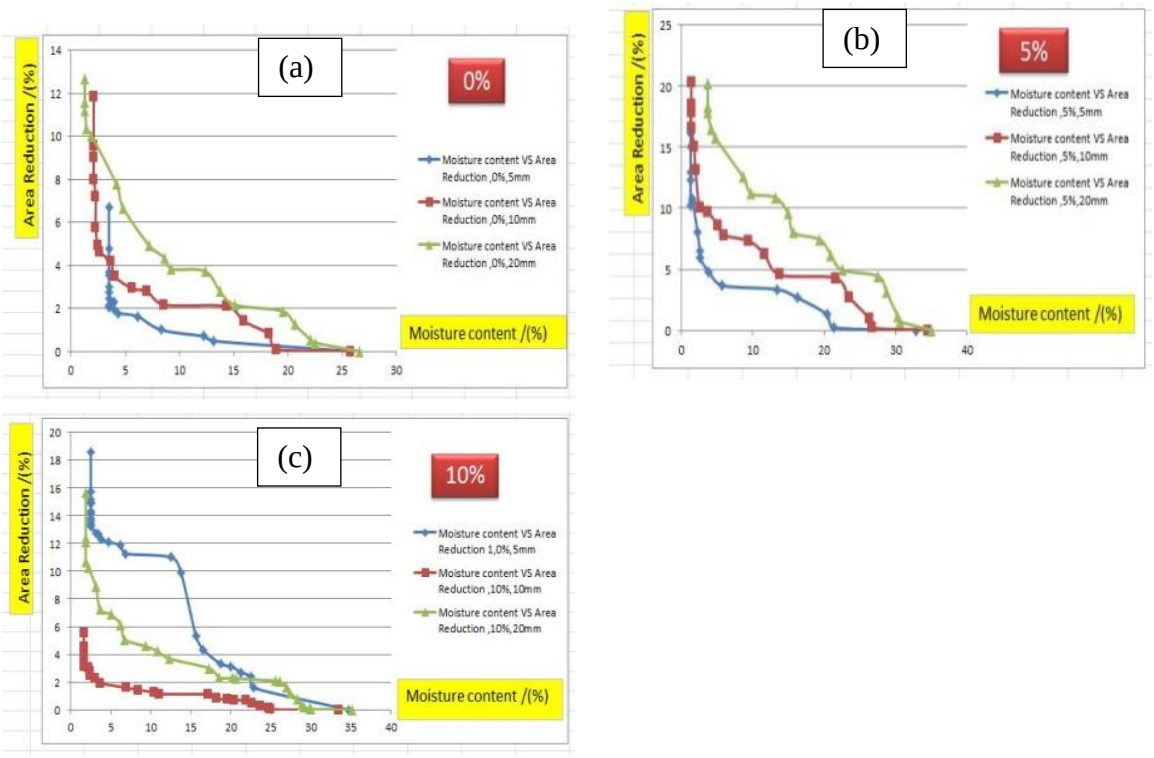


Figure 8. variations of percentage area reduction with moisture content of the specimens with an initial voids ratio corresponding to that of specimens consolidated under a pressure of 46.2 kPa

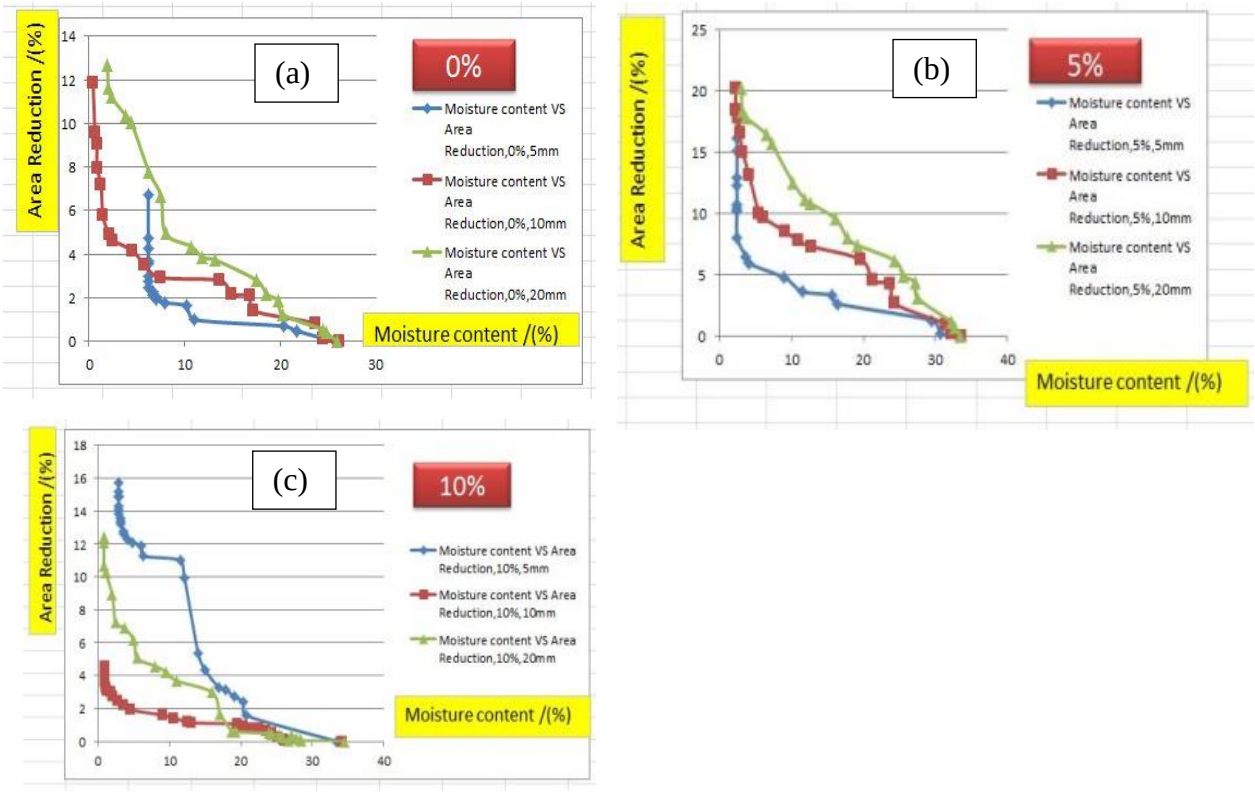


Figure 9. Variations of percentage area reduction with moisture content of the specimens with an initial voids ratio corresponding to that of specimens consolidated under a pressure of 92.4 kPa

Fig.8 shows the variations of percentage area reduction with moisture content of the specimens with an initial voids ratio corresponding to that of specimens consolidated under a pressure of 46.2 kPa and Fig.9 shows the same variations of the specimens with an initial voids ratio corresponding to that of specimens consolidated under a pressure of 92.6 kPa. It is seen that although the moisture content is reduced gradually during the drying process of specimens, most of the reduction of area took place once the specimens reached the equilibrium moisture content.

It appears that the shrinkage taking place during the drying stage of the specimens does not reduce the area significantly and once the equilibrium moisture content is reached, the specimens continue to undergo contraction at the equilibrium water content. As the drying takes place, the large amount of water absorbed into the diffuse negatively charged layer in montmorillonite clay mineral present in Moragahakanda expansive soil and in Bentonite (Herath, 1993) would be released, thus causing the reduction in the distance between the layers.

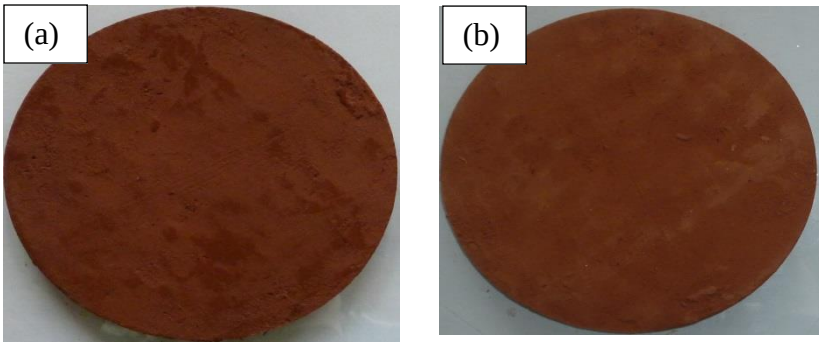


Figure 10. View of the sample (a) before drying, (b) after drying

Fig.10 shows that for all the 18 specimens, although the reduction of surface area occurred, no shrinkage cracks were developed. This observation is a very important as during dry season the liner will lose moisture as the leachate generated is minimal, and it is very advantageous for the liner not to develop shrinkage cracks, although expansive soils have the characteristic of self healing the cracks when wetted.

4. CONCLUSION

This study was carried out to investigate the shrinkage behaviour of candidate liner materials prepared using expansive soils from Moragahakanda and that amended by 5% and 10% of Bentonite. The above candidate materials were found to conform to hydraulic criteria of liner materials by a previous study and therefore, the specimens in this study was prepared to the same degree of compaction having the same voids ratio. Based on the outcome of this study, the following conclusions can be made.

(1) Greater thickness of the specimen dries out at a lower rate.
(2) Area reduction takes place, even after reaching the equilibrium water content during drying.
(3) No shrinkage cracks within the surface of the specimens were visible, thus making the materials highly suitable as a liner material, provided that other criteria to determine the suitability as a liner material such as hydraulic conductivity and durability issues are satisfied.

5. ACKNOWLEDGMENTS

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Effect of Gas Transport Parameters of cover soil in Optimizing Landfill Final Cover Thickness in Dry Zone of Sri Lanka

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Abstract: *Landfill final cover plays a vital role in separating waste from the surrounding. Therefore landfill final cover should able to minimize infiltration of moisture and to control harmful gas emissions through it. In dry zone of Sri Lanka, due to less precipitation, infiltration is less. Thus more attention should be given regarding harmful greenhouse gases, produced with in the landfill as a result of anaerobic decomposition. Thereforeit is important to develop a proper mechanism to transfer oxygen through final cover and hence to enhance aerobic decomposition of refuse.The gas exchange process through final cover soil is controlled by both diffusive and advective gas transport properties. Based on laboratory experiments it was realized that gas transport parameters are high in dry side of optimum of compaction. And gas transport parameters were gradually increased with the air content. Further, by combining both hydraulic performances and gas transport parameters final cover thickness was decided.*

Keywords: *Landfill gases, Landfill final cover, air permeability, soil-gas diffusivity*

1. INTRODUCTION

Final cover is an essential part of all municipal solid waste landfill facilities. A well designed final cover controls moisture infiltration from the surface into the closed facilities, limits the formation of leachate, prevents direct exposure of waste to the atmosphere and controls gas emission and odour. Regulations have been implemented to design and construct the final cover of the landfill both in local and international context in order to achieve above requirements (USEPA, 2005).

Conventional approach has been implemented in most of the countries, consisting of number of layers such that erosion layer, drainage layer, infiltration layer, gas collection layer and foundation layer in order to achieve different outcomes (USEPA, 2005). However this design approach is not economical for a country like Sri Lanka, since it is difficult to construct and maintain a complex layer system for landfill final cover (Wickramarachchi *et* al, 2010). Therefore it is important to identify a simple cover system to meet the required functioning of the landfill final cover.

As the second approach of designing final cover systems, Alternative Final Cover (AFC) system can be identified which equally performs as the conventional approach. The AFC design process is flexible and is predicated on scientific and engineering principles and practices. Geosynthetic Clay Liner (GCL), capillary barrier and Evapotranspirative cover (ET) are some examples for alternative approach (ITRC, 2003). It is required to permit the AFC approach with necessary regulations and guidelines and performance goals.

Once the solid waste is placed in a landfill and covered with a soil fill, a complex series of biological and chemical reactions is occurred within the landfill. The anaerobic degradation of organic waste in landfill which is occurred under oxygen limited condition produces a mixture of gases that contains methane, carbon dioxide and various other toxic compounds (IPCC, 2007). Since Sri Lanka is a middle developing country, recovery of landfill gases is not in practiced. As a result, landfill gases are emitted to atmosphere through final cover soil which creates numerous environmental problems. Therefore, it is economical to design the landfill final cover to promote gas (oxygen) exchange between the atmosphere and the waste layer to maintain the aerobic condition and methane oxidation in the waste layer (Wickramarachchi *et* al, 2010).

Nevertheless, selection of the final cover soil for landfill should be done giving a special care to both minimize generation of greenhouse gases and to minimize leachate production. Therefore, it is necessary to select high gas exchangeable and less water permeable material for final cover of a landfill (Wickramarachchi *et* al, 2010). However, finding such a material with two opposite properties is not an easy task. Since infiltration of rain water through soil in dry zone is very less, the generation of leachate in dry zone is comparatively less. Therefore, when selecting material for the final cover of a landfill in dry zone, gas exchangeable parameters become dominant.

In this research study, a suitable cover material for an alternative final cover of an engineered landfill in dry zone of Sri Lanka was selected based on gas exchangeable properties. Further, results were compared with the existing predictive models for the gas transport parameters and final cover thickness was decided according to Sri Lankan regulations, by combining gas transport parameters and hydraulic performance of the selected material.

1.1. Numerical Models for Gas Transport Parameters

1.1.1 Diffusion

Diffusion is a principal mechanism in the interexchange of gases between soil and the atmosphere. The soil-gas diffusion coefficient (D_p) governs the gas transport which takes place due to soil-gas concentration gradient. Fick's law is used to calculate the gas diffusion coefficient (D_p) and it can be expressed mathematically as;

$$\varepsilon \frac{dC}{dt} = D_p \frac{d^2C}{dz^2} \qquad (1)$$

where C is the gas concentration, Z is the elevation difference of the sample, t is time period and ε is the soil-air content. Generally diffusion is represented as a ratio of gas diffusion coefficient of soil (D_p) with respect to gas diffusion coefficient in free air (D_o). This is termed as soil-gas diffusivity (D_p/D_o).

Since determination of the gas diffusion in laboratory is time consuming and difficult, researchers have introduced mathematical formulae to determine the gas diffusivity as a function of soil type and the soil-air content (ε). Buckingham(1904) suggested that the soil-gas diffusivity follows a power law function of the soil-air content as,

$$\frac{D_p}{D_0} = \varepsilon^x \qquad (2)$$

where*x* is an exponent characterizing pore connectivity-tortuosity (Buckingham, 1904). For this simple model Buckingham (1904) suggested $x = 2$ whereas Marshall (1959) suggested x = *1.5*. Penman(1940) further modified the model proposed by Buckingham and proposed a relationship between soil-gas diffusivity and soil-air content as,

$$\frac{D_p}{D_0} = 0.66\varepsilon \qquad (3)$$

1.1.2 Advection

Advection is a gas transport mechanism induced by soil-air pressure gradient. To model the advection flow, a parameter called coefficient of air permeability (k_a) is used. Air permeability coefficient is computed based on Darcy's law as,

$$q = -\frac{K_a}{\eta}\left(\frac{dP}{dz}\right) \quad (4)$$

where *dP/dz* represents the pressure gradient across the sample, *A* is cross sectional area of the soil sample, η is the dynamic viscosity of the air (1.86 x 10^{-5} Pas) (Wickramarachchi*et* al, 2010).

In general existing predictive models for air permeability are also based on a power law function of soil-air content (ε), similar to air diffusion. The generalized form of such model proposed by Buckingham (1904) can be written as,

$$K_a = \alpha\varepsilon^x \quad (5)$$

whereα is a constant to pore connectivity and *x* is a power law exponent labelled as a water blocking factor for k_a. Buckingham (1904) suggested *x= 2* and Marshall (1959) suggested *x = 1.5* based on several laboratory experiments, Kawamoto *et* al (2006) suggested that α =700.

2. MATERIALS AND METHODOLOGY

Laterite soil commonly available in dry zone of Sri Lanka was selected as the candidate material for this study. Physical properties of laterite soil are presented in Table 01.

For sample preparation 2.36 mm sieved soil was used and compaction was done in 100 cm^3 molds. By adding different sand percentages by 5% increments (5%, 10%, 15%, 20%, 25% and 30% by soil weight) to natural laterite soil, different soil sand mixtures were prepared. Then compaction curves were developed for each and every soil sand mixture. The compaction curves of soil-sand mixtures are depicted in Figure 1.

Table 01 Physical properties of laterite soil

Property	Value
Liquid Limit (%)	56
Plastic Limit (%)	30
Plasticity Index (%)	26
Specific Gravity	2.67
OMC (%)	23
Maximum dry density (kN/m^2)	16.43

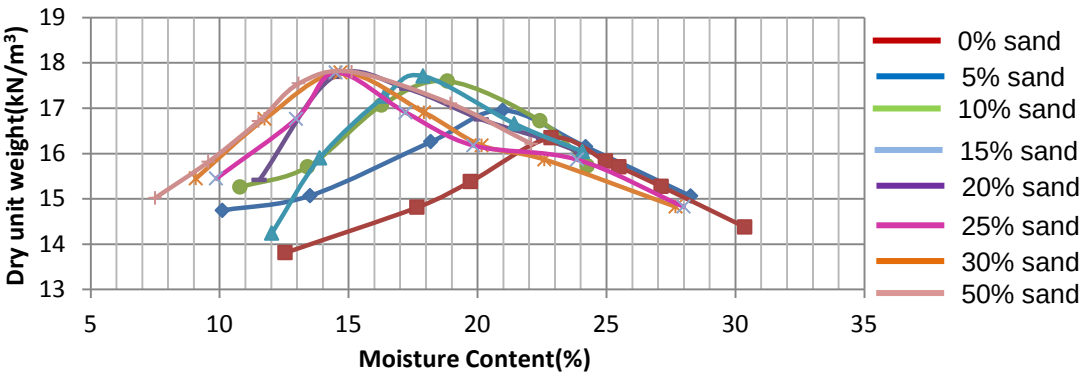

Figure 1 Compaction curves for different soil-sand mixtures of laterite soil

In order to check gas transport parameters, three samples from dry side and three samples from wet side were taken for a particular soil sand mixture.

2.1. Determination of Gas Transport Parameters

k_a was measured in the laboratory using air permeability setup illustrated in Figure 2, in which air is flowing at a given inlet pressure through soil samples .Using the pressure gauge, pressure difference between top and bottom of the sample was measured to calculate k_a value based on Darcy`s law (Curie, 1960).

D_p was measured using the diffusion chamber method as illustrated in Figure 3. Initially the closed chamber was flushed with Nitrogen and all the oxygen inside the chamber was removed by opening the outlet.Then the soil sample was fixed to the apparatus keeping the top surface of the sample open to the atmosphere. Once the chamber was opened atmospheric air is diffused to the chamber through the soil sample. Oxygen was used as the tracer gas. Change of oxygen concentration was measured as a function of time using the oxygen sensor connected to the diffusion chamber through a data logger, and then D_p was calculated (Wickramarachchi *et* al., 2010). In this study, the gas diffusion coefficient of oxygen in free air (D_o) at 20^0 C was taken as 0.20 cm^2/s(Curie, 1960).

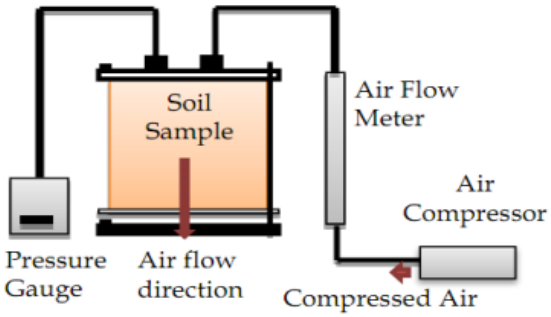

Figure 2 Air permeability setup

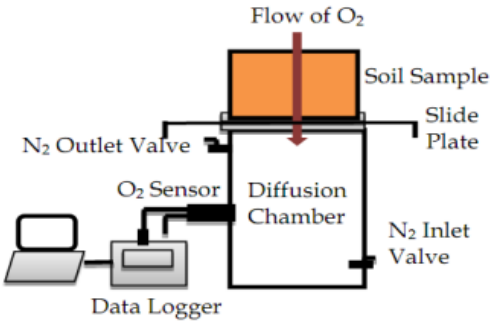

Figure 3 Gas diffusion setup

3. RESULTS AND DISCUSSION

3.1. Effect of Soil Air Content on Soil Gas Diffusivity

The measured diffusivity (D_p) was normalized with respect to diffusivity in free air (D_o) and variation of D_p/D_o over air content (ε) is illustrated in Figure 4.

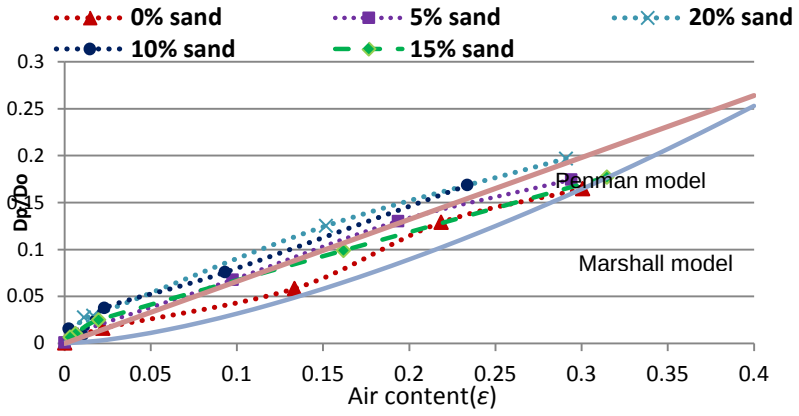

Figure 4 Soil-gas diffusivity vs. air content

It can be seen that D_p/D_o increases with ε irrespective of sand content. However, when sand content of cover material increases, D_p/D_o increases. This implies that, D_p/D_o is having an effect of by soil structure. Further, test results were compared with Marshall and Penman models (Wickramarachchi *et* al, 2010). It can be seen that natural laterite soil is agreed with Marshall model where as amended soil with sand is agreed with Penman model.

It is well understood that moisture content is significantly affected on the air presence in the compacted soil. Compacted soil in the dry side of OMC has higher air content than that of wet side of OMC. The variation of D_p/D_o with moisture presence in both dry and wet side of OMC is illustrated in Figure 5.

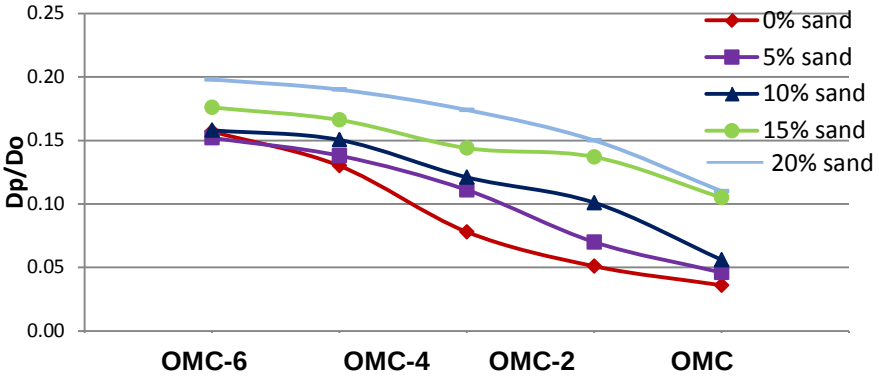

Figure 5 Soil-gas diffusivity vs. OMC

It can be seen that D_p/D_o significantly increases in the dry side of OMC whereas D_p/D_o gradually decreases in the wet side of OMC irrespective of sand content. Under most dry condition, soil amended with 20% of sand depicts the highest diffusivity.

3.2. Effect of Soil Air Content on Air Permeability

The variation of coefficient of air permeability (k_a)of different soil sand mixtures as a function of soil-air content (ε) is illustrated in Figure 6. Further, results have been compared with Marshall model (Wickramarachchi*et* al, 2010).

It can be noted that $k_a$value has been increased in the dry side of optimum, when ε values are high, irrespective to the sand content. Especially when sand percentage was increased beyond 20% a significant increase can be noted. This is due to the reduction of tortuosity of soil with addition of sand (Curie, 1960). This implies that k_a is highly affected by soil structure.

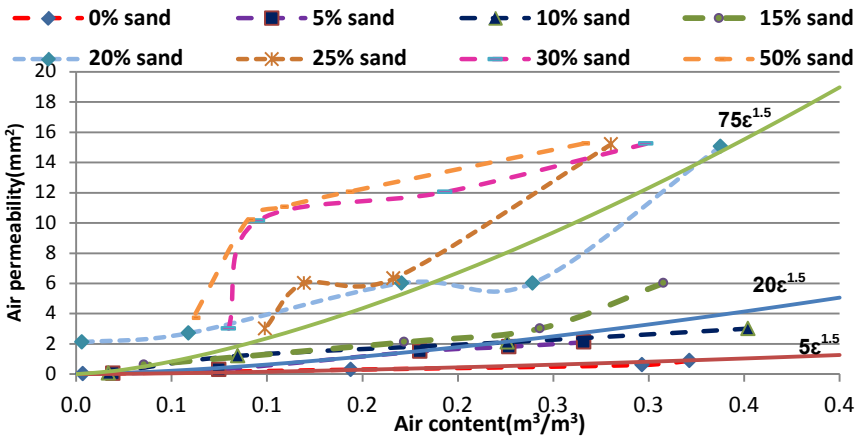

Figure 6 Variation of k_a with ε

Based on the test results, it can be seen that air permeability has been significantly increased with the addition of sand. Even though air content of the soil increases, there is not any improvement in air permeability in all samples up to a certain extent of the soil-air content.

After this particular point, air permeability of all soil samples has been significantly increased. This particular point is identified as OMC of the particular soil sample. Based on these results, it is very clear that there is no any noticeable improvement in air permeability in wet side of OMC whereas there is a significant improvement in air permeability in dry side of OMC with the increase of air content. Further, this behaviour is clearly shown in Figure 7, where variation of air permeability over moisture content is presented.

It can be observed from the graph that air permeability in dry side of OMC is significantly higher than that in wet side of OMC. With the addition of sand, total porosity of the material is increased, which enhance the air permeability. This implies that air permeability is strongly affected by soil structure.

Further, test results were compared with Marshall model (Equation *(5)* in section 1.1.2) in Figure 6. It can be seen that natural laterite soil is agreed with Marshall model when α=5 where as amended soil with sand (5 %, 10% and 15%) is agreed with Marshall model when α=20.

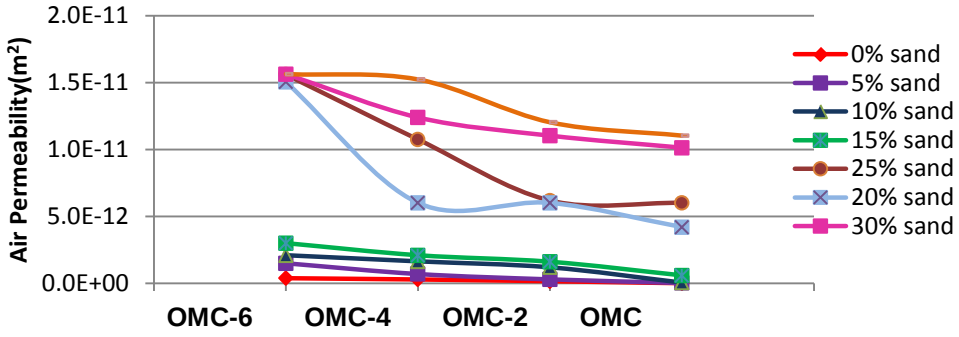

Figure 7 Variation of k_a with OMC

4. Relationship Between Gas Transport Parameters and Saturated Hydraulic Conductivity

Based on the above study, it was realized that gas transport parameters of cover material is significantly improved when the cover soil compacted in dry side of OMC. However, according to Central Environmental Authority (CEA) guidelines (CEA,1992), landfill cover materials should be satisfied the hydraulic conductivity requirements, i.e. hydraulic conductivity should be less than 1 x 10^{-7} m/s. Therefore a relationship has been developed between gas transport parameters and hydraulic conductivity, in order to select the most suitable landfill cover material for dry zone.

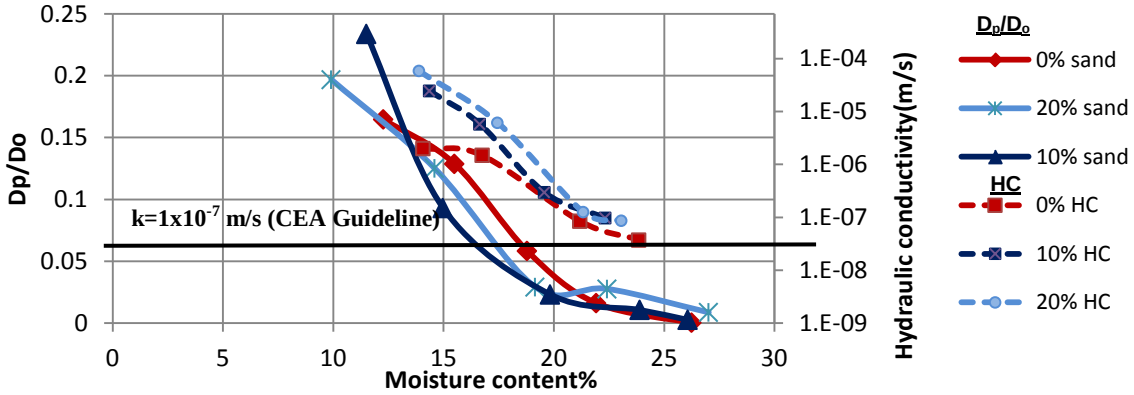

Figure 8Variations of D_p/D_o and k with Moisture Content

Variation of soil-gas diffusivity and hydraulic conductivity over moisture content is shown in Figure 8, whereas variation of air permeability and hydraulic conductivity over moisture content is shown in Figure 9. Further, effect of addition of sand on gas transport parameters and hydraulic conductivity is presented. The landfill cover material requirement with respect to hydraulic conductivity (i.e. 1 x 10^{-7} m/s) are also presented in Figure 8 and Figure 9.

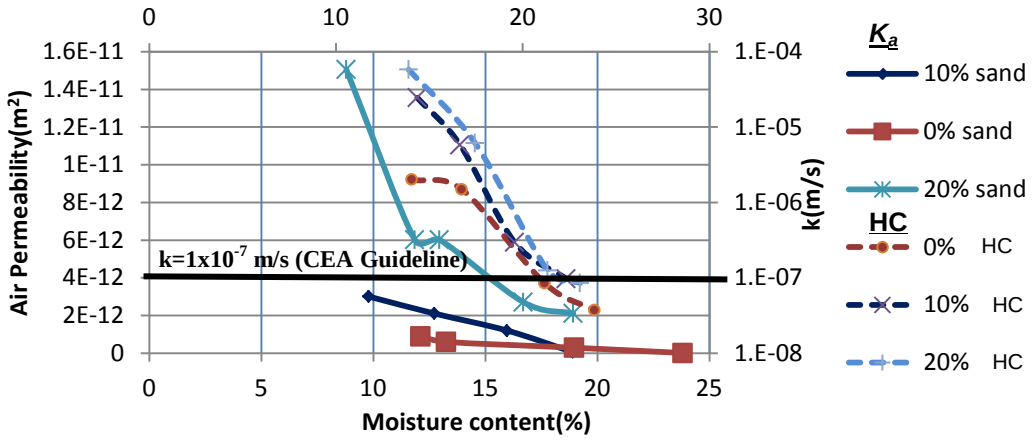

Figure9 Variation of k_a and k with Moisture Content

Based on the results, it can be seen that both gas transport parameters and saturated hydraulic conductivity have been decreased with increase of moisture content. It is very clear that, with the addition of sand to cover material, both saturated hydraulic conductivity and gas transport parameters have been increased.

5. CONCLUSIONS

Evapotranspirative cover system was identified as the most suitable final landfill cover system for dry zone of Sri Lanka. Series of laboratory experiments were conducted in order to investigate gas transport parameters and hydraulic conductivity of cover material. Based on the results, it was realized that both $k_a$and $D_p/D_o$are increased with soil-air content. In other words, $k_a$and D_p/D_o are significantly increased in dry side of OMC of compaction irrespective of sand content in the cover material.

In order to satisfy CEA guidelines of hydraulic conductivity of the final cover material, 1 x 10^{-7} m/s (CEA, 1992) unamended laterite soil should be compacted in OMC and wet side of optimum. In the range of 2-4% less than OMC, hydraulic conductivity of the soil is matching with the required regulations. Thus this region can be identified as the optimum region when designing the landfill final cover in dry zone of Sri Lanka considering both gas transport parameters and hydraulic performances. According to the standards, cover thickness can be selected as 60cm.

If gas transport parameters were not matched with the required hydraulic conductivity standards, cover thickness can be increased. Hence the head loss can be increased and hydraulic conductivity can be reduced as required.

6. ACKNOWLEDGMENTS

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experiments is greatly acknowledged.

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Development of a Finite Element Method to Evaluate Slope Stability of Municipal Solid Waste Landfills using Probabilistic Approach

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Abstract: *In Sri Lanka the most sought after method of disposal of Municipal Solid Waste (MSW) is unfortunately, open dumping. Unplanned dumping of MSW has resulted in disastrous shear failures. The purpose of this study is to determine shear strength parameters of MSW and to use it in proposing a method to evaluate slope stability for design of MSW landfills in Sri Lanka. Landfilled solid waste samples were collected from dump site at Hambantota, a rapidly developing area. Shear strength parameters tested by direct shear apparatus produced cohesion ranging from 14kPa to 74kPa while the friction angle ranged between 25° and 48°. Reliability analysis via response surface method was adopted to evaluate failure probability of various slopes. Failure probabilities below 2% were observed for slopes ranging from 3:1 to 0.5:1.*

Keywords: *Finite Element Method, Municipal solid waste, Reliability analysis, Shear strength*

1. INTRODUCTION

Municipal Solid Waste (MSW) generation has increased in mammoth proportions in the past few decades. According to Hoornweg *et* al (2013) based on the World Bank review of solid waste management, global city residents (3 billion people) and commercial institutions dispose of staggering 3.6 million tonnes of MSW per day. Furthermore, the percentage of city dwellers is expected to increase by 70%~80% in the next few decades worsening the MSW amount to be managed. As a result, current average daily per capita MSW generation of 1.2 kg is further expected to increase with rapid urbanization of the globe.

Cities in low to middle lower income countries tend to have collection rates below 50% of waste generation, most of which is the waste that is not recovered for recycling. It is well known that the most common disposal method for collected waste in these countries is open dumps. Waste disposal in developing countries such as Sri Lanka, has not been given any significant attention over the past few decades. This had lead to unregulated disposal of solid waste primarily in urban and suburban areas. Lack of attention given to solid waste disposal had lead to the poor practice in disposing of solid waste in open dumps. Sri Lanka is among the largest MSW producing countries in terms of daily per capita generation.

Open dumps are the primary mode of disposing MSW in the country due to the lack of technical knowledge and financial constrains encountered in solid waste management. In addition, lack of sufficient land has lead to dumping waste in high mounds and in steep slopes resulting in catastrophic failures in MSW slopes.One example for such failure can be taken as the Payatas Landfill in Manila, Philippines which was subjected to a shear failure in the year 2000 where 1.2 million m^3 of waste of

the top layer slid down the slope causing 230 reported deaths and over 800 people reported missing (Jafari *et* al., 2013)

An example for one such recent failure in Sri Lanka can be taken from the 80foot high Bloemendhal open dump in Maligawatta, Sri Lanka which took place in mid 2009. Although, no harm for human life was recorded, the shear failure of the dump resulted in a considerable amount of property damage to the nearby dwellings in the adjoining area. Evaluation of MSW landfill slope stability in Sri Lanka has not been previously done and it is a vital component which needs to be fulfilled for the design of engineered landfills.

Unlike soil, shear strength parameters of MSW tend to change over time and with location due to bio-chemical degradation taking place within the waste body (Zhan *et* al, 2008; Reddy *et* al, 2009). As a result evaluation of slope stability of MSW landfills has proven to be a tedious and unreliable task if done by adopting conventional Factor Of Safety (FOS) method. Considering the high variability of shear strength parameters of MSW landfills a probabilistic approach was adopted in this study. To evaluate the stability of slope a methodology incorporating the uncertainties involved in determining shear strength characteristics of landfilled MSW, which can be adopted in determining slope stability of MSW landfills, is presented in this paper.

2. SAMPLE COLLECTION

Waste dump site at Hambantota Municipal Council, a rapidly developing area, was selected as the study area to obtain representative samples of a landfill in the dry zone of Sri Lanka. Spanning across an area of 1 acre, the open dump site is located along Hambantota-Mattala main road. Dump site can be mainly categorized in to two zones with old and new waste. Schematic diagram of Hambantota Municipal Council waste dump site is depicted in Figure 1. Old dump areas are indicated by 'A' and 'B'. The old waste dump area indicated by 'A' is about 13 years in age and was not selected for sampling as it is outside the elephant fence. The old waste area indicated by 'B' is about ten years in age. The new waste area indicated by 'C' is less than two years in age. The area indicated by 'D' is allocated for future landfill and intact area 'E' is assumed to be uncontaminated with leachate.

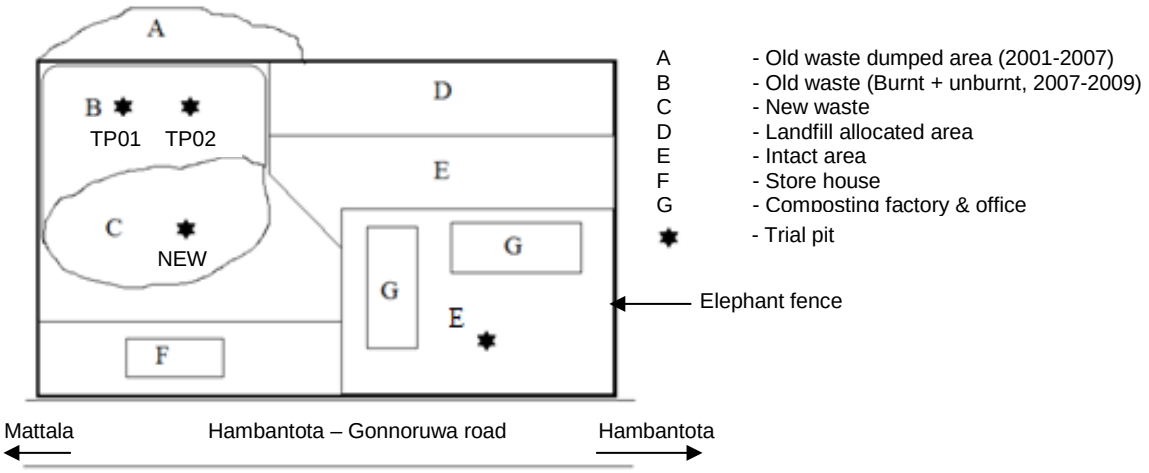

Figure 1 Layout of Hambantota dump site

MSW samples have already been obtained from both new and old dump locations as indicated in Figure 1 at depths of 0.5 m, 1.0 m and 1.5 m to represent variation of MSW characteristics with time. Two trial pits were excavated at the old dump and one trial pit at the new dump to obtain undisturbed samples by means of box sampling. At each test pit excavated in the waste area, samples were obtained at every 0.5 m depth interval up to a depth of 1.5 m starting from the waste surface. Beyond this depth in the parent stratum under the waste, soil samples were obtained at 0.5 m and 1.5 m depths starting from ground level. This procedure was followed for both new and old waste areas. From the intact soil, samples were obtained at 0.5 m depth. Two box samples were collected at each depth for all test pits.

Conventional techniques used in sampling cannot be used to collect undisturbed samples from the solid waste landfill in the presence of large particle sizes. Moreover, as waste consists of particles like polythene and plastics it is difficult to obtain undisturbed well representative samples using boreholes. Thus, a large scale 300 mm × 300 mm × 300 mm box was used to extract samples. First ground surface was smoothed and the steel box driven in to waste. Then a trench around the box was carefully excavated and the sample was cut from bottom layer.

3. LABORATORY TESTING PROGRAM

3.1. Composition of landfill solid waste

Initially MSW components were sorted as bio-degradable and non bio-degradable by visual observation. The residual fines contained some inert fraction, but it was difficult to quantify this by visual observations. Therefore, air dry samples were heated in the oven to 440 ^{0}C to determine the organic content of MSW. The solid waste material obtained had a higher percentage of fine particles. Majority of the solid waste consisted of fine particulate matter which were indistinguishable. Thus, in advance to sorting the solid waste sample was sieved through a 4.75 mm standard sieve to separate the fine component. A portion of intact samples from box sampling was oven dried at 60 ^{0}C to determine the natural moisture content. Shown in Figure 2 is the variation of Composition of MSW with respect to location and age.

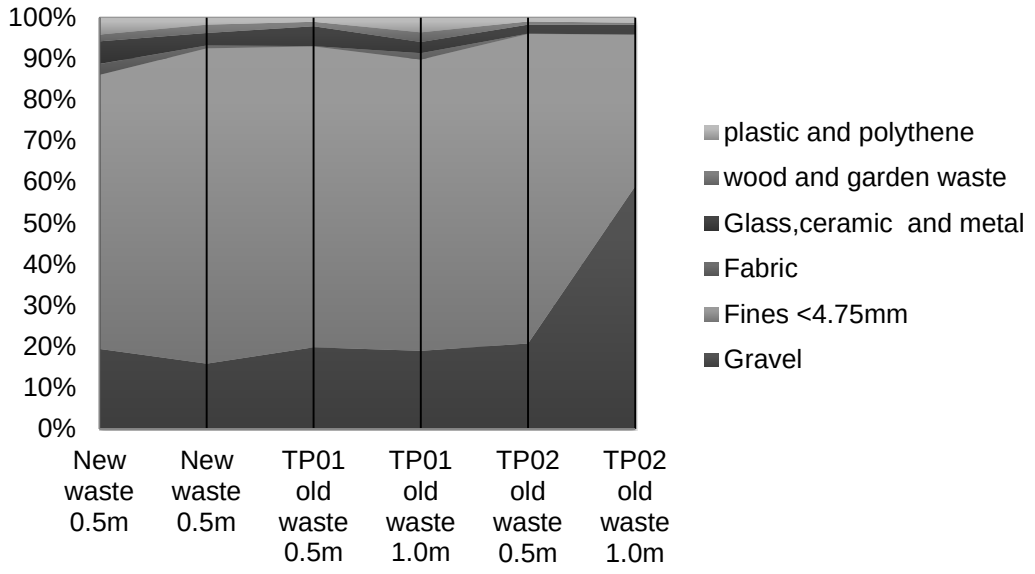

Figure 2 Variation of MSW composition with age and location

3.2. Compaction Characteristics

Conducting large scaled testing for MSW samples was not available in this study and direct shear tests conducted were limited to a maximum diameter of 100 mm and 45 mm depth. Thus, MSW samples had to be shredded and remolded in order to conduct direct shear testing. For this purpose, compaction curves were developed by conducting standard Proctor compaction tests for shredded MSW samples of 4.75 mm maximum particle size. Maximum particle size of 4.75 mm was used for two main reasons. First reason is to eliminate the obstruction for movement of components in the direct shear apparatus by limiting maximum particle size to about one tenth of the smallest dimension of the sample prepared. This particle size is further justified because as shown in Figure 2 nearly 60% of the waste mass consists of particles which are less than 4.75 mm. Figure 3 illustrates the compaction curves developed for different solid waste samples. Higher dry unit weights were

observed for compacted wastes. Values obtained in this study are highly unlikely compared to maximum dry unit weight of 5.9 kN/m^3 obtained by Reddy *et* al (2009). Unlike typical MSW (with larger proportion of plastic, paper and organic waste), samples obtained from Hambantota consisted of a higher percentage of gravel and sandy material which can be explained as the reason for such high compacted dry unit weights.

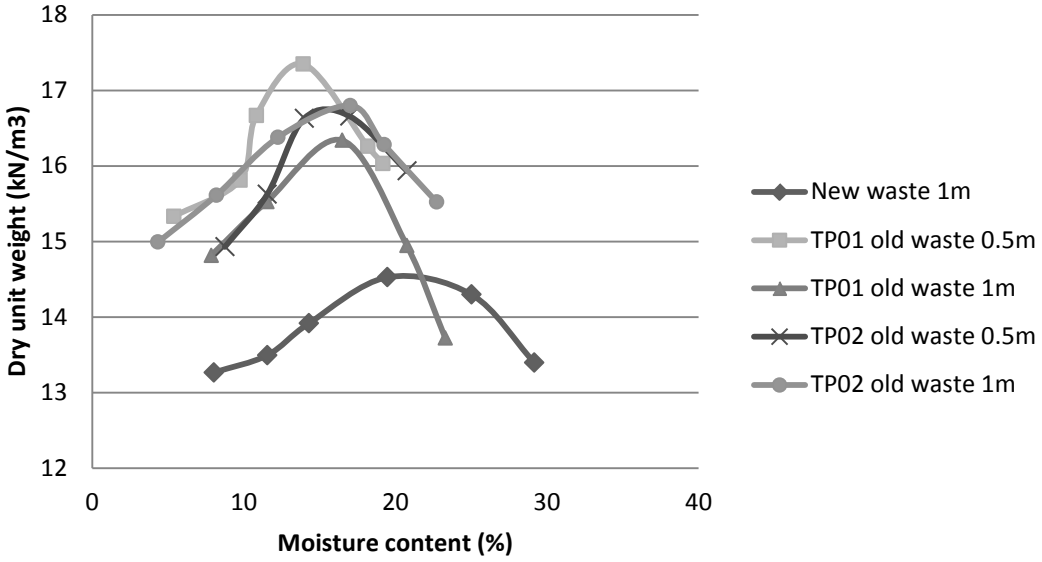

Figure 3 Compaction curves developed for MSW

3.3. Direct Shear Test

3.3.1. Sample Preparation

Shear strength parameters for MSW samples collected was conducted using a direct shear test apparatus. Because the Shear box has a diameter of 100 mm and a depth of 45 mm particles had to be shredded and recompacted in to the mould in order to be sheared. Similar methodology was adopted by Reddy *et* al. (2009) and Zekkos *et* al. (2010). Specimens were prepared by air drying the waste samples until a constant weight was observed and shredded into particle size no more than 4.75 mm. Air drying was adopted to avoid deformation of particles which would have resulted if oven drying was used. The dried waste was prepared with optimum moisture content (OMC), OMC+3%, OMC+6%, OMC-3% and OMC-6% to obtain representative samples from the entire compaction curve.

3.3.2. Shearing of samples

All direct shear tests were conducted under drained condition to obtain effective shear strength parameters. Samples were sheared at different vertical stresses of 25 kPa, 50 kPa and 75 kPa. For locations where soil was present under waste layers undisturbed samples were extruded from a circular cutter and sheared under the same vertical stresses.

Where samples showed no peak shear stress, the stress at 15 % of shear strain was obtained to produce the Mohr-Coulomb failure envelope. Similar methodology was adopted by Reddy *et* al (2009) in defining shear strength parameters for MSW and the concept of defining shear strength parameters at strain levels considered critical was further mentioned by Canizal *et* al (2011). Methodology suggested by Zekkos *et* al (2010) was adopted in conducting staged direct shear tests. Shown in Figure 4 is the variation of Effective Cohesion (C') and Effective Friction Angle (ϕ') with respect to moisture content at different locations.

Both friction angle and cohesion showed an erratic behaviour with respect to moisture content. Also no pattern in the variation of new and old MSW was observed in the summarized results in Figure 4. This is an indication of the unpredictability of shear strength Parameters of landfilled MSW. The effective cohesion ranged from 14 kPa to 74 kPa whereas the friction angle fluctuated between 25° and 48°. Reason for such extreme variation of shear strength parameters are due to the inherently heterogeneous nature of MSW composition. These results further reinforce the need to resort to a probabilistic approach in evaluating the slope stability of landfills.

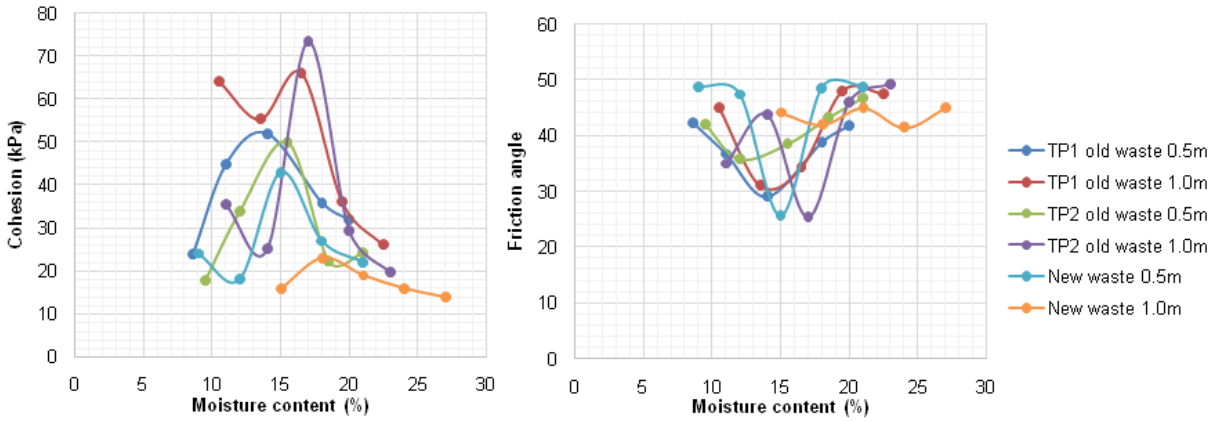

Figure 4 Variation of shear strength parameters with moisture content

4. SLOPE STABILITY ANALYSIS

The results obtained by laboratory testing was used to evaluate the slope stability in varying slopes of 0.5:1, 1:1, 2:1 and 3:1. For the purpose of evaluation, Reliability Analysis via Response Surface Method proposed by Xu and Low (2006) for embankments was adopted. Finite Element Modeling was adopted to evaluate the slope stability of each design slope angle.

4.1. Reliability Analysis Based on Finite Element Method

The conventional methods adopted in evaluation of slope stabilities of embankments cannot be used in the context of MSW landfills, simply because the values obtained for the factor of safety will be inconsistent and will not provide a comprehensive measure of risk. The reasons for the inconsistencies can be due to changes in shear strength parameters, errors in testing and environmental conditions. These uncertainties can be accounted for and a more consistent measure of slope stability can be obtained by resorting to a probabilistic analysis method (Xu and Low, 2006). The failure probability of a slope is given by equation (1).

$$p_f = P[G(X) \leq 0] = \int_{G(X)\leq 0} f(X)dx \quad (1)$$

Where, *G(X)* is the performance function defined by;

$$G(X) = F(X) - 1 \quad (2)$$

Where, *F(X)* is the function for the factor of safety and *f(X)* is the probability density function of the basic variable vector, X. Due to the difficulty in identifying and integration of the probability function an approximate 2nd order polynomial function is defined for the performance function. Reason for defining a second order polynomial function and the procedure for reliability analysis is further elaborated by Low (1998) and Xu and Low (2006). The second order polynomial function which defines the approximate performance function can be written as;

$$G'(X) = l + \sum_{i=1}^{r} m_i x_i \sum_{i=1}^{r} n_i x_i^2 \quad (3)$$

Where $x_i, i=1,.....,r$ are random variables, l, m_i, n_i are parameters which need to be determined. For a function with n number of random variables 2n+1 number of unknown coefficients need to be determined to define the performance function.

4.2. Modeling of Two Layered Slope and Performance Function

For the evaluation of slope stability of the landfill two layered model was created using PLAXIS finite element modeling software. As indicated in Figure 5 a waste layer of 5 m and a soil layer of 6m were considered. This selection was done based on site observations where the parent stratum was to be homogeneous to a depth of 6m and the waste layer with the current composition is not likely to reach beyond 5m in height. The cohesion, friction angle and the bulk density of each layer, altogether 6 parameters were considered as random variables which govern the factor of safety. Thus, the performance function for the factor of safety consisted of 13 (2x6+1) terms which meant 13 unknown coefficients had to be determined to define the performance function G(X).The input data set and the FOS obtained for each slope angle using PLAXIS Finite Element Software is shown in Table 1. Shown in Figure 6 are the incremental displacements obtained by PLAXIS, of the four different slopes at the critical stage.

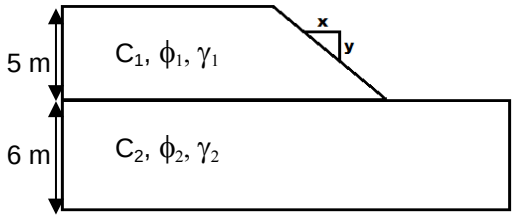

Figure 5 Two layer slope model for landfill

Table 1 Input sampling points for performance function based on Finite Element Method

Set	Input variables						FOS			
	C_1 kPa	ϕ_1	γ_1 kN/m^3	C_2 kPa	ϕ_2	γ_2 kN/m^3	0.5:1 Slope	1:1 Slope	2:1 Slope	3:1 Slope
1	18	47.48	17.69	30	21.8	15.85	2.24	3.008	3.802	4.393
2	23	41.98	16.99	36	49.7	17.25	2.709	3.506	4.714	6.112
3	27	48.57	18.70	49	36.9	16	2.734	3.683	5.474	6.816
4	16	41.42	17.85	30	21.8	15.85	1.877	2.547	3.58	4.199
5	43	25.64	18.4	36	49.7	17.25	2.841	3.695	5.169	6.568
6	45	36.75	18.53	32	35.75	18.00	3.25	3.972	4.994	5.954
7	55.24	31.02	18.16	32	35	14.92	3.522	4.145	5.031	5.851
8	36	38.86	19.17	24	39	17.435	2.824	3.473	4.514	5.457
9	36	48	18.88	32	35.75	18.00	3.217	3.692	5.039	5.968
10	33.8	35.75	17.69	32	38.67	17.11	2.754	3.588	4.917	6.053
11	25	43.7	19.04	41	47	15.64	2.409	3.242	4.77	6.184
12	22.29	43.2	19.43	14	35	16.96	2.171	2.703	3.5	4.258
13	29.27	45.99	19.2	32	38.67	17.11	2.721	3.612	4.877	5.91
mean, m_i	**31.51**	**40.64**	**18.44**	**32.31**	**37.29**	**16.72**	**2.71**	**3.45**	**4.64**	**5.67**
Std. Deviation, σ_i	**11.44**	**6.89**	**0.72**	**8.15**	**8.68**	**0.96**	**0.46**	**0.46**	**0.63**	**0.86**

The calculated factors of safety and corresponding input variables were used to construct a response surface G(X) which represents the performance function for each slope considered in the analysis. Simple matrix multiplication can be used to solve the coefficients for the function for the Factor of safety. It should be noted that the response surface in this case is a six dimensional function which passes through all thirteen points selected. Using the performance function defined the reliability index is calculated by using the First Order Reliability Method as explained by Low (1996).

4.3. Reliability Index and Probability of Failure

The reliability index is defined as the minimum distance from the point of mean values of the random variables to the boundary of the limit state function in units of directional standard deviations. The reliability index β, is given by equation (4).

$$\beta = min_{X=F}\sqrt{\left(\frac{X_i - m_i}{\sigma_i}\right)^T (R)^{-1} \left(\frac{X_i - m_i}{\sigma_i}\right)} \quad (4)$$

Where R = correlation matrix, m_i = mean of the random variable and σ_i=standard deviation of random variable x_i. The reliability index is calculated based on the limit state function when, G(X) = 0. For the analysis of this study the correlation matrix was considered to be a 6 x 6 identity matrix because the parameters had no correlation with one another. Microsoft Excel's built-in Solver optimization tool can be used to minimize the reliability index (Low, 1996). Based on the reliability index the probability of failure, P_f can be obtained by the equation (5).

$$P_f = 1 - \phi(\beta) \quad (5)$$

Where ϕ = Cumulative Distribution Function (CDF) of the standard normal variable. The random variables should be normally distributed and the limit state surface should be planar. The failure probability used here means the probability that the performance function *G*(X) =*F*(X) −1 is equal to 0.

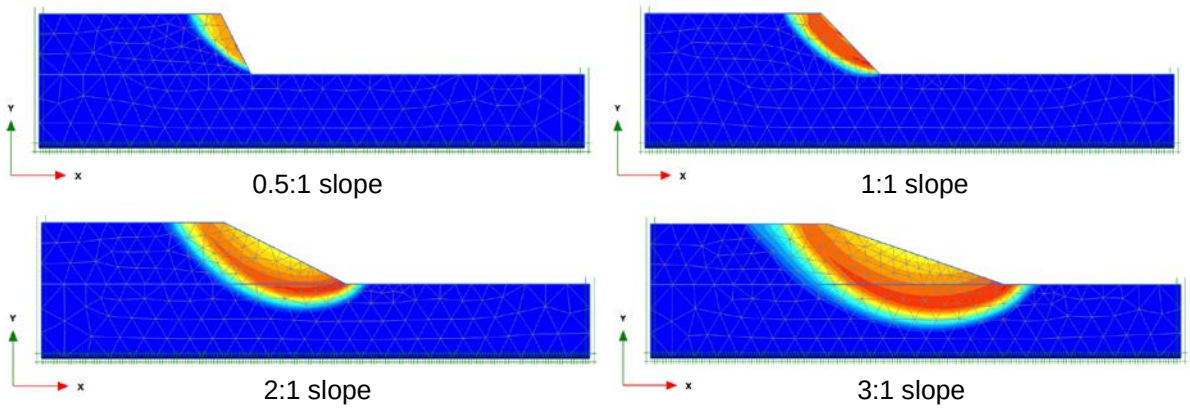

Figure 6 Incremental displacements of slopes at critical stage

Summarized in Table 2 below are the values obtained for the reliability index and corresponding probability of failure for each slope angle. According to the results a clear trend can be observed where the failure probability is increased from 0.001% to 1.995% with the increasing slope angle. According to Gibson (2011) the evaluated slopes can be categorized as follows;

- Slope 3:1 and 2:1 - P_f < 0.5%, for very long term slopes (no monitoring required)
- Slope 1:1 and 0.5:1 - P_f 1.5% to 5%, for semi permanent medium term slopes (incidental superficial monitoring required)

Table 2 Variable values, reliability index and probability of Failure

Slope angle	Variable value at critical point						β	P_f %
	c_1	ϕ_1	γ_1	c_2	ϕ_2	γ_2		
3:1	9.28	41.38	18.42	19.29	7.37	16.64	4.27	0.001
2:1	18.8	46.47	18.3	1.048	45.27	16.6	4.19	0.004
1:1	24.2	34.86	18.84	32.72	34.62	18.41	2.14	1.626
0.5:1	10.3	36.39	18.44	28.62	40.84	16.52	2.05	1.995

5. SUMMARY AND CONCLUSIONS

Based on the laboratory experiments and the results obtained from FEM analysis, following conclusions can be drawn;

- Shear strength parameters showed erratic behaviour for samples tested while direct shear test results proved to be very high. The reason behind such high cohesion and friction angle values can be explained by the fact that solid waste tested had a high content (about 60% by dry mass) of fine particles (<4.75 mm) and gravel (about 20% by dry mass)
- Slope stability of MSW landfills must be evaluated by adopting a probabilistic approach. Conventional FOS methods only consider a single set of input data when producing a factor of safety for a slope. Thus, variation in shear strength parameters of the waste layer and uncertainties involved in conducting tests are not accounted for. By using the reliability analysis via response surface method, the uncertainties involved with the spatial distribution of landfilled MSW shear strength parameters can be accounted for. The proposed model can be easily adopted in evaluation of slope stability of MSW landfills in the country and is compatible with various deterministic stability methods (both finite element modelling and limit equilibrium method) with readily available software without the need for additional coding.
- The method can be extended for analysis in the wet zone of the country incorporating pore water pressure within the waste mass as another random variable, enabling the evaluation of stability of a solid waste slope under saturated conditions. This will further increase the number of sample points required to determine the performance function for the factor of safety. It is recommended to conduct testing for waste samples with larger particle sizes depending on the average particle size of the waste body to obtain more accurate shear strength parameters as input variables.

6. ACKNOWLEDGMENTS

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Costal and Lagoon Environment

Applicability of Salinity Stratification Estimation by New Bulk Model for Two Choked Coastal Lagoons in Sri Lanka

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Abstract*: Two coastal lagoons in Sri Lanka, Koggala and Rekawa were studied to examine the applicability of a new bulk model to estimate relative stratification of choked lagoons. Two lagoons selected for the study represent two subcategories of choked coastal lagoons; permanently open choked coastal lagoons (POCCLs) and intermittently open and closed choked coastal lagoons (IOCCCLs). Application results for Koggala Lagoon demonstrate that δS/S estimation capability of the new bulk model is superior to Fischer's model, suggesting that the new bulk model would be suitable to apply for POCCLs. On the other hand, when the model was applied to Rekawa Lagoon, it demonstrated poor δS/S estimation capability compared to the POCCL case. This might have caused due to the variation in the time scale required for the IOCCCLs to reach the quasi equivalent state related to mixing process as the quasi equivalent state assumption is trivial in R_{iL} derivation.*

Keywords*: Coastal lagoons, salinity stratification, Estuarine Richardson Number, wind induced mixing, Lagoon Richardson Number, Quasi-equivalent state*

1. INTRODUCTION

Sustainable use of coastal lagoons in Sri Lanka is becoming essential as the lagoon environments are being rapidly and indiscriminately exploited by anthropogenic activities over the recent decade (IUCN, 2006). A comprehensive knowledge on the various aspects of coastal lagoons, including their physical process is a prerequisite for their sustainable use while preserving their naturalness. One such important physical process is the vertical mixing/stratification process and the level of stratification of coastal lagoons due to saline water intrusion. These can be crucial in determining the changes on the inherent nature of coastal lagoons as the vertical fluxes of water properties such as dissolved oxygen and nutrient elements depend on them (Simpson et al, 1990).

Simple bulk models could be useful for estimating the degree of stratification of coastal lagoons as they can be easily applied with minimum field requirements, unlike numerical models. These models become more useful in situations where there are no proper systems to monitor large number of hydrologic/riverine parameters on a regular basis, such as what is commonly experienced in coastal lagoons in Sri Lanka. Despite the controversies over the identification of coastal lagoons from estuaries, the number of coastal lagoons situated along the entire coast of Sri Lanka is estimated to

be 45.

Coastal lagoons have been considered as a class of estuaries in many estuary classification schemes (Pritchard 1955; Hansen and Rattray, 1966) as they share the common characteristics such as having one or many restricted connection to the ocean and both of which the water circulation is driven by tides, river discharge, wind stress and heat balance. Yet, there is a considerable difference among the two types with respect to their geomorphologies which cause the two types to respond unequally. The typical planner view of a coastal plain estuary could be approximated by a funnel shape where the width of the estuary gradually decreases towards the upstream. On the other hand, typical planner view of a coastal lagoon would be a larger idealized rectangular or elliptic shape representing the main water body which connects with relatively smaller idealized rectangular shapes representing the lagoon mouth channels. However, this study focuses only on choked coastal lagoons which are one of the coastal lagoon types defined by Kjerfve (1989). According to Kjerfve, choked coastal lagoons are usually characterized by a long and narrow entrance channel where the ratio of the entrance channel cross section to surface area of the lagoon is small. Additionally, the long residence times, wind forcing and domination by the hydrologic/riverine cycles are some of the other characteristics of choked coastal lagoons (Kjerfve, 1989).

This paper aims on examining the applicability of a new bulk model (Perera et al, 2015) to estimate the relative stratification ($\delta S/S$) of choked coastal lagoons in Sri Lanka. The new bulk model used in this study is a modified version of Fisher's model (Fischer, 1972) which was originally developed to estimate the relative stratification of estuaries. In this new version of the model, one of the bulk parameters included in Fisher's model, the Estuarine Richardson Number ($R_{iE,}$) was replaced by a new bulk parameter, the Lagoon Richardson Number ($R_{iL,}$) which is a theoretical enhancement of $R_{iE.}$ The significant difference between these two parameters R_{iE} and R_{iL} is the inclusion of wind induced mixing energy in R_{iL} as such it would characterize the mixing process of choked coastal lagoons more ideally than R_{iE} which was originally derived to characterize the mixing process of estuaries.

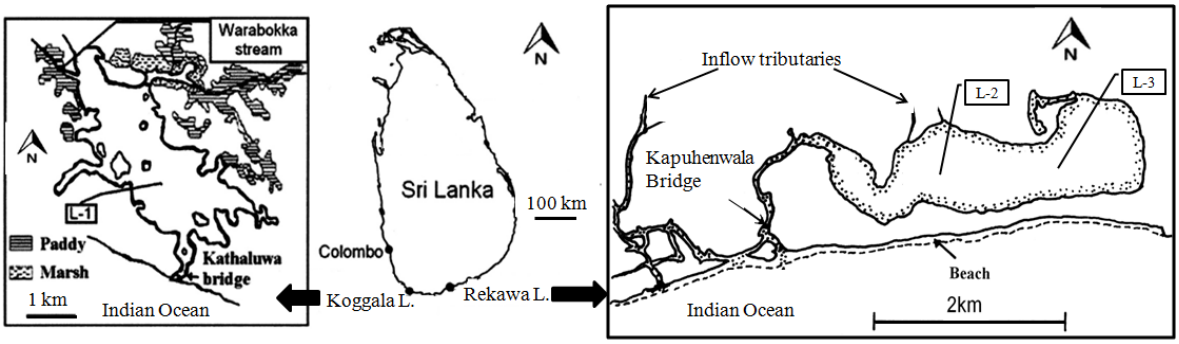


Figure 1 Map of Koggala and Rekawa Lagoons

2. MATERIALS AND METHODS

2.1. Sites Description

Two coastal lagoons, Koggala Lagoon and Rekawa Lagoon (Figure 1),which are located along the southern coast of Sri Lanka, were selected for the case studies. These two lagoons were selected to represent two different subcategories of choked coastal lagoons, permanently open choked coastal lagoons (POCCLs) and intermittently open and closed choked coastal lagoons (IOCCCLs).Above two subcategories were formed based on the different natures of lagoon mouth conditions as the two names suggest. Out of the two lagoons Koggala Lagoon represents POCCLs while Rekawa Lagoon represents IOCCCLs.

2.1.1. Koggala Lagoon

Koggala Lagoon (5°58' 6° 20' N and 80°17' 80°22' E) is situated about130 km south from the country's trade capital, Colombo. Prior to 1995, Koggala Lagoon was an IOCCCL with a freshwater ecosystem. Recent anthropogenic activities such as unplanned sand removal and structural intervention at the lagoon mouth area turned Koggala Lagoon to a POCCL, consequently shifting its ecosystem towards more saline conditions. This transformation has generated various environmental and socio-economic problems (Priyadarshana et al, 2007). Having a waterway area of 7.27 km^2, Koggala Lagoon is approximately 4.8 km long and 2 km wide. The hydro-catchment area of Koggala Lagoon is approximately 55 km^2 consisting mainly homesteads and cultivated fields of paddy, coconut, tea, rubber and other plantations (Priyadarshana et al, 2007). The depth of the lagoon ranges from 1.0 to 3.7 m and it can vary according to seasonal changes in rainfall. Koggala Lagoon usually experiences a mean annual rainfall of between 2,000 and 2, 500 mm (IUCN, 2006). Although it receives freshwater influx from several streams, more than 88% of the stream water is received to the lagoon through Warabokka stream. Lagoon's main water body is enclosed with 14 islets and the largest one is located in the south east corner of the lagoon. Lagoon mouth channel that connects the lagoon with the sea is located at the south east corner. Two bridges that was built as a part of the Colombo-Matara main road and rail track lie across the lagoon mouth channel of which the approximate width closer to rail track bridge is about 100 m.

2.1.2. Rekawa Lagoon

Rekawa Lagoon (50° 58'N and 80° 47E) is situated about 200 km south from the country's trade capital, Colombo. The hydro-catchment area of Rekawa Lagoon is approximately 225 km^2 consisting of a large tract of paddy fields, mangrove and scrub forest and homestead (Rekawa special area management coordinating committee, 1996). Lagoon's surface extends up to approximately 1 km in width at the widest point and about 3.3 km in length along the shoreline, claiming to a surface area of 2.38 km^2 (Priyadarshana & Aseada, 2006). Kirama oya stream which connects with the lagoon mouth channel 0.7 km from the sea has been the main inflow tributary of the lagoon, especially during the mouth closed phase of the lagoon. Additionally, the lagoon receives freshwater from another inflow tributary which connects at downstream end of the lagoon's main water body from the landward side and operates mostly in rainy season. This significant positioning of inflow tributaries together with relatively deep 2 km long meandering lagoon mouth channel makes Rekawa Lagoon a unique coastal water body. Although Rekawa Lagoon claims to a relatively larger catchment area, the numerous irrigation structures constructed along the inflow tributaries of Rekawa Lagoon has been significantly reducing the freshwater flow into the lagoon (Rekawa special area management coordinating committee, 1996). Rekawa Lagoon mouth is periodically closed, especially in dry season, as the winds and constant waves on the shoreline give rise to dispositional sand dunes along the coast and occasionally opened in rainy season naturally or artificially by local community to prevent flooding when higher amount of rainfall is received to the catchment area. The water depth of the lagoon can vary depending on the seasonal variations in rainfall as well as the open or close state of the lagoon mouth. Kapuhenwala causeway constructed across the lagoon mouth channel in 1984 has been drastically altering the natural rhythm of the lagoon as it impedes the water circulation of the lagoon (Rekawa special area management coordinating committee, 1996).

2.2. Field Observations

Field observations were conducted to investigate salinity stratification characteristics of the two lagoons during the time period starting from November 2011 to February 2013 under different seasonal, tidal and mouth conditions (Table 1).

Vertical profiles of salinity, dissolved oxygen, water temperature and flow velocity were measured with 0.5 m vertical intervals at several points covering all the key parts of the lagoons, lagoon mouths, main water bodies and main inflow tributaries. However in this study, the data collected at the selected main representative survey point of the central parts of the lagoons are presented. The selected survey point to represent the central part of Koggala Lagoon was L-1 while it was L-2 for all the cases of Rekawa Lagoon, except for the case named R21May12DC where the data collected at L-3 point was

used (see Table 1). A water quality measuring equipment (multi probe) YSI Model 55 and KENEK VP 100 flow meter were used to measure the water quality parameters and flow velocity, respectively.

Table 1 Survey timing and conditions with corresponding survey case names

Lagoon	Season	Mouth	Timing	Case Name
Koggala	Rainy	Open	22Nov2011	K22Nov11RO
		Open	30Nov2012	K30Nov12RO
		Open	15Sep2012	K15Sep12RO
		Open	15Oct2012	K15Oct12RO
	Dry	Open	16Mar2012	K16Mar12DO
		Open	14Feb2013	K14Feb13DO
Rekawa	Rainy	Open	21Oct2012	R21Oct12RO
	Dry	Close	17Mar2012	R17Mar12DC
		Close	21May2012	R21May12DC
		Close	27Jan2013	R27Jan13DC

2.3. Application of the New Bulk Model

Fischer's model uses two bulk parameters which characterize a typical estuary to estimate its relative stratification ($\delta S/S$; the ratio of surface and bottom salinity difference to mean salinity of the considered cross section). Those two bulk parameters are R_{iE} and the Densimetric Froude Number (F_m) (Hansen & Rattray, 1966) which were defined as;

$$R_{iE} = \frac{(\Delta\rho / \rho) g (Q_f / b)}{U_t^3} \tag{1}$$

$$F_m = \frac{(Q_f / bd)}{\sqrt{gd(\Delta\rho / \rho)}} \tag{2}$$

where $\Delta\rho$ is the density difference between river inflow (ρ_l) and sea water (ρ_S), g is the acceleration of gravity, Q_f is freshwater inflow rate, U_t is the r.m.s tidal velocity, b is the width and d is the depth of the estuary. Here, R_{iE} specifically characterizes the balance between the two energy contributions for the mixing/stratification process of estuaries (the potential energy input by the tributaries and the kinetic energy input by the tide during a tidal period) while F_m determines the magnitude of the vertical circulation (Fischer, 1972). Considering the differences in the energy contributions for mixing process of estuaries and coastal lagoons, Perera et al (2015) replaced R_{iE} from a new bulk parameter, R_{iL}, and used it with F_m to estimate $\delta S/S$ for POCCL. In addition to the two energies considered in $R_{iE,}$ the energy input by the wind force is also constituted in R_{iL} as expressed in the following equation.

$$R_{iL} = \frac{\Delta\rho g Q_l d_{Ls}}{\rho_S (b_M d_M) U_t^3 + \sqrt{4 C_D{}^3 \frac{\rho_a}{\rho_w}} \rho_a (b_L l_L) U_h^3} \tag{3}$$

where Q_l is the inflow rate of the stream water along with the surface runoff from the catchment area of the lagoon, C_D is the drag coefficient, ρ_w is the density of water at the mean temperature of the lagoon, ρ_a is the air density, U_h is the wind speed at 10 m above the water surface. b_M, d_M, b_L and d_L are the mean width and mean depth of the lagoon mouth and lagoon, respectively. Additionally, d_{Ls} is the theoretical effective surface depth of the lagoon which is calculated using 99% of the flushing time of the lagoon (T_r), lagoon surface area (A_L) and Q_l according to the following equation;

$$d_{Ls} = (Q_l T_r / A_L) \tag{4}$$

The new bulk model with the two bulk parameters R_{iL} and F_m is applied to Koggala and Rekawa Lagoons in order to examine its applicability to choked coastal lagoons with two different lagoon mouth characteristics. The monthly mean inflow rate from the stream and catchment of Koggala Lagoon was

used as Q_I for calculating the bulk parameters included in the model. When calculating Q_I values a linear relationship was assumed with the monthly precipitation (Gunaratne et al, 2010a) (Figure 2.) Similarly, the mean monthly inflow rate from the tributaries were used as Q_I for Rekawa Lagoon which were calculated assuming a linear relationship with monthly precipitation and mean monthly surface runoff rates estimated by Gunaratne et al (2010b). The water densities corresponding to each survey were calculated according to McCutcheon et al (1993) by using observed salinity and temperature data. Furthermore, the entrance width and the mean depth of the lagoon mouth channels were used as parameters b_M and d_M , respectively. Moreover,, 1.3×10^{-3} was used as the value of C_D which is recommended by Fischer et al (1979) for most of the engineering calculations of the same kind. The corresponding mean monthly inflow rate of the tributaries was divided by the mean cross section of Kirama oya river to calculate the U_t values for mouth closed phase of Rekawa Lagoon. This calculation should be reasonable since Kirama oya stream acts as the main inflow tributary during the dry season while the functioning of other tributaries cease due to lack of rainfall. Additionally, it may be reasonable to not consider Kirama oya stream as an inflow tributary during the mouth open phase as it connects with the lagoon mouth channel at the downstream end, much closer to lagoon mouth. The wind speed data required for the calculation of R_{iL} for Koggala and Rekawa lagoons were collected from Galle and Hambantota Meteorological stations, respectively.

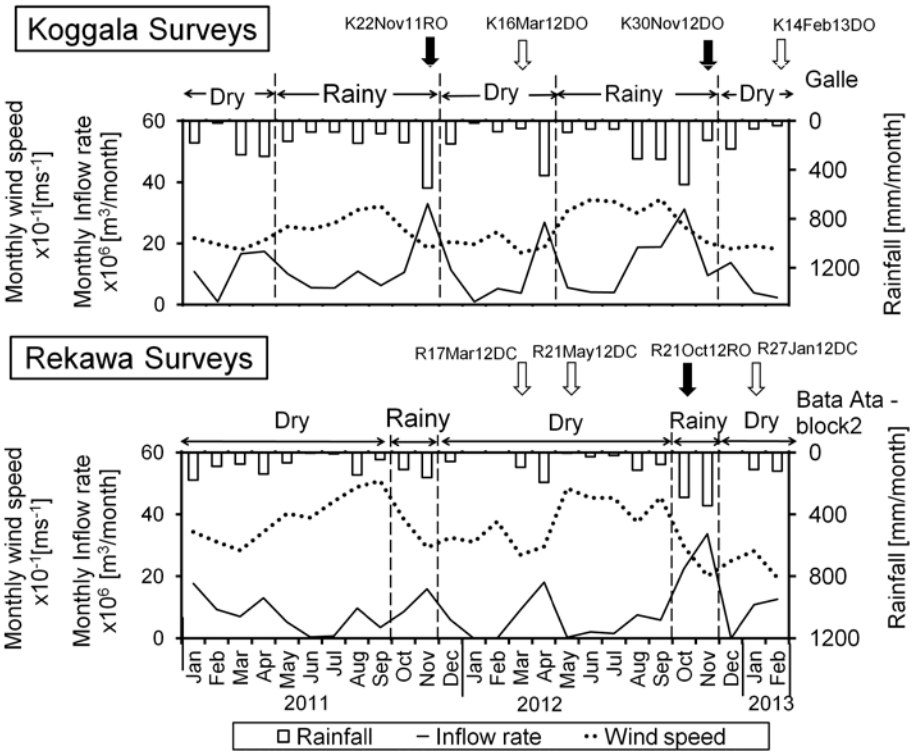


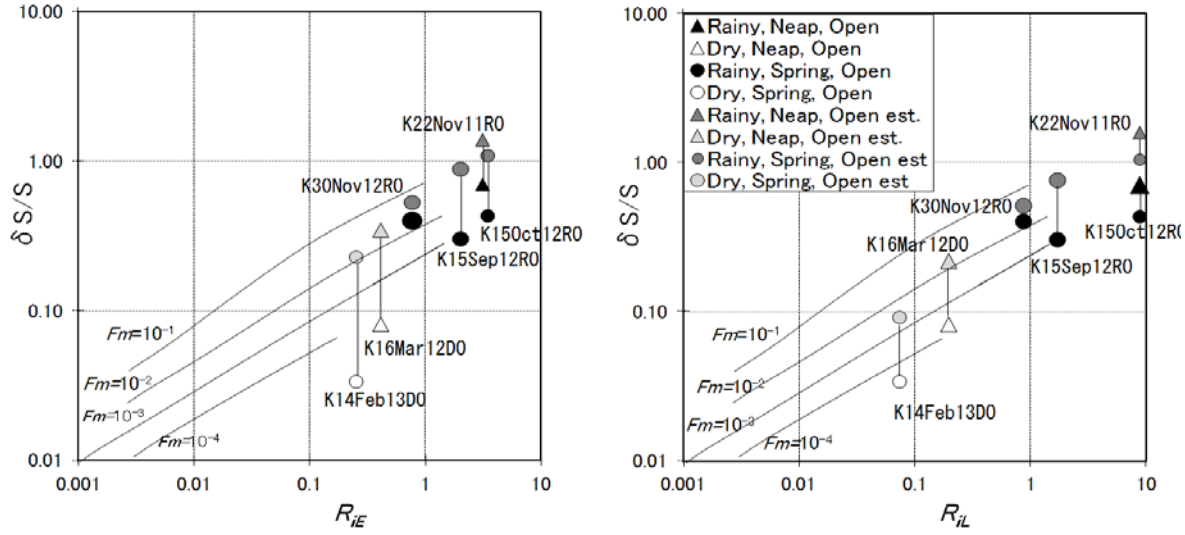
Figure 2 Monthly rainfall (Galle and Bata Ata-block2 rain gauge stations for Koggala and Rekawa Lagoons, Department of Meteorology, Sri Lanka) and monthly inflow rate from the stream and catchment of lagoons for the period from January, 2011 to February, 2013 with survey timing (Black color arrows; surveys conducted in rainy season, white colour arrows; surveys conducted in dry season).

3. RESULTS

3.1. Koggala Lagoon (POCCL)

Figure 3 shows a comparison of the Fischer model and new bulk model application to Koggala Lagoon. Results demonstrate that $\delta S/S$ estimating capability is improved in the new bulk model application with the introduction of R_{iL}, particularly for dry season. However, the deviation of the observed and estimated $\delta S/S$ values in rainy season (Figure 3; black and corresponding gray colour

plots) are not significantly reduced because the energy supplied by the wind for the mixing process is not remarkably large compared to the energy supplied by the inflow stream during rainy season. On the other hand, the deviation of observed and estimated δS/*S* values in dry season (Figure 3; white and corresponding gray colour plots) are significantly reduced as the magnitude of potential energy supplied by inflow stream is remarkably reduced so that the effect of the energy supplied by wind stress becomes significant during dry season. Thus, it would be safer to infer that the new bulk model would provide good estimation results of relative stratification for POCCLs than Fisher's model for both seasons accordingto this case study.



(i) (ii)

Figure 3 Comparison of the application results of bulk models to Koggala Lagoon (i) Fischer's model (ii) The new bulk model. Legend entries ending with the abbreviation "est." corresponds to estimated results represented as gray color plots. White and black color plots represent dry and rainy season results, respectively.

3.2. Rekawa Lagoon (IOCCCL)

Figure 5 shows a comparison of the application results of Fischer model and new bulk model to Rekawa Lagoon. Although the new bulk model shows better estimation in rainy season (mouth open condition) the deviation of observed and estimated *δS/S* results in dry season (mouth closed condition) are almost similar to the results obtained by applying Fischer's model. Additionally, the estimated relative stratification status corresponding to the survey case name R21May12DC is completely different for the two models where the Fisher's model estimates stratified condition and the new bulk model estimates more mixed condition while the actual salinity profile indicates partially stratified condition (see Figure 4). This contrasting result together with the large deviation in observed and estimated *δS/S* values under mouth closed condition indicates that it is not safer to use any of these models to estimate relative stratification of IOCCCLs, particularly during the mouth closed phase (dry season). Thus, the cause of this problem should be common to both models. Going back to the basic assumptions considered in deriving the bulk parameters used in both models, suggests that the assumption which is most likely to have cause this problem would be the quasi-equivalent state assumption which is trivial in the derivation of R_{iL} as well as R_{iE}

Quasi-equivalent state of a lagoon can be altered with respect to different time scales depending on the significant variations in the dominant forcing factors such as wind, rainfall and tide etc. According to Furusato et al (2013) the quasi-equivalent state of POCCLs related to its mixing state would last throughout a season (several months) without changing remarkably though it can be changed quite regularly for IOCCCLs depending on the timescale related to the opening and closing event duration of the lagoon mouth. Thus, POCCLs such as Koggala Lagoon are inherent with longer time scales of

quasi-equivalent states related to the mixing process due to permanently open nature of the lagoon mouth and would show better estimation results when new bulk model is applied. On the other hand, when the new bulk model is applied to IOCCCLs such as Rekawa Lagoon, the estimation capability of the model could vary depending on the situation whether the IOCCCL is in quasi-equivalent state or not.

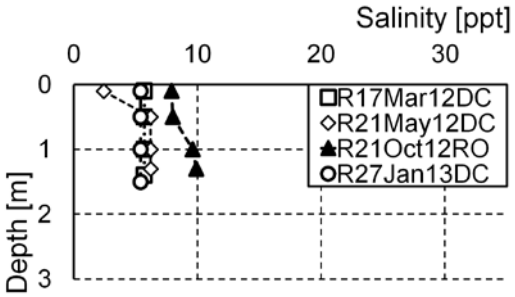
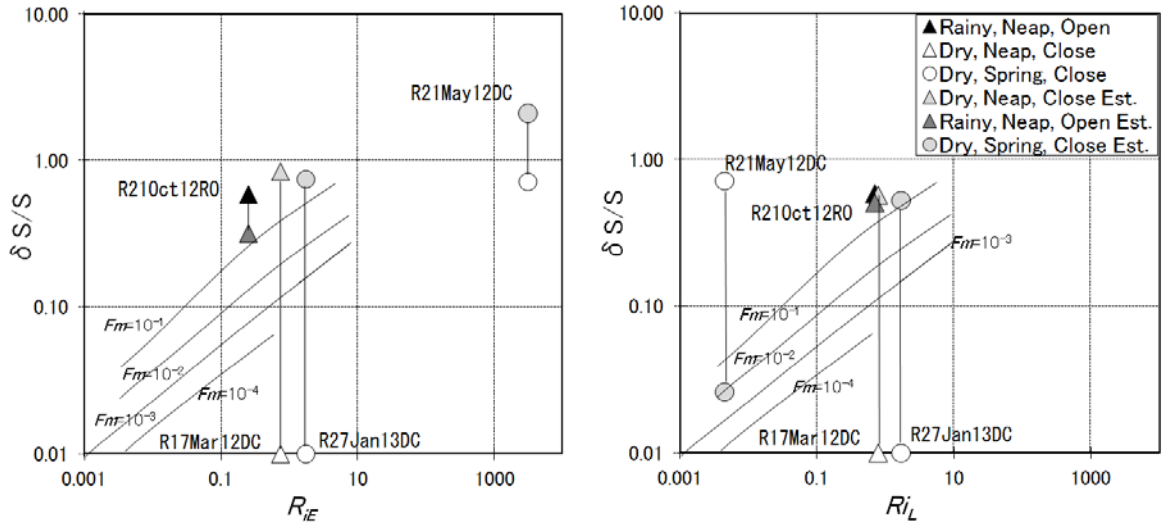


Figure 4 Vertical profiles of salinity at the central part of Rekawa Lagoon (L-2 & L-3). Colours of the plots are as described in Figure 3



(i) (ii)

Figure 5 Comparison of the application results of bulk models to Rekawa Lagoon (i) Fischer's model (ii) The new bulk model. Legend is as described in Figure 3

4. CONCLUSION

The new bulk model with R_{iL} which characterize the mixing/stratification process of coastal lagoons was applied to Koggala and Rekawa Lagoons in order to examine its applicability to choked coastal lagoons in Sri Lanka. The new model provides better estimation results than Fischer's model for Koggala Lagoon suggesting that the new model would be suitable to apply for POCCLs. However, when it is applied to Rekawa Lagoon the improvement of the estimation capability is quite low compared with the previous case. One of the probable causes for this problem would be the difference in the time scales required for IOCCCLs to reach its quasi equivalent state as it a trivial assumption in the derivation of R_{iL}. Thus, R_{iL} should be further enhanced in order to apply for IOCCCLs by including a parameter to represent the time scale. However, the validity of this explanation together with the effectiveness and applicability of the proposed model to IOCCCLs and POCCLs needs to be further verified by applying it to more choked coastal lagoons in the future.

5. ACKNOWLEDGMENTS

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Coastal Flooding Caused by Storm Surges: A Risk Assessment for the City of Tangalle in the South Coast of Sri Lanka

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Abstract: *Coastal flooding due to tropical cyclone generated storm surges has caused considerable damage and loss of life in the North Indian Ocean region including in Sri Lanka. This paper is concerned with an assessment of the risk of the storm surge hazard for the city of Tangalle on the south coast of Sri Lanka, as a pilot study. The present hazard assessment utilizes a database of historical events of tropical cyclones in the North Indian Ocean region. A statistical analysis of the past events has been carried out to identify an appropriate storm surge scenario. Numerical models comprising a parametric cyclone model and a hydrodynamic model based on shallow water equations have been employed to simulate cyclone wind velocity and pressure fields as well as coastal flooding due to the storm surges. The spatial distribution of the depth of inundation as well as maps of the risk to population and dwellings in the study area are presented and discussed.*

Keywords: *Tropical cyclones, Hazard assessment, Numerical modeling, Surge height, Depth of Inundation.*

1. INTRODUCTION

Sri Lanka is vulnerable to cyclones generated mostly in the southern part of Bay of Bengal, and to a lesser extent, to those in the southeast of Arabian Sea. The cyclones which form over the southernmost part of Bay of Bengal at low latitudes mainly move west or west to northwestwards into the Gulf of Mannar across the coast of Sri Lanka. These cyclones generally form during the later part of the post-monsoon season or early part of the pre-monsoon season. On the other hand, cyclones that form over Arabian Sea mostly move north or north-easterly while a few travel west or northwestwards (SAARC, 1998).

However, due to atmospheric dynamics associated with cyclones and the relative proximity of Sri Lanka to the equator, a large proportion of cyclones generated in Bay of Bengal and Arabian Sea, fortunately, do not make landfall in Sri Lanka. Yet, sixteen cyclonic or severe cyclonic storms have made landfall in Sri Lanka during the last century according to the Department of Meteorology, Sri Lanka (DMSL, 2010). The loss of lives and damage and destruction to property and the environment caused by land-falling tropical cyclones could be due to some or all of the following phenomena associated with such intense weather systems, namely, extreme winds, storm surge, heavy rainfall, and sometimes, excessive rainfall leading to landslides.

It must be added that, usually, much of the death toll and damage to property in coastal areas is as a result of cyclone-induced storm surge causing inundation of low-lying onshore lands (Dube, 2003; Lin et al., 2010). A storm surge is a rise above the normal water level along a shore resulting from strong onshore winds and/or reduced atmospheric pressure. The worst impact occurs when the storm surge arrives on top of a high tide. The height of the storm surge depends on cyclone dynamics such as the wind speed, the translation speed, the angle of attack at landfall, the pressure drop and also on coastal and shelf morphological factors such as the bathymetry and the shape of the coastline (Jelesnianski, 1992). Further, the severity and the extent of onshore inundation depend primarily upon the surge height and the prevailing tide as well as the elevation, the slope and the surface roughness of the terrain (Wijetunge, 2013).

However, unfortunately, no detailed assessment of the risk of flooding caused by storm surges has been carried out for the coastline of Sri Lanka. Accordingly, the present research employs numerical modeling tools and risk assessment methodologies together with necessary field work to carry out a hazard, vulnerability and risk assessment of coastal flooding due to tropical cyclone generated storm surges in the city of Tangalle on the south coast of Sri Lanka, as a pilot study.

2. METHODOLOGY

2.1. Study Area

The city of Tangalle is located in the Tangalle Divisional Secretariat of the Southern Province of Sri Lanka (Figure 1). The study area comprises 45 Grama Niladhari (GN) divisions with a population of about 45,000. The coastal belt of the city consists of wide-open beaches, lagoons and its associated waterways and a fishery harbour. Fishing and agriculture are the main livelihoods in this area.

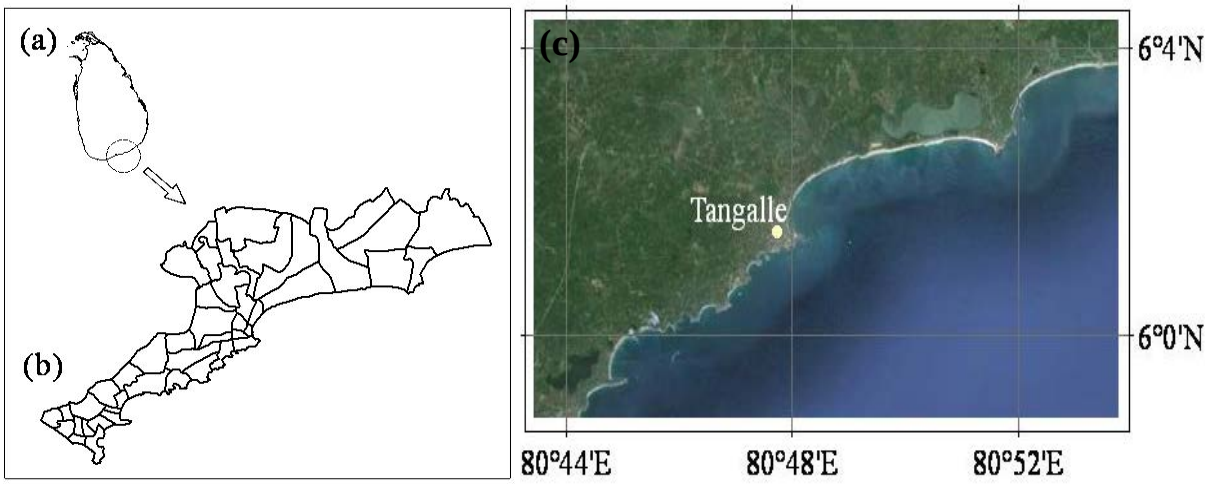

Figure 1 Study area of Tangalle: (a) Map of Sri Lanka, (b) Map of Grama Niladhari (GN) Divisions in the study area, and (c) Google Earth Image of the study area.

2.2. Statistical Analysis

A database of historical tropical cyclone events was compiled for the North Indian Ocean (NIO) region for the period 1900-todate using 'best-track' data from several sources including Joint Typhoon Warning Centre (JTWC) of the US Navy and SAARC Meteorological Research Center (SMRC). An observation window or a 'scan-box' bounded by 4-11°N and 78-93°E covering probable cyclone generation and feeder regions in southern portions of both Bay of Bengal and Arabian Sea was demarcated and all cyclones that had either formed or crossed the scan box during the above period were considered to have the potential to make landfall in or in the vicinity of Sri Lanka provided that necessary atmospheric forcing satisfied the requirements for the same. Of the subset of 201 independent cyclone events found to be falling within the scan-box mentioned above, the portion of the data prior to satellite observations (i.e., 1945), as well as those events for which it was not possible to assign reliable maximum wind speeds were excluded. Accordingly, the peak annual wind speeds corresponding to the remaining 59 independent cyclonic events were then statistically analysed using Gumbel's (1958) method, following Rupp and Lander (1996) for tropical cyclones in Guam, and several others. The fact that, of the 201 cyclonic events in the database since 1900, only about 8% have made landfall in Sri Lanka, was also incorporated into the probabilistic analysis by employing the multiplication rule. Figure 2 shows the resulting plot of wind speed against the return period based on the Type-I extreme value distribution. The recurrence interval for different wind speeds could thus be

deduced, and accordingly, we select a maximum sustained wind speed of 270 km/h with an estimated recurrence interval of 300 years as a 'worst-case scenario' for the present hazard assessment.

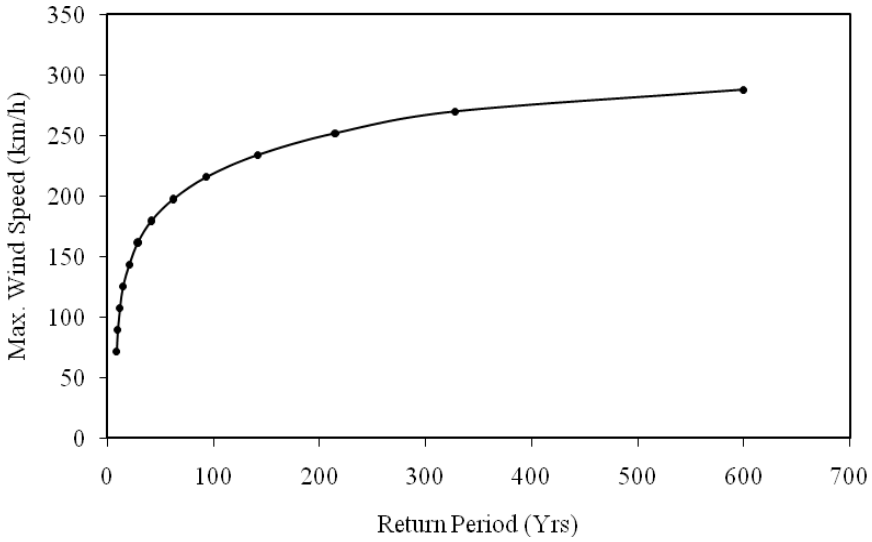

Figure 2 Maximum sustained wind speed and corresponding recurrence interval for cyclones that make landfall in Sri Lanka.

2.3 Numerical Model Set-Up

The computational domain for the present study was selected based on consideration of past studies of storm surges in the NIO region covering Sri Lanka, for example, those of Henry and Murty (1992) and Chittibabu et al. (2002). Accordingly, a rectangular region extending from 77°E – 85°E and 4°N - 12°N was selected as the 2430 m spatial resolution outermost grid of the 5-level nested grid set-up. The resolutions of the inner grids are 810 m, 270 m, 90 m and 30 m.

The bathymetry for the computational grid of 2430 m spatial resolution was at first interpolated from 30 arc-sec GEBCO data and was then updated with data from navigation charts. These navigation charts typically covered depths down to about 3000–4000 m at scales 1:150,000 or 1:300,000. The depths in navigation charts were reduced from Chart Datum (i.e., Lowest Astronomical Tide) to Mean Sea Level (MSL). The topography of the grids was constructed using Light Detection and Ranging (LIDAR) data of horizontal resolution 1 m and vertical resolution not less than 0.3 m.

A widely-accepted hydrodynamic model, DELFT3D, based on the quadratic wind friction formulation and depth-averaged, non-linear equations of conservation of mass and momentum was employed to compute the water surface elevation due to cyclone induced forcing of space- and time-varying wind and pressure fields. The wind and pressure distributions due to the cyclone were computed using an axisymmetric parametric model formulation similar to that of Holland's (1980), and were then used as input to force the hydrodynamic model.

2.4 Model Calibration and Verification

Two past cyclone events that resulted in storm surges in some parts of the coastline of Sri Lanka have been utilized to calibrate and verify the numerical model, namely, the severe cyclonic storms of 1978 and 1964, respectively.

The model verification run with 1964 cyclone as the forcing was carried out with the same values of model parameters such as wind friction factor and Manning's coefficient as in the simulation for 1978 cyclone. The computed maximum surge heights were then compared with available records of observed surge heights due to the cyclones of 1978 and 1964.

2.5 Numerical Simulation of Selected Hazard Scenario

For the selected hazard scenario, the landfall location and the approach of the cyclone at the coastline was varied to cover a range of probable tracks and an array of separate model simulations was carried out for each hypothetical track. The models for these cyclone scenarios were integrated with a maximum pressure drop of 80 hPa and a radius of maximum wind of 40 km.

The track that is likely to cause the highest impact was identified based on the computed peak surge heights immediately offshore of the study area. The detailed inundation simulations were then carried out for the identified track mentioned above.

The numerical output of the model simulations gives the space- and time-varying water surface elevation from which the distribution of flow depth as well as the extent of inundation could be found.

Since tidal dynamics is not incorporated in the model, following NOAA (2007) in the case of tsunami inundation and in keeping with the present objective of hazard mapping based on the worst-case scenario, Mean High Water (MHW), which is on average 0.3 m above MSL for the study area, was used as the baseline vertical datum in developing digital elevation models for the simulations. The depths in navigation charts were also reduced from Chart Datum (i.e., Lowest Astronomical Tide) to the same vertical datum, representing MHW.

2.6 Risk Analysis and Assessment

Risk is a function of the hazard frequency and the severity, the exposed element or elements at risk, and the vulnerability of that element, and may be expressed as follows:

$$\text{Risk} = \text{Hazard} \times \text{Vulnerability} \quad (1)$$

The hazard due to flooding depends on several factors, such as the depth of inundation, the flow velocity, the duration of flooding, and the debris carried by the flow. However, following Wijetunge (2014) in the case of tsunami flooding, we utilize the mean depth of inundation as the primary parameter to quantify the hazard to the population and residential buildings. The vulnerability of the coastal community was estimated based on data available from the Department of Census and Statistics of the Government of Sri Lanka, viz., the population density and the building density, as details concerning other indicators of vulnerability such as the age, the gender, etc, were not available at the desired spatial resolution. The relative risk was then computed using Eq. (1). Supplementary data pertaining to buildings as well as vital infrastructure in the city of Tangalle and its immediate surroundings were also collected through field studies, however, that part of the vulnerability analysis is not presented here due to space constraints.

3. RESULTS AND DISCUSSION

3.1 Model Verification

Table 1 compares the maximum values of the simulated surge levels and the corresponding observed maximum storm tide levels at several locations in Sri Lanka and in South India. The locations of observed storm tides have been identified only by the general area of the city or village, so in the absence of exact coordinates of these locations of observed surge levels, we give a range of simulated maximum surge heights in the vicinity of the general area of each location; the source of information regarding observed maximum storm tide levels is also given.

We see in Table 1 that the ranges of computed storm surge levels are, on the whole, in reasonable agreement with the observed storm tide levels. It must be noted that the simulated surge levels do not include the effects of the tide and waves whereas the observed surge levels include the storm surge and the effects of the tide at the time as well as the wave set-up.

However, it must be added that the verification of the model was not possible for the particular study area or its immediate surroundings as observed storm surge data corresponding to any past cyclone events are not available.

Table 1 Comparison of simulated and observed maximum surge heights.

Cyclone event	Location	Observed storm tide level	Simulated maximum storm surge level
1964-Cyclone	Rameswaram and Madanpan	3.0 – 4.2 m*	3.0 – 4.0 m
	Pamban and Nagapattinam	3.0 – 5.0 m*	3.0 – 3.7 m
	Tondi	3.0 – 6.0 m**	3.2 – 5.8 m
	Dhanushkodi	3.0 – 6.0 m^{+}	2.8 – 3.4 m
	Mannar	4.8 - 5.2 m^{++}	4.6 m
1978-Cyclone	Batticaloa	1.0 – 2.0 m^{+++}	0.8 – 1.6 m
	Tondi and Devipattinam	3.0 – 5.0 m*	2.4 – 3.3 m

*Ramesh (2004);**Murty et al. (2004);$^{+}$SMRC (2004);$^{++}$CDM (1964);$^{+++}$DMSL (2010)

3.2 Assessment of Risk of Coastal Flooding

The spatial distribution of onshore floodingn due to tropical cyclone induced storm surge hazard corresponding to a maximum sustained wind speed of 270 km/h is shown in Figure 3. Note that, the inundation depths shown in Figure 3 correspond to the high-tide and the classification of inundation utilized is also shown alongside together with the waterways and water bodies shown in blue. We see that the low-lying northern areas and parts of the central localities of the city of Tangalle and the surrounding are likely to be inundated most with flood depths exceeding 2 m. The model simulations also indicate that the waterways and low points in elevation along the coastline are the primary conduits of storm surge flooding into areas further inland.

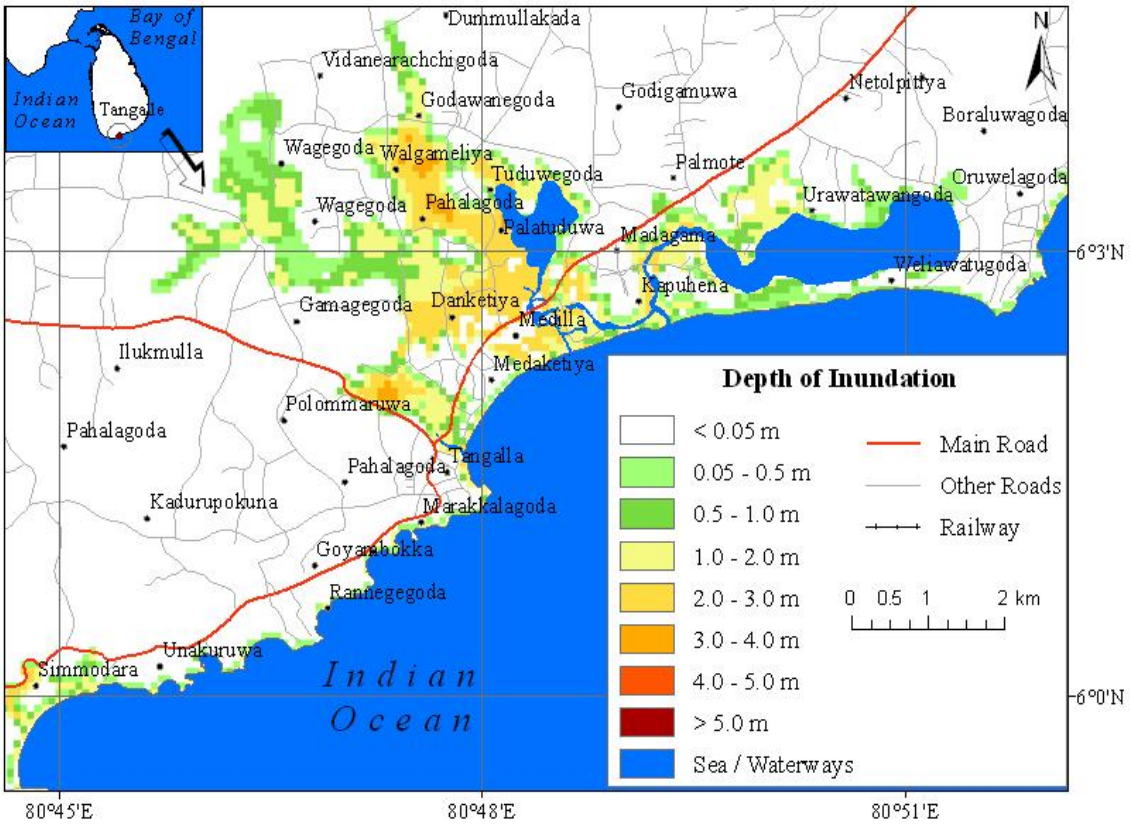

Figure 3 Spatial distribution of inundation in Tangalle, Sri Lanka due to the storm surge caused by a tropical cyclone of wind speed 270 km/h.

Figures 4 and 5 depict the spatial distribution of the computed relative risk to population and dwellings, respectively, in connection with potential inundation caused by a storm surge due to the above cyclone event at high tide. The risk in both cases, computed using Eqn. (1), has been classified into four levels as low, medium, high and very high.

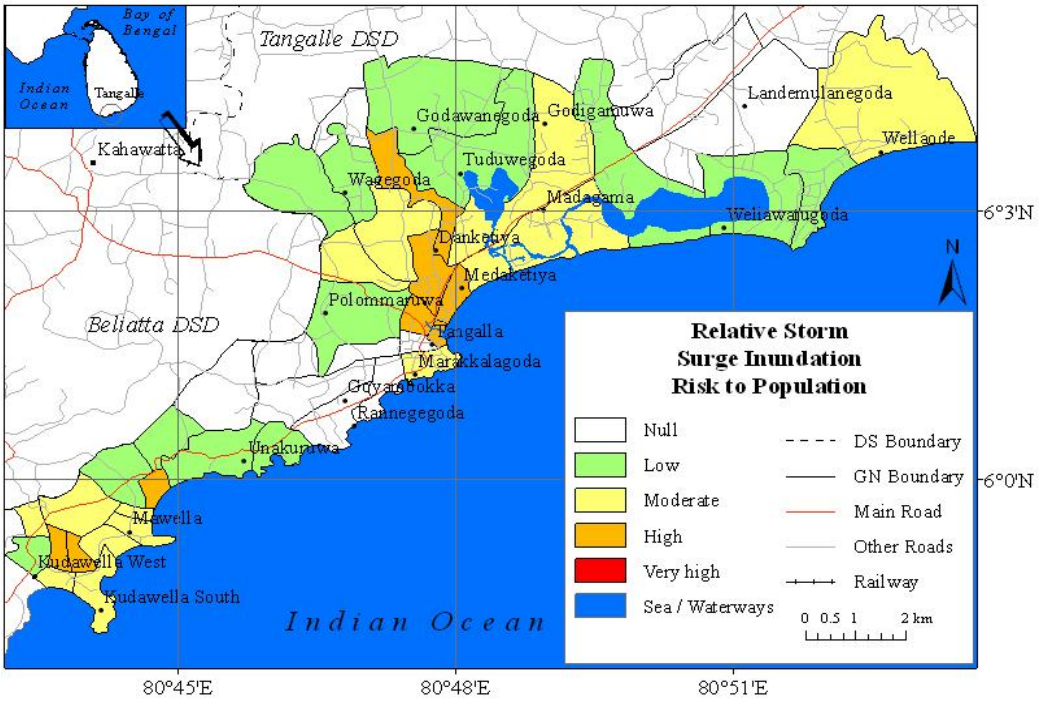

Figure 4 Spatial distribution of the relative risk to population in Tangalle due to flooding caused by a storm surge generated by a tropical cyclone of maximum sustained wind speed 270 km/h.

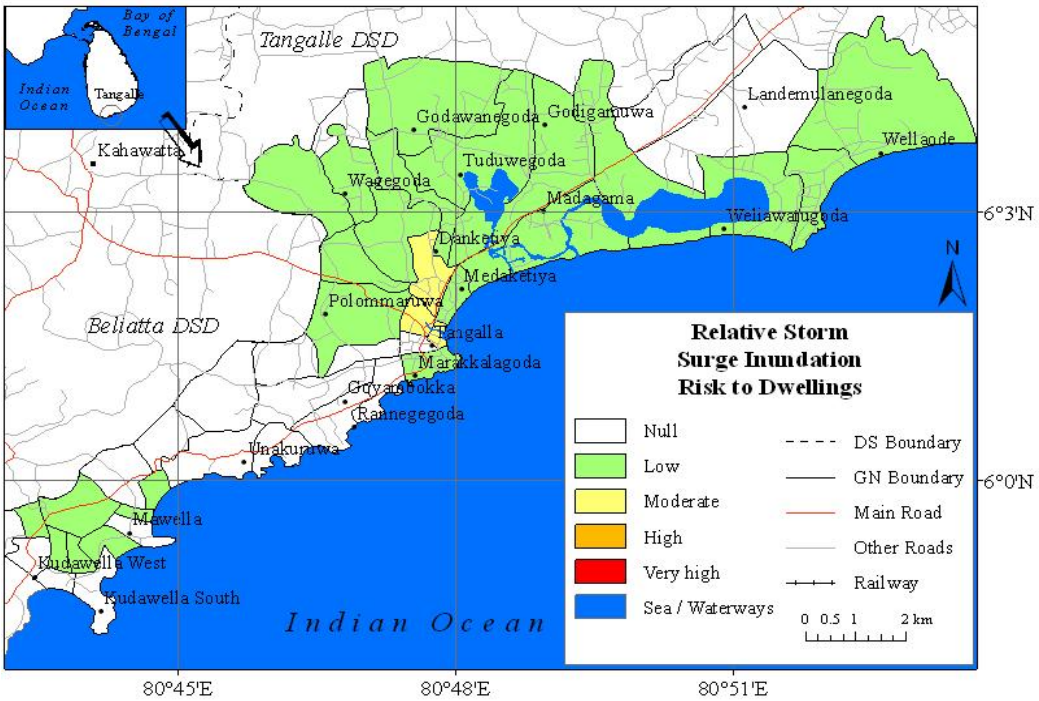

Figure 5 Spatial distribution of the relative risk to dwellings in Tangalle due to flooding caused by a storm surge generated by a tropical cyclone of maximum sustained wind speed 270 km/h.

A field survey of the study area revealed that part of the inundation zone is heavily populated with many residential buildings and commercial establishments; moreover, vital infrastructure such as hospitals, fire stations, electricity sub-stations, schools, etc are also located in the inundation zone.

It is proposed that the storm surge risk mitigation strategy for the pilot study area should comprise cyclone and storm surge forecasting, provision of early warning to vulnerable communities to enable their evacuation as well as education and awareness programs at the community level.

Finally, there are certain limitations inherent in a study of this nature. One limitation is that the resolution of the modeling is no greater or more accurate than the bathymetric data used. Moreover, the tide has been linearly superimposed on the computed storm surge levels on a conservative basis although the tide-surge interaction is non-linear. It must also be added that the set-up due to wave breaking has not been incorporated in the present model simulations. It must also be added that depth-averaged models assume a uniform velocity profile across the flow depth and neglect vertical accelerations. Moreover, the mathematical formulation employed in the present model does not explicitly account for all means of energy dissipation. For instance, although energy dissipation due to bottom friction is included in the present model, dissipation due to turbulence is not explicitly formulated.

4. CONCLUSIONS

Numerical simulations have been performed to compute the spatial distribution of onshore flooding due to tropical cyclone induced storm surges in Tangalle on the south coast of Sri Lanka, corresponding to a 'worst-case' scenario as a pilot study. The effect of the tide has also been incorporated in inundation computations. The simulated flood depths as well as the available vulnerability indicators have been used to delineate the distribution of relative risk to population and dwellings. The effects of the terrain as well as the presence of waterways and water bodies can be seen in the flood distribution in the study area.

5. ACKNOWLEDGEMENT

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An Experimental Study on Berm Breakwater Recession

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Abstract: *Berm Breakwaters are allowed to reshape its seaward profile of the breakwater to a more stable profile that could effectively withstand the wave action and the reshaped profile parameters are very crucial when designing a berm breakwater. Berm Recession can be identified as one of the most important profile parameters and In this study, influence of various parameters on berm recession was studied based on the two dimensional model tests carried out at the Faculty of Engineering, University of Peradeniya. A new formula for recession was derived using dimensional analysis and it was compared with the existing formulae.*

Keywords: *Berm Breakwaters, Recession, Physical Model Tests, Waves*

1. INTRODUCTION

Berm breakwaters are different from other conventional breakwaters because conventional breakwaters are almost statically stable whereas, the seaward slope of berm breakwaters are allowed to reshape to a stable profile. Smaller sizes of stones can be used for berm breakwaters because they are allowed to reshape. Compared to conventional rubble mound breakwaters, 2 to 10 times smaller stones can be used for berm breakwaters. Berm breakwaters can also be constructed by using relatively easy, land based methods and less specialized construction equipment compared to conventional breakwaters, therefore, low cost can be considered as an added advantage. As a berm breakwater dissipates much of the wave energy with minimum reflection, smaller vessels can navigate very close to berm breakwaters.

Though berm breakwaters have many advantages, very few berm breakwaters are reported to be constructed around the world. According to the report of working group 40 of the PIANC (2003), only around 60 berm breakwaters had been constructed before 2004. This type of breakwaters is mainly used in Iceland, Norway and Denmark. Lack of experience in berm breakwaters and the consideration of higher risk of failure compared to conventional breakwaters due to excessive reshaping and changes of sections as a result of not having clear design guidelines, could be some of the reasons for less popularity of berm breakwaters. Only limited numbers of studies have been done so far on the stability and reshaping characteristics of berm breakwaters. Therefore, in order to understand some of these phenomena that could eventually be used for improving the design guidelines of berm breakwaters, further studies related to above phenomenon are necessary.

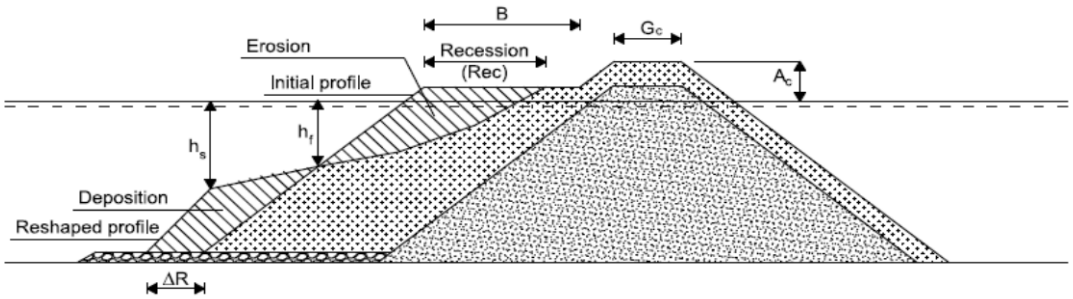

Figure 1 Reshaped profile parameters (after Andersen, 2006)

Figure 1 shows the typical profile parameters used to describe a reshaped berm breakwater. The most important measure for the reshaping phenomenon can be identified as the recession of berm (Rec), because the width and materials of the berm usually decides the stability and failure of the breakwater. The extension of berm breakwater towards the ocean is determined by the berm recession. So the scour protection, bedding layer, design depth and some other parameters depend on the magnitude of recession and the extension of the breakwater (Andersen, 2009). Not to mention the fact that when recession is more than the berm width, the breakwater failure would occur.

There are several common indices which are used to classify berm breakwaters. In stability classification, stability parameter (H_0) and combined wave height-wave period stability parameter (H_0T_0) are mostly used.

$$H_0 = \frac{H_s}{\Delta D_{n50}} \tag{1}$$

where, H_s = Significant wave height,
Δ = ρ_s/ρ_w -1 = Relative buoyant density,
ρ_s = Mass density of stone material,
ρ_w = Mass density of water
D_{n50} = Median stone diameter = Volume$^{1/3}$

$$H_0 T_0 = \left(\frac{H_s}{\Delta D_{50}}\right)\left(T_m\sqrt{\frac{g}{D_{50}}}\right) \tag{2}$$

T_m = Mean zero up-crossing wave period.
g = Gravitational acceleration.

When considering the reshaping of berm breakwaters, they are classified into three main categories in PIANC (2004). Little movement (H_0<1.5-2, H_0T_0 <20-40), Limited movement during reshaping - statically stable (1.5<H_0<2.7, 40<H_0T_0<70), Relevant movement, dynamically stable (H_0>2.7, H_0T_0>70). In this study, all of these stability regimes were considered and analysed.

2. PREVIOUS STUDIES ON BERM RECESSION

Several research studies have been conducted so far and several methods and formulae to estimate the recession were given in those studies. Several of those studies are based on physical models and some are from numerical analysis. In this study, several existing formulae are used to calculate the recession of the model tests and then compared with the model test results. They are from the studies Hall and Kao (1991), Tørum and Krogh (2000), Sigurdarson et al (2007), Moghim et al (2009), Andersen and Burcharth (2010), Sigurdarson and van der Meer (2013) .

The formula given by Hall and Kao (1991) is given in equation 3. The formula was formed based on simple regression analysis. It can be seen that the effect of the percentage of rounded stones is included.

$$\frac{\mathrm{Re}\,c}{D_{50}} = -10.4 + 0.51H_0^{2.5} + 7.52\,f_g - 1.07\,f_g^2 + 6.12P_R \tag{3}$$

where, P_R = Fraction of rounded stones in a sample, f_g= D_{n85}/ D_{n15} = Gradation factor of stones. (D_{n85} is the nominal diameter of stone for which 85% of the total sample mass is of lighter stones, D_{n15} is the nominal diameter of stone for which 15% of the total sample mass is of lighter stones)

The formula given in the Tørum and Krogh (2000) based on simple regression models is as given in equations 4. It can be seen that influence of the gradation of the stones and the water depth are included.

$$\frac{\mathrm{Re}\,c}{D_{50}} = 0.0000027\left(H_0T_0\right)^3 + 0.000009\left(H_0T_0\right)^2 + 0.11\left(H_0T_0\right) - f_{grading} - f_d \tag{4}$$

where, $f_{grading}$=-10.5 + 23.9f_g - 9.9f_g^2

f_d = depth factor = -0.16(h/D_{n50}) + 4.0
(h = water depth). This formula is valid for 1.3<f_g<1.8 and 12.5< h/D_{n50} <25 only.

Sigurdarson et al (2007) has been derived a simple formula for the calculation of the recession.

$$\frac{\mathrm{Re}\,c}{D_{n50}} = 0.037\left(H_0T_0 - S_c\right)^{1.34} \tag{5}$$

where, S_c = Scattering of the recession measurements . $Rec/D_{n50} = 0$ for $H_0T_0 < S_c$.
Based on physical model test results Moghim et al (2009) proposed following formulae for the estimation of berm recession. For 9 < $\sqrt{H_0^2T_0}$ < 22 and 500 < N <6000, (N= Number of waves)

$$\frac{\mathrm{Re}\,c}{D_{50}} = \left(1.03\sqrt{H_0^2T_0} - 7.47\right)\left\{1.61 - \exp\left(-2.2\left(\frac{N}{3000}\right)\right)\right\} \quad \text{for } \sqrt{H_0^2T_0} < 17 \tag{6}$$

$$\frac{\mathrm{Re}\,c}{D_{50}} = \left(0.43\sqrt{H_0^2T_0} + 4.22\right)\left\{1.61 - \exp\left(-2.2\left(\frac{N}{3000}\right)\right)\right\} \quad \text{for } \sqrt{H_0^2T_0} > 17 \tag{7}$$

Sigurdarson and van der Meer (2013) derived a simple formula using experimental data.

$$\frac{\mathrm{Re}\,c_{av}}{D_{50}} = 1.6\left(\frac{H_s}{\Delta D_{50}} - 1.0\right)^{2.5} \tag{8}$$

$$\frac{\mathrm{Re}\,c_{av}}{D_{50}} = 0 \tag{9}$$

When using the above formulae for the same input parameters, it was found that the predicted recession values differ considerably to each other, indicating that their applicability to real life problems difficult. In addition, each of these formula suggested for computing berm recession has several limitations which constraints them from using for various field conditions. Also there are considerably high limitations for each formula which constraints them from using for various conditions. Therefore, in order to get a better understanding of the reshaping phenomenon and to derive an improved formulation to predict berm recession, a series of physical model tests were carried out.

3. EXPERIMENTAL PROCEDURE

3.1. Experimental Set-Up

The experimental study was carried out in the wave channel in the Hydraulics laboratory of the Faculty of Engineering, University of Peradeniya. Experimental set-up consists of a steel framed 12.75 m long 0.71 m deep and 0.52 m wide rectangular 2D wave channel.

Two resistant type, twin-wire wave probes were used to measure the wave parameters. Analogue-to-digital (AD) converter was used to convert the analogue output from the wave probes into digital form. Lab View programme in the Personal Computer was used to obtain measurements using wave probes and save data for further analysis. The data files saved in the PC was subsequently analysed by employing a MATLAB programme to obtain the required wave parameters.

Distance between wave gauges and the distance from the model to the gauges were changed according to the Goda and Suzuki (1976) study, in order to obtain the wave parameters.

Model breakwaters were constructed at a distance of 8 m from the wave generator as shown in Figure 2. The models were with homogeneous armours and less permeable core, as shown in Figure 3. Armour stone sizes and the seaward slope of the breakwater models were systematically changed. Stones were painted as in Figure 4, in order to reduce the excessive friction among stones and to identify the stone displacement.

Regular waves were generated at the upstream end of the channel and the total number of waves generated for each test runs was 2000.

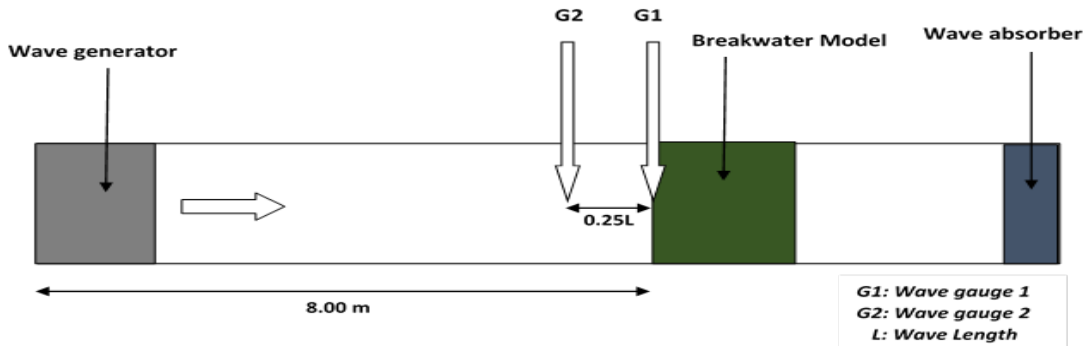

Figure 2 Plan View of the Wave Channel with the Breakwater and Wave Gauges

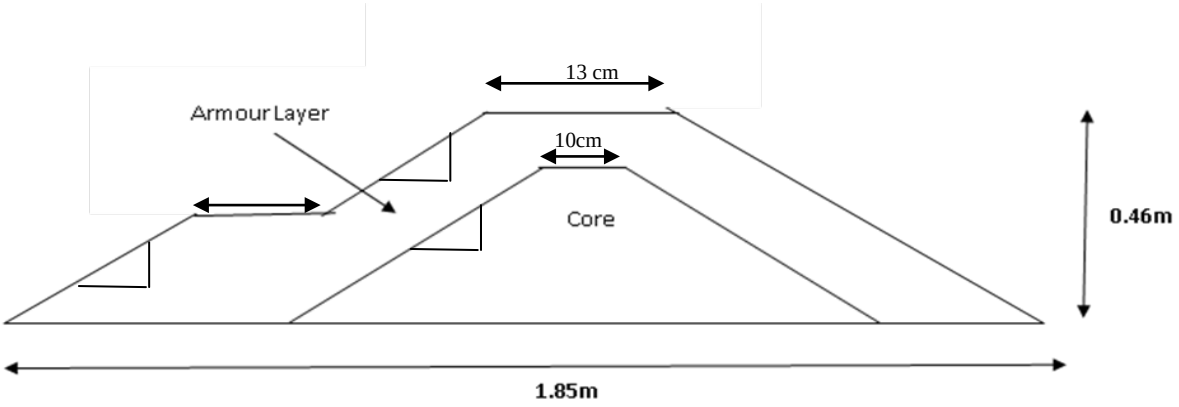

Figure 3 Schematic Diagram of the Model with 1:2 Seaward Slope




Figure 4 Painted Berm Breakwater Section

3.2. Model Runs

Varied model parameters during the experimental runs with their magnitudes are given in Table 1. Though 108 experimental runs were conducted, some of the test runs did not produce any reshaped profiles in the breakwater surface, indicating that there is a threshold condition for the occurrence of reshaping phenomena. Only 77 test runs could generate reshaped profiles of the model and these results were used in the data analysis.

Table 1 Parameters used in Model Tests

Variables	Values	Number of test runs
Seaward Slope	1:2.5, 1:2 , 1:1.5	3
Wave Period (s)	1.07, 1.00, 0.91, 0.86	4
Wave Height (cm)	6 , 10 , 16	3
Stone Grading D_{50} (mm)	13, 10.3, 8	3
D_{85}/D_{15}	1.63, 1.67, 2.33	
		Total = 108

4. DATA ANALYSIS

4.1. Evaluation of Existing Methods in Estimating Berm Recession

Each of the existing formulae discussed in Sec 2, were used to calculate the recession values for the model tests and compared with the model recession values. Figure 5 compares the computed recession values using previous methods with the measured values obtained in this study.

From this analysis, it can be noted that the predictive capability of berm recession using previously derived methods is very much limited, as the data points are highly scattered. In addition, some of these methods predict recession values for certain input data sets where there are no absolute recessions measured for the corresponding test runs. This seems to be one of the major drawbacks of most of the previous formulae derived for computing berm recession. Therefore, a new formula was developed using a semi-empirical approach.

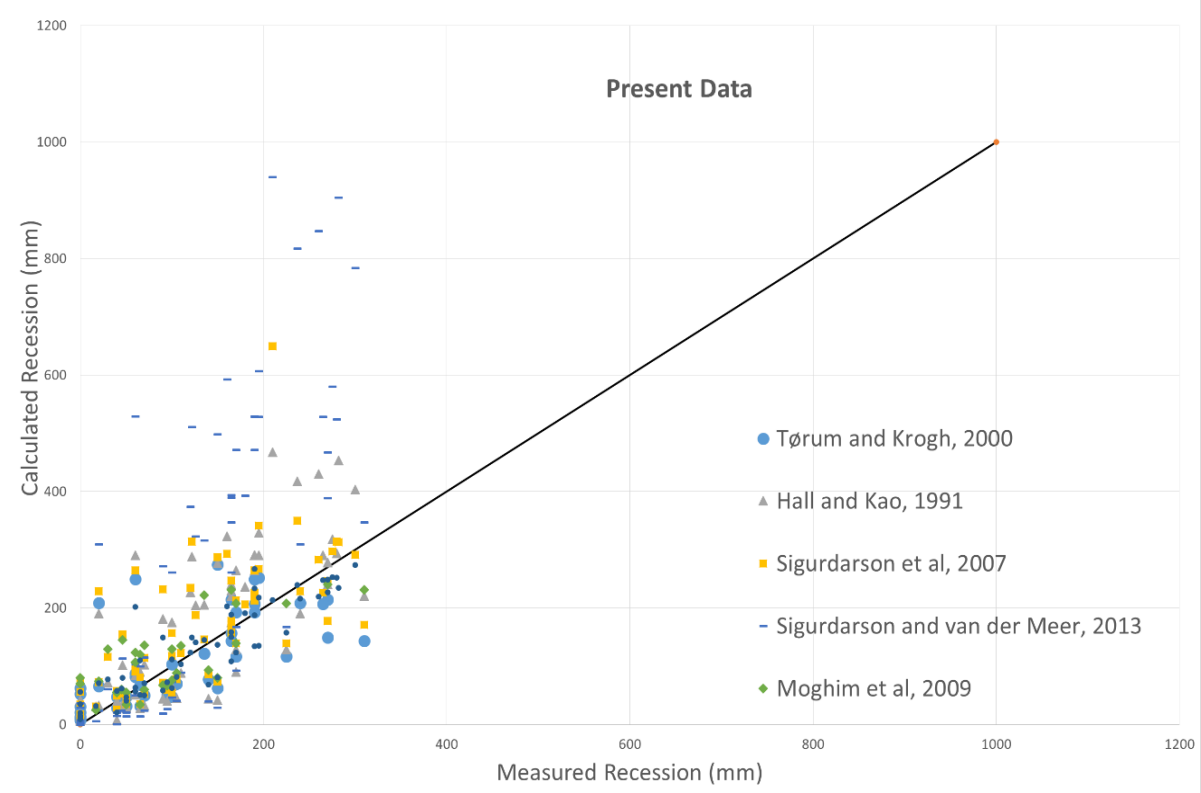

Figure 5 Comparison of present test results with results obtained from existing methods

4.2. Development of a New Formula to Predict Recession

Dimensional analysis was carried out in order to formulate the dimensionless parameters which influence the berm recession. Selection of the parameters was done with the help of the previous studies and coastal engineering concepts. Following parameters were selected as the governing parameters influencing the process associated with the recession in berm breakwaters.

$$Rec = \phi\,(H_s, T_m, N, D_{50}, f_g, \rho_w, \rho, h, v, g, h_{berm}, \tan\alpha) \quad (10)$$

where, v = Kinematic viscosity, h_{berm} = Berm height, α = Seaward slope angle.
In the present study, the number of waves (N) was kept constant for each model test as N = 2000. Therefore, dimensionless parameter representing N was neglected. Similar action was taken for h and h_{berm} as well. Since the kinematic viscosity does not influence the reshaping of berm breakwaters in turbulent flow conditions such as used in the model tests, the effect of viscosity was also neglected. Hall and Kao (1991) suggests that v can be ignored if D_{50} is selected according to, $g^{0.5}D_{50}^{1.5}/\,v > 3 \times 10^4$, where the maximum value of D_{50} should be limited to approximately 45 mm, and this was satisfied in the present model tests.

Using the dimensional analysis, following relationship was derived.

$$\frac{\mathrm{Re}\,c}{D_{50}} = \Phi\left(H_0 = \left(\frac{H_s}{\Delta D_{50}}\right), T_0 = \left(T_m\sqrt{\frac{g}{D_{50}}}\right), f_g\ , \tan\alpha\right) \quad (11)$$

In order to study the influence of each one of these dimensionless parameters on the *Rec/D*$_{50}$, linear regression analysis was used to establish suitable relationships between *Rec/D*$_{50}$ and other dimensionless parameters, as shown in Figure 6. These dimensionless parameters were again rearranged to form new dimensionless quantities and then, using multi variable linear regression analysis, the following formula was obtained for the estimation of the recession.

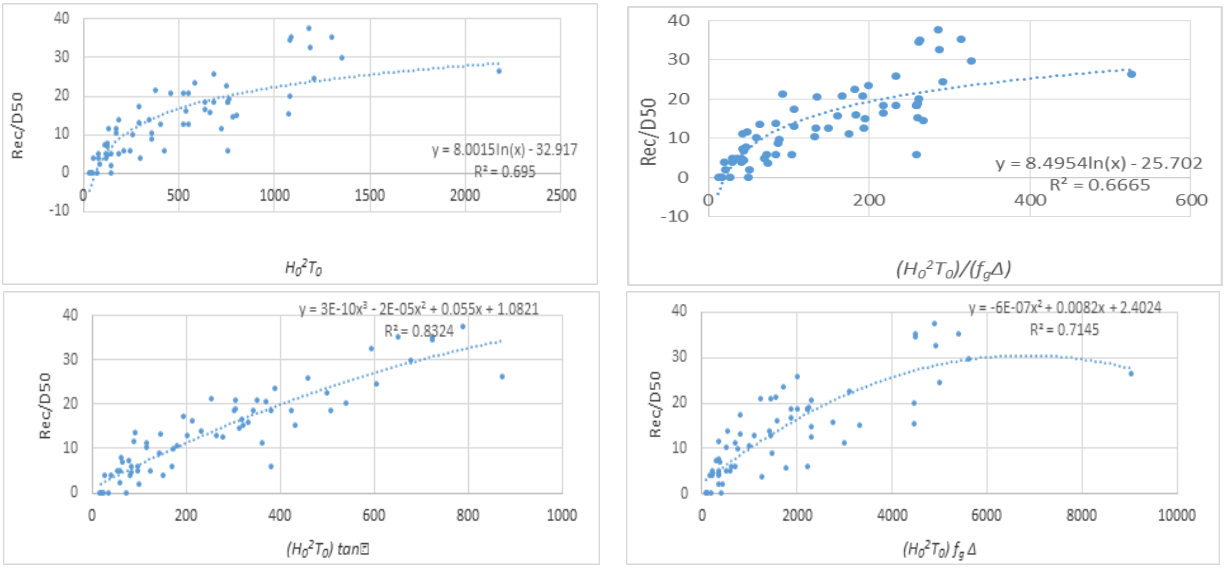

Figure 6 Influence of some dimensionless parameter combinations on *Rec/D*$_{50}$

$$\frac{Rec}{D_{50}} = 2.21 + 0.015\left(H_0^2 T_0\right) + 0.14\left(\frac{H_0^2 T_0}{f_g \Delta}\right) - 0.057\left(\frac{H_0^2 T_0}{f_g \Delta \tan\alpha}\right); \quad H_0^2 T_0 > 100$$

$$\frac{Rec}{D_{50}} = 0; \quad H_0^2 T_0 < 100 \tag{12}$$

The experimental data obtained in this study was used to compute recession using previously suggested methods and the present formula. The standard deviation from the above computations are given in Table 2.

Table 2 Standard deviations of *Rec/D*$_{50}$ for present data

Study	Standard Deviation of Rec/D50
Torum & Krogh (2000)	5.07
Hall & Kao (1991)	9.81
Sigurdarson et al (2013)	29.01
Sigurdarson et al (2007)	10.03
Moghim et al (2009)	4.94
Present Study	3.98

It can be clearly seen that the present method gives results with less deviation from the observed data for the present test results.

5. CONCLUSION

The present formula derived for the recession seems to be superior to the existing formulae considered in this study based on the following reasons.

- The threshold condition where zero recession occurs in actual cases is addressed and solved to a certain degree in the present formula.
- Gives more accurate results for the model test data sets considered.

However, the present formula could be further improved using more test results. Following improvements are recommended.

- The proposed formula should be validated using independent test results.
- The effect of the parameter such as *N, h,* h_{berm}, etc. needs to be incorporated.
- Actual test conditions used to derive different formulae considered in this study should be taken into consideration in the comparison of results.

6. ACKNOWLEDGEMENT

Sincere gratitude goes to Mr. N.M.T.K Revel for his contribution through advising on the concept of the research and helping with wave data extraction and data analysis.

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Climate Change Impacts on Seasonal Wave Climate of the Western Coast of Sri Lanka

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Abstract: *Climate change and climate change driven impacts are most widely argued topics among contemporary researchers and scientists. Broadly there are two schools of philosophies that process entirely contrasting concepts about this whole concept of climate change and its impacts. While one of the concepts state about frequently varied climate change and occurrence of extreme weather events the others are banking upon the concept that there is no climate change and it is only the indifferences in occurrence of weather and climate events. The title of this study itself implies the fact that this research study supports the concept of climate change and its probable impacts, thus leaving out the other approach on climate change. Extensive concentrations of green house gasses emitted to the earth's atmosphere and vast amounts of aerosols govern the majority of anthropogenic causes for climate change, while many of the natural causes such as changes in solar radiation also contribute immensely to earth's climate change. Absence of detailed studies carried out on investigating probable impacts on wave climates due to projected climate changes is one of the major drawbacks in handling the unique coastal eco systems in Sri Lanka. As a country where coastal resources play a major role in its development, it is absolutely necessary to have a clear idea about the probable impacts that could arise on its coastal areas. This detailed investigation provides vital information on probable impacts that might cause on the western coast of Sri Lanka under the influence of global climate change.The outcomes of this study indicate that there is certain threat to the coasts in Sri Lanka due to a considerable increment in mean wave height and shifted wave directions. It also indicates that not only the number of extreme wave events increase very significantly but also its intensity upsurges in a considerable amount as well, while indicating a major shift in seasonal wave climate that prevails along the western coast of Sri Lanka. This probable shift of seasonal wave climate of the western coast should be considered seriously, since many of its socio-economic activities are directly related to this seasonal wave climate variation.*

Keywords: *Climate Change impacts, seasonal wave climate shift, mean wave height and direction*

1. INTRODUCTION

Changes in the Earth's climate and its possible impacts belong to most widely discussed scientific problems in the present era. These climate changes occur due to both internal variability within the climate system and external factors, which can be either anthropogenic or natural. Increasing concentration of atmospheric greenhouse-gases tends to warm the Earth's surface and its lower atmosphere, while an increase in some types of aerosols tends to cool it (Houghton et al., 2001). Natural factors, such as changes in solar output or explosive volcanic activity, can also cause radioactive forcing and hence influence the Earth's climate. Complex climate models are required to provide detailed estimates of feedbacks and regional features in the climate system. Although confidence in the ability of these models to provide useful projections of future climate has improved on a range of space and time-scales, the present-day climate models cannot yet simulate all aspects

of climate (Houghton et al., 2001). For example, there are particular uncertainties associated with clouds and their interaction with radiation and aerosols. Besides, climate model simulations are a combination of a forced climate change component together with internally generated natural variability. The internal variability of the global and regional climate system adds a further level of uncertainty in the evaluation of a climate change simulation.

As an important element of the climate system, ocean wave heights (among many other ocean surface characteristics) could be affected by anthropogenic forcing. However, ocean wave heights are not directly available from the output of global climate models. Useful projections of future wave height climate need to be produced through dynamical or statistical ‘‘downscaling’’ approaches, just like other regional climate change information. Therefore, there are various sources of uncertainty in the generation of ocean wave height climate change projections (Hamer et al., 2010a).

Within the context of climate change, one of the recurrent questions is how this change could impact waves and thus wave-dominated coasts. As an example, the Western Coast of Sri Lanka is bounded by the Indian Ocean and it is characterized by intense human activities, such as sea transport, fishing, coastal shipping, ports, seaside resorts, touristic sandy beaches and surfing areas. Since the said coast is completely open to the ocean, its wave climate is characterized by swells and storms generated by strong winds in the Indian Ocean. In the work done by Wang & Swail, 2006, it was mentioned that the changes in wave conditions could even modify the coastal morphology and hence impact the human activities as well. In the context of global warming, significant climate changes at the oceanic basin scale could modify the wave climate. General Circulation Models (GCM) indeed project atmospheric changes such as a poleward shift of storm tracks (Yin, 2005) or a decrease of the total number and intensity of cyclones in the Northern Hemisphere (CATTO et al., 2011). Occurrence of such kind of changes in wind can make a significant impact on the resulting wave climate, in terms of wave height, period and direction.

Concerning the future wave climate, work done by Christensen et al., 2007 have highlighted the fact that there is a vast lack of information on potential changes in regional wave climate. However, a significant work is yet to be carried out with respect to the possible climate change driven wave climate changes in the Indian Ocean, which will indicate the possible impacts to the western coast of Sri Lanka. The aim of the study carried out was to provide more rigorous projections of the offshore ocean wave climate of the western coastline of Sri Lanka, to provide a suitable dataset, which can be used to assess the possible coastal impacts of climate change in the region, under the forcing of future climate scenarios.

2. OBJECTIVES AND METHODOLOGY OF THE RESEARCH STUDY

2.1. Objectives of the Research Study.

(i) Set up and calibrate a wave model that is capable of predicting the off-shore (deep water) wave climate around Sri Lanka, given the wind field over the relevant portion of the Indian Ocean.
(ii) Application of the said wave model for different scenarios of climate change and environment.

2.2. Research Methodology.

(i) Selection of an appropriate wave generation model.
   a. Possible models are MIKE SW, SWAN, WAM and WW3, and out of these WW3 and SWAN models were used as the model.
(ii) Acquisition of necessary wind data sets for wave modelling process.
   a. Possible sources for actual (historical) wind data are re-analysed wind data from global models (ECMWF, NOAA), Satellite measurements (Quickscat, Adeos) and Wind data from NOAA was used in the study as the historical wind data set.
   b. Predicted wind data (CCAM wind data) was provided by the CSIRO, Australia. The same wind model was used to generate the wind data for the Thailand study as well.
(iii) Acquisition of necessary deep-water wave data for the research domain to calibration and validation of the wave model.

a. The possible sources are Satellite altimetry (TOPEX/POSEIDON, JASON), SAR measurements by satellites (ERS), Wave hind casts from global models (ECMWF, NOAA) and limited deep-water wave measurements obtained off Sri Lanka.

(iv) Analyses of predicted wind data sets and determine the bias correction procedure if that is necessary.

(v) Set up the wave model to cover an appropriate domain around Sri Lanka, relevant to the data availability and the needs of the study. (As shown in Figure 1)
   a. For this study 300 x 300 area will be taken as the domain (Latitude -8^0 to 22^0 and Longitude 65^0 to 95^0)

(vi) Run the model with the predicted future wind field to predict the future wave climate scenarios.

(vii) Analyzing the differences in the wave climates for the current and future scenarios and assessing their impacts on various uses of the coastal seas.

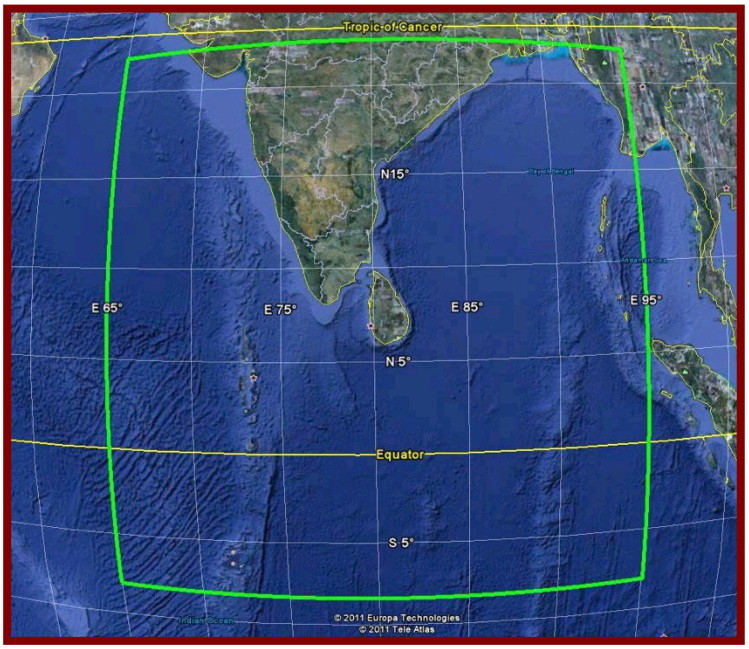

Figure 1. Domain Area of the Study

3. APPROACH TO THE RESEARCH PROBLEM

One of the most critical issues in wave modeling with future climate change projections is the uncertainty related to the projection of Regional Climate Changes. McInnes *et al.*, 2007 has pointed out the fact that there are three main sources of uncertainty to be considered when producing projections of global warming-induced climate change for a region for a given year in the future.

(i) The uncertainty in the future evolution of greenhouse gas concentrations in the atmosphere.

(ii) The uncertainty in how much the global average surface temperature will respond to increases in atmospheric greenhouse gas concentrations.

(iii) The uncertainty in how changes to the climate as a result of global warming will vary spatially and hence how the climate of the region under consideration will respond to an increase in global average surface temperature.

The first uncertainty can be addressed by considering different plausible storylines of future global demographic, economic and technological change. The Intergovernmental Panel on Climate Change's Special Report on Emission Scenarios has provided greenhouse gas emission scenarios associated with a suite of such storylines. The second uncertainty can be addressed by considering the future rates of future global warming in simulations of different climate models forced with increases in atmospheric greenhouse gas concentrations arising from emission scenario corresponding to the Special Report on Emission Scenarios (SRES). The third uncertainty, the uncertainty in the response of the regional climate to a given global warming value, can be addressed by considering the response of the climate of the region of interest to global warming in multiple climate models.

Although the work done relating to wave climate with future climate changes are rare to find for the Sri Lankan context, there are number of successful benchmark attempts that have been made in various parts of the world to address the said issue. Almost in all such research work carried out, the outcomes claim a significant relationship between the future wave climate and its forcing conditions that are to be caused by future climate change scenarios.

The work done by Hamer et al., 2010a emphasize the fact that insufficient projections of wave climate were available to assess the effects of probable climate change impacts on erosion of the world's coasts. According to the said work, at present, a considerable research effort is placed into regional ocean wave projections, with forcing conditions derived from a few selected emission scenarios from a few selected GCMs. It has identified several shortcomings of such an approach, as the following.

(i) Limitations of statistical confidence in the projections.
(ii) Additional effort requirements in modelling due to repeated model runs.
(iii) Laps in the global coverage, including the crucial areas that are most likely to be at risk due to changing wave conditions.

The propose solution given in the said study for this problematic scenario was to shift to global projections. The aim should be to use different wave models and statistical downscaling approaches to produce ensembles of wave projections that correspond to climate projections from different climate models for different emission scenarios. When combined, a distribution of projections will be available that will allow an assessment of all three levels of uncertainty (associated with forcing, climate models, and downscaling methods, respectively), presenting projections within statistical confidence intervals. The proposed design will provide suitable data on a global scale for carrying out surface ocean wave projections, focusing on mid- and late-twenty-first-century time slices, to service the increasing demands of the coastal impacts community. Future studies could include dynamic coupling of wave processes into coupled ocean–atmosphere global climate models (Hamer et al., 2010a).

According to work done by Swail & Wang, 2005, results of ocean wave climate change scenarios for the northern hemisphere oceans for the twenty-first century show that significant changes can be anticipated in both the North Atlantic and the North Pacific under all the three forcing-scenarios. The rate and sign of the projected future wave height changes are not constant throughout the 21st century and in some regions, these appear to be very much dependent on the forcing conditions as well. The rate of change appears to have a positive relationship with the rate of increase in the greenhouse-gases forcing.

4. USE OF WAVEWATCH III AND SWAN MODELS TO MODEL FUTURE WAVE CLIMATES

The basic scientific philosophy of SWAN is identical to that of WAM (Cycle 3 and 4) and it uses the same formulations for the source terms. On the other hand, SWAN contains some additional formulations, primarily for shallow water. Moreover, the numerical techniques are very different. WAVEWATCH III not only uses different numerical techniques but also different formulations for the wind input and white capping (The Wave Model Development and Implementation Group, 1988).

When SWAN is nested with WAVEWATCH III, it was noted that the boundary conditions for SWAN provided by WAVEWATCH III are not consistent, even the same physics are being used. The potential reasons are manifold such as the differences in numerical techniques employed and implementation for geographic area (spatial and spectral resolutions, coefficients, etc.) (Chawla & Tolman, 2007), (Tolman, 2010), and (Cavaleri et al., 2007).

To overcome this issue, the deep-water boundary of SWAN nest was located in WAVEWATCH III, where shallow water effects do not dominate. This is primarily important to avoid the existence of large discontinuities between the two models. At the same time, the spatial and spectral resolutions were adjusted so that it will not differ largely from one another, in order to avoid additional modifications of the model layout (Cavaleri et al., 2007), and (Chawla & Tolman, 2007).

Since SWAN in coded in such a manner, so that the scientific findings with another wave model could

be shared with SWAN. Similarly, SWAN could be readily nested in WAVEWATCH III for collaborative modelling of wave climates from Deep Ocean to shoreline. In this research study, the said similarities have been extensively exploited, in order to couple the WW3 and SWAN together to model the necessary outputs. (Cavaleri et al., 2007), (Chawla & Tolman, 2007), (The SWAN Team, 2010), (The SWAN Team, 2011 (c)), and (The Wave Model Development and Implementation Group, 1988).

5. MODELLED WAVE OUTPUTS

The outputs obtained via wave modelling are for three different time spans, could be identified as the following.

(i) Past wave climate - year 1981 to 2000
(ii) First future wave climate – year 2041 to 2060
(iii) Second future wave climate - year 2081 to 2100)

In order to match with the overall project requirements, outputs were obtained for a point near Colombo. This was also governed by the fact of available wave measurements. Most of the available wave measurements are for Colombo, thus it is convenient to compare the modelled wave output against the available wave measurement.

The comparison of the variation in wave climates were carried out with respect to the modelled past wave climate and the comparisons made reviled the fact that there might be a considerable differences in future wave climates along the western coast of Sri Lanka. Comparison of mean wave climates indicates an increment of about 6.5% for projected mean wave height during year 2041 – 2060 and 8.5% increment for the same during year 2081 – 2100. The mean wave direction of the first future wave climate shows an increment of about 8% and that is slightly reduced during the second projected wave climate. Although these are not very alarming in magnitudes, it indicates a significant variation, when considered with respect to the usual seasonal wave climates prevailing in Sri Lanka.

The predicted wave climate changes could be considered in correspond to main rainy seasons prevailing in Sri Lanka. Sri Lanka has two major monsoon periods and two inter-monsoon periods, where the time periods are to be considered as the following.

- Southwest monsoon – May to September
- Northeast monsoon – December to February
- First inter-monsoon – October to November
- Second inter-monsoon – March to April

Investigating the predicted changes during these major seasons is particularly important, due to the fact that the wave condition along the western coast of Sri Lanka is expected to exhibits major seasonal variations, especially during the southwest monsoon period. Table 1 indicates the predicted wave climates during the two major monsoon periods.

Table 1 Comparison of predicted wave climates during the monsoon periods

		Southwest Monsoon			Northeast Monsoon		
		1981	**2041**	**2081**	**1981**	**2041**	**2081**
Wave Heights	Mean	1.06	1.12	1.11	1.06	1.14	1.18
	Var	0.2047	0.2558	0.2808	0.2401	0.2317	0.2372
	Std	0.45	0.51	0.53	0.49	0.48	0.49
Wave Directions	Mean	249.88	269.84	267.32	247.47	270.12	272.27
	Var	279.51	294.81	371.58	291.79	335.40	383.66
	Std	16.72	17.17	19.28	17.08	19.31	19.59

Similarly, it is necessary to have a comparison of the predicted wave climates for the two inter-monsoon periods as well. Table 2 consists of the necessary information for the said comparison of

predicted wave climates.

Table 2 Comparison of predicted wave climates during the inter-monsoon periods

		First Inter-Monsoon			Second Inter-Monsoon		
		1981	**2041**	**2081**	**1981**	**2041**	**2081**
Wave Heights	Mean	1.06	1.09	1.13	1.02	1.14	1.16
	Var	0.1975	0.1757	0.2648	0.2755	0.3081	0.2620
	Std	0.44	0.42	0.51	0.52	0.56	0.51
Wave Directions	Mean	249.30	264.48	272.14	254.44	277.92	267.17
	Var	282.35	429.79	291.65	384.07	362.05	265.55
	Std	16.80	20.73	17.08	19.60	19.03	16.30

The above two tables indicate the variation of predicted wave climates during the two main monsoon periods and the two inter-monsoon periods prevailing in Sri Lanka. According to the modelled results, it can be observed that wave heights are higher during the Northeast monsoon period than those during the Southwest monsoon, which is not compatible with the expected monsoonal variations. However, it could be also noted that the inter-monsoonal wave heights are also at higher magnitudes. Since these observations are at variance with the existing pattern of wave heights, it could be concluded that the starting time period of the two main monsoons, as well as the two inter-monsoon periods have shifted slightly (i.e starting time period is being delayed slightly).

However, it is also possible that the inter-monsoon periods providing higher magnitudes of wave heights and the effects of the inter-monsoons extending over a longer period than that at present, thus the effects of the two main monsoon periods are being over shadowed by the two inter-monsoon periods. The following plots also support the said arguments and provide a better understanding about the variations in wave climates during the monsoonal periods.

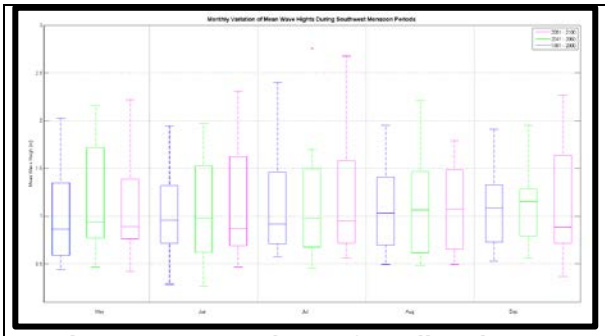

Figure 2. Comparison of predicted wave heights during Southwest monsoon

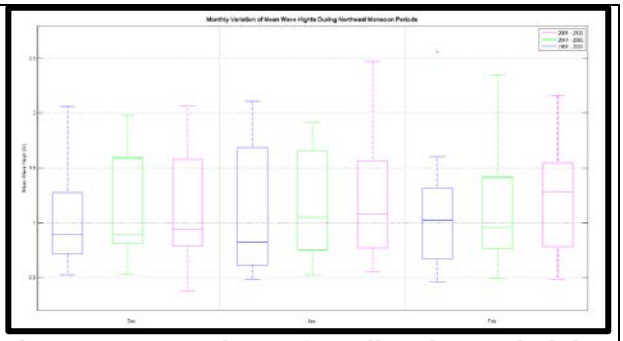

Figure 3. Comparison of predicted wave heights during Northeast monsoon period

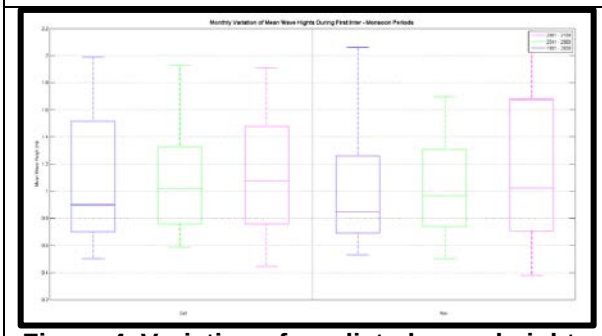

Figure 4. Variation of predicted wave heights during the first inter-monsoon period

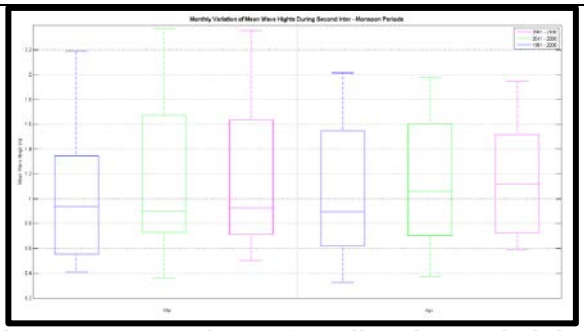

Figure 5. Comparison of predicted wave heights during the second inter-monsoon period

6. CONCLUSIONS AND FUTURE DEVELOPMENTS

Beside that fact that percentage values are not so significantly high for predicted mean wave climates, corresponding increment in mean wave height for the first projected wave climate is 7 cm and that value increases by another 2 cm for the second projected wave climate, resulting in a total of 9 cm increment for the mean wave height by the end of 21st century. This is a considerable amount of increment, given the fact that it may cause increased erosion and disruption along the heavily populated western coast of Sri Lanka.

The probable variations in the seasonal wave climate are also with considerable significance, because western coastal belt of Sri Lanka is heavily dependent upon the seasonal variations of the wave climate. Almost all the economic activities, such as fishery and tourism are functioning according to the prevailing wave conditions. If a shift in usual wave climate is to be observed, it can cause negative influences on the existing socio-economic system of the area. On the other hand, if the duration and the severity of extreme wave climate is to be extended over longer periods, it also causes major setbacks to the usual day to day life of the people, who are solely dependent upon fishery and tourism industry.

On the other hand, modelled wind data proved to be a very good input to force the wave models, as it was available at a higher resolution (0.5 degree) for the domain area of study. Therefore, it is also concluded that although there are certain issues that exist with the consistency of modelled wind data, together with necessary modifications to wave model, it could be used to force wave models to obtain very useful information.

The domain area for the study was restricted by the project requirements. However, if a more detailed study to be carried out for the Indian Ocean region, wind data should be available for a larger domain, covering a lager area of the Indian Ocean. This research study was not carried out at such a high detailed level of the entire Indian Ocean, thus the availability of modelled wind data was sufficient for this particular research study.

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Environmental Engineering and Management

Influence of Chromolena odorata and Mikania micrantha Plant Litter Extracts on Germination and Early Seedling Development of Bidens pilosa

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Abstract: *Allelopathy is explained as any direct or indirect effect of one plant on another through the production of chemical compounds that escape into the environment. This study aimed to examine the effect of leaf and root litter extracts od Chromolaena odorata and Mikania micrantha on percent germination and seedling growth of Bidens pilosa, specifically focusing allelochemical activity. The results of the present study showed that germination of Bidens pilosa seeds was affected by root debris extract of both invasive plants. Conversely, target species growth and development only inhibit by M.* m*icrantha aqueous extracts while C. odorata extracts were not influenced on the growth process.*

Keywords: *Chromolaena odorata, Mikania micrantha, Bidens pilosa, allelopathy*

1. INTRODUCTION

Spreading of alien plants is a well known phenomenon in almost all parts of world (Vitousek et al., 1997). Alien plant invasion is serious threat to the natural and semi natural ecosystems worldwide (Mgidia et al., 2007). High competitive ability of alien species has mentioned as a key factor promoting invasive success (Roy, 1990). Competitive ability depends on several factors incorporated with the invasive plants and allelopathy is one of the often discussed. Allelopathy is explained as any direct or indirect effect of one plant on another through the production of chemical compounds that escape into the environment (Rice, 1974). However, allelopathic effect varies depending on target plant species (McEvan et al., 2010).

There are several ways that allelochemicals reach to the soil, either as a secrete from living plant tissue or by decomposition and leaching of plant residues (Reigosaet al., 1999). The release of allelochemicals from litter decomposition could inhibit the establishment of seedlings not only of native tree species but also of herbaceous vegetation. Germination and seedling establishment are the most critical stages for plant population dynamics and success in the plant community. Exposure to allelochemicals in this stage may inhibit or retard growth and that may affect badly on the development of plant community.

Two invasive species prominently found in Sri Lanka are mile minute weed *(Mikenia micrantha)* and Siam weed *(Chromolaena odorata). Mikenia micrantha* is an invasive fast growing weed belonging to the Asteraceae family. This weed listed as one of 100 invasive species in world (Lowe et al 2001). This notorious weed spread to Mauritius, India, Sri Lanka, Bangladesh, south East Asia and pacific (Deng et al., 2004). On the other hand *Chromolaena odorata* is another species that has invaded natural ecosystems, that also belongs to same family, and in nature it is perennial herbaceous plant that native to Central America.

Our target species, *Bidens pilosa* is native species often found in temperate and tropical regions and interestingly belongs to same family Asteraceae. The studies focused on understanding the effects of

these Asteraceae family plant allelochmicals interactions on the growth and plant physiology has been less. In this study we especially incorporated both root and leaf for compare strength of separate sources that valuable for evaluation. Moreover, root mediated allelopathy is one of the neglected area in invasive plant studies (Roberts and Anderson 2001; Skulman et al. 2004).

We hypothesized that the presence of leaf and root extracts of *C.odorata* and *M.micrantha* would reduce the germination and early seedling development of *B.pilosa.* The purpose of this experiment was to examine the effect of leaf and root litter extracts on percent germination and seedling growth of *B.pilosa* specifically focusing allelopathic activity.

2. MATERIALS AND METHODS

2.1 Plant Material

Decomposed leaves and roots of respective two species were collected in nearby heavily infested site within Faculty of Engineering, University of Ruhuna, located in Galle ($6^{o}4'$ 4.72" N, $80^{o}11'$ 11.5" E) during February-March 2014. After collection of dead leaf and root they were dried at 40°C for 72 hrs and ground manually using mortar and pestle at room temperature (25-26°C). 50g of ground root and leaf matter was dissolved in 500ml distilled water for prepare separate solutions from two species. The extract was left for incubation about 24 hrs and filtered through 2 layers of filter paper (Whatman No.1) and stored in dark conical flasks and then stored in a refrigerator until use.

2.2 Collection of *Bidens pilosa* Seeds

Healthy uniform seeds of (*Bidens pilosa* L., family: Asteraceae) was collected from the same site above mentioned. The seeds were soaked in distilled water for one hour. Then the seeds were surface sterilized with 1% H_2O_2 solution for 5 minutes, and rinsed with double-distilled water several times for complete removal of the sterilant.

The effects of the extracts on the germination and seedling growth of *B. pilosa* was tested by placing 10 seeds in Petri dishes.(6 replications) containing 2 layers of Whatman No.1 filter paper saturated with the extracts (5 ml). A separate control series was also set up using only distilled water. The total numbers of 36 petri dishes were utilized for two experiment and moisture in the petri-dishes was maintained by adding about 2 ml of extract/distilled water (control) as and when required.

Total number of seeds germinated after 1 day were counted both in Petri-dishes and compared with the control respectively up to every day for 10 days. Root and shoot length (average of 10 plants) and dry weight (of 10 plants) were taken and compared with the control. The lengths of radicle and hypocotyl of seedlings were measured on a single day when its seeds had reached the maximal germination rate in separate treatments i.e., on day 8 for *C.odorata* and extracts, and 7 day for *M. micrantha* extracts exposed seedlings. Corresponding Germination percentage was calculated using below given formula.

Germination percentage= Number of germinated seeds/Total number of seeds×100

2.2 Statistical Analysis

The data were subjected to homogeneity (Levene) tests. When this assumption was met, one factor analysis of variance (ANOVA) was performed for all corresponding variables: root dry weight, shoot dry weight, root to shoot ratio etc, for one fixed factor (aqueous extract). The raw data were used to compute the mean± standard deviation. All Statistical analysis were performed by using SPSS version 16.0 (SPSS, Chicago,IL,USA).

3. RESULTS

When exposed to *C. odorata* extracts, *B. pilosa* seeds reached an average number of 93% in the control by the end of experiment while significantly least number of seeds germinated at the root extract. Furthermore, compared to leaf extract seeds germinated at root extracts were significantly decreased by 47%. In contrast, when exposed to *M .micrantha* extracts, *B. pilosa* seeds reached an average of 91% compared to the no-extract control by the end of the experiment. However, seed germination percentage between control and the *M.micrantha* leaf extracts were not significantly different. Interestingly, *M. micrantha*, root extracts also highly reduced the seed germination that near to 44% compared to the 81 % germinated at leaf extracts.

Hypocotyls length was affected significantly *(ANOVA, $p < 0.05$)* by root and leaf extracts of *M.micrantha* debris. The highest hypocotyl length was observed in control (9.66mm) and the lowest was observed under *M. micrantha* leaf extract exposed seedlings (4.5mm). However, seedling height difference between the leaf and root debris of *M.micrantha* was trivial *($p>0.05$)*. In contrast, the effect of *C.odorata* extract on *B. pilosa* seeds showed a different pattern and final hypocotyls length of three different treatments were not significantly different from the root leaf or control. On the other hand seedlings dry weight was not significantly influenced by the any of the aqueous extracts of both invasive species *($p>0.05$)*.

Among the *M. micrantha* root and leaf extracts, more significant radical inhibition was observed in the root extracts exposed seeds of *B. pilosa* compared to the control *($p<0.05$)*. However, none of the extracts of *C.odorata* were able to significantly change radical elongation of *B. pilosa* seedlings. The total dry weight of *B. pilosa* seedlings were not affected by root and leaf extracts of *C.odorata* and in the same way *M.micrantha* root and leaf extracts not effected to corresponding dry weight.

Table 1 Radicle length, Hypocotyl length, and total dry weight under *C.odorata and M.micrantha* leaf and root debris extracts. Data are mean±SD,n= 10

variables	*C. odorata* extracts			*M.Micrantha* extracts		
	Leaf	**Root**	**Control**	**Leaf**	**Root**	**Control**
Radicle length(mm)	221.83±5.19	26.0±3.84	27.00±4.83	15.83±2.48[a]	31.83±6.67[a]	10.5±3.93[b]
Hypocotyl length(mm	7.16±1.72	6.5±2.16	9.66±3.38	7.0±1.26[a]	4.5±1.64 [a]	9.66±3.38[b]
Total dry weight(g)	0.019±0.002	0.019±0.003	0.019±0.001	0.017±0.002	0.016±0.002	0.019±0.001

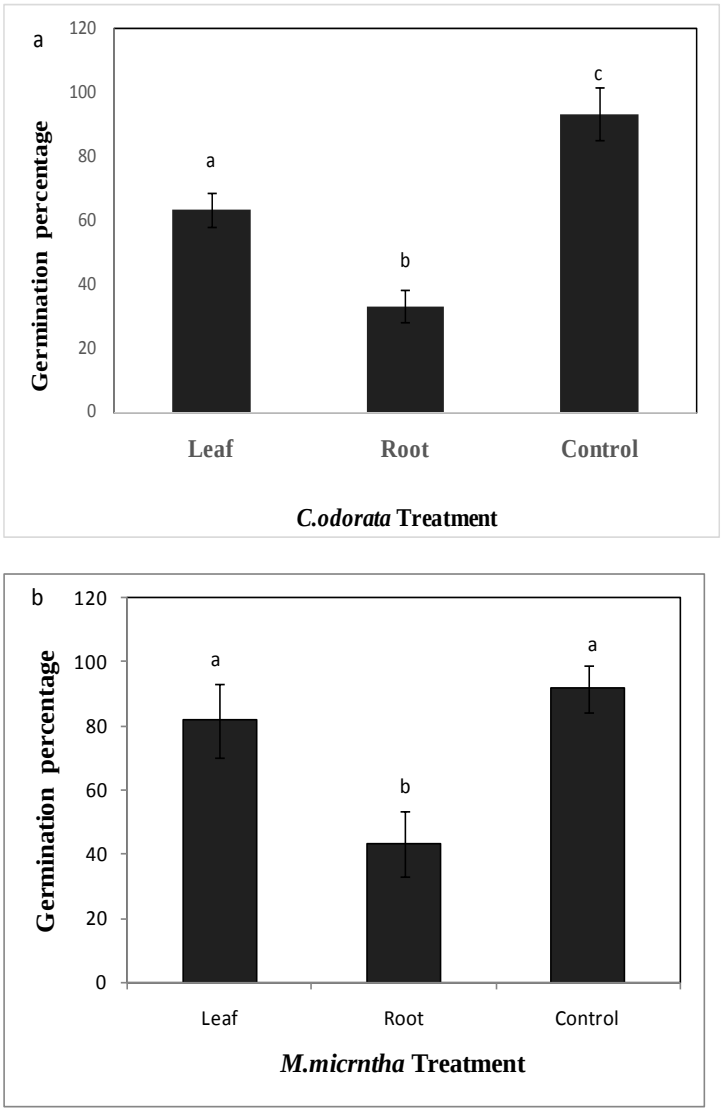

Figure 1. Influence of a *C.odorata* plant extracts, b *M.micrantha* plant extracts *on B.pilosa* seed germination, Mean± SD, n=10. Different letters mark significant differences, (p<0.05)

4. DISCUSSION

In this study, it was found that a strong overall inhibitory effect of *M. micrantha* and *C.odorata* on *B.pilosa* seed germination. By using native species as target species, we provide more insight into interactions that could occur in the Sri Lankan field conditions. In our study also it is proven that roots are more sensitive to allelochemicals than other parts of the plants, as earlier explained by others for similar species such as *Parthenium hysterophorus* (Wakjira et al., 2005). In both assays, root growth was more sensitive indicator of growth inhibition than seed germination or emergence (percentage or speed). This is in agreement with the results of several authors (Oliveira and Campos 2006; Ruprechtetal. 2008) because more processes were affected during the seedling growth than in germination. We have observed that *B.pilosa* was sensitive to extracts of *M.micrantha* and *C.odorata*, a pattern that was quite complex phenomenon that yet not clearly understood. However, allelopathic effects of each invasive species also varied based on the radical emergence and plumule elongation. Extracts of *C.odorata* showed trivial effect on *B.pilosa* seedling development while both root and leaf debris solutions of *M.micrantha* reduced the target species growth process more consistently.

This results is somewhat contradicted with the research findings (Ambika, 2002; Hills and Ostermeyer, 2000) observed that the leaves of *C. odorata* contain a large amount of allelochemicals, which may retard the growth of crop plants while considering the earlier researchers findings

compatible with leaf leachates of *M. micrantha* reduced the root and shoot elongation of *Oryza sativa and Triticum aestivum* etc. (Baral et al.,2011).

5 CONCLUSION

The results of the present study showed that germination of *Bidens pilosa* seeds was affected by root extracts of both invasive plants. Conversely, target species growth and development only inhibit by *M. micrantha* aqueous extracts while *C. odorata* extracts were not influenced for the growth process. It seems that mechanisms of adaptation to alleopathy in both species are not the same and *Chromolaena odorata* is less allelopathic than to *Mikania micrantha* in case of *Bidens pilosa*.

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A Pilot Study of Water Quality and People's Importance Level towards Sustainable Management of Kandy Lake Basin, Sri Lanka

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Abstract: *This study aimed to clarify the factors affecting people's preferences for sustainable management of Kandy lake basin, Sri Lanka. We conducted water sampling surveys to clarify the current situation on water quality of the lake and related water bodies, and an interview survey to investigate preferences of local people and tourists on functions of Kandy Lake. As the result, the lake was highly eutrophicated because of nutrients inflow by domestic wastewater, as local residents had recognized. Regardless of respondents' property, high importance levels were observed in functions as a spot of religion, culture, history and tourism. Younger generation emphasized recreational functions like boating and bird watching, more than older generation. It would be important to watch a generational change of preferences for sustainable management of a lake. Communication among residents, scientist and local government and would be more important to improve the future lake environment.*

Keywords: *sustainability assessment, decision making, water resources management, ecotourism, lake basin development, knowledge sharing process*

1. INTRODUCTION

Urbanization sometimes drastically changes materials flow and its balance in a certain area even within a short period (Hara et al., 2010). Such rapid urbanization often results in severe environmental deterioration (Hiramatsu et al., 2009; Honda et al., 2010) because countermeasures, which need to wait for policy-makers' decision, cannot catch up with the change. Sustainability assessment is one of the important key tools to develop and evaluate policies for a sustainable society. Various indicators have been developed to evaluate sustainability of environment (Hsu et al, 2014; Bausch et al., 2014). Though a number of parameters are to be evaluated in assessment of sustainability from a wide range of aspects like sanitation, ecosystem, etc., many of the indicators developed so far attempt to integrate each parameter by giving a certain weighing formula, which is often decided by experts or scientists including developers of the indicator. However, importance on each aspect often varies dependent on

evaluators or stakeholders. Therefore, in developing such polices for a sustainable society, it is important to clarify people's preferences to decide resources allocation for implementation.

Kandy Lake is an artificial lake built in 1807 besides the Temple of the Sacred Tooth Relic, in Kandy City, a UNESCO world heritage site and one of the famous tourist spots in Sri Lanka.. However, due to the rapid urban development and increase of tourists, severe pollution of the lake is recently concerned. In 2009, a massive die-off of fish was reported in Kandy Lake (Kawakami et al., 2011). However, because of the symbolic nature of the lake, opinions of various stakeholders, including local residents and tourists, should be carefully reflected in making policies for sustainable management of Kandy Lake. Their thought and preferences on the lake management may vary dependent on of the nature of stakeholders;local residents or tourists, age, education level, occupation, etc.

To measure people's preferences, Hirayama et al. (2011) reported that Analytic Hierarchy Process (AHP) was useful to clarify relative importance levels of lake functions by residents in Biwa Lake basin, Japan. Moreover, it was clarified that willingness-to-pay (WTP) of each policy could be calculated as the product of the WTP including people's value for lake and that the weight of the lake function could be evaluated by residents and government officers with AHP and contingent valuation method (Hirayama et al, 2005). In this study, we investigated people's preference on lake functions to reveal factors which affect deference in importance level among people. That would be useful for sustainable management of Kandy Lake with good consensus of people and for development of a better methodology of sustainability assessment on water environment.

2. MATERIALS AND METHODS

2.1. Target Area

Kandy Lake and surrounding area were selected as the target area (Figure 1). Kandy Lake is an artificial lake built in 1807 besides the Temple of Tooth in Kandy City, Sri Lanka. Its surface area is 14.7 ha and perimeter is 3.3 km. Kandy City is located in a tropical rainforest area. Average annual precipitation is 1,840 mm. Kandy City had 125,400 of population and 4,600 persons/km^2 of density in 2011. Sewage treatment system in Kandy was currently under planning and not in service at the moment.

2.2. Water Quality

To investigate the current conditions of water quality in the target area, water samples were collected at total 18 sites: 8 locations in Kandy Lake (L1-L8), 4 inlet water channels (IN1-4), 3 locations in outlet canal (O1-3) and 3 locations in Mahaweli River, which receives water from the outlet canal (R1-3) (Figure 1). Samples were kept in a cool condition until analyzed, except during travel from Kandy to Tokyo for about 16 hours. Total organic carbon (TOC), dissolved organic carbon (DOC), total nitrogen (TN) and dissolved nitrogen (DN) concentrations were analyzed by a TOC analyzer (TOC-V, Shimadzu, Japan). The major cations and anions were analyzed by an ion chromatography system (LC-20A Series, Shimadzu, Japan). These analyses were conducted within 6 days after sampling, the minimum period of shipping from Kandy to Kanazawa, Japan, where samples were analyzed for chemical parameters. Heavy metals were analyzed by an ICP-MS (Agilent 7700, USA). Fourteen samples were selected for total coliform and *E. coli* analyses by colony counting on the very day of sampling in Sri Lanka, by colony counting on Chromocult Coliform Agar ES (Merck Millipore, USA).

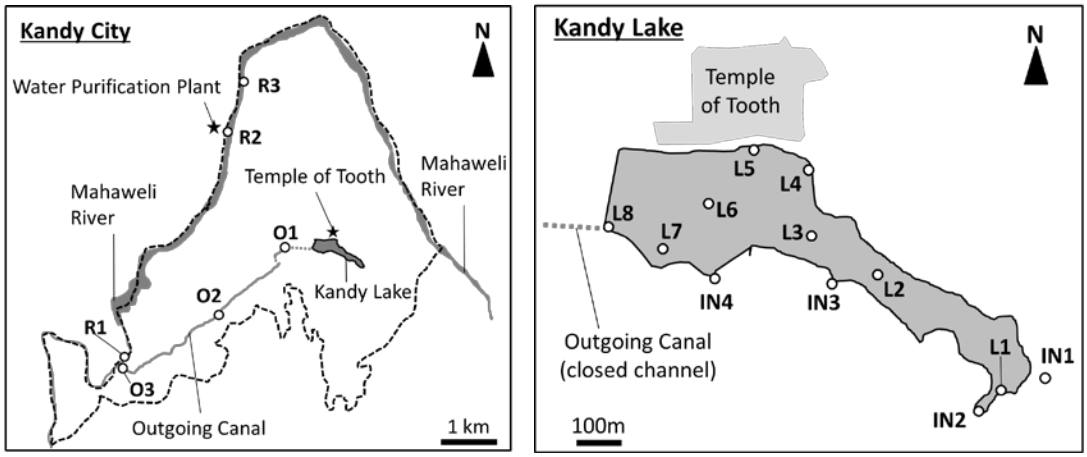

Figure 1 The target site and sampling points in Kandy City (left) and Kandy Lake (right).

2.3. Interview Surveys

Questionnaire-based interview was performed to investigate individuals' preferences on functions of Kandy Lake. In this survey, 20 items were set as the functions of Kandy Lake (Table 1). Each respondent was asked to evaluate each function as "very important", "moderately important", "a little important", "not important", or "no idea". Personal data of the respondents like age, sex, occupation, home location, relationship with the lake, stay period in Kandy city and frequency of visiting lake was also inquired.

Table1 Lake functions raised in the questionnaire

1.	Domestic water usage	11.	Angling / recreational fishing
2.	Industrial water usage	12.	Boat and canoeing
3.	Use for irrigation	13.	Birds / Wild animals watching
4.	Fishing (for food)	14.	Place for education
5.	Flood control	15.	Tourist spot
6.	Natural purification of wastewater / urban drainage	16.	Local Symbol / landmark
7.	Place for event (festival, religious events)	17.	Historical attraction
8.	Habitat of plants and animals	18.	Cultural spot
9.	Swimming	19.	Religious spot
10.	Water amenity	20.	Landscape / scenery

Two separate surveys were conducted, targeting local resident stakeholders and lake-side walkers respectively.
In the survey with local resident stakeholders, 21 stakeholders living in Kandy City were selected which include 4 politicians, 3 officers of local government offices, 4 representatives from hotel business sectors, 3 representatives from residents were interviewed. The respondents were asked which they think is important in the past, present and future of Kandy Lake basin among the 20 lake functions (as in the Table 1). After the questionnaire survey, free discussion among the stakeholders was also held to understand people's relations and thoughts on sustainable management of Kandy Lake. In the second survey, 45 persons who visit the lake side were randomly selected, including domestic and foreign tourists, and local residents.

We examined the data by each question and made clear the difference in importance level of lake functions by cross tabulation. Regarding importance level, collected data were analyzed after translating "very important", "moderately important", "a little important" and "not important" into the score of 3, 2, 1 and 0, respectively.

3. RESULTS AND DISCUSSION

3.1. Current Situations on Lake Water Quality

Kandy Lake was found to be highly eutrophicated. Higher TN concentrations greater than 1 mg N/L were observed in all the samples of Kandy Lake as in the Figure 2. Over saturation of DO concentration at 8.1 – 10.5 mg/L in lake water samples also indicated active primary production by microalgae (Figure 3). Phosphorus was a limiting factor of microalgae bloom because N/P ratios were higher than 13.5 (Figure 4). The TOC concentrations were in the range of 3.2 – 4.4 mgC/L (data not shown).

Kandy Lake received water from the catchment through 4 major inlet channels (IN1~IN4) as well as small water channels connected to the lake. Water from those channels was supposed to contain wastewater from domestic residences and hotels. Especially at one of inlet channel (IN3), high concentrations of coliform and ammonium (Figures 2 and 5) were observed, suggesting high contamination with sewerage. Phosphorus was also detected in IN2 and IN3, which had relatively higher inorganic nitrogen concentrations than other inlet channels (IN1 and IN4). The main source of phosphorus was probably derived from sewerage rather than phosphorus-containing detergents, because their N/P ratios were 25 and 35 at IN2 and IN3, respectively. No significant contamination with heavy metals was found in the lake and inlet waters.

Currently, water in Kandy Lake was mainly used for landscaping, and gardening of plants at the Temple of Tooth. Therefore, current water quality probably do not result in any significant health problem. However, it is at risk of due to high eutrophication which may cause harmful microalgae bloom or oxygen depletion in the bottom layer. It is important to monitor and control phosphorus loading through domestic wastewater to prevent the harmful microalgae bloom.

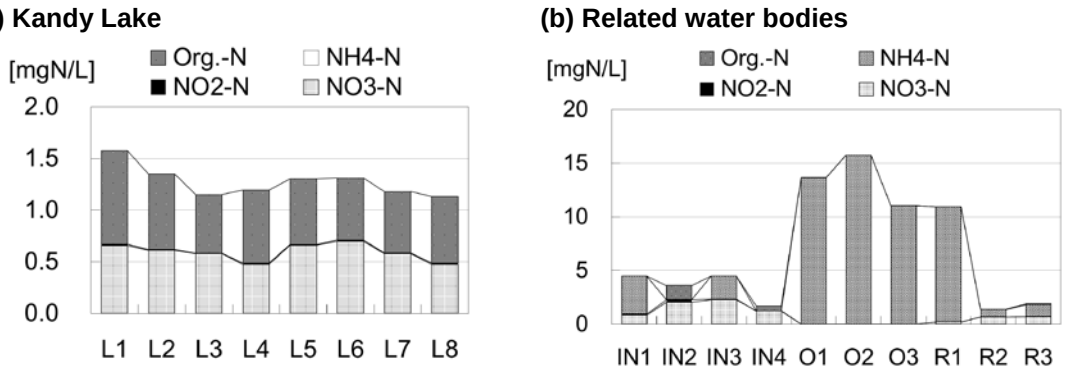

Figure 2 Nitrogen concentrations in (a) Kandy Lake and (b) related water bodies.

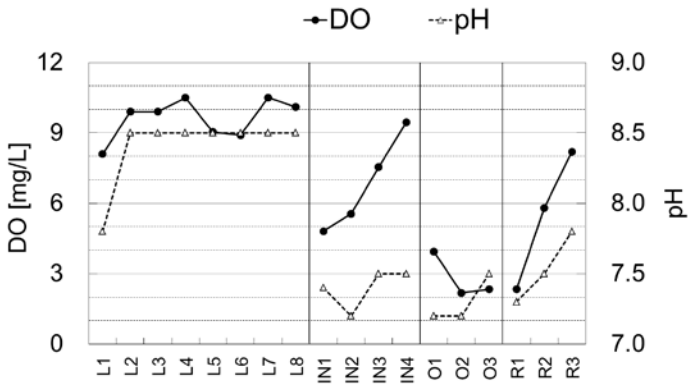

Figure 3 Dissolved oxygen (DO) concentrations and pH in Kandy Lake and related water bodies.

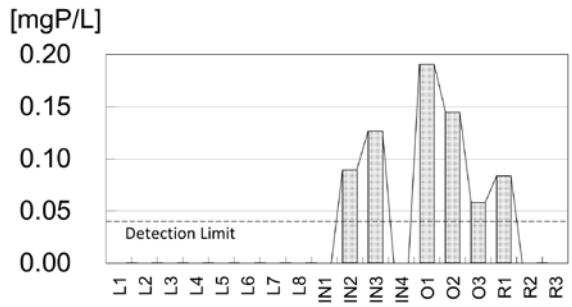

Figure 4 Phosphorus (PO_4-P) concentrations in Kandy Lake and related water bodies.

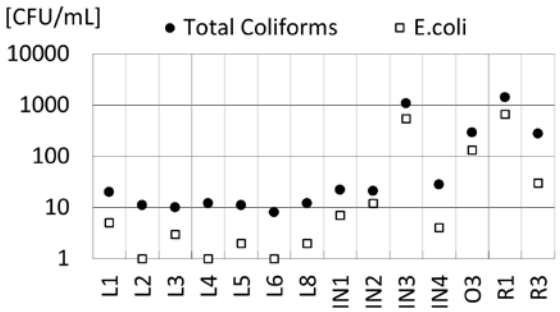

Figure 5 Total coliform and *E. coli* concentrations in Kandy Lake and related water bodies.

Kandy Lake have only one outlet canal which meets Mahaweli River at around 5km down. The outlet canal also receives untreated domestic wastewater from several point and non-point sources. High concentration of ammonium and coliform were observed at points in the canal (O1, O2 and O3) and near the canal discharge point to the Mahaweli River (R1) (Figures 2 and 5). Contamination of this outlet canal should be seriously concerned because downstream Mahaweli River is used for irrigation and most importantly for water intake of water treatment plant is located at R2 to supply tap water to Kandy City.

3.2. People's Preferences on Lake Functions

Generation gap was one of the factors to indicate distinct difference in preferences on lake functions. The recreational functions like 'boating' and 'birds and animals watching' were put more importance with each succeeding generation. On the other hand, older generation didn't put importance on 'swimming' and 'angling' more than younger generation (Figure 6). From comparison by location of residences, it was implied that Sri Lankan residents from outside of the lake basin put more importance on natural environment of the lake than the basin residents, because score of 'animals/birds and animals watching' was remarkably higher. The lake basin residents had similar perception with older generation. Both groups didn't emphasize 'swimming' and 'angling' (Figure 7) as important functions. This is probably because most of the residents of Lake Basin belonged to older generation. Other interested point is that tourists from foreign countries answered 'domestic use' and 'irrigation' are more important than Sri Lankan residents. High importance was found in 'purification and amenity' at present and in the future than in the past (Figure 9). There was no significant trend based on frequency of the visit (Figure 8). However, it was noted that younger generation shows their preferences towards some functions related to a water-friendly park, such as 'swimming', 'education', 'amenity' and 'purification'.. Younger generation may expect lake to bring another benefit in addition to as a 'religious' and 'historical' spot. Therefore, It would be important to take into account the generational change of preferences for sustainable management of a lake in future.

These outcomes brought a glimpse of people's preferences on management of Kandy Lake. However, the sample size were not large enough to conclude any particular suggestions. If we could apply this methodology to get the bigger and balanced data by age, sex, nationality and relationship with the lake, we will be able to discuss about how to connect these people's preferences and policy strategy. In addition, the conclusion should be share among stakeholder including government and set the vision of future Lake Kandy on round table. The sharing and discussing process are most important for lake basin management.

For better understanding of residents' thought and values of Kandy Lake, we also studied the residence perception toward the water pollution and enhancing the beauty around the Kandy lake. Many residents around the lake believed "lake water quality is getting worst", "wastewater treatment facilities are not enough", "improper solid waste discharge deteriorate lake quality". Some residents referred about "utilization rule is needed", "we have to do something for next generation." Residents were possibly afraid of ignorance about future lake environment. Experts, local government officers

and residents have to enhance their communication. Continuous sharing of information and discussion among key stakeholder is very important for sustainable lake management.

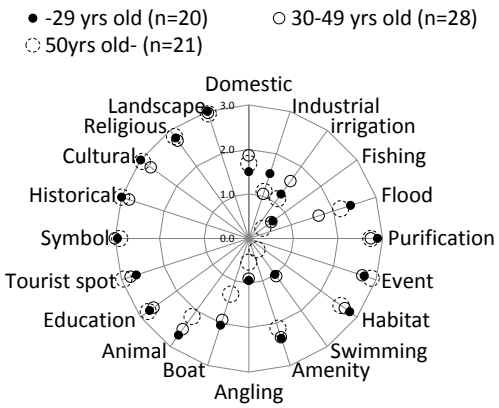

Figure 6 Difference of importance level by age.

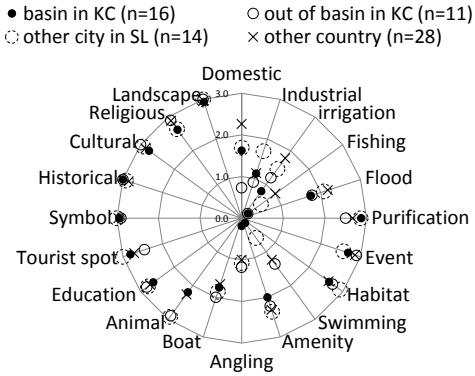

Figure 7 Difference of importance level by location of residences.

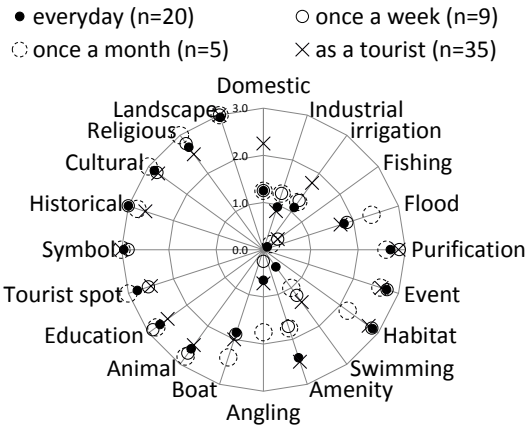

Figure 8 Difference of importance level by frequency of visit.

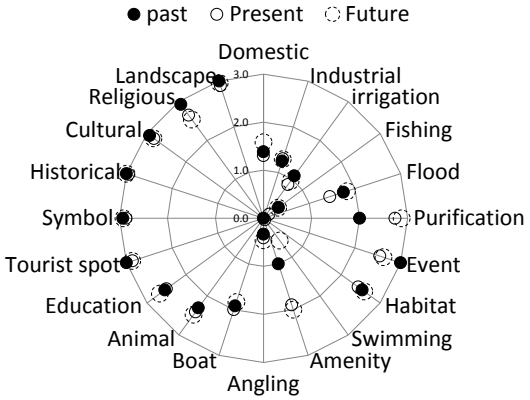

Figure 9 Difference of importance level by timeline, scored by local residents stakeholders (n=24).

4. CONCLUSIONS

Regardless of stakeholder type, all the stake holders of Kandy Lake identified the lake as an importantspot of religion, culture, history and tourism. Younger generation emphasized recreational functions like, boating and bird watching, more than older generation. On the other hand, older generation and lake basin residents didn't put importance on 'swimming' and 'angling', compared with younger generation. It would be important to take in to account a generational change of preferences for sustainable management of a lake. We also found that the lake was highly eutrophicated because of nutrients inflow by domestic wastewater. Communication among experts, local government agents and residents would be more important to improve the future lake environment. Additionally the figures which visualized people's difference thought and value may be useful in sharing or discussion process.

5. ACKNOWLEDGMENTS

We appreciate the local resident stakeholders and all interview respondents for their kind cooperation to our survey**.**

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Spatiotemporal Variation of Water Quality Around and Inside an Open Solid Waste Dumpsite in Sri Lanka

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Abstract: *Burial of municipal solid wastes in landfills is the most common disposal alternative in most countries. Electrical conductivity (EC) provides an index of the total dissolved electrolyte in leachate which is generated by percolation of excess rainwater through the waste layers in a landfill. The objective of this study is to observe the groundwater and retained water in an open dumpsite in order to formulate the stabilization of abandoned waste with time and to minimize the environmental impact of solid waste dumping sites. This paper describes the investigation of EC and/or cations on water samples in an open dumpsite in the Central Province, Sri Lanka. Investigation of 13 boreholes for water samples made on two transects line along the slope each in old site and new site was done about one year. Although EC values of groundwater in new site near the river were decreasing rapidly, it did not change in old site. The different conditions of geological information and water levels between these two sites showed that the deep fossil valley around new site caused washing of a permeable layer by a large amount of river-bed water. Since EC values of the retained water in new site were increasing during this study, washing out of ingredients included in the abandoned waste will be continue for a while. The elapsed time induced decreasing of the concentration of NH_4^+, Na^+ and K^+ in comparison with Ca^{2+} and Mg^{2+}.*

Keywords: *solid waste dump site, borehole log, retained water, groundwater, electrical conductivity, cation*

1. INTRODUCTION

Dumping of municipal solid wastes in landfills is the most common disposal alternative in most of the developing countries. By percolation of excess rainwater through the waste layers in a landfill, leachate is generated, which is one of the major sources of pollutants from landfills. Leachate has organic and inorganic substances which contains various types of chemicals. Electrical Conductivity (EC) provides an index of the total dissolved electrolyte leached from abandoned waste. According to the investigation of EC and details of inorganic substances in the leachate from organic waste *(Guo, 2001)*, the trend of sum equivalent concentrations of cations such as Na^+, K^+, Ca^{2+}, Mg^{2+} and NH_4^+ were similar with that of EC. There was no such kind of investigation in tropical countries where high

rainfalls are experiences.

Mowjood et al (2013) have shown that, there is perched water (retained water) at the bottom of waste layer which has no direct continuity with groundwater aquifer in the dumpsite. The objective of this study is to observe the groundwater and retained water in an open dumpsite in order to formulate the stabilization of abandoned waste with time and to minimize the environmental impact of solid waste dumping sites. This paper describes the investigation of inorganic ingredients on groundwater and retained water in an open dumpsite in the Central Province, Sri Lanka.

2. MATERIALS AND METHODS

2.1. Location

The Udapalatha dumpsite (N 7° 09', E 80° 35') is located near the right bank of Mahaweli river in Udapalatha Pradeshiya Sabha (PS) in the Central Province of Sri Lanka. The average annual rainfall is above 2000 mm with an average annual temperature of 24.5 ºC *(Department of Meteorology, 2012)*. The dumpsite has been used by both Gampola Urban council and Udapalatha PS for around seven years. The site has been abandoned in 2011. The dumpsite has two sections, old and new with 7 and 0.5 years of operation. Although majority of landfilled waste was domestic garbage, construction and demolition waste also was found. Old site was almost covered with thin loam soil and was overgrown with weeds. New site was covered partially with loam and weeds.

2.2. Monitoring Wells

Thirteen boreholes were made on two transects line along the slope each in old site and new site as shown in figure.1. A hydraulically operated rotary drilling machine was used for installation of monitoring wells (BH) of groundwater. Dry rotary boring was used for establishing monitoring wells (PBH) for retained water and landfill gas with 127 mm of PX size casings. Furthermore, acquiring coordinate values (X, Y) and a height value (z) were measured before boring.

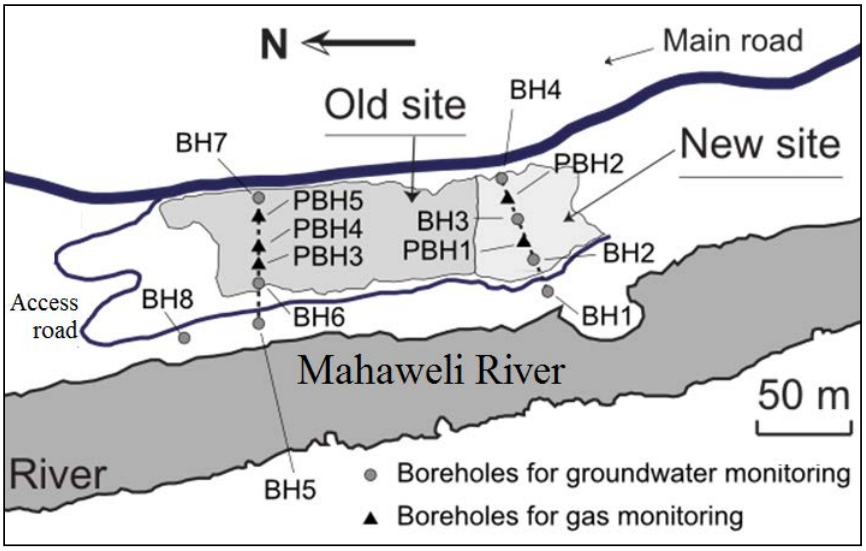

Figure 1 Locations of boreholes in the dumpsite.

The details of the monitoring wells are shown in figure 2. Inner and outer diameters of the PVC and High-Density Polyethylene (HDPE) pipes were 50 mm and 63 mm, respectively. The bottoms of the cases were perforated up to 0.5 m above the bed rock boundary with the perforated holes of 6.0 mm diameter (Porosity of PVC pipe is 5%) for groundwater monitoring. Perforated PVC pipes were covered with a mesh. For retained water monitoring, the cases were with similar perforation but for the entire depth of waste layer from the original ground. Perforated HDPE pipes were not covered with a mesh to prevent clogging of meshby microorganisms.

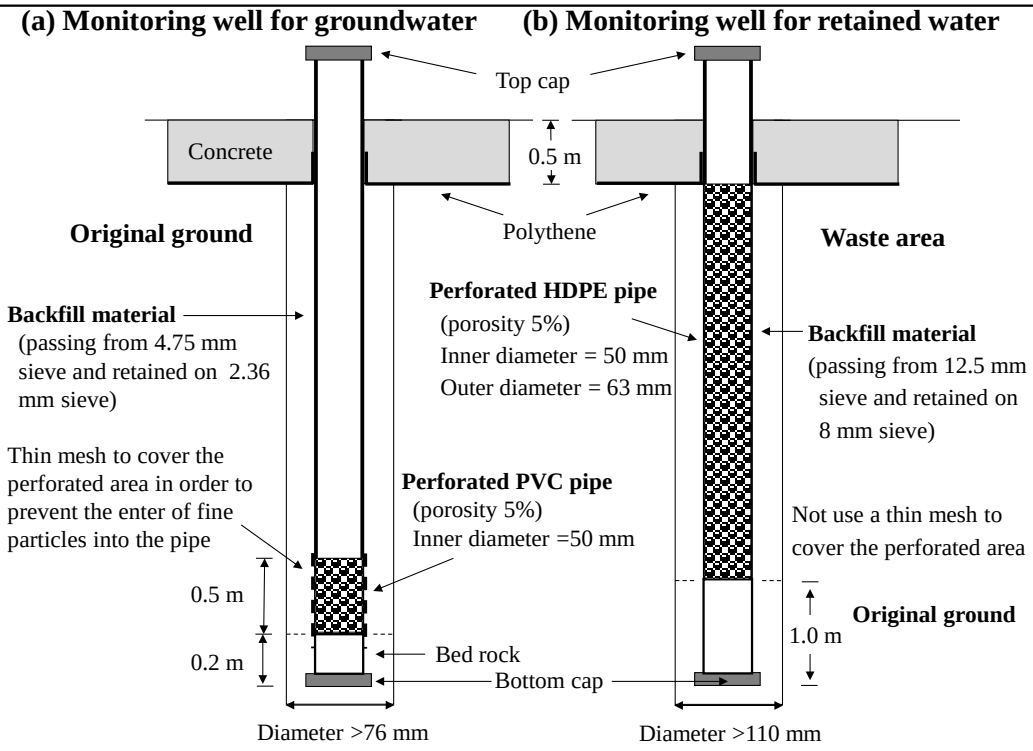

Figure 2 Schematic diagram of monitoring wells

2.3. Sample Collection and Analysis

The groundwater and retained water were monitored for one year from March 2013 with around one month interval for water level, EC, Na^+, K^+, Ca^{2+}, Mg^{2+} and NH_4^+. Water samples were collected from the monitoring wells (BHs and PBHs) after water levels were measured. Water in the monitoring wells were pumped before collecting the samples.

EC was measured in-situ using a multi-parameter meter (D-54 HORIBA scientific). Water samples were taken to the laboratory, filtered through 0.45 µm membrane filters for measuring Na, K, Ca and Mg. These four cations were analysed with Atomic Absorption Spectrophotometer (AA-7000 Shimadzu Corporation). Ammonium ion was analysed using a spectrophotometer (DR 2700 HACH Company).

3. RESULTS AND DISCUSSION

3.1. Geophysical Log Descriptions

Figure 3 shows the topography of the site, location of boreholes and geomorphology of the log of monitoring wells. Altitude of old and new sites ranges 487-509 m and 484-504 m amsl (above mean sea level). BH8 in the intact zone was at the same altitude as of BH5.

As for most BHs, a sandy soil layer was found just above the bed rock. The boundaries between a sand layer and a bed rock of each BH were connected (discontinues line in Fig. 3) in order to visualize the down stream of groundwater. Although the overall altitude of bottom boundary decreased towards the river, it was revealed that the this boundary altitude increased just at the river as shown in between BH 5 - BH6 and BH1-BH2 (Figure 3). A fossil valley might be made along the course of the old river. The valley of position BH2 at new site was especially deep. Furthermore, waste was included in BH2, BH5 and BH6 which assumed that waste had not been dumped before boring.

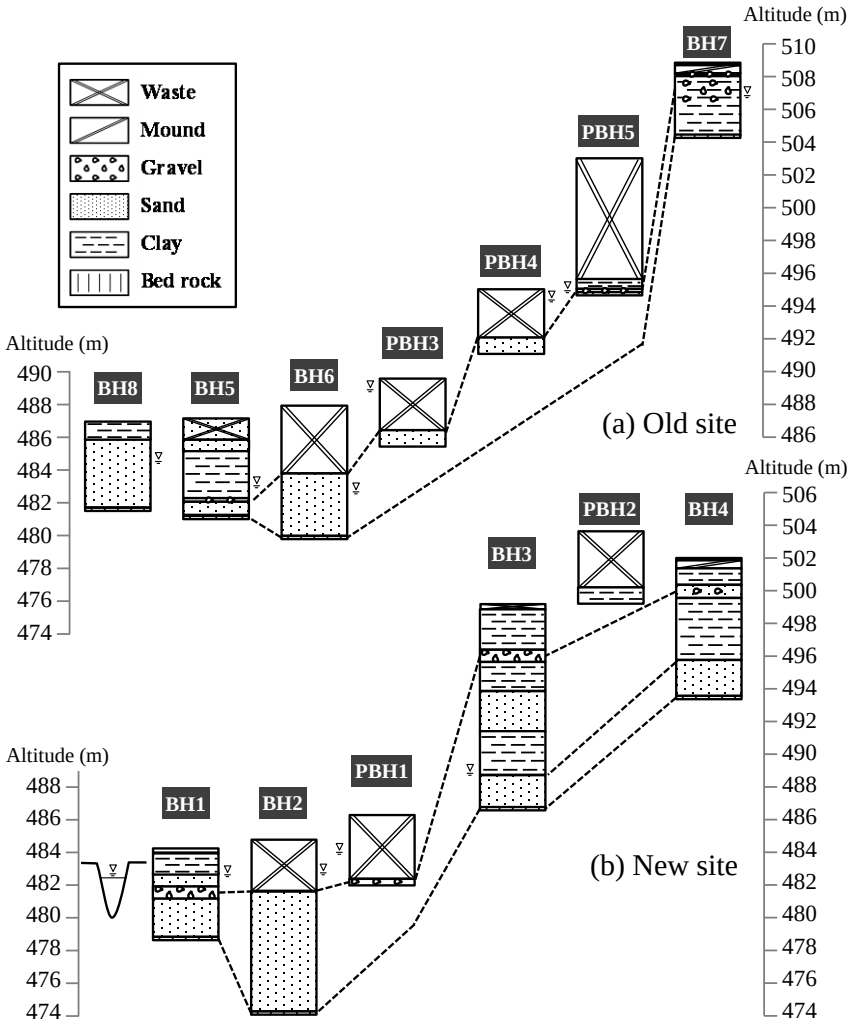

Figure 3 Borehole settings in the dump sites

Temporal variations in water levels in boreholes are shown in figure 4. Although water level of the river had a possibility to influence the down slope boreholes (BH1, BH2, BH5, BH6 and BH8), all water levels of BHs in old site (BH5, BH6, BH7 and BH8) changed almost equally with time. On the other hand, trend of water levels of BHs in new site was different among the upper and down slope. Because near the down slope boreholes in new site had relatively high water levels, it appears that the deep fossil valley around new site caused washing of a permeable layer such as sand or waste by a high amount of river-bed water.

As for PBHs at the waste layer, PBH3 and PBH4 in old site had a high level of retained water within 1 meter from the surface. PBH5 positioned on the top of site had low water level, thus little retained water. The water levels of PBH1 and PBH2 were 1.5-2.2 meters and 1.8-3.3 meters, respectively. Unlike the tendency of the groundwater level, the retained water levels of new site were deeper than that of old site.

3.2. Water Quality

3.2.1. Electrical conductivity of groundwater and retained water

The temporal variations of EC of borehole water are shown in figure 5. EC values of river and BH8 ranged 4-14 mS/m and 14-75 mS/m, respectively.

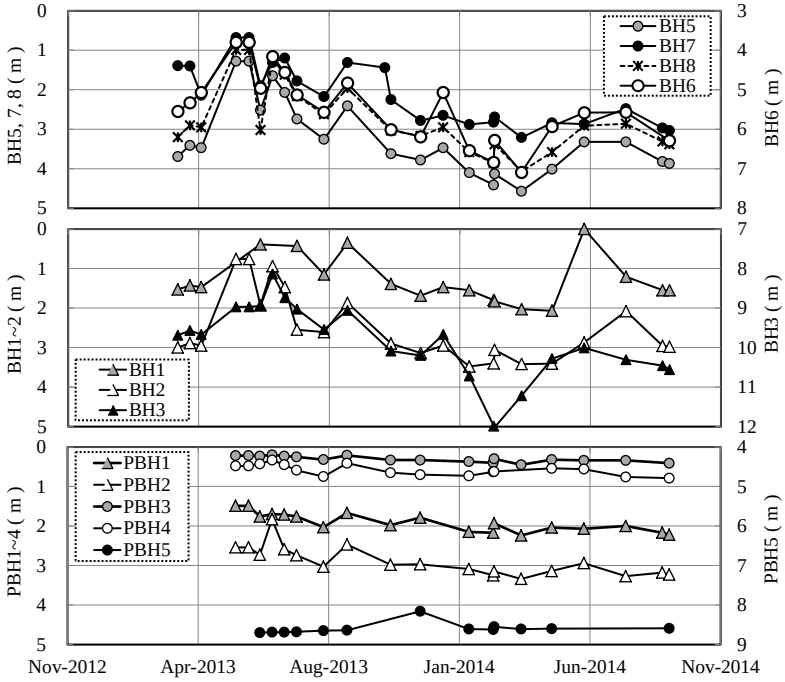

Figure 4 Temporal variation of water levels in boreholes; BHs and PBHs.

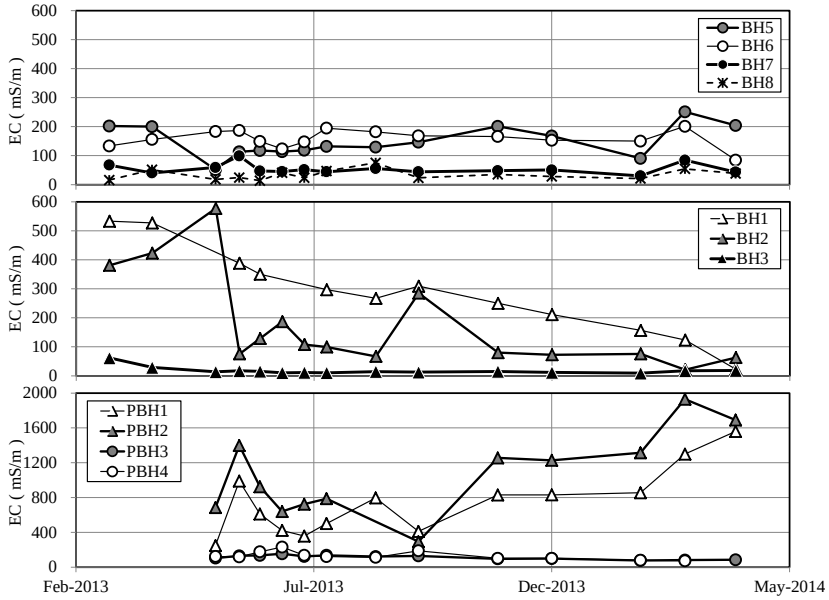

Figure 5 Temporal variation of EC of water in boreholes.

Although EC of groundwater at the upper slope (BH3 and BH7) and BH8 at intact zone were relatively low values, the down slope (BH1, BH2, BH5 and BH6) had high EC. It is presumed that the groundwater of downstream was high EC because of penetration of the leachate from the buried waste. The data show that around 9-99 mS/m of the groundwater at the upper slope (BH3 and BH7) and BH8 at intact zone were background levels.

EC values of BH5 and BH6 in old site ranged 52-250 mS/m and 85-200 mS/m, respectively. In new site, the maximum EC values of BH1 and BH2 were 530 mS/m and 580 mS/m, respectively. It is assumed that effect on wash-out of salts differed according to the dumping age. As for new site (BH1 and BH2), EC values were gradually decreased. This shows that the water levels were relatively high and a large amount of river-bed water washed of salts around down slope boreholes.

EC values of the retained water in new site for PBH1 and PBH2 continued to rise from 250 mS/m to 1560 mS/m and from 680 mS/m to 1690 mS/m in May 2013 and April 2014, respectively. It was found that salts which were attached to or were included in the abandoned waste had been washing steadily out as of April, 2014.

3.2.2. Composition of cations in retained water

Although the majority of dissolved electrolyte in the retained water includes anions and cations, this paper focuses upon only cations. The temporal variations of measured values of cation equivalent in retained water with EC are shown in figure 6. The major cations of the retained water in PBH2 were Na^+, K^+, Ca^{2+}, Mg^{2+} and NH_4^+ with equivalent concentrations ranging from 7.2 to 23, 9.2 to 16, 3.2 to 5.7, 2.9 to 6.9, and 33 to 140 N, respectively.

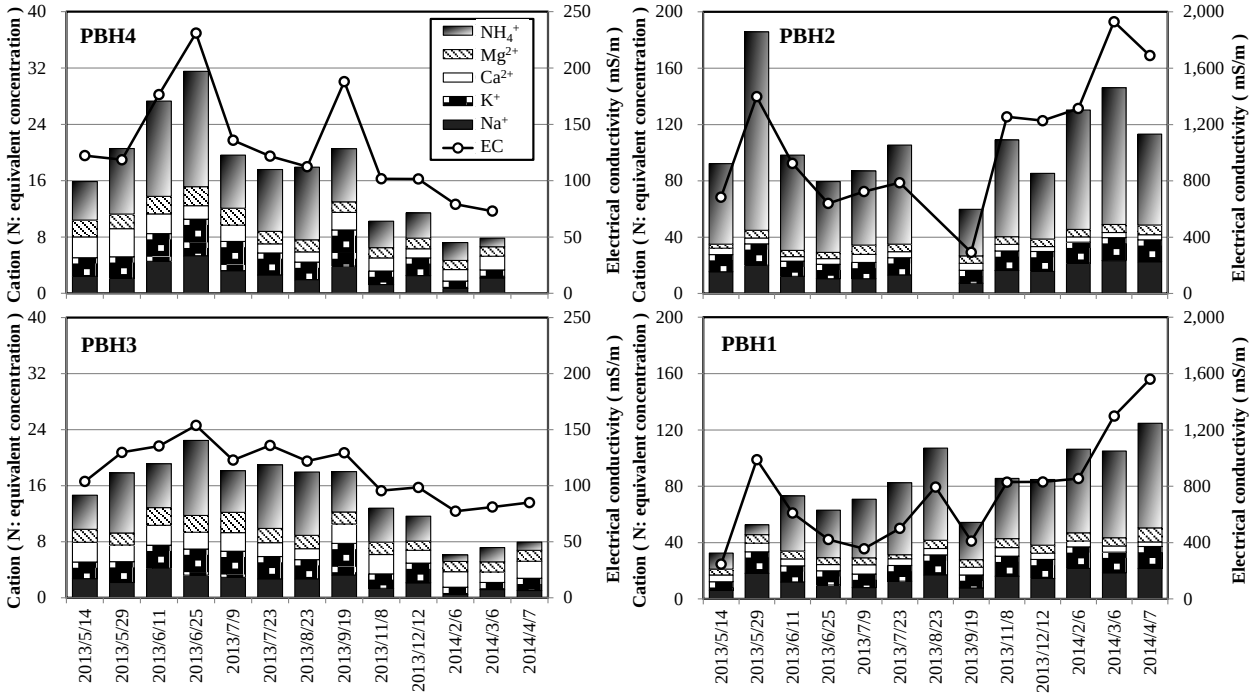

Figure 6 Temporal variations of cation equivalent concentration of retained waters.

The tendency of the total cation equivalent concentration was similar to that of EC roughly. Relationship between EC value and total cation equivalent concentration are shown in table 1. The high determination coefficients (r^2) of PBH3 and PBH4 showed that the total equivalent concentration of 5 ingredients expressed total dissolved electrolytes. The total cation equivalent concentration of PBH1 and PBH2 which had the low r^2 were up to about 30 times in comparison with that of old site. There were few relations of EC value and total cation equivalent concentration in new site which had high EC value, since the higher the concentration of electrolyte rises, the lower the activity coefficient of the ion becomes. *(Miller, 1988, McCleskey, 2012, Ikeura, 2012)*.

The ratio of each ingredient except NH_4^+ was not so changed throughout the year. Since NH_4^+ may be present in retained water due to fermentable organic matter with high concentrations of proteins, ammonification might be enhanced with saturated anaerobic condition during rainy season except around January in old site. EC values of old site were not so high in May 2013 and decreased with time for the duration of this study. The results in 2014 showed that most of the substance from abandoned waste of old site was washed out.

Median ratios of each ingredient for the total cation equivalent concentration are shown in table 2. It was listed in order of higher ratio in total equivalent concentration for PBH1 and PBH2 of $NH_4^+ > Na^+$, $K^+ > Ca^{2+}$, Mg^{2+}. The equivalent concentration of each ingredient except NH_4^+ was almost the same in old site. Because differences of operation more than six years between old site and new site induced

decreasing of the concentration of Na^+ and K^+ in comparison with Ca^{2+} and Mg^{2+}, it was shown that Na^+ and K^+ were critically affected from abandoned waste.

Table 1 Relationship between EC value and total cation equivalent concentration.

Borehole number	Determination coefficient (r^2)	n
PBH1	0.57	13
PBH2	0.53	12
PBH3	0.96	13
PBH4	0.84	12

Table 2 Median ratio of each ingredient for the total cation equivalent concentration.

Borehole number	Na^+	K^+	Ca^{2+}	Mg^{2+}	NH_4^+
PBH1	0.17	0.16	0.07	0.07	0.55
PBH2	0.14	0.13	0.04	0.05	0.63
PBH3	0.15	0.17	0.15	0.13	0.33
PBH4	0.16	0.16	0.12	0.11	0.38

4. CONCLUSIONS

Investigation of 13 boreholes for groundwater and retained water made on two transects line along the slope each in old site and new site was done for one year. Although EC values of groundwater in new site near the river were decreasing rapidly, it did not change in old site. These two sites had different conditions of nature of the soil and strata. The deep fossil valley around new site caused washing of a permeable layer by a large amount of river-bed water. On the other hand, EC values of the retained water in old site were found that salts which were attached to or were included in the abandoned waste had been washing steadily out for about 10 years after the dumping. Regarding the cationic ingredients, the elapsed time induced decreasing of the concentration of NH_4^+, Na^+ and K^+ in comparison with Ca^{2+} and Mg^{2+}.

5. ACKNOWLEDGMENTS

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Problems Related to Sanitary Labours in Solid Waste Management: A Case Study in Matale Municipal Council

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Abstract: *As a developing country, in Sri Lanka, solid waste has become one of the most serious national problems, especially in urban areas since; there is no proper mechanism to manage solid waste. However in solid waste management process than other tasks, local authorities give prior place to waste collecting that is more labour intensive exercise. Nevertheless, still local authorities fail to provide proper waste collecting service due to various problems of labours. Within this context, this study focused on the problems related to sanitary labours, who engage in waste collecting and scavenging activities in Matale Municipal Council, represent lowest position in social strata. 15 sanitary labours were included to snowball sample and eight Municipal Council officers related to solid waste management were purposively selected for the study. Qualitative methods such as in-depth interviews and focus group discussions were conducted to gather data from the respondents. In addition a simple data sheet was used to collect basic socio-economic characteristics of sanitary labours. Shortage of labours, high number of labour absentees and frequent drop outs, irresponsibility, health and safety equipment related problems were major problems found in the study. Engage in part time jobs, temporary nature of the job, inadequate recourses and welfare facilities, lack of education, political influence and family related problems of labours were reasons for the revealed problems. Therefore the study suggests the importance of improving labours' awareness on health and safety, and labour welfare, physical and human resource of municipal council related to solid waste management. In addition assure the social recognition of sanitary labours by changing the job title will be important in solving such problems.*

Keywords: *Solid Waste, Solid Waste Management, Local Authorities, Sanitary Labours*

1. INTRODUCTION

Solid waste (SW) has become an influential dilemma in socio-economical and environmental aspects around the world, mainly due to rapid intensification of urbanization process. Within this context, Solid Waste Management (SWM) is proceed to be a foremost challenge and issue in urban areas in the world, particularly in the rapidly urbanizing areas of developing countries (Shukor et al, 2011; Alwis, 2006). With huge waste generation amounts in each year, Sri Lankan Local Authorities (LAs) are facing difficulties in its' systematic management.

According to the provisions of Local Government Act, LAs in Sri Lanka are responsible for collecting and disposal of waste generated by the people within their territories (Bandara, 2008). In waste collecting activities, local authorities use sanitary labours support who are doing essential service for SWM. Since their service is influence to cleanness and well being of cities. Nevertheless LAs have faced various problems when obtaining the service from them. Those problems are negatively impact to total SWM process of LAs.

Sanitation, it includes management of liquid and SW for personal, domestic and environmental hygiene (Narayanaswamy and Sachithanadam, 2010). With the expansion of human settlements, cities were emerged. When people concentrate more toward cities, sanitation become as a problem. Therefore LAs had to find ways to remove these waste generated by both human and animal within their territories. In generally, people view towards "cleaning work" is a "dirty task". Due to this social stigma

finding labours for cleaning was difficult. So that, some countries had to import labours. For an example in Sri Lanka they were descendants of emigrants from India sent by the Indian government during the British colonial era. During this era these communities were settled different places in Sri Lanka where LAs existed. Time later, those places were separated from mass society based on their ethnicity, class and caste. 'Sanitary labouers' are known as different names in different occasions as such, 'Health Labouers', 'Manual Scavengers'. They are mainly working and involving in street cleaning, waste carrying, drainage and toilet cleaning in the cities.

In Sri Lanka Sanitary labourers belong to urban low income, depressed caste community (Silva et al, 2009b). Studies have been revealed that in Sri Lanka they are marginalized ethnically and socially due to their actual and caste specific association with scavenging, overcrowding, poor housing, notoriety for bad behaviours (alcoholism, drug addiction, drug peddling, crime, petty theft and prostitution) and low social esteem attributed to the place itself. Furthermore, they are faced discriminations that cover all domains of their lives such as, basic services, education, employment, land market and political participation (Silva et al, 2009a; Silva et al, 2009b). In world context also sanitary labours are experienced socially and economically very poor status in the society. As literature revealed women sanitary labours are faced many adjustment problems when they play a dual role at their working place as well as their homes. Furthermore, illiteracy, low income, propertylessness, debts, homelessness or no own place to live, no basic facilities in their dwelling place, health problems and heavy works are the problems that they faced (Shanthi, 2010).

Nevertheless, their service is more important in the SWM process in present than past, which is done by LAs with the high waste generation amount. In this process LAs specially, Municipal Councils (MCs) face key constrains include, lack of expertise knowledge, limited financial resources, complex institutional responsibilities, lacks in regulatory and institutional framework lacks in community participation and lack of labours (Bandara, 2008 ; Mannapperuma & Basnayake, 2007 ; Wijerathne et al, 2012b ; UNEP, 2008 ; Karunasena & Wickramasundara, 2012).

However, still there some gaps that have not covered through literature related to SWM in LAs with its fast growing population and extensity of economic activities day by day. Therefore this study intends to examine the problems faced by LAs related to SWM in Sri Lanka with special reference to sanitary labours, from sociological point of view. Sanitary labours represent the bottom strata in the organization hierarchy of SWM section. Yet, their occupational role is very much important in waste collecting activity which belongs to one of the prominent duties in SWM process of LAs for a clean environment.

2. OBJECTIVES

The study was carried out in order to achieve two main objectives and one specific objective. One of the main objectives of the study was to examine the problems related to sanitary labours in SWM of LAs and find out the reasons behind this situation. The second main objective was to identify the impacts of them in SWM process in LAs. Specific objective was to make suggestions to solve them.

3. RESEARCH LOCATION AND METHODOLOGY

3.1Research Location

According to the Census and Statistics data, central province is the second most populated province in Sri Lanka (Census and Statistics, 2012). Parallel to the population, SW generation is also increasing day by day within the province. It is the third most SW collected province in Sri Lanka (Database of MSW in Sri Lanka, 2005). Out of the three MCs (Kandy, Matale and Nuwaraeliya), Matale is the second highest waste collected MC per day in central province (Database of municipal solid waste in Sri Lanka, 2005). SWM is one of the principal responsibilities of MC administration out of others. Also, Matale is one of the towns with sanitary labourer settlements under MC authority.

3.2 Data Collection and Analysis

The sample was selected for the study using two sampling methods. 15 sanitary labours representing 5 temporary labours and 10 permanent labours (represented one female labour) were included to snowball sample. In-depth interviews were carried out to gather data. In addition a simple data sheet was used to collect basic socio-economic characteristics of the respondents. There were two focus groups discussions separately conducted for temporary and permanent sanitary labours.

Similarly, eight in-depth interviews were carried out with responsible officers related to SWM using purposive sampling method. They were Mayor, Engineer, Public Health Inspector, Superintend of Work, two Field Supervisors, Transport Officer and Medical Officer of Health. Published reports, research articles and internet sources also used in the study. Thematic analysis method was used to analyze collected data.

4. RESULTS AND DISCUSSION

4.1Problems Faced by Matale MC Related to Sanitary Labours in SWM

Matale MC has failed in proper SWM process mainly due to one of the problems, related to sanitary labours, although MC allocated the highest amount of money from MC budget for salaries (Matale MC Budget, 2014). Sanitary labours are the people who engage in waste collecting activities in MC boundaries both in residential and business areas. The study found various kinds of problems in SWM related to sanitary labours in Matale MC. They are as follows.

1. Shortage of labours for work

Shortage of laborers for work was the major problem faced by MC related to sanitary labours. There were 105 labour carders for SWM in 2014. Among them 75 were permanent and 30 were temporary. Although there were 75 permanent labours, disabled, sick labours were among them who cannot perform well. Additionally, around 15 members are working in several other sectors even they have appointed as labours. Therefore daily attendance was usually lower than the actual requirement. Moreover daily requirement of labours for each duty of the SWM section further classified as follows.

- 6 labors – Drivers
- 2 labors – Supervisors
- 4 labors – Over time work
- 6 labors – Sweepers
- 30 labors – Daily labor work

Nevertheless often there is a labour shortage to appoint above duties. These have caused to increase the work load for labours and face difficulties to get off their duties.

2. High number of labour absentees and frequent drop outs

Although there were 105 carders, only 70-80 labours report to work daily. Usually there is a 15% of absenteeism among labours. Labours should inform their absences before three days to the municipal authority except sick leaves. Alike other governmental officers, labours have 45 days of leave per year. Nevertheless, in reality labours are not following any of these rules and mostly they do finish their leaves within first six months of the year and worked as no pay labours for every other absent. Moreover, once a temporary labour taken the appointment as a permanent labour, their absence rate becomes higher than earlier. Also they have turned over the job without any notice. When they continuously absence without any notice to work, MC considers it as a drop out. So that once a labour comes to work again, their service duration breaks. However there are no documented data related to labour drop outs. Through 2014 state budget of November all the labours those who have been worked more than 180 day have considered as permanent labours. Therefore in 2015, 108 permanent labours and two temporary labourers are working in MC.

Similarly, labours, who recruited as sanitary labours, are unofficially allocated to other duties such as drivers, supervisors, office workers and library workers. But they have received labour level salaries. Though they get lower salary, they prefer to join with such duties rather than labour work.

3. Labours are finished their works untimely and leave quickly

Labours have two shifts, day time and night. Daytime shift starts at 8.00 am and it exists until 2.00 pm. Night shift continues 6.00 pm to 11.00 pm. Labours are not followed the proposed routs and they are not punctual. Nevertheless labours often try to finish their work before their working hours and leave quickly.

4. Lack of safety equipments for labours

MC provides gloves, boots, raincoats, jackets for labours. Nevertheless MC does not have enough amounts of safety equipments. And sometimes it can be taken time to supply them. Boots, gloves and rain courts are provided according to the requirement of sanitary labour once the previous pair gets damaged. Moreover, gloves which are provided by MC are not in real quality and can be used nearly one week of time. Similarly, MC provides only gloves for hand cart labours and for the tractor drivers provide only raincoats.

5. Lack of care on safety and health among labours

Labours are dislike to use gloves and boots when they are working. Though municipal officers give awareness about them, labours are not used them. Also labours mentioned they are in difficulty when they wear gloves and boots in work. But they wear them once some important person visits them. As MC officers revealed, some of the labours who are appointed to work in public toilets are taking food at the same premises. Also labours who are working at gully service are not wearing the gloves. Labours who are working at drains are not wearing boots though officers forced them to wear.

MC carries out a vaccination program (for Typhoid) for labours (including drivers). Below table shows the information on it.

Table 1:No.of Vaccinated labours

Year	No of vaccinated labours
2006	106
2010	62
2011	41
2014	73

(Source: Matale MC data, 2014)

Nevertheless labours have low intensity to take the vaccination.

4.2 Reasons for the Problems

According to the revealed data reasons for above mentioned problems are as follows.

1. Labours are engaged in another jobs other than in MC work

Labours are doing part time jobs other than their main income at MC such as, jobbing. Majority of them are young labours, in between the ages of 20-40. They live in labour quarters situated in Iggolla. These houses were built by MC under a loan system. Therefore labours pay Rs.810 from their monthly salary. As MC Officers mentioned they do not make any restrictions to collect and sell recyclable items to labours. Labours keep separate bags beside the tractors and they are sorting recyclable items while they are travelling.

2. No any special welfare for labours

MC does not provide any special welfare facilities for labours except vaccination. As labours revealed, even bathing or cleaning facilities were not provided by MC. They have gone directly to their homes after finishing their duty. Moreover, as revealed data, municipal authority attention is not adequate for SWM. They just put their weight on it once the community protest occurs related to SW. It caused to increase the problems in SWM section in MC.

3. Political influence and personal relationships

Around 15 labours are working in several other sectors even they have appointed as labours, using their private relationships and political influence. As such, office works, library works, supervisors and tractor drivers.

4. Lack of education

Due to their less educational level sanitary labours do not have much idea about sanitary living conditions, working conditions, health and safety. They have received their job generation to generation.

5. Temporary nature of the job

Since their lack of educational qualifications, they are in difficulty to enter the permanent carder position. For that, they have pass grade 8. So that they have to work as temporary labour at all that mainly caused for the absenteeism.

7. Family related problems and alcohol addiction among labours

As revealed data, most of the sanitary labours addicted to alcohol and they spent more money for that from their earnings. That has caused for their family problems such as, domestic violence, economic deficiencies. On the other hand people also have unfavourable attitude towards sanitary labours as they are drug addicted. Those are caused for labour absenteeism, lack of intensity to work, irresponsibility and social sigma to sanitary labours.

4.3 Impacts of the Problems

Below shows the negative impacts that can be resulted from above revealed problems.

1. MC fails in executing efficient waste collecting service within MC

As a result of lack of labours, MC fails in executing proper waste collecting service with other problems of SWM section in Matale MC. While 38.7 tons of SW is generated from various sources in Matale MC per day (UNEP, 2008), these labour problems aggravates the efficiency and efficacy of the waste collecting service.

2. Risk of diseases

As municipal authority mentioned, some of the labours have skin diseases like eczemas due to their lack of intensity on sanitation. Nevertheless, there are no documented data related to this matter.

3. Disorganization of working arrangement in MC

When labours are engaging other duties like supervisors, office workers and library workers it caused to disrupt the proper working arrangement inside the MC.

4. Risks for accidents

Once the labours unofficially working as a tractor driver, it has a risk for accidents especially in the night shift due to their lack of experience.

5. CONCLUSION

The study found the problems related to sanitary labours in SWM process, reasons for problems and impacts of them. Sanitary labours joined with decisive work role related to cleanness of city though they represent the bottom strata of society and their occupation hierarchy which majority of people do not like to serve. Nevertheless, efficiency and efficacy of their work important for whole SWM process of MC. Hence, cleanness of the city reflects from waste collecting and scavenging activities which is done by sanitary labours. Therefore, it is important to find solutions for the sanitary labours related problems for a better SWM process in a city.

6. SUGGESTIONS

1. Provide continuous awareness programs on health and safety using resource persons to labours
2. Appoint a relevant person for supervision of health, safety and awareness of labours in SWM section or formalized existing responsibilities among the MC officers in this regard
3. Expand the labour welfare facilities and beneficiaries to improve their living condition and to reduce the turnovers and absenteeism by providing sanitary facilities and annual bonus.
4. Purchasing quality safety equipments as necessity of labours requirements by allocating more money for SWM section from MC budget
5. Make visible labour requirement policy to recruit relevant persons for relevant positions based on the qualifications
6. Take actions to assure the social recognition of sanitary labours by changing the job title as ‘Community Health Assistant’

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Suitability of Natural Red Earth as a Reactive Material for Permeable Reactive Barriers to Remove Copper from Ground Water Contaminated with Leachate

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Abstract: *In developing countries like Sri Lanka, still the most common method of solid waste disposal is open dumping. This causes major hazards by polluting air, water and soil alike. When rain water or any other liquid gets contacted with this uncovered waste, leachate containing many harmful substances is generated. Permeable Reactive Barrier (PRB) is an emerging and promising technology for the treatment of contaminated ground waters which are economical and require less maintenance. Natural Red Earth (NRE) occurring in the Coastal Zone of Sri Lanka was selected as a suitable candidate material for PRB and experiments were conducted with to assess its ability to remove Copper from contaminated ground water. The Red soil samples were tested for basic physical and chemical properties. NRE showed almost 100% Cu removal within a short period. Thus, it can be stated that NRE is a promising candidate material to remove Cu from aqueous solutions.*

Keywords: *Leachate, Heavy Metals, Copper, Permeable Reactive Barrier, Natural Red Earth*

1. INTRODUCTION

In developing countries like Sri Lanka, still the most common method of solid waste disposal is open dumping. This causes major hazards by polluting air, water and soil alike. When rain water or any other liquid gets contacted with this uncovered waste, leachate containing many harmful substances is generated. With no bottom lining, this leachate can percolate through soil or runoff and mix with ground and surface water. Thus, in an environment where open solid waste dumping is common, it is essential to find solutions to treat leachate generated from SW dumps in-situ before it spread to outside environment. This shows the potential contamination hazard landfill leachate from unlined landfills can pose on environment and Arneth *et* al (1989) present several reported causes of ground water pollution from landfill leachate.

Where open dumping is done, pump and treat method for leachate contaminated groundwater treatment is not possible. A very promising alternative to these situations is Permeable Reactive Barriers (PRB) which is being used extensively in developed countries. Typically, contamination is carried into the PRBs under natural gradient condition (creating a passive treatment system) and treated water comes out of the other side. In theory, the field application of PRBs presents a series of advantages in relation to pump-and-treat, including low operation costs, low maintenance, and low on-going energy requirements (Park *et* al, 2002).

Heavy metals are natural components of the Earth's crust. They cannot be degraded or destroyed. Heavy metal ions such as Cu(II), Cd(II), Hg(II), Zn(II), Pb(II) etc., have long been recognized as ecotoxicological hazardous substances and their chronic toxicities and accumulation abilities in living organisms have been of great interest in the last years. (Cay *et al*, 2004). Therefore, the removal of

heavy metals from waters and waste waters are important to protect public health and wildlife. (Ajmal *et* al, 1998)

Sources such as electronic goods, electroplating waste, painting waste, used batteries etc, when dumped along with municipal solid waste increase the heavy metal contents in dumpsites. Slow leaching of these heavy metals under acidic environment during the degradation process leads to leachate with high metal concentration (Esakku *et* al, 2003). Exposure of heavy metals may cause but not limited to blood and bone disorder, kidney damage and decreased mental capacity and neurological damage.

The contamination of ground water and soil is the major environmental risk related to unsanitary land filling of solid waste. The most common pollutants involved are metals like copper, lead cadmium, mercury etc. High risk groups include the population living close to a waste dump and those whose water supply has become contaminated either due to waste dumping or leakage from landfill sites. (Raman & Narayanan, 2008)

For low concentrations of metal ions in wastewater, the adsorption process is recommended for their removal. The process of adsorption implies the presence of an “adsorbent” solid that binds molecules by physical attractive forces, ion exchange, and chemical binding. It is advisable that the adsorbent is available in large quantities, easily regeneratable, and cheap (Demirbas, 2008).

The main objectives of this study have been to investigate the adsorption characteristics of Natural Red Earth (NRE) as a suitable candidate material for PRB for the removal of Copper from aqueous solutions. During the study kinetic and equilibrium experiments were performed under different conditions.

2. MATERIALS AND METHODS

2.1 Copper

Copper is common in leachate found in Sri Lanka. A recent study by Sewwandi *et* al, 2012 show high levels of Cu in many waste dumps in Sri Lanka. The Cu levels observed during above study shows values ranging from 58 to 740 µg L^{-1}.

2.2 Natural Red Earth

Natural Red Earth (NRE) is a naturally-occurring Fe coated quartz sand, which is a mixture of different minerals such as ilmenite, rutile, zircon and others. It has been shown that NRE is composed of high Fe^{3+}, up to 6% (Rajapaksha *et* al, 2011). NRE naturally occurs abundantly in the north-western coastal belt of Sri Lanka underlain by Miocene limestone sequences (Nikagolla *et* al, 2013).

Natural red earth samples used in the study were collected from the Aruwakkaru limestone quarry site in the northwestern part of Sri Lanka (latitudes and longitudes of $8^0 14' 50'' N$ and $79^0 45' 45'' E$). It mainly consists of SiO_2 (54.15 %), Al_2O_3 (20.73 %) and Fe_3O_2 (12.39 %), where SiO_2 is present in crystalline form while Al (as Al_2O_3) and Fe (as Fe_2O_3) exist as an amorphous coating around the silica grains (Vithanage *et* al. 2006). Natural red earth occurs as rounded and well-sorted quartz sand in a red clayey matrix with accessory ilmenite and magnetite. The brick red color of NRE indicates oxidizing conditions for the formation of red hematite. The NRE contains 0–1 % Fe^{2+} and typically a higher (>2.0 %) Fe^{3+} content (Nikagolla *et* al, 2013). As NRE consists of two main surface sites (>AlOH and >FeOH), it can be more promising as a good adsorbent species (Mahatantila *et* al, 2011).

2.3 Physical and Chemical Properties of NRE

A series of laboratory experiments were conducted to understand the properties of NRE. Physical properties like particle density, particle size distribution, moisture content and BET surface area were tested and chemical properties like pH and Electrical conductivity were tested. All testes were carried out according to relevant BS or ASTM standards.

2.4 Surface Titration

To determine the Point of Zero Charge (pH_{zpc}), surface titrations were carried out using the Automatic Potentiometric Titrator. A quantity of 5 g/L of <75 µm fraction of NRE suspension was equilibrated well in 100 ml distilled water for 24 h. The initial pH value was 7.61 and by drop wise addition of 0.00588 M NaOH it was raised to 9.00. Three titration experiment were performed on the basis of different electrolytic concentrations (0.1, 0.01, and 0.001 M NaNO3) (Vithanage *et* al., 2006) by addition of 0.01710 M HCL until the pH reached 4.0.

2.4 pH Value

The pH for all experiments was maintained at 5.5. This pH value was selected from the previous studies done on adsorption, which indicated to be the most promising pH range for Cu adsorption. No research is still done for Copper removal by NRE, thus pH value was selected as 5.5 arsenic adsorption on NRE recorded as high in between pH 4 and pH 7 and an average pH was selected for the control experiments (Vithanage *et* al., 2006). According to the in-situ pH values taken at the bore holes as Udapalatha dump site in Gampola, Sri Lanka over an year, it is evident that the pH value of ground water contaminated with leachate is between 5.4-8.7. Afterwards, control experiments in that pH range was also conducted and they showed high removal rates. Thus, adsorption is preferable for removing Cu from the ground water.

2.5 Initial Experiments

To determine the ability of NRE in removing Cu at aqueous environment, a set of batch experiments under the following conditions were completed. Sieved NRE samples of particle size less than 75 µm were taken for the experiments unless in experiments for varying particle sizes. The measured amounts of soil were added to distilled water to create necessary concentration and the samples were agitated with Automatic Potentiometric Titrator for 2 hours to homogenize the solution. Then, the pH of the solution was adjusted to 5.5 by drop wise addition of 1M HCl and 1M NaOH solutions. Then 20 ml aliquots of solution were taken out and cu was spiked using micro pipettes to prepare the necessary Cu solution. Solutions were put in *AT12R Thomas* shaking incubator at 25^0C agitating at 150 rpm for 24 hours for the reactions to take place. The final pH was also recorded. All experiments were done in triplicate. After completion, supernatant from each tested sample was taken into tubes, centrifuged at 16,000rpm for 10 minutes in *Suprema 21* High Speed Refrigerated Centrifuge and filtered with 0.45µm filter in preparation for analysis. All samples were analyzed for remaining Cu using Shimadzu AA 7000 Atomic Adsorption Spectrophotometer (AAS). All tests were done in triplicate.

To determine the optimum amount of NRE for Cu removal, further experiments were carried out with different NRE amounts but at the same Cu concentration. NRE amount was varied from 1g/L to 25g/L. Tests were done in triplicate using sieved soil particle with sizes less than 75µm in pH 5.5. Cu solution was prepared and different soil amounts were introduced to 100 ml of 1 ppm Cu solution to achieve the desired NRE concentration. The sample containers were kept in shaking incubator at 25^0C agitating at 150 rpm for 24 hours. On completion, the supernatants were taken in to tubes and was analyzed for Cu concentration remaining using AAS.

2.6 Kinetic Experiments

In this study, two sets of kinetic experiments were completed to determine the removal rate of Cu with time. Tests were done in triplicate. Sieved soil samples with particle sizes less than 75µm were used for the experiment. Number of 100 ml samples with 5g/L NRE concentration was prepared using pH 5.5 1ppm Cu solution. All samples were kept in shaking incubator under continuous agitation and samples were taken out soon after 5, 10, 20, 30 45, 60, 90, 120, 180, 300, 480, 1440 minutes respectively. The supernatant from each sample of these was taken to tubes immediately, centrifuged at 16,000 rpm and were analyzed for remaining Cu with the AAS.

The second kinetic experiment was conducted to assess the Cu removal level during the initial 20 minutes of reaction. Samples were made in triplicate, prepared similarly (100ml volume 5g/L NRE samples using pH 5.5 1ppm Cu solution) and was incubated under continuous agitation. Samples

were taken out at 1, 2,3,4,5, 10 and 15 minutes for analysis. The supernatant were taken to tubes immediately, centrifuged at 16,000 rpm and analyzed for remaining Cu with the AAS.

2.7 Cu Adsorption with Different Particle Sizes of NRE

Particles size distribution in Red soil show only 33% of particles are less than 75µm in size. Since particle size is directly propotional to the available surface area and is an influential factor in adsorption, experiments were carried out using three particle size ranges; sizes less than 75µm, sizes between 75µm and 4.25mm and sizes between 4.25mm and 9.52mm to assess the removal efficiencies. NRE was crushed and sieved to obtain required sizes. Tests were done in triplicate in 100ml volume 5g/L NRE samples using pH 5.5, 1 ppm Cu solution. After 24 hours of incubation under continuous agitation, samples were removed and analyzed for remaining Cu with the AAS.

3. RESULTS AND DISCUSSIONS

3.1. Physical and chemical properties of NRE

The following physical and chemical properties (Table 1) were determined through laboratory experiments. According to the results obtained the BET surface area obtained for NRE is low, 15.59 m^2/g. However some literature has indicated much higher surface area values estimated under different methods. Particle size distribution show nearly two thirds particles above 75 µm. pH and electrical conductivity values are moderate, particle density and wet basis moisture contents are acceptable.

Table 1 Physical and chemical properties of NRE

Parameter	Value	Method
Particle density	1.96 g/cm^3	BS 1377 Part II-1990
Particle size distribution	33.20% less than 75µm	BS 1377 Part II-1990
Moisture Content Wet Weight Basis	0.73	BS 1377 Part II-1990
BET Surface area	15.59 m^2/g	ASTM D3663-03 (2008)
pH value	6.81	BS 1377 Part III-1990
Electrical conductivity	283.5 (µS/cm)	BS 1377 Part III-1990

3.2. Batch Experiments

The batch experiments carried out to determine the NRE ability to remove Cu in solution show an average 81.31% Cu removal rate for a 2 g/L Red soil concentration. This removal rate is acceptable for a promising candidate material on PRB to remove Cu from aqueous solutions.

3.3. Surface Titrations

Figure 1 shows the variation of surface charge of NRE, as a function of pH in 0.1, 0.01, and 0.001 M NaNO3. The pH_{ZPC} of NRE was estimated experimentally as 9.0. Vithanage *et* al, 2006 has obtained pH_{ZPC} of NRE as 8.8 and Vithanage *et* al, 2007 states the pH_{ZPC} of NRE as 8.5. As these values are close, pH_{ZPC} of NRE can be accepted as 9.0.

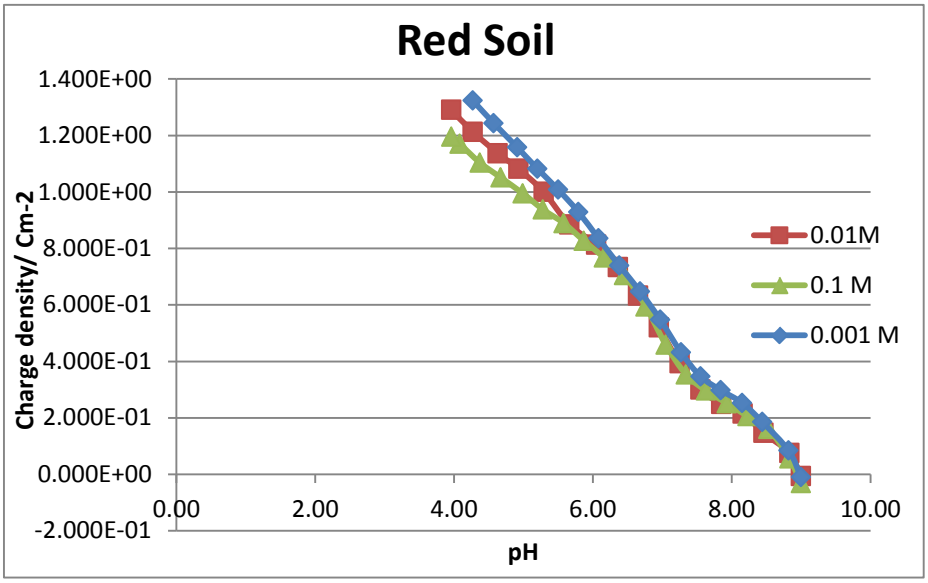

Figure 1 Point of Zero Charge of NRE

3.4. Kinetic Experiments

According to the results obtained (Figure 2), it is evident that the Cu adsorption into NRE is rapid.

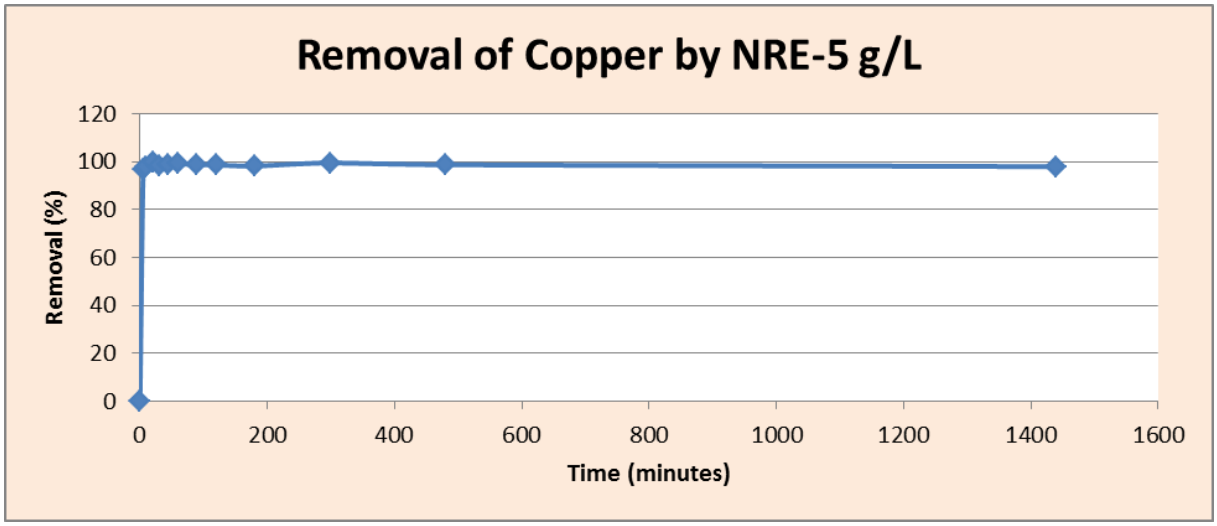

Figure 2 Removal of Cu by 5g/l of NRE with time

The adsorption is immediate and over 95% Cu in the solution was adsorbed onto NRE within minutes of contact and reached an apparent adsorption plateau within the first 10 minutes. Similar result was also reported by Rajapaksha *et al*, 2011 where adsorption of arsenite and arsenate onto NRE was investigated.

In figure 2 below Cu removal within the initial reaction stages show that NRE can achieve a removal efficiency of 93% within the first minute of reaction. Further removal efficiency attains almost 100% in 10 minutes with 5g/L NRE in 1ppm Cu solution at pH 5.5. Similar results for Cr removal is reported in Nikagolla *et* al, 2013.

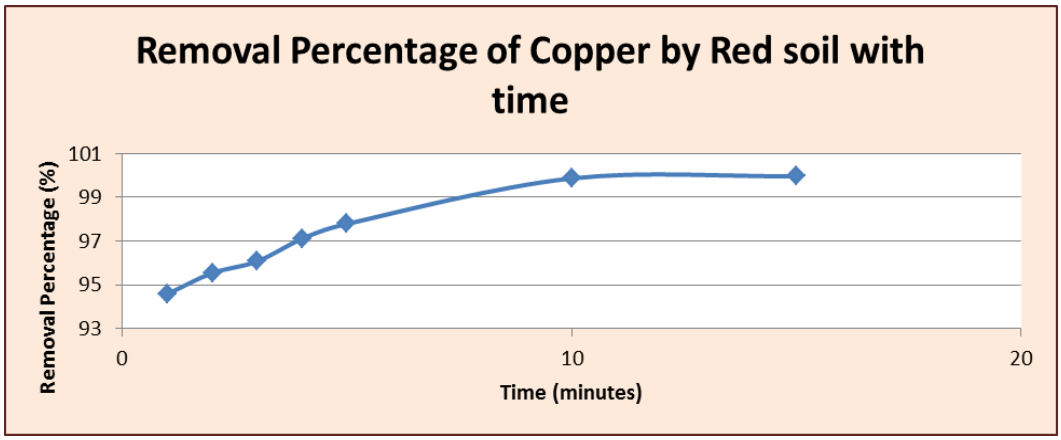

Figure 3 Removal of Cu by NRE – initial reaction time

3.5. Experiments to Optimize the NRE Dosage

According to results obtained (figure 4), the minimum NRE dosage for almost 100% Cu removal was 5g/L. Mahathantila *et al,* 2011 too in her findings has stated that Pb(II) adsorption reached equilibrium at 5 g/L NRE dosage and remained constant at values greater than for 2.41 and 8.92 µmol/L initial Pb(II) concentrations. Therefore, the proper NRE dosage for effective Cu removal 1ppm concentration was decided as 5g/L.

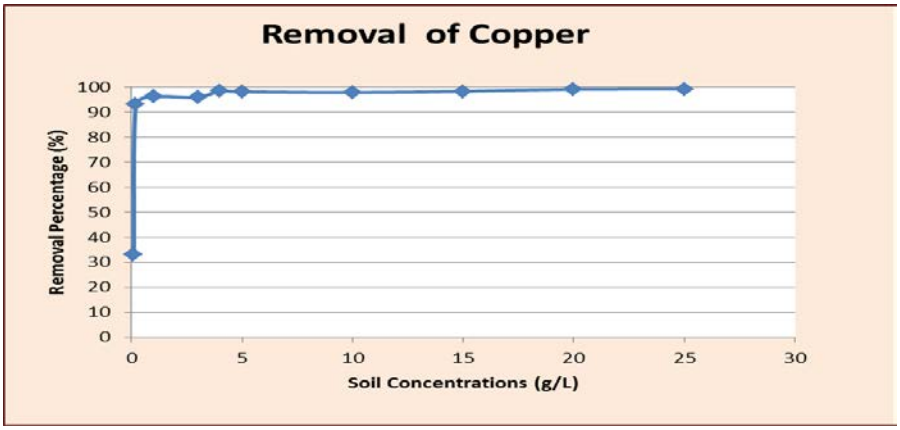

Figure 4 Removal of Cu with varying NRE dosage

3.6. Cu Adsorption by Different Particle Sizes of NRE

Investigations for NRE particles size dependency on Cu removal show that the Cu removal decrease with particle size as expected. Obtained results show that 98%, 75% and 70% copper removal was achieved with NRE particles less than 75µm in size, particles between 75 µm and 4.25 mm in size and particles between 4.25 mm and 9.52 mm in size respectively. This information is valuable in finalizing the PRB modelling.

4. CONCLUSION

The Natural Red Earth yielded promising results removing Cu metal effectively in aqueous solutions. Results show that on average 81.31% of Cu is removed from 1ppm Cu samples at 5.5 pH and at a NRE concentration of 2 g/L. Further studies suggest that total Cu removal from 1ppm Cu samples at

5.5 pH require a minimum 5g/L NRE dosage. In all experiments the Cu adsorption was very rapid and within the initial 10 minutes, over 99% removal efficiency achieved for 5g/L NRE dosage.

As an overall conclusion, the above studies indicate that Natural Red Earth found in North-Western coastal areas of Sri Lanka is a promising candidate material to remove Cu from aqueous solutions especially from low concentration solutions such as leachate contaminated groundwater. Further studies are being carried out with NRE to check its potential to remove other heavy metals found in leachate and to develop a prototype PRB unit using the Natural Red Earth.

5. ACKNOWLEDGEMENTS

The authors would like to express their sincere gratitude the JICA and JST for financially supporting this research under its Science and Technology Research Partnership for Sustainable Development (SATREPS) project.

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Electromagnetic Survey (GEM-2) for Monitoring of an Open Dumpsite in Sri Lanka

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Abstract: *Open dumping of municipal solid waste creates series of environmental and social problems. Monitoring of dumpsites is important in deciding and designing the mitigation measures to reduce the risk. Direct and continuous monitoring of dumpsite is difficult and requires time, money and labour for both sample collection and laboratory analysis. Geophysical methods such as electromagnetic (EM) technique can be used for subsurface investigation rapidly without in-situ drilling and sampling. GEM-2 is a Multi-frequency, handheld EM sensor. This produces EM waves at pre-set multi frequencies and receives secondary EM waves from eddy current of substrate depending on the apparent electrical conductivity (EC_a) of the substrate. Thus EC_a of the underneath substrate can be mapped. An open dumpsite in Udapaltha Pradeshiya Sabha, Central province, Sri Lanka was surveyed using GEM-2 and validated with field measurement. For validation, waste samples were collected up to 30 cm depth and electrical conductivity (EC) was measured at the laboratory and EC_a was calculated. Spatial and temporal variation of EC_a and delineation were clearly depicted in maps produced by GEM-2. Maximum reported EC_a in Old-bottom and New-top of dump site were 88 mS/m and 180 mS/m, respectively. Measured EC_a and EM surveyed EC_a were correlated with simple linear regression. Best correlations were obtained at Old-bottom with 93 kHz wave length while 85 kHz for New-top. It can be concluded that EM survey is a powerful technique for dumpsite monitoring with cautious interpretation.*

Keywords: *Apparent electrical conductivity, Electromagnetic survey Municipal solid waste, Open dumpsite.*

1 INTRODUCTION

Municipal Solid Waste (MSW) has been identified as one of the major pollution problems of the world, and become one of the major challenges in developing countries. Most common disposal method of MSW in Sri Lanka is open dumping. Wastes in dumpsites undergo decomposing processes results in several organic and inorganic substances. Most of these compounds leave the dumpsite as leachate and gas and pollute the environment.

The open dumpsites have to be monitored for better management since the gas emission and leachate generation varies spatially and temporally. Conventional monitoring of MSW dumpsite required drilling, sampling and laboratory analyzing that costs and time consuming. Investigation on spatial and temporal variation is also difficult with spot measurements.

In this context, the geophysical methods such as Electromagnetic (EM), Ground penetrating radar (GPR), Seismic refraction, Electrical resistivity, Gravity, Borehole geophysical logging, Induced polarization and Thermal infrared are used to monitor and evaluate the dumpsites features (McDowell, *et al* 2002; McGinnis, *et al*2011, Ayolabi, *et al* 2013). Geophysical methods are best options to investigate the dumpsites, because of effective sampling strategy, reduced assessment cost as reducing the number of borings/wells needed and investigating large area(Zungalia, *et al* 1989; Letellier, 2012; Wijesekara, *et al* 2014). These methods are generally used in preliminary investigations with verification of direct method. However geophysical data interpretation is quite difficult and needs special expertise (Sharma & Reddy, 2004). High initial capital investment is one of the main factors hindering the wider use of this technology in landfill survey in Sri Lanka.

Apparent electrical conductivity (EC_a) is the depth weighted average of the bulk soil electrical conductivity. It can be measured in both ways such as contact and non-contact methods. It is commonly measured by using EM techniques (Cook & Walker, 1992). EC_a is good indicator of soil physical and chemical properties. GEM-2 is a handheld, digital, programmable, multi-frequency, broadband EM sensor which widely uses to geological, environmental and geotechnical surveys. This sensor produces EM waves at pre-set multi frequencies and receives secondary EM waves from eddy current of substrate depending on the EC_a of the substrate. Thus EC_a of the underneath substrate can be mapped. Open dumpsites in Sri Lanka are rarely monitored by geophysical methods, due to its availability and validation. This study was conducted to reveal the applicability of EM survey in an open dumpsite at Udapalatha.

2 METHODOLOGY

2.1 Study area

An uncontrolled landfill in Udapalatha Pradeshiya Sabha (PS), central province, Sri Lanka was selected for the study. It is located at 7^0 08' 40.8" N and 80^0 34' 43.2" E and at altitude of 492 m amsl (above mean sea level). The site is at a steep slope towards the Mahaweli River from the Gampola-Kotmale Road with 15 m elevation difference within 50 m length. The dumpsite has two sections: Old and New with 7 and 0.5 years of operation, respectively. The municipal waste collected from the Udapalatha Pradasheiya Shabha and Gampola Urban council were dumped until 2011. The site was not in operation for last 3 years and the land was covered by vegetation during the study period.

Comparatively flat areas at the bottom of old section (Old-bottom) and top of new section (New-top) were selected for EM survey (Figure 1). Old-bottom was nearby the Mahaweli River and New-top was more close to the main road. Both areas have been cleared from vegetation and interrupting materials.



Figure 1 Dump site and locations GEM survey

2.2 GEM-2 Electromagnetic Survey

As shown in Figure 2, selected areas were divided into grids and the grid lines were marked with a spacing of 1 m. Survey area with the dimensions of 8 x 2 m and 11 x 4 m at Old-bottom and New-top, respectively were marked. The base period for the sensor was selected according to power line of the area. Therefore 25 Hz was selected for 50 Hz power region. The output sample rate is 10 Hz. Other inputs for GEM-2 such as coil separation and sensor height were 1.66 m, and 1 m, respectively. Frequency of the EM waves has to be specified by the users. Using more than five frequencies simultaneously is not recommended, because of power being distributed equally between frequencies. High number of frequencies may weaken the signal. Therefore, five frequencies, 55, 65, 75, 85 and 93 kHz were selected at a time for the survey (maximum frequency that can be used from the GEM-2 was 93).

The survey was conducted on 3 days (13th, 24th November and 2nd December 2014). GEM-2 was hold horizontally and surveyed starting from line 1 and walked steadily along the line then stopped at end of the line 1. Automatically the data were stored in the SD card memory as well as internal memory of the GEM-2. U-turn was taken at end of the line 1 and repeated the same along the line 2, continued the same until finish the survey as shown by Figure 2.

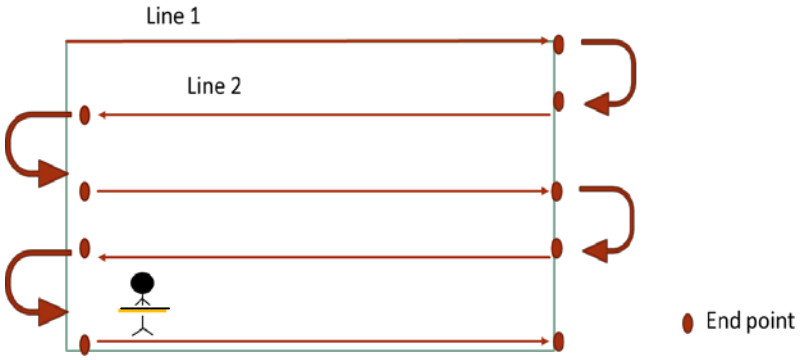


Figure 2 Schematic diagram of grid lines and the procedure of survey

Data was downloaded and converted as .CSV file using EM Export software. Quadrature data were converted to EC_a using Invertor software. The EC_a variation was plotted in the horizontal plane by Surfer 11 for each frequency.

2.3 Field Sampling and Analysis of EC

Waste sampling points were selected for validation at both sites, based on the horizontal map generated by GEM-2 at preliminary test. The samples were collected on 13th November 2014 in five points within the survey area up to the depths of 15 and 30 cm from the surface. Electrical conductivity (EC) was measured in the SATREPS Environmental Laboratory, Faculty of Engineering, University of Peradeniya using standard method (Allison, *et al* 1954). Measured EC at different depths were used to calculate depth weighted EC as apparent EC. The EC_a from both GEM-2 and field measurements were correlated. The linear regression analysis was done for both old and new sections, separately.

2.4 Measurement of Rainfall

Daily rainfall was measured from the weather station at Meewathura farm, University of Peradeniya, Peradeniya during the study period.

3 RESULTS AND DISCUSSION

3.1 Rainfall During the Study Period

Figure 3 shows the daily rainfall from 12th November to 2nd December 2014. The rainfall varied between the surveys. It is reported that EC_a varies with the moisture content of the substrate (Grellier et al 2007).

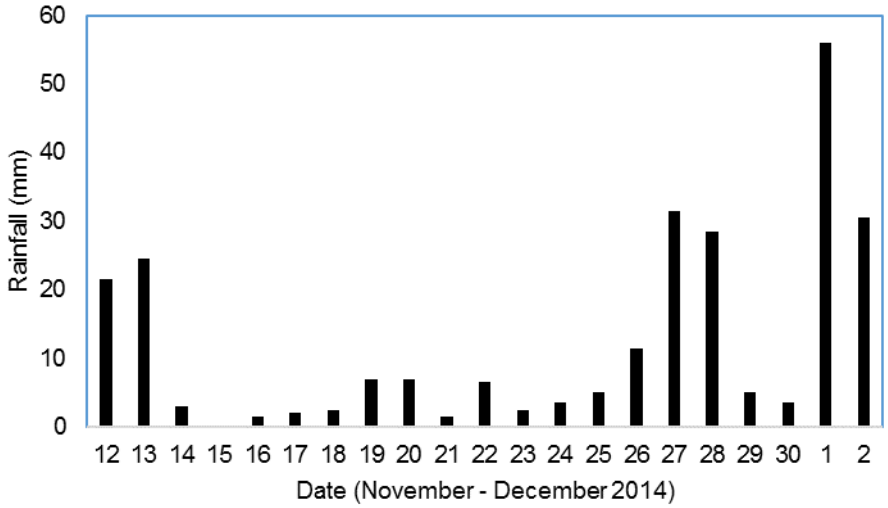


Figure 3 Daily rainfall during the period of the study

3.2 Spatial and Temporal Variation of EC_a

GEM-2 EM sensor measures EC_a at multiple frequencies and magnetic susceptibility at the lowest selected frequency (Geophex Limited, 2004). The survey produces raw data as quadrature and in-phase components representing EC_a and magnetic susceptibility, respectively (Won *et al* 1996). EC_a maps for each frequency were shown figure 4 and 5. Each frequency map represents a horizontal layer up to a depth. Lower the frequency the higher the layer thickness surveyed.

Figure 4 shows the EC_a variation at 85 kHz frequency in both sites on 3 days. Lower EC_a was observed towards river side in Old-bottom. Higher EC_a was observed towards the river in New-top. This may be due to the difference in the stage of waste stabilization process between old and new sites. The temporal variations were able to observe with the GEM-2 EM survey maps on 3 days.

Figure 5 shows the EC_a maps with 5 different frequencies for old and new site on 2nd December 2014. Horizontal layers at different depths of dumpsite are depicted. Shallow depth has higher EC_a compared to deeper layers. There is a consistency in the spatial variation in EC_a as the thickness of the survey increased. The compactness of the waste layers in the landfill sites may be one of the reasons for the EC_a variations although it was not measured in this study.

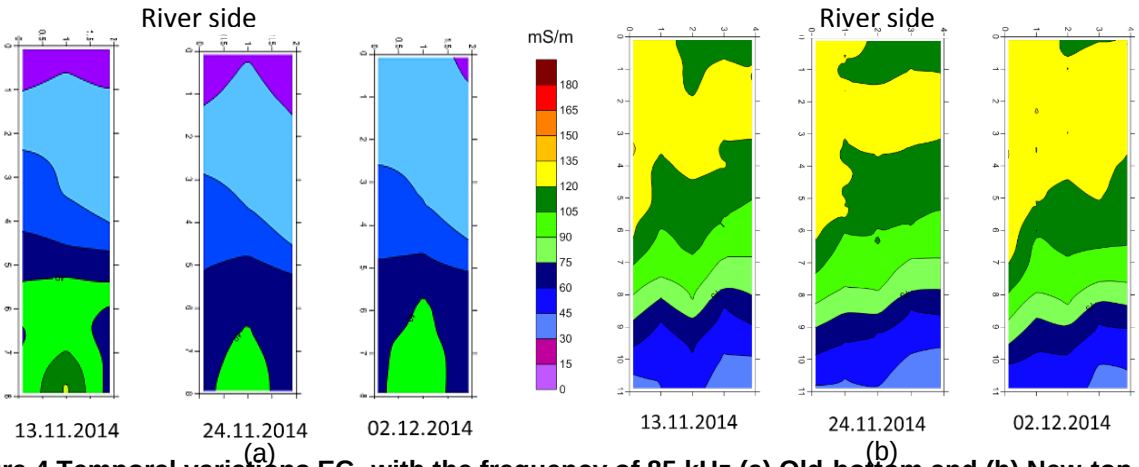


Figure 4 Temporal variations EC_a with the frequency of 85 kHz (a) Old-bottom and (b) New-top

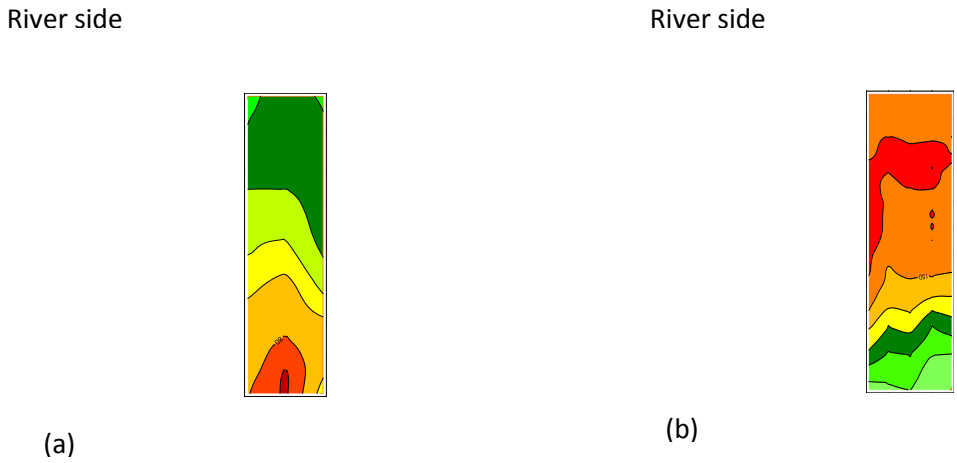
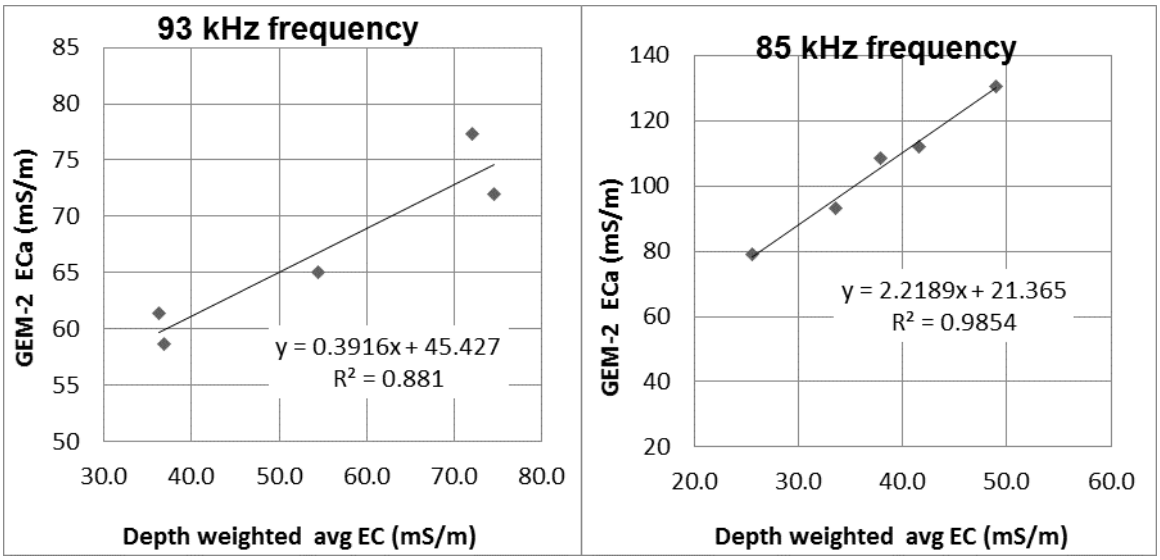


Figure 5 EC_a map with different frequency on 02.12.2014 (a) Old-bottom (b) New-top

3.3 Validation of GEM – 2 EM survey

Depth weighted EC measured at the laboratory was correlated with EC_a at each frequency maps of GEM-2. Figure 6 shows the correlation for Old-bottom (a) and New-top (b). In both sections, high correlation was obtained in all frequencies tested (Table 1 and 2). Best correlations were obtained at Old-bottom and New-top with 93 kHz and 85 kHz wave length, respectively.

Old site has low EC thus high resistivity than new site. Wave penetrates deeper in high resistive site than low resistive site. The Old site has high resistivity so that 93 kHz EC_a was correlated at 30 cm depth weighted average EC. New site has lower resistivity than old site thus 85 kHz EC_a was correlated at 30 cm depth weighted average EC.



(a) (b)

Figure 6 Correlation of EC_a from GEM - 2 and laboratory (a) Old-bottom (b) New-top

Table1. Simple linear regression output of Old-bottom

Frequency	Depth 15 cm		Depth 30 cm		Depth weighted avg	
	R^2 (%)	P-value	R^2 (%)	P-value	R^2 (%)	P-value
93 kHz	70.5	0.075	77.6	0.048	88.1	0.034
85 kHz	64.1	0.104	75.8	0.055	78.6	0.045
75 kHz	65.5	0.097	74.9	0.058	78.3	0.046
65 kHz	62.6	0.111	72.9	0.065	75.9	0.054
55 kHz	60.1	0.123	73.6	0.063	75.6	0.055

Table 2 Simple linear regression output of New-top

Frequency	Depth 15 cm		Depth 30 cm		Depth weighted avg	
	R^2 (%)	P-value	R^2 (%)	P-value	R^2 (%)	P-value
93 kHz	78.5	0.045	73.5	0.063	94.4	0.006
85 kHz	59.9	0.124	89.6	0.015	98.5	0.001
75 kHz	59.0	0.129	88.2	0.018	97.0	0.002
65 kHz	58.4	0.132	87.5	0.019	96.2	0.003
55 kHz	52.5	0.166	91.5	0.011	96.1	0.003

4 CONCLUSIONS

Spatial and temporal variation of EC_a was mapped using GEM-2 with different frequencies. Waste distribution and delineation of dumpsite were clearly depicted on horizontal plane maps with different layers in both Old-bottom and New-top of Udapalatha dumpsite. Maximum exhibited EC_a in Old-bottom and New-top were 88 mS/m and 180 mS/m, respectively by EM survey. GEM-2 EM surveyed EC_a was highly correlated with weighted average EC which was calculated using measured EC of waste samples at the depth of 15 and 30 cm in both sites. Best correlation was found with 93 kHz and 85 kHz wave length for Old-bottom and New-top, respectively. This may be due to differences in characteristics between old and new sections of the dumpsite..GEM-2 EM sensor can be used for waste stratification of dumpsite with careful interpretation. Detail site investigation can be carried out by incorporate with other geophysics technique.

5 ACKNOWLEDGMENTS

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Estimation of Leachate Generation Using HELP Model in an Open Dumpsite in Sri Lanka

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Abstract: *Managing leachate is one of the problems associated with municipal solid waste landfills. Leachate generation highly varies with type of waste, climate, site and surface condition. Several mathematical computer models have been used for prediction of leachate for controlled sanitary landfills. However, the prediction of leachate for an open dumpsite is rarely reported. The applicability of Hydrologic Evaluation of Landfill Performance (HELP) model which is based on water balance method was evaluated in this study for Udapalatha open dump site in the Central Province, Sri Lanka. Input weather data (rainfall, temperature, Relative Humidity, wind velocity, solar radiation) and site specific data (area, depth, profile characteristics) were obtained from nearby weather station and site investigation, respectively. Model output leachate was validated (quasi) with changes in groundwater level in percussion boreholes which were installed at the dump site and monitored from May 2013.The leachate generation was 2759 mm when the rainfall was 3270 mm in 2013. Thus, 84% of the precipitation is contributed to leachate annually. The trends in temporal changes of water level in monitoring well and estimated leachate with rainfall were similar. Monthly leachate generation shows that the leachate is more than rainfall in few months where heavy rain was recorded in the previous month. This may be due to the release of water stored in the waste layer in the previous month. HELP model satisfactorily produce the annual and monthly leachate in the open dump site tested.*

Keywords: *HELP model, Leachate, Municipal solid waste, Open dumpsite***.**

1. INTRODUCTION

Leachate is formed by percolation of rainfall through an open solid waste landfill or through the cap of a completed site when the refuse moisture content exceeds its field capacity (El-fadel*et al* 2010). Leachate contains of organic and inorganic compounds, microorganisms, humic, fulvic substances, carcinogenic materials thus, a source of pollutant for surface and groundwater contamination.

One of the problems associated with landfill is managing the leachate generation. The quantity of landfill leachate can exhibit considerable temporal variations due to many factors such as climate, refuse characteristics and internal process of landfill. Climate is the main factor that governs the

leachate production compared to other factors. Sri Lanka belongs to warm climatic region receives rainfall in two seasons due to north-east and South-west monsoons. Higher amount of rainfall was obtained in north-east monsoon from October to February. Therefore, estimation of leachate quantity is important in designing the leachate collection system and other operations.

There are very limited data available on the leachate generation in landfills or open dumpsites in Sri Lanka. Most of the studies focussed on the quality than the quantity of the leachate generated. There are several models used for leachate quantification (Abdulrahman, *et al* 2013, Adeolu, *et al* 2011, Aziz, *et al* 2011, Pantini *et al* 2013, Velasquez *et al* 2003). HELP (Hydrological Evaluation of Landfill Performance) model is one of the methods used to estimate the leachate in landfills (Schroeder *et al* 1994, Berger *et al* 1996,Berger, 2013). It is originally developed to evaluate the hydrologic performance of liner of landfills and to assist the design alternatives of sanitary landfills. HELP model can be tested for an uncontrolled landfill. The objectives of this study were to assess the applicability of HELP model for leachate quantification in uncontrolled landfill site and to understand the temporal variability of leachate generation.

2. MATERIAL AND METHOD

2.1 Location of the study

Open dumpsite, located in UdapalathaPradeshiyaSabha(PS),Central Province, Sri Lanka was selected. The dumpsite (N 7˚ 09', E 80˚ 35') is at the right bank of the Mahaweli River (Figure1). The dumpsite has two sections with different period of operation. Section that was abandoned in 2007 considered as old section. The other section was considered as new section since it was operated until 2011. The topography of the site is sloppy with 15 m elevation difference in 50 m width. Total length of dumpsite was 200 m.

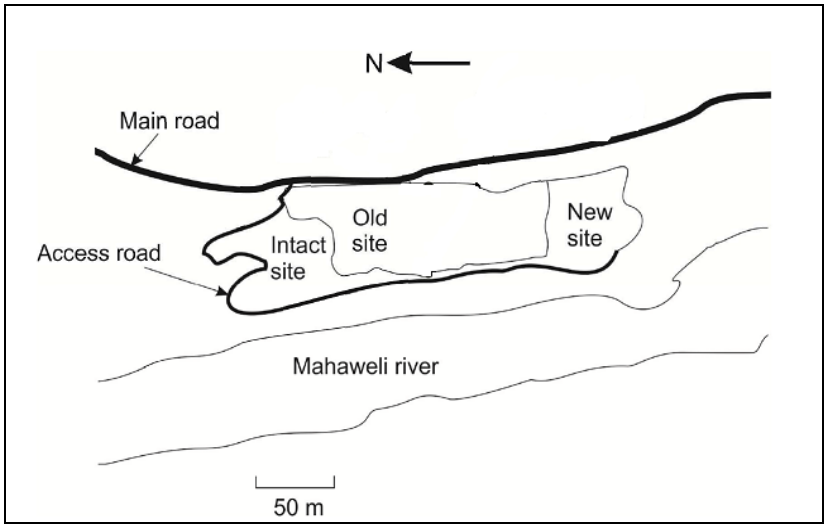


Figure1 Schematic diagram for the location of dump site

2.2 Input Data for HELP Model

The HELP model is a quasi-two-dimensional model that performs the daily water balance over a one-dimensional column of landfill. Vertical drainage is modelled by Darcy's law and saturated lateral drainage is modelled by the Boussinesq equation employing Dupuit-Forcheimer assumptions. Darcy's low is used to measure saturated hydraulic conductivity of each layer. Unsaturated Hydraulic Conductivity is used to measure flow when soil is not fully saturated (Schroeder *et al* 1994).

Evaporation data should be provided as input on daily basis with evaporative zone depth of the landfill. The HELP model computes the evaporation coefficient using Ritchie's equation. The coefficient is limited to 3.3 to 5.5 in the model. Potential evapo-transpiration is estimated using the

modified Penman method. Evaporation of interception and surface water is based on the energy balance method, and interception is modelled by a method proposed by Horton (Schroeder *et al*1994). The runoff process is modelled using the Soil Conservation Service (SCS) curve number method (Mark, 1995).

Three types of input data (weather data, soil characteristic data and site information data) required by HELP model.

2.2.1 Weather Data

Error! Reference source not found.shows the input data of model. Rainfall, temperature, and solar radiation data on daily basis and relative humidity data on seasonal basis are required. Maximum leaf area index, evaporative zone depth and average wind speed are needed to be entered to the model. Rainfall data collected from close by rain gauge at Evalgolla tea estate (N 7° 06', E 80° 21'). Temperature andbright sunshine hour data were collected from weather station at Gannoruwa (N 7° 17', E 80° 35') since those are not available in Evalgolla.

Table 1 Weather data required for the HELP model

Data Type	Parameter	Input value	Unit
Weather data	Rainfall	Daily	Mm
	Temperature	Daily	C˚
	Solar radiation	Daily	MJ/m^2
	Maximum leaf area index	3.5	-
	Evaporative zone depth	76.2	Cm
	Relative humidity	Seasonally	-
	Average wind speed	3.75	km/h

Rainfall is the main source of water for an open dump site and one of the important input parameter for the model. Figure 3 shows the monthly rainfall recorded at Evagolla weather station for the study period 2013. High rainfall was recorded in the months of June, July and September in 2013.March and December were dry months with less rainfall. The total annual rainfall was 3270 mm in 2013.

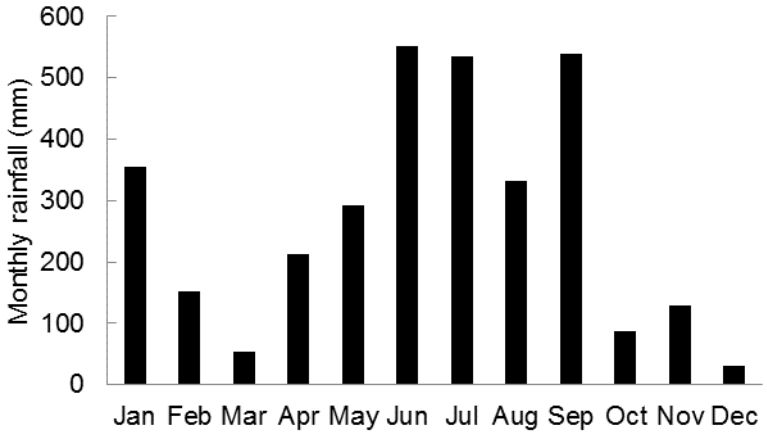


Figure 3 Monthly rainfall at Evalgolla in 2013

2.2.2 Soil and Waste Characteristics

HELP model requires soil characteristics identified for each layers and each layer is considered as homogeneous. Soil type, layer thickness, bulk density and initial moisture content have to be fed to the model. Accordingly, the model generate hydraulic conductivity, porosity, field capacity and permanent wilting point.
Model was run for new and old section separately since soil characteristicswere different. Table 2 shows the soil types with depth up to bed rock from waste layer.This data were obtained from the

log book maintained during the drilling of bore holes and installation of the wells. Table 3 show the properties of each soil and waste layers.

Table 2 Soil and waste information used for the model

New Site		Old site	
Depth from ground surface (m)	**Soil type**	**Depth from ground surface (m)**	**Soil type**
3	Dump material	1.3 - 4.2	Dump material
4	Loam	4.3- 7.2	Fine sand
5.5	Fine sand	5.2 - 8.2	Loamy fine sand
10.2	Coarse sand	5.3- 8.3	Coarse sand
10.4	Bed rock	6.1 - 9.1	Coarse sand

Table 3 Soil and waste properties

Soil type	Total porosity (vol/vol)	Field capacity (vol/vol)	Wilting point (vol/vol)	Saturated hydraulic conductivity (cm/sec)
Waste material	0.671	0.292	0.077	1×10^{-3}
Coarse sand	0.417	0.045	0.018	1×10^{-2}
Fine sand	0.457	0.083	0.033	1×10^{-3}
Loamy fine sand	0.457	0.131	0.058	1×10^{-3}
Sandy Loam	0.453	0.190	0.085	7.2×10^{-4}
Loam	0.463	0.232	0.116	3.7×10^{-4}

2.2.1 Site Information

Model requires dumpsite information such as dumping area, percentage of runoff area, runoff curve numbers (CN), slope, slope length andtype of vegetation. The model also capable to computing runoff curve number by using given data.

2.3 Groundwater Monitoring Well

Groundwater was monitored to validate the HELP model. Percussion Bore Holes (PBH)was installed inside of the buried waste in landfill area up to the bottom of the waste layer. Pipe in PBH is perforated up to the top of the well from the bottom as shown in Figure 2. Old section and new section were monitored from 2 PBH and 3 PBH, respectively. Groundwater level in PBH was measured using water level sensor from May 2013 onwards with monthly interval. Depth of well and water level were measured. Accordingly, depth of water in the each well was determined.

2.4 Running the Model

HELP 3.07 was run in the DOS environment. Evaporation, precipitation, temperature and solar radiation data were in ACSII data input format. Then, the information of the waste layer was entered. The unit of the output, period of the output, interval of the output were specified.

2.5 Model Validation

Leachate quantity predicted by HELP model was compared with boreholes (PBH) water level changes during the time period of May 2013 to December 2013. Four monitoring wells were selected for comparison in new and old sections, respectively.
The model output was checked with the observed water level data and adjustment was made to the texture of waste and soil, thickness of the layer.Model was run repeatedly. This procedure was followed until the model predicted result reasonably with observed data. It was assumed that texture was uniform throughout the layer.

2.6 Model Simulation

After the validation of the model, it was used to simulate the leachate for the past 5 years period from 2009-2013 for old and new section separately.

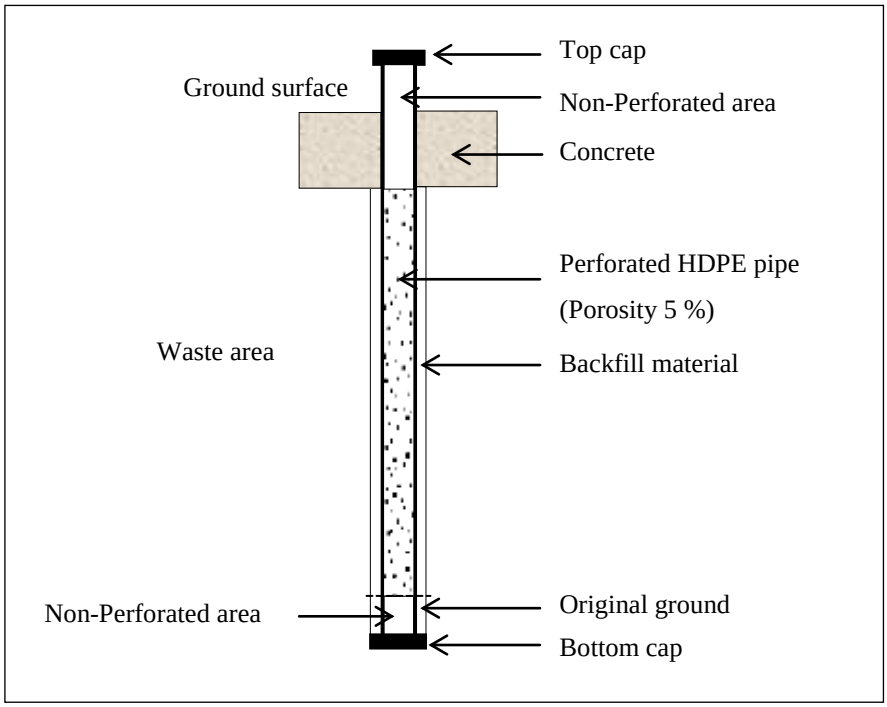


Figure 2 Schematic diagram of PBH

3. RESULTS AND DISCUSSION

3.1 Model Output

Figure 5 shows the output of the HELP model. Leachate quantity as percolation, evapo-transpiration, surface runoff and change in moisture content in the waste layer were quantified in depth (mm).These outputs can be obtained as daily, monthly or yearly and the peak values during the simulation period.

Figure 5 shows the total summary of the components of the water balance during the study period. Large portion (84%) of rainfall has been contributed for leachate production in 2013.Evapo-transpiration was 18% of the rainfall. Negative storage, 2.7%, was found as water released from waste. Surface runoff was lesser than 0.1% due to vegetation cover at the Udapalatha dumpsite.

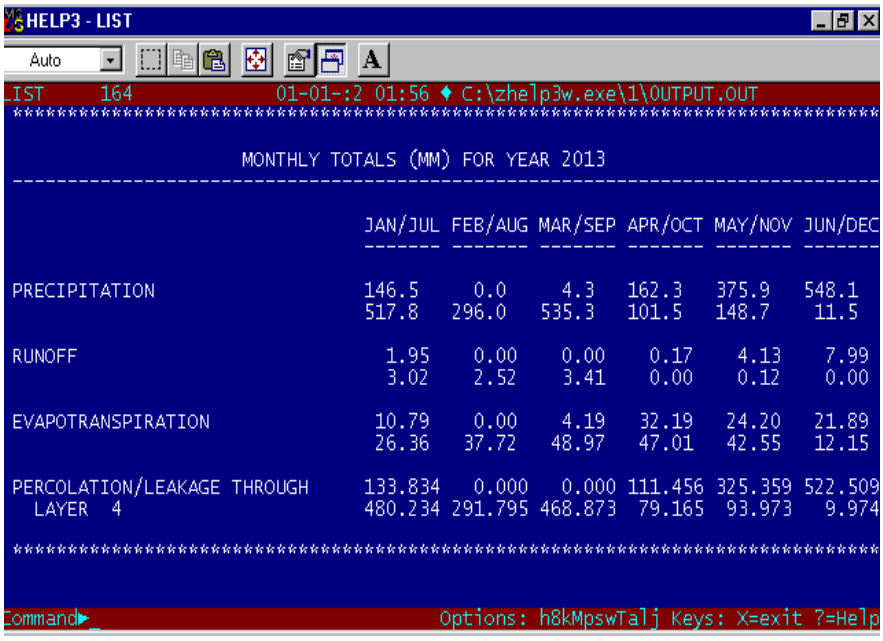


Figure 4 Model output

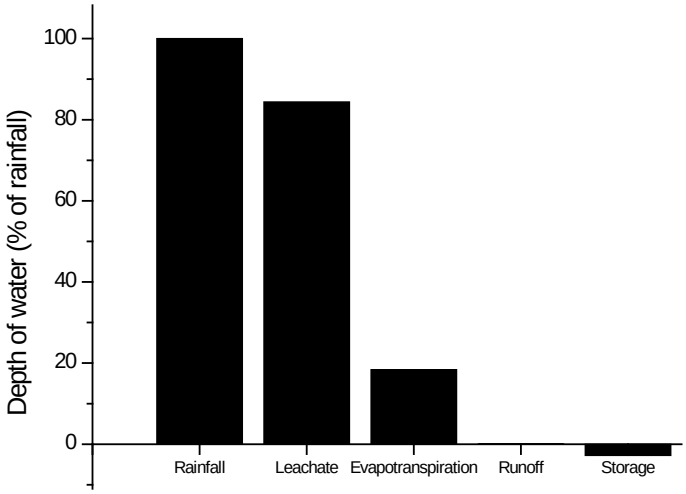
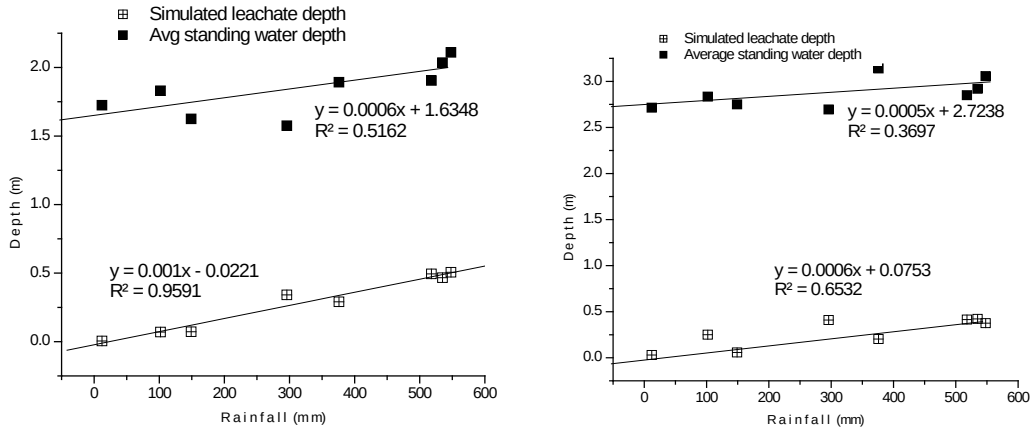


Figure 5 Summary of output of the model

3.3 Rainfall, Leachate and Water Levels in PBH

Figure 6 shows the monthly average water level in PBH and leachate quantity estimated by HELP model. Both leachate quantity and water level in PBH reduced with less rainfall and increased with high rainfall. The trends in changes of water level in monitoring well and estimated leachate generation with rainfall were similar. The leachate generated by HELP model is reflected by change in groundwater levels both in new and old sections.



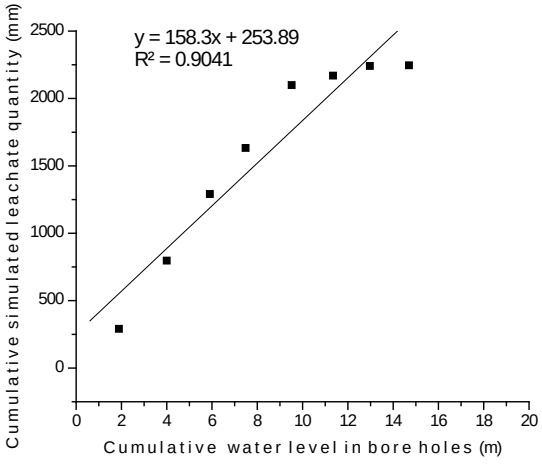
(a) (b)

Figure 6 Leachate and water depth in PBH with rainfall in new (a) and old (b) sections

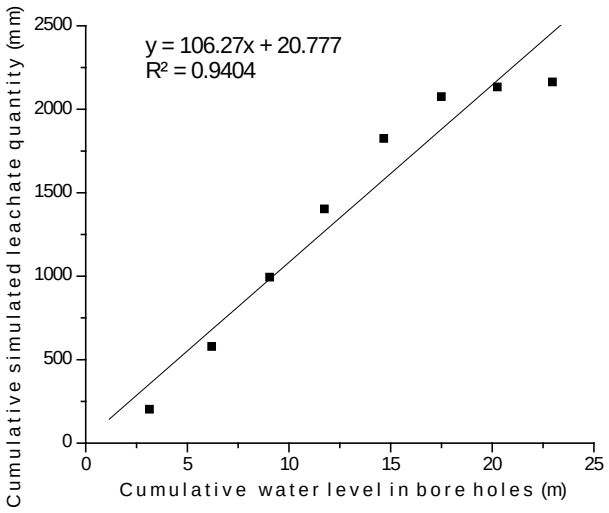
3.4 Model Validation

HELP model output was validated with change in water level for the months fromMay to December 2013 since the water level monitoring was begun from May 2013. Figure 7, shows the cumulative leachate generation and the water level change in PBH for new and old section. Since all the boundaries of the site are not hydrologically defined, 1:1 relation cannot be expected. The dumpsite was not in operation during the validation period, the effect of loading at the bottom strata due to compaction is not considered. However, the results show a good validation for estimating the leachate in Udapalatha dumpsite.

Two trends were observed for dry and wet period (Figure 7). In the later part of the year of this time series data, higher changes were observed in water level but with less generation of leacahte. This may be due to the dry condition from October to December. The increase in water level in borehole may be from outside of the dumpsite. Soil and waste characteristics, measured in the previous year were used (secondary data source). In addition, the model assumes that the waste layer is uniform horizontally. These assumptions may be the reason for difference between water level in PBH and leachate generation. Therefore, this validation can be considered as quasi validation.



(a)



(b)

Figure 7 Relationship between cumulative changes in water level in bore holes and cumulative leachate estimated by model (a) New sectionand (b) old section.

3.5 Effect of Rainfall on Leachate Quantity

Figure 8 shows the effect of monthly rainfall on leachate generation. In another study (Rafizul & Alamgir, 2012) also have shown similar results between rainfall and leachate generation. Highest leachate quantity shows during the high rainfall period. However, two deviations were observed in month of August (new and old sites) and October (old site) where leachate generation is higher than the monthly rainfall. It may be due to change of water storage in waste in the previous months. When there were high rainfall waste layer get saturate and release the water gradually in the following dry period. Although the Leachate generation depends on the rainfall, the retention and releasing capacity of the waste layer also plays a major role. Figure 9 shows the simulated leachate for past 5 years from 2009 to 2013. Highest leachate is produced in 2009 and 2010. The lowest leachate was found in the following years 2011 and 2012. Therefore, the leachate management options should take into account this temporal variation.

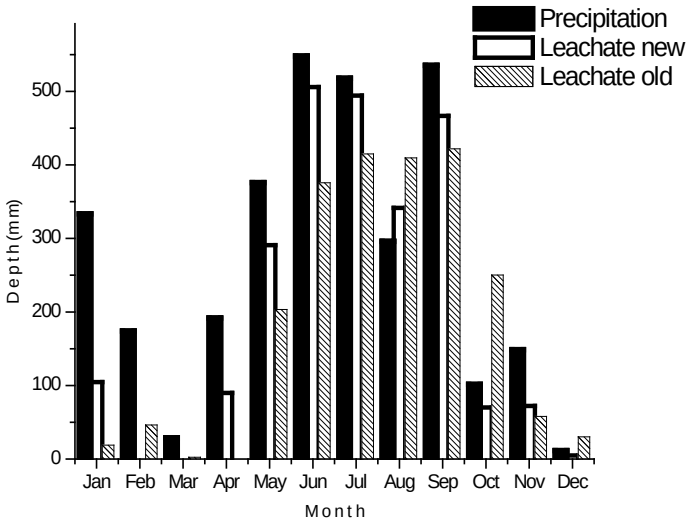


Figure 8 Monthly Rainfall and leachate quantity

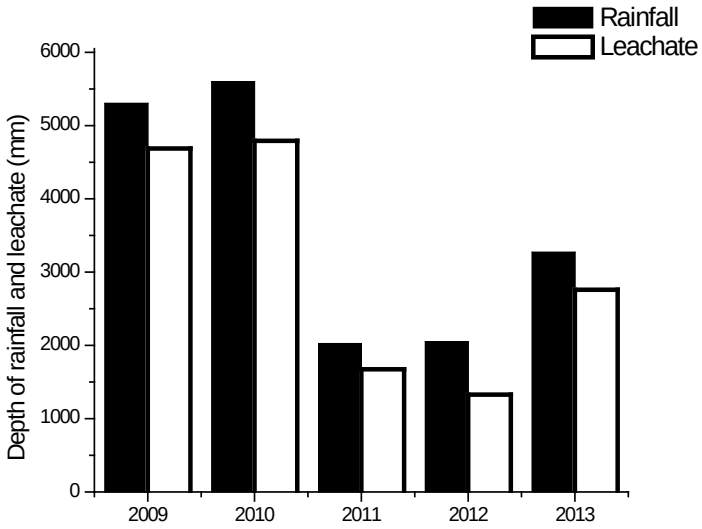


Figure 9 Annual rainfall and leachate production

4. CONCLUSIONS

Leachate, evapotranspiration and runoff were generated using weather, soil and site information by HELP model for a dumpsite. Large portion (84%) of rainfall has been contributed for leachate production. Evapotranspiration was 18% of the rainfall. The change in moisture storage in the waste was -2%, as moisture content decreased.The trends in temporal changes of water level in monitoring well and estimated leachate generation with rainfall were similar. The leachate generated by HELP model is reflected by change in groundwater levels both in new and old sections. A quasi validation was done for leachate generation by model with the cumulative change in water levels. Simulation of leachate in last 5 years shows that although the Leachate generation depends on the rainfall, the retention and releasing capacity of the waste layer also plays a major role. Increasing the number of layers using wider range of soil classification may improve the model output.

5. ACKNOWLEDGMENTS

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Nitrogen Removal from Municipal Leachate by Combined Aerobic-Anaerobic Treatments

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Abstract: *Combined nitrification and de-nitrification have proven for the effective nitrogen removal from landfill leachate with minimum process controls compared with other nitrogen removal methods such as ANAMMOX. Main aim of this study was to obtain the optimum hydraulic retention time (HRT) which gives maximum nitrogen removal efficiency. Behalf of that, several water quality measurement parameters were recorded throughout the process and their variations were interpreted based on the theories of microbial action.Nitrification was carried out in a suspended growth, aerated reactor. The reactor was maintained with 2 day and 1 day HRTs and mixed liquor suspended solids of 2400 mg/L. Ammonium nitrogen removal efficiency of the reactor were up to 33% and 21% respectively. The effluent from aerobic reactor was fed to an anoxic, attached growth reactor. It was maintained with 1 day and 0.5 day HRTs. An external carbon source (methanol) was added as an electron donor to promote de-nitrification and methanol dosage was maintained at 3.6 mg COD/mg NO_3^-N. The overall nitrogen removal efficiency was up to 61% and 56% while total ammonia nitrogen removal was up to 41% and 29% respectively with above mentioned HRTs.*

Keywords: *Nitrogen removal, Aerobic reactor, Anaerobic reactor, Hydraulic retention time*

1. INTRODUCTION

Landfill leachate causes serious problems in ground water and surface water sources due to its various contaminants. Nitrogen is one of the contaminants which create a lot of posers in water bodies with exceeding amounts. Ammonium nitrogen is dangerous for fishes, causes unexpected aquatic growth creating further problems and depletes oxygen. Additionally nitrate nitrogen causes methemoglobinemia (blue baby syndrome) in infants (Horan, 1997; Metcalf and Eddy, 1991). Therefore nitrogen level in the treated leachate discharges need to be maintained below the maximum allowable limit. There are four phases of nitrogen in leachate as ammonium nitrogen (NH_4^+-N), nitrate nitrogen (NO_3^--N), nitrite nitrogen (NO_2^--N) and organic nitrogen. Even though nitrogen can be removed as nitrogen or ammonium gas, the best way is to remove as nitrogen gas. It is because of the odor and other environmental impacts. Therefore microorganisms should be controlled in the way of producing nitrogen gas. Among the biological treatment methods combined aerobic nitrification and anoxic de-nitrification has proven to be more feasible for leachate treatments.

This study was carried out in a series aerobic and anaerobic reactor which ensures de-nitrification followed by nitrification. There are favorable conditions which should be maintained in the bioreactors to sustain the microbial activity. However sufficient hydraulic retention times for the nitrification and de-nitrification is very important in order to achieve maximum nitrogen removal. Therefore one of the objectives is to investigate the sufficient hydraulic retention time of the reactors by comparing NH_4^+ - N, NO_3^- - N, NO_2^- - N, Total nitrogen removal efficiencies. Additionally some of the selected water quality parameters such as BOD, pH, TDS, conductivity and ORP (Oxidation Reduction Potential) were recorded throughout the treatment. Then an interpretation was carried out based on the theories.

1.1. Nitrogen Removal Processes

Chemotrophs are the ones involved in nitrogen removal processes. They would catalyst the breakdown of compounds in leachate. In a chemical breakdown, two half reactions happen as oxidation and reduction. Oxidation is the donation of electrons to a substance while reduction is the acceptance of the electrons from a substance. Oxidation release energy while reduction absorbs energy. Therefore biological redox reactions are arranged such a way that they proceed with a net yield of free energy. This available energy is consumed by the organisms (Zhou, 2001; Metcalf and Eddy, 1991; Rittmann, 2001; Emily, 2013). Suitable microorganisms and environment should be maintained in the reactors to achieve conversion of all phases of nitrogen into the nitrogen gas.
It is called aerobic when oxygen is the electron acceptor and anaerobic when another electron acceptor. Additionally it is called anoxic when the nitrite or nitrate as electron acceptors under anaerobic condition (Zhou, 2001; Metcalf and Eddy, 1991; Rittmann, 2001).

In this study main three nitrogen treatment processes were studied. Those were aerobic nitrification, anoxic de-nitrification and anaerobic ammonium oxidation (ANAMMOX). In landfills, the organic nitrogen would degrade into the ammonia gas under anaerobic condition. Ammonia is highly soluble in water establishing equilibrium as,

$$NH3 + H2O \rightleftharpoons NH3.H2O \rightleftharpoons NH4+ + OH-$$

Therefore ammonium removal should start with NH_4^+ removal or conversion since the percentage is very high in leachate.

1.1.1. Aerobic nitrification

Nitrification is generally carried out by aerobic, autotrophic bacteria. NH_4^+ is converted to NO_2^- and then NO_2^- further converted to NO_3^- with molecular oxygen as the electron acceptor. (Khin and Annachhatre, 2004; Mulder et al., 1995; Schmidt et al., 2003). The relevant bacteria involvement and reactions are show in Table 1 given below (Sliekers et al., 2002; Sliekers, 2003; Horan, 1997; Schmidt et al., 2003).

Table 1 Aerobic nitrification

	Conversion of NH_4^+ to NO_2^-	**Conversion of NO_2^- to NO_3^-**
Bacteria	Ammonia oxidizing bacteria (AOB) β proteobacteria – Nitrosomonas Nitrosospira Γ proteobacteria – Nitrosococous	Nitrite oxidizing bacteria (NOB) δ γ α proteobacteria - Nitrobacter, Nitrococcus Nitrospira phylum
Enzymes	Ammonium monooxygenase (AMO) Convert NH_4^+ to hydroxylamine Hydroxylamine oxidoreductase (HAO) Convert hydroxylamine to NO_2^-	Nitrite oxidoreductase (NO2-OR) Convert NO_2^- to NO_3^-
Electron donor half reaction	$NH_4^+ \longrightarrow N^{3+} + 4H^+ + 6e^-$	$NO_2^- \longrightarrow N^{3+} + O^{2-} + e^-$
Electron acceptor half reaction	$O_2 + 4e^- \longrightarrow 2O^{2-}$	$O_2 + 4e^- \longrightarrow 2O^{2-}$
Overall reaction	$NH_4^+ + 1.5O_2 \rightarrow NO_2^- + H_2O + 2H^+$	$NO_2^- + 0.5O_2 \longrightarrow NO_3^-$

Overall reaction is,

$$NH_4^+ + 1.83O_2 + 1.98HCO_3^- \longrightarrow \underset{\text{(New cells)}}{0.021C_5H_7NO_2} + 1.04H_2O + 0.98NO_3^- + 1.88H_2CO_3$$

The microorganisms involve in the nitrification are obligate aerobes. They utilize molecular oxygen for their needs. Therefore Dissolved oxygen concentration is the critical parameter (Horan, 1997). There are some optimum operational conditions which should be maintained in the nitrification reactor and they are given in Table 2.

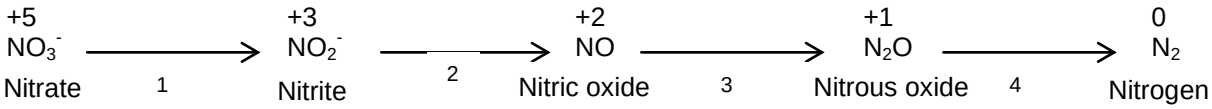
Table 2 Operational considerations of nitrification process

Factor	Description on effect
Dissolved oxygen concentration (DO)	DO concentration > 1.2mg/L
pH	pH range - 7.2 to 8.6. pH has to be maintained by adding some alkaline.
Residence time	Aerobic SRT range - 6 to 8 days and possibly higher
Temperature	Temperature range - 5° to 40°C. Maximum rate gives at the range - 28^0C to 35^0C.

(Carrera et al., 2003; Schmidt et al., 2003; Zhu et al., 2008; Khin et al., 2004; Metcalf and Eddy, 1991; Kulikowska and Klimiuk, 2003)

1.1.2. Anoxic de-nitrification

The denitrification is the reduction of NO_3^- and NO_2^- to gaseous nitrogen. Heterotrophic facultative bacteria are involved (Horan, 1997; Zhu et al., 2008; Knowles, 1982). Nitrogen oxides serve as electron acceptors and hence a suitable electron donor is required such as organic matter, reduced sulfur compounds and molecular hydrogen etc. The reduction of NO_3^- to N_2 happens in four steps as follows.



Reactions and enzymesinvolved in these reactions are given in Table 3

Table 3 Reduction reactions in the de-nitrification process

Step	Reduction reaction	Enzymes involved
1	$NO_3^- + 2H^+ + 4e^- \longrightarrow NO_2^- + H_2O$	Nitrate reductase
2	$NO_2^- + 2H^+ + e^- \longrightarrow NO + H_2O$	Nitrite reductase
3	$2NO + 2H^+ + 2e^- \longrightarrow N_2O + H_2O$	Nitric oxide reductase
4	$N_20 + 2H^+ + 2e^- \longrightarrow N_2 + H_2O$	Nitrous oxide reductase

The final reaction with a carbon source as electron donor

$NO_3^- + 1.08CH_3OH + H^+ \longrightarrow 0.065C_5H_7O_2N + 0.47N_2 + 0.76CO_2 + 2.44\ H_2O$

Bacteria involved in the anoxic de-nitrification are genera of Alcaligenes, Pseudomonas and Rhodobacter, Rubrivivax and Psedomonas subgroups respectively in α, β, and γ purple subdivision (Etchebehere et al., 2001; Graaf et al., 1996). Operational considerations for anoxic de-ntrification process are summarized in Table 4.

Table 4 Operational considerations of de-nitrification process

Factor	Description on effect
Dissolved oxygen concentration (DO)	DO concentration < 0.5 mg/L
pH	Optimum range - 7.0 to 8.0. But, may occur up to pH 11. Decreasing pH and increasing O_2 tend to decrease the overall de-nitrification rate.
Temperature	Temperature range - 10 to 35°C
Carbon requirement	2.86 g COD/ g NO_3^- -N
Retention time	Low retention time due to high biomass yield.

(Carrera et al., 2003; Schmidt et al., 2003; Zhu et al., 2008; Khin et al., 2004; Metcalf and Eddy, 1991)

2. MATERIALS AND METHODS

2.1. Leachate Feed

Leachate which was used in this study was collected from a municipal landfill located in Guhagoda (near Kandy, Sri Lanka). The biochemical oxygen demand (BOD5) and pH was 300 – 400 ppm O2 and 8 – 8.6 respectively. Typical NH4+ - N were 3200-3300 mg/L. The leachate was collected once per week.

2.2. Aerobic Nitrification Reactor

Leachate was continuously fed to the aerobic suspended growth bioreactor which was maintained with 2 day and 1 day hydraulic retention times. Volume of the reactor was 8 L. Mixed liquor suspended solids was maintained 2400 mg/L. DO concentration was maintained above 1.2 mg/L, so that air is pumped and aerated with leachate. Temperature and pH were maintained at 280C (ambient temperature) and 7.2 - 8.6 respectively. Reactor was operated for 5 weeks for each HRT.

2.3. Anoxic De-nitrification Reactor

Aerobic bioreactor effluent was continuously fed to a anaerobic bioreactor with 1 day and 0.5 day hydraulic retention times. Reactor volume was 5 L. DO concentration, pH and temperature were maintained below 0.5 mg/L, 7 – 8 and 28^0C (ambient) respectively. Additionally methanol was added to maintain 2.86 g COD/ g NO_2^--N in the reactor. Reactor was operated for 5 weeks for each HRT. Experimental parameters which were maintained are shown in the Table 5, below.

Table 5 Control factors for the experiments

Experiment	Aerobic HRT /day	Anaerobic HRT / day
1	2	1
2	1	0.5

2.4. Analytical Method

NH_3 - N, NO_2^- - N, Total nitrogen, COD, BOD, pH, turbidity, conductivity and ORP (Oxidation Redox Potential) were measured in leachate feed, aerobic and anaerobic bioreactor effluent weekly. Methods which were followed for the analysis of the above parameters are shown in the Table 6. Variation of selected water quality parameters with different hydraulic retention times of leachate were graphed to compare the results and to investigate optimum HRT which gives maximum nitrogen removal.

Table 6 Analytical methods

Parameter	Method
NH_3 - N	Salicylate method, HR (0.4 to 50.0 mg/L NH_3 – N)
NO_2^- - N	USEPA Diazotization, LR (0.002 to 0.300 mg/L NO_2^- - N)
Total nitrogen	Total Organic carbon analyzer
COD	USEPA Reactor Digestion Method, 20 to 1500 mg/L COD
BOD	BOD Incubator method
pH	pH meter
turbidity	Turbidity meter
conductivity	Conductivity meter
ORP(Oxidation Redox Potential)	ORP meter

3. RESULTS AND DISCUSSION

Reactors were operated for 5 weeks for each HRT. The results obtained are shown below.
NH_3-N removal percentages were considered to find the optimum HRT of the aerobic reactor. Figure 1(a) shows the variation of NH_3–N removal percentages of leachate in the aerobic reactor with experiment 1 and 2. NH_3–N removal efficiency was simply calculated using the equation 1.

$$NH_3\text{-N removal efficiency} = \frac{(Influent\ NH_3\text{-N}) - (Effluent\ NH_3\text{-N})}{(Influent\ NH_3\text{-N})} \quad ------(1)$$

Ammonium nitrogen removal efficiencies in the nitrification reactor with experiment 1 and 2 were up to 33% and 21% respectively. Therefore experiment 1 show much NH_3-N removal than the experiment 2 showing that 1day HRT may not enough for the nitrification. Figure 1(b) shows the total ammonia nitrogen removal percentage after the two reactors. It was up to 41% and 29% with experiment 1 and 2 respectively resulting high ammonia removal with experiment 1.

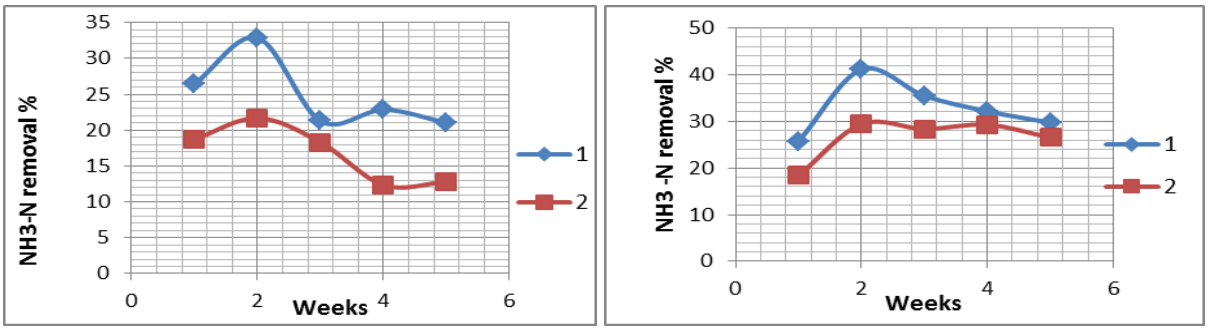


Figure 1 (a) Variation of NH3 – N removal percentage in the aerobic reactor (b) Variation of NH3 – N removal percentage by the treatment

Furthermore total nitrogen (TN) removal efficiencies were compared for the investigation of overall performance of the combined aerobic anaerobic treatment. Figure 2 shows variation of TN removal efficiencies with experiment 1 and 2 and they were up to 61% and 56% respectively. Hence, Experiment 1 gives much removal of nitrogen compared to experiment 1.

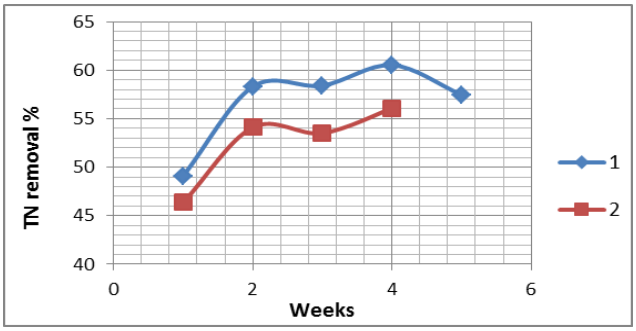


Figure 2 Variation of Total nitrogen removal percentages

When comparing NH_3-N removal and total nitrogen removal percentages, HRTs of experiment 1 are much prefer for the nitrogen removal than the HRTs of experiment 2. Thence, the optimum HRT for combined aerobic and anaerobic reactors are 2 day and 1 day respectively from the tested HRTs. 1 day and 0.5 day HRTs are not sufficient for the nitrification and de-nitrification processes.

Then variation of some of the water quality parameters with the optimum retention times (HRTs of experiment1) were investigated. Figure 3(a) shows the NH_3–N variation of feed and two reactor effluents. Ammonia nitrogen concentrations have reduced in both reactor outlets, but it has not become steady. It may due to the composition change in the raw leachate from week to week since substrate concentration changing. Ammonium nitrogen concentration has dropped after the aerobic reactor showing that nitrification has taken place. But, a drop of NH_3-N has given in the anoxic reactor

also. If only de-nitrification took place in the anoxic reactor, ammonia concentration should stay still. Hence NH_3 also has participated in a reaction and it may ANAMMOX.

Figure 3(b) shows NO_2^-–N variation of leachate feed and two reactor effluents. Nitrite concentrations in leachate are small compared with ammonia nitrogen. At the 5th week, NO_2^--N in the aerobic reactor effluent has reached very high value. High nitrite concentration may due to the absence of nitrite oxidizing bacteria or low oxygen demand in the reactor. However dissolved oxygen concentrations were at preferable limits. Therefore it should be due to the absence of nitrite oxidizing bacteria.

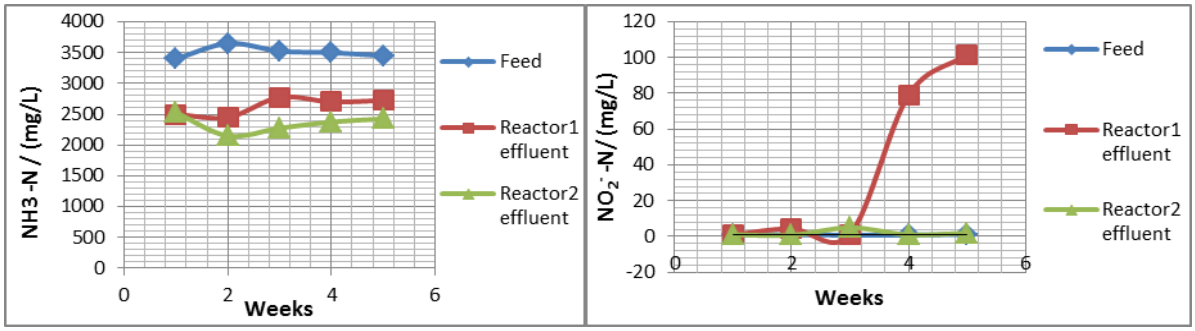


Figure 3 (a) NH_3 – N variation of leachate feed, aerobic and anaerobic reactor effluents (b) NO_2^-–N variation of leachate feed, aerobic and anaerobic reactor effluents

Figure 4(a) shows total nitrogen variation in leachate feed, aerobic and anaerobic reactor effluents. It is clearly visible that the total nitrogen content has reduced with the treatment. Figure 4(b) shows BOD_5 variation in the leachate feed and the two reactor effluents. BOD_5 have decreased after the treatment with a removal percentages of 86.96 – 96.33 %. Even though experiments were carried out only to remove nitrogen, BOD_5 level also has come to a preferable level standing as an added advantage.

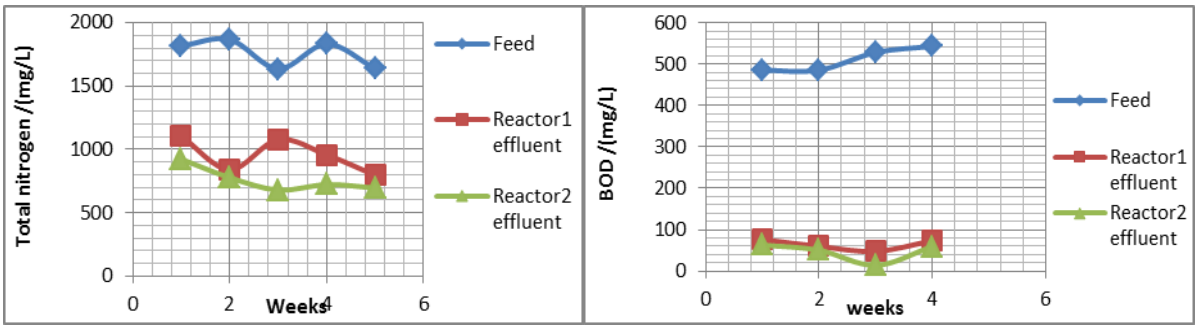


Figure 4 (a) Total nitrogen variation of leachate feed, aerobic and anaerobic reactor effluents (b) Variation of BOD_5 in the feed and two reactor effluents

Figure 5(a) shows the pH variation of leachate feed and two reactor effluents. pH has increased with aerobic treatment and further by anaerobic treatment. According to the microorganisms' metabolism pH should be reduced after the aerobic reactor according to nitrification reaction shown below.

$$NH_4^+ + 1.83O_2 + 1.98HCO_3^- \longrightarrow 0.021C_5H_7NO_2 + 1.04H_2O + 0.98NO_3^- + 1.88H_2CO_3$$

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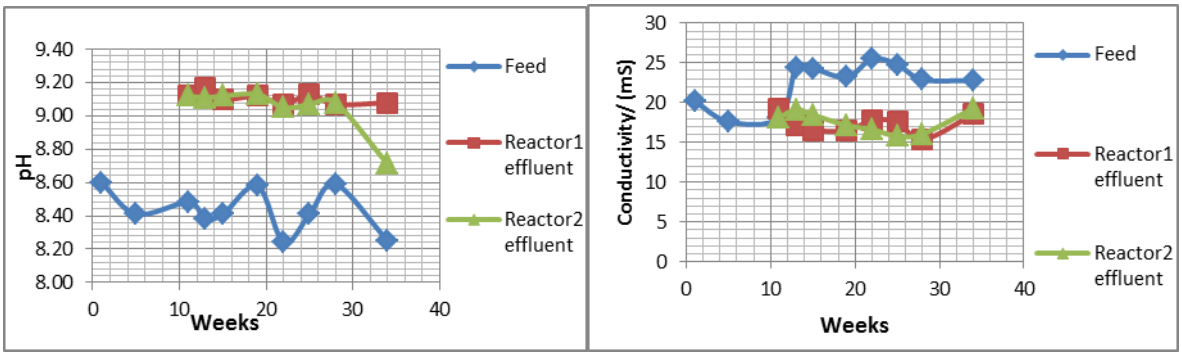

Figure 5 (a) pH variation of leachate feed, aerobic and anaerobic reactor effluents (b) Conductivity variation of leachate feed, aerobic and anaerobic reactor effluents

However, pH has shown an increment after the nitrification process which is not possible according to theories and literature. Therefore, to verify this variation further studies have to be carried out.

Figure 5(b) shows the conductivity variation of leachate feed and two reactor effluents. Conductivity gives an idea how well a solution conducts electricity due to the presence of charged particles or ions. Ions are from electrolytes dissolved in water such as salts (NaCl, $MgSO_3$), Acids (HCl, acetic) and Bases (NaOH, Ammonia). Conductivity has reduced up to 36% with the treatment due to the reduction of charged particles in leachate.

Figure 6(a) shows the Total Dissolve Solid (TDS) variation of leachate feed and two reactor effluents. Total dissolved solid content has reduced due to the treatment with a removal percentage up to 40% as one more added advantage with the combined nitrification and de-nitrification. Figure 6(b) shows Oxidation Reduction Potential (ORP) variations of leachate feed and two reactor effluents. ORP has reduced with the treatments. ORP measures the water's ability to oxidize contaminants. Higher ORP means higher the number of oxidizing agents. With the treatment ORP has reduced showing lower number of oxidizing agents.

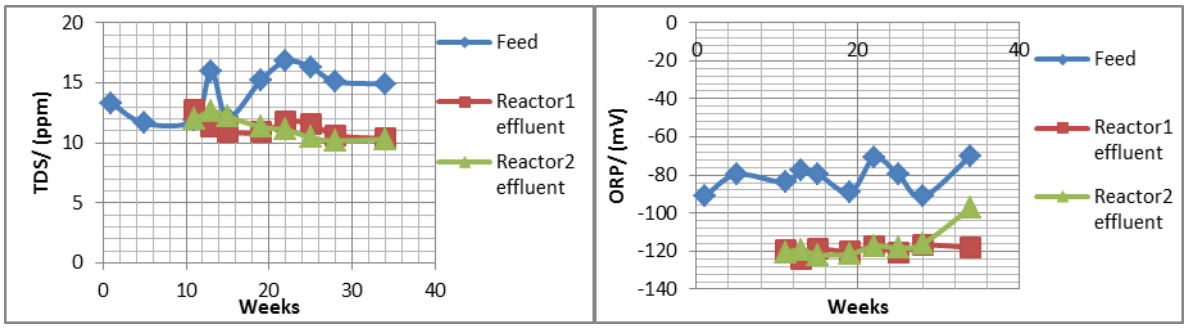

Figure 6 (a) Total dissolve Solid (TDS) variation of leachate feed, aerobic and anaerobic reactor effluents (b) ORP variation of leachate feed, aerobic and anaerobic reactor effluents

4. CONCLUSIONS

According to the results of the study,

i. Optimum nitrogen removal can be achieved at leachate hydraulic retention time 1day and 2day, the average ammonium concentration in the effluent 200 mg NH3-N/dm^3.
ii. Complete nitrogen removal can be achieved by changing leachate hydraulic retention time, since complete nitrogen removal was not received (maximum nitrogen removal was 61%) from the study.
iii. BOD_5 removal is an added advantage with combined nitrification and de-nitrification process.

Raw leachate with high ammonium and BOD_5 concentrations say 3500 mg/L and 550 mg/L respectively was used for the study. Effluent concentrations were reduced up to 200 mg/L and 60 mg/L with the treatment. Hence this combined process can be practiced in landfill dumpsites to treat

leachate, before releasing to water bodies. Then negative effects on environment can be reduced. Additionally this is very cost effective since this requires very law critical maintenances compared with other nitrogen removal methods such as ANAMOX. However due to the leachate feed variation from week to week, results were not steady and gave low yields than expected. Therefore an equivalent tank may have to use before the aerobic bioreactor since composition change in feed could cause to inhibit the microorganism activity and give low yields in reactors.

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Vector Based GIS Applications to Select Suitable Land for Landfill Sitting in Kandy, Sri Lanka

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Abstract: *The accumulation of solid wastes around urban centers has become a worldwide environmental problem. Therefore determining the location of landfill sites that are situated in urban areas is an urgent necessity. Aimed to select suitable, orpotential landfill areas from Kandy district, Sri Lanka, minimization of the hindrance to integration of socio-economic determinants using vector based geographical information systems (GIS) analysis was performed. Grama Niladari (GN) divisions having lowest geographic population in Kandy district were selected to acquire the highest spatial associations between land use and population density. Land use including home garden, buildup areas and paddy fields were evaluated as socio-economic factors for successful landfill sitting in the Kandy district. Vector based approach in GIS was used since it is able to perform spatial suitability analysis more precisely. Vector form of the data obtained from GIS analyses indicated that 53.40% of the total land area has a low population density (<400 persons per square kilometer). However 7.56% of this low population density area was excluded due to home gardens, buildup areas and paddy fields and 35.20% of was under both environmental and social influenced in order to minimize the human health and socio-economic conditions and risks associated with land filling. The rest of these lower population density areas (29.80%) were suggested to be more suitable for landfill sitting in the Kandy district. Future studies will be focused on the application of geospatial and hydrogeological factors in the above selected areas for sitting landfills.*

Keywords: *Landfill sitting, Spatial model, Vector-based data, GIS*

1. INTRODUCTION

Exponential population growth subjected to enhance the accumulation of human being around the built-up areas (Kontos et al., 2003). Due to inappropriate consumption pattern with high per capita ecological foot print in urbanized culture, the tremendous increase in waste generation has been reported (Marín et al., 2012). Therefore, municipal solid waste management (MSW) problems have been assumed a huge proportion in the less developed parts of the world (Rahman et al., 2008). In addition, solid waste management is a major global concern at present that is increasing day by day (Gorsevski et al., 2012).

The most common MSW management practice is open dumping. Most of the developing countries are still focusing on open dumping due to advantages of ability to manage a huge quantity of MSW generated per day (Paul & Krishnagar, 2012). However, open dumps are responsible several environmental problems including surface water and groundwater contamination and severe health issue due to air and water pollution (Ruben & Paralik S., 2006). Hence, it is really important to locate the landfill in an appropriate place which reduces the environmental pollution.

Evaluation of the most applicable location for a new landfill is a complex process that requires undertaken of various criteria (Chang et al., 2008, Gorsevski et al., 2012). Among those criteria economic, ecological and social effects are often considered for enhancing the planning process and for setting guidelines that minimizes environment and public health risks. Some of the locational standards are not compatible when evaluating the entire land mass of the regional selected area at a one time. These criteria are site specific and should be used when evaluating a single site or a limited number of potential sites (Reinmuth, 2004). Therefore siting process included in this plan is intended as preliminary step before going to wide scale site specific siting study. Central Environmental Authority (CEA) in Sri Lanka has established general guideline for the landfill site selection (CEA, 2005) provides. Nevertheless, lack of macro level spatial data influences for proper quality risk assessment in Sri Lanka.

To evaluate preliminary aspect of the landfill siting, assigning simple sum of weight method is considered as a very useful tool. In addition, it is one of common methodologies used for site selection in general, and for siting landfill in a selected region (Al-Hanbali et al., 2011). The socioeconomic concern, geodemogrophy provide more spatial data for landfill siting. However some of demographics characterize as follow, high, medium, and low minority, poverty and population density (Gragg III et al., 1996), among the parameters population density data are very useful attribute for spatial analysis (Weerakoon, 2013). The (Luo, 2005) reviled that, compatibility of the census data and the administrative units avoid spatial adjustments. In Sri Lankan census, it is geographically divided by 25 districts within 9 provinces. Each district is divided into DS (Divisional Secretariat) divisions and each DS division is sub divided into GN (Grama Niladari) divisions. The lower geographic area of census data is the GN division level. In population analysis, GN level is mostly suitable for representative macro level visualization.

For gaining high suitability of landfill siting, the sites has to also satisfy the land use criteria (Paul & Krishnagar, 2012), specifically home garden, paddy fields and water bodies to be assign.

Vector based data are comprised of points lines and polygons which express geo reference attributes of the real world more precisely than the low resolution raster data. (Lin & Kao, 1998, Gorsevski et al., 2012, Marín et al., 2012)Vector based GIS model is capable of processing digital spatial data efficiently and precisely to facilitate landfill siting analysis.

Therefore this study aimed to select preliminary suitable landfill site using the vector based Geographic Information System (GIS) and weighted sum analysis method in Kandy District.

2. MATERIALS AND METHODS

2.1. Study Area

The Kandy Municipal Council (KMC)is located in the central part of the country. The location of the study area is shown in Figure 1.

District upper and lower limit of the extent given below (upper left 80° 25' 21.32" E 7° 29' 32.318" N, upper right 81° 0' 48.282" E 7° 29' 33.549" N, lower right 81° 0' 48.204" E 6° 56' 29.356" N and lower left 80° 25' 23.662" E 6° 56' 28.09" N) which geography derive mean altitude of around 900 m above mean sea level in central of Sri Lanka.

The KMC covers an area of 1934.29 km^2 and consists of 20 Divisional Secretariat divisions also sub divided in to minor divisions as Grama niladari (GN) divisions representing residential population nearly 139,000 and floating population of over 50,000 visit Kandy daily, mainly due to tourism and temple of Lord Buddha's tooth relic located in the middle of the city (Wijesekara et al., 2014, Statistical report 2012). Generation of 130 tons/day of MSW including food waste and industrial waste mainly generated from Kandy (Balasooriya et al., 2013). The average annual precipitation varies around 3175 mm and it has an average temperature range from 21 to 26 ºC and in dry periods the temperature of

the area rises up to 29-30 ^{o}C. The complex geomorphological terrain of different landscapes, soil types, geology and their respective land use patterns, paddy fields, home garden and several others can be observed.

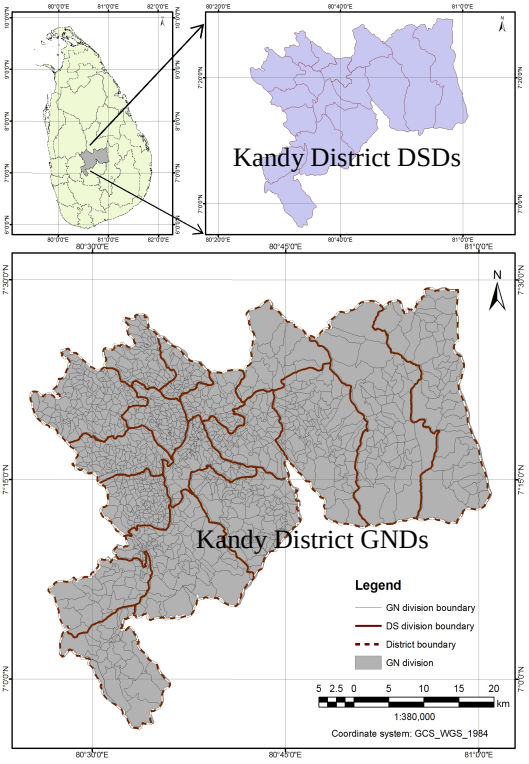

Figure 1 Location of the study area.

2.2. Methodology

In this study, two map layers such as 1:50,000 land use, GN level population density data was processed and digitized within the area of district. Suitable landfill site was analysed by using the GIS together with assigned simple sum of weight method (Al-Hanbali et al., 2011). Figure 2 shows the flow diagram of the method used for the study.

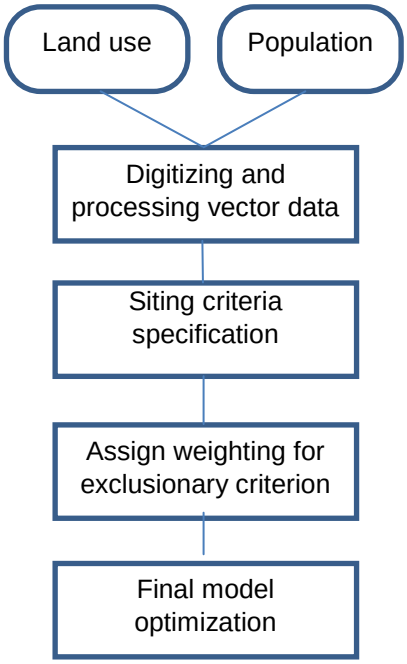

Figure 2 A flow chart illustrating the methodology used in landfill siting.

Table 1 Summarized assign Weights in attributes.

Attribute	Hazard type	Criteria	Weight
Population (W_1)			
Population Density (per km^2)	Social	<400	1
		400 - 800	2
		800 - 1600	3
		>1600	4
Land use (W_2)			
Home Gardens/ Built up Area	Social		4
River/ Water Bodies/ Tank (Working)/ Tank (Abandoned)/ Reservoirs	Environmental		4
Nature reserve/ Forests/ Marsh			3
Paddy	Social		3
Tea/ Other Plantation	-		2
Grassland/ Scrub barren/ Chena	-		1

The layers, the criteria used and there weights are summarized in Table 1 Population data in Kandy district was collected from census and statics department (Statistical report 2012) and population density was calculated foe each GN division. Density higher than 400 per sq km weighted as high risk (weight = 4) area and lower than that weighted as low risk areas (weight = 3, 2 and 1). Low risk areas were considered to be suitable for waste disposal. In the exclusionary criteria of land use map in the area was identified and home gardens, built up area, river, water bodies, tank (working), tank (abandoned), reservoirs were weighted (weight = 4) as high risk area for landfill site selection and other low risk area weighted according to Table 1 (CEA, 2005).

Weighted values were incorporate to each layer according to criteria and analysis was finally optimized by using ArcGIS 9.3 software.

3. RESULTS AND DISCUSSION

Taking into account of the mentioned considerations, the district was screened for preliminary potential landfill sites. Pattern of land use is considered as necessary criteria for the landfill site selection, accordingly the some unsuitable land uses were excluded for landfill site selection as follows; home gardens, built up area, river, water bodies, tank (working), tank (abandoned), reservoirs (CEA, 2005, Environmental Protection Agency, 2006.). Chena, grassland and scrub barren land were considered as very low risk areas for landfill site selection considered 1:50,000 land use map (Figure 3).

Population density of the area was considered as another criterion with representation of demographic attribute. Surround area of the Kandy Four Gravest, all GN divisions has highest density range 1600-4400 person per sq km, higher value compared to the other regions. Therefore, it is considered as high risk region and also the high amount of waste generated area (Figure 4).

Suitability of sites was classified into seven categories (Abessi & Saerdi, 2010, Vogt, 1999), high risk area (in dark red colour), low risk land use and socioeconomically affected area (gradiently red to green colour) and low land use and socioeconomic impacted area (in dark green colour). Potential landfill sites in terms of suitability within the province are illustrated in Figure 5.

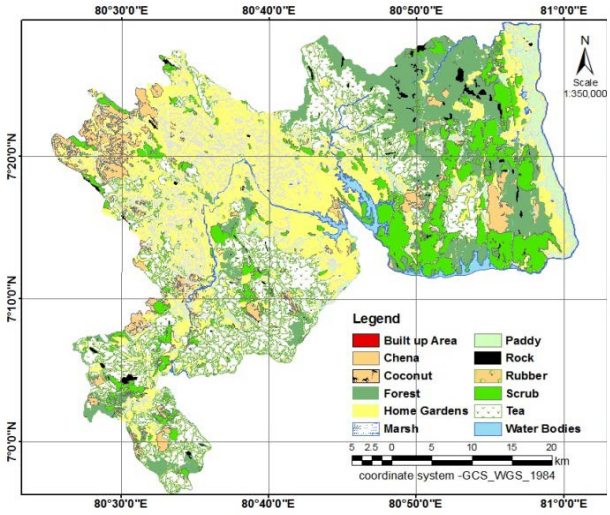

Figure 3 1:50,000 land use map in Kandy district.

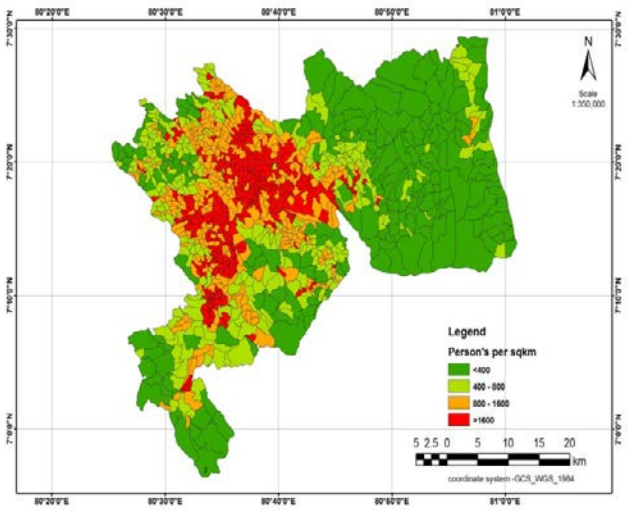

Figure 4 GND level Population density map in Kandy district.

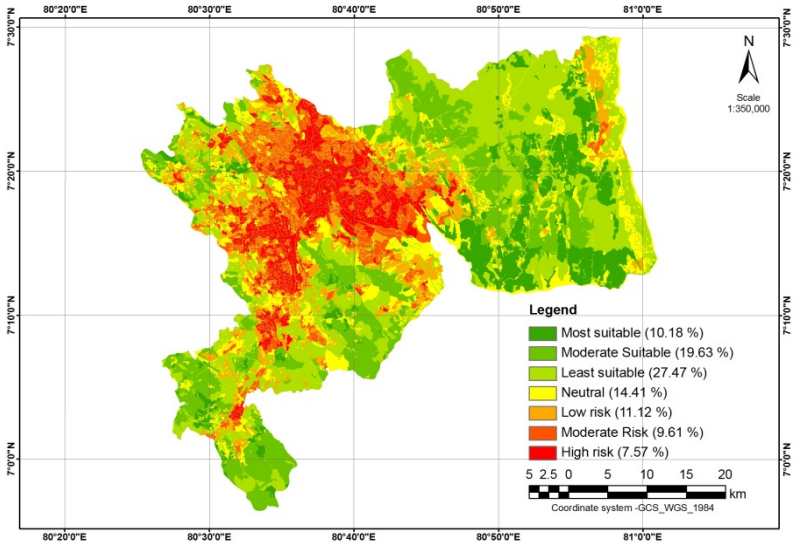

Figure 5 Risk categorization map of Kandy district.

That vector form of the data obtained from GIS analyses indicated that 53.40% of the total land area has a low population density. However 7.56% of this low population density area was excluded due to home gardens, river, water bodies, tank (working), tank (abandoned), reservoirs and 35.20% of was under both environmental and social influenced in order to minimize the human health and socio-economic conditions and risks associated with landfilling.

The rest of these lower population density and lower land use impacted areas (29.80%) were suggested to be suitable and under the above percent 10.18% was most suitable zone (dark green colour) for landfill sitting in the selected area.

4. CONCLUSIONS

According to preliminary consideration of above demographic and environmental attributes, potential amount of land (moderate and most suitable) area 29.80% can be further investigate for site specific landfill selection. The least criteria of selected attributes and without buffer zones also provide significant portion of surrounding under risk area category. Then the sensitivity of this area can be concluded as a quite high with respect to subjected socioeconomic attribute rather than environmental attributes. Further investigation on following areas (most suitable) including hydrogeological pedological and geotechnical investigation can be applicable and useful for prior engineering selection.

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In-situ Thermal Disturbances Affected Groundwater Quality in a Marine Sediment Aquifer (Saitama, Japan)

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Abstract: *Subsurface temperature increase has been observed below several mega-cities worldwide. Subsurface thermal disturbances will likely be more common and pronounced in the future due to combined effects of global warming, urbanization, and increasing utilization of geothermal installations such as ground source heat pump (GSHP) systems for indoor heating and cooling. This could affect the subsurface physical, chemical, and biological environment and may cause changes in groundwater quality. In this study, in-situ thermal heating and natural cooling experiments were performed at a 50-m deep U-tube installation at Saitama University Campus, and the influence of the artificial thermal disturbance on groundwater quality in a marine sediment aquifer was investigated. A significant increase in several chemical components including boron, silicate, potassium, and dissolved organic carbon was observed together with a marked increase in subsurface temperature during 13 months of thermal heating. The chemical component concentrations and subsurface temperature decreased to approximate the original platform levels (before thermal heating) during the subsequent 14 months of natural cooling. Linear relationships between changes in subsurface temperature and changes in groundwater chemical concentrations were found.*

Keywords: *Subsurface thermal disturbance, ground source heat pump (GSHP) system, subsurface temperature*

1. INTRODUCTION

Recently, a temperature increase (thermal disturbance) in subsurface environment has been observed below several mega-cities worldwide. The subsurface thermal disturbance may be strongly associated with combined effects of global warming, urbanization (Harris and Chapman, 1997; Perrier et al., 2005; Taniguchi et al., 2007; Kooi, 2008), and increasing utilization of geothermal installations such as ground source heat pump (GSHP) systems for indoor heating and cooling (Sowers et al., 2006; Haehnlein et al., 2010; Hartog et al., 2013).

For instance, the subsurface thermal disturbance may cause changes in groundwater quality due to changes in solubility, chemical reaction rate, and adsorption-desorption reaction in an aquifer (Bonte et al., 2011; Haehnlein et al., 2013; Saito and Komatsu, 2014). However, there are few studies regarding in-situ, field-scale experiment, and the influence of the subsurface thermal disturbance especially on groundwater quality is not understood sufficiently.

Therefore, in this study, in-situ thermal heating and natural cooling experiments were performed at a 50-m deep U-tube installation at Saitama University Campus, and the influence of the artificial thermal disturbance on groundwater quality in a marine aquifer was investigated.

2. MATERIALS AND METHODS

At the established experimental site in Saitama University Campus in the Arakawa Lowland near the Tokyo metropolitan area of Japan, approximately 40°C hot water was circulated inside a 50-m deep U-tube type heat exchanger as an experimental GSHP system for 13 months (Figure 1). After this thermal heating test, the experimental site was continuously under natural cooling condition for 14 months.

The subsurface temperature and groundwater quality were sequentially monitored during both the thermal heating and natural cooling tests at installed four monitoring wells at 1 m (W1), 2 m (W2), 5 m (W5), and 10 m (W10) distance from the U-tube for two confined aquifers (around 17 m and 39 m depths) (Figure 1). Since these aquifers deposited in different sedimentary environments (marine and non-marine), they have completely different hydrogeological properties.

Field measurements of pH, electric conductivity (EC), dissolved oxygen (DO), and oxidation-reduction potential (ORP) were carried out using portable analyzers. Hydrogen sulfide (H_2S) was also measured using PACKTEST (KYORITSU CHEMICAL CHECK Lab., Corp.), which is based on a simple colorimetric method. In the laboratory, major ions (Na^+, NH_4^+, K^+, Mg^{2+}, Ca^{2+}, Cl^-, NO_2^-, NO_3^-, SO_4^{2-}) dissolved inorganic carbon and organic carbon (DIC and DOC), heavy metals and trace elements (Li, B, Al, Si, Cr, Mn, Fe, Ni, Cu, Zn, As, Sr, Cd, Sb, Pb), and dissolved methane (CH_4) were measured using Ion Chromatograph, TOC analyzer, ICP-MS, and GC-FID (with head space method), respectively.

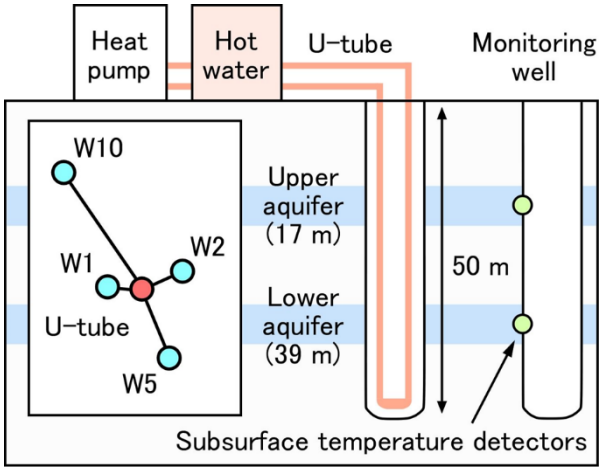

Figure 1 Experimental setup established in Saitama University Campus in the Arakawa Lowland near the Tokyo metropolitan area of Japan

3. RESULTS AND DISCUSSION

The temporal variations of the subsurface temperature observed at four monitoring wells in both two aquifers are presented in Figure 2. The increase in the subsurface temperature was more significant in nearer monitoring well during the 13 months of thermal heating, showing finally 7.2°C and 7.8°C increase in the upper and lower aquifers, respectively. After this heating period, the increased subsurface temperature decreased to approximate the original platform levels (before thermal heating) under natural cooling condition. Especially for W10, there was almost no significant change in the subsurface temperature throughout these field experiments, in case of comparing to other three monitoring wells (W1, W2, and W5). Therefore, this W10 well is assumed to be a reference monitoring well in this study site.

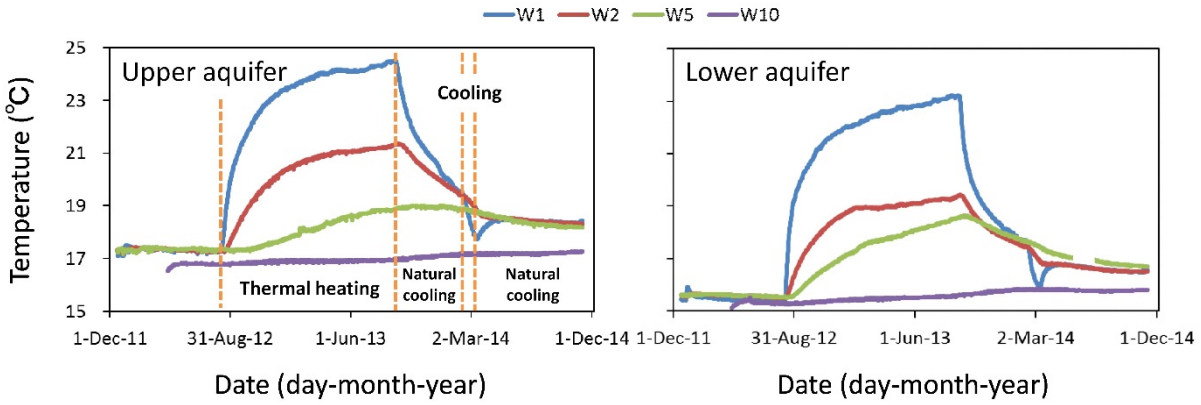

Figure 2 Temporal variations of subsurface temperature observed at four monitoring wells (W1, W2, W5, and W10) in the upper and lower aquifers during first term thermal heating and natural cooling tests

During the thermal heating test, for example, boron concentration in groundwater taken from the upper marine aquifer increased markedly at W1 together with the marked increase in the subsurface temperature (Figure 3a). After the 13 months heating period, the increased boron concentration decreased to the original platform levels together with the decrease in the subsurface temperature at W1 monitoring well. Other groundwater chemical components such as silicate, potassium, and dissolved organic carbon showed also increases in their concentrations. On the other hand, it seemed to show no significant changes in field measurement parameters (pH, EC, DO, and ORP) as well as dissolved gases (H_2S and CH_4) in the upper marine aquifer.

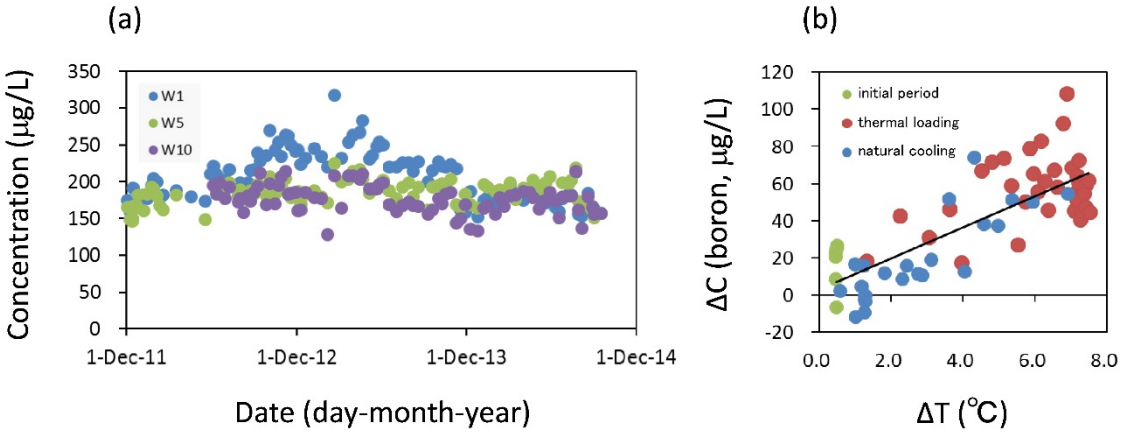

Figure 3 (a) Temporal variations of boron concentration at the monitoring wells W1, W5, and W10 in the upper aquifer, (b) ΔT and ΔC diagram analyzed for boron in the upper aquifer

Based on these observed results, the relationships between changes in the subsurface temperature (ΔT) and the chemical concentrations in groundwater (ΔC) were analyzed. This analysis was performed by subtracting the data at W10 as a reference monitoring well from the data at W1 as a most thermally-disturbed well for these two continuous thermal disturbance tests. The groundwater chemical components that show changes in their concentrations demonstrated linear relations, as shown in figure 3b as an example of ΔT and ΔC diagram analyzed for boron in the upper marine aquifer. On the other hand, it was difficult to conclude the influence of the thermal disturbance in the lower non-marine aquifer in this study.

The second term in-situ thermal heating experiment is now being performed by circulation of approximately 60°C hot water inside the 50-m deep U-tube heat exchanger to make clear the

influence on the groundwater quality of the lower non-marine aquifer. The temporal variations of the subsurface temperature observed at four monitoring wells in both two investigated aquifers during second term thermal heating are illustrated in Figure 4. Especially at the monitoring well W1, the subsurface temperature is increasing markedly, and the change in the subsurface temperature is more significant, as compared to first term field experiment. The subsurface temperature is further increasing and the monitoring for groundwater quality is being continued.

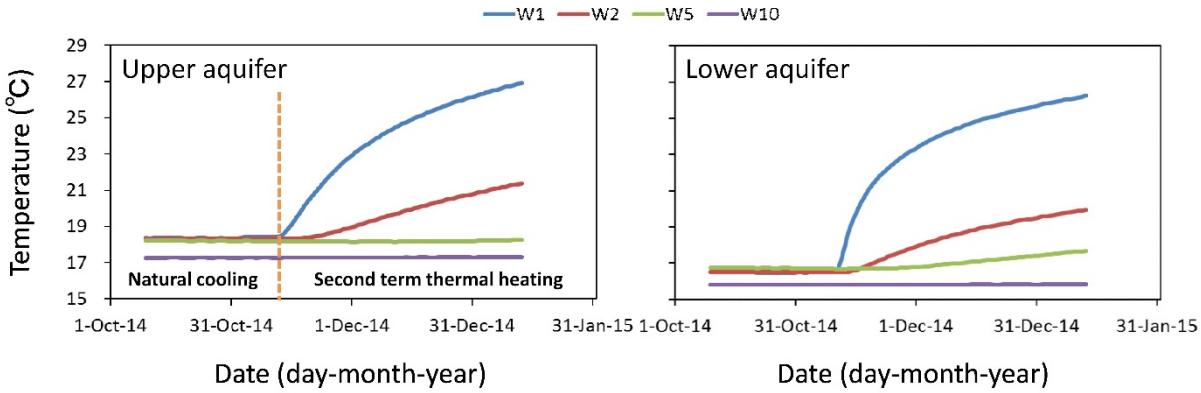

Figure 4 Temporal variations of subsurface temperature observed at four monitoring wells in the upper and lower aquifers during ongoing second term thermal heating test

4. CONCLUSIONS

At an established experimental site in Saitama University Campus in the Arakawa Lowland near the Tokyo metropolitan area of Japan, subsurface temperature and groundwater quality were simultaneously monitored in a marine sediment aquifer during in-situ thermal heating (13 months) and subsequent natural cooling (14 months) periods. A significant increase in several groundwater chemical components including boron, silicate, potassium, and dissolved organic carbon was observed together with a marked increase in subsurface temperature during 13 months of thermal heating. The chemical component concentrations and subsurface temperature decreased to approximate the original platform levels (before thermal heating) during the subsequent 14 months of natural cooling. Linear relationships between changes in subsurface temperature and changes in groundwater chemical concentrations were found.

5. ACKNOWLEDGMENTS

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Adsorption Characterization of Pb, Cu and Ni and Municipal Solid Waste from an Open Dump Site, Sri Lanka

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Abstract: *Solid waste open dumpsites are a source for groundwater and surface water contamination with heavy metals. The heavy metal transport and mobility has to be monitored and predicted to minimize the impact. The adsorption capacity and retardation factor are important parameters in contaminant transport calculations. Kinetic and isotherm studies were conducted for Pb, Cu and Ni for two different age waste materials in Udapalatha, Central province, Sri Lanka. The waste was characterized prior to adsorption experiments and high organic carbon content was found in both old and new dump as 11.84 and 21.21%, respectively. Higher adsorption was found in old waste for all heavy metal in kinetic studies. The adsorption isotherm studies carried out at 25 ^{0}C, 100 rpm for Pb, Cu, and Ni separately for both waste samples. Freundlich isotherm model was best fitted all three for both waste samples compared to linear and Langmuir model as the waste has higher organic carbon content. The adsorption constant for Pb in Freundlich model (K_f) for old and new dump were 2.74 and 0.94 mgg^{-1}, respectively. The K_f for Cu for waste in old and new dump section were 1.31 and 2.17 mgg^{-1}, respectively. The adsorptive capacity for Ni in Freundlich was 0.47 and 0.23 mgg^{-1} with new and old dump section, respectively. These isotherm constants can be used for the prediction of contaminant transport in similar dump sites.*

Key words: *adsorption, contaminant transport, heavy metals, isotherm studies, solid waste.*

1. INTRODUCTION

Uncontrolled open dumping of municipal solid waste is a common practice in most of the South Asian countries including Sri Lanka. This creates serious impacts to the environments particularly to the surface and groundwater resources. Leachate, the liquid drains from the waste layer has several contaminants including heavy metals. Heavy metal toxicity of leachate has been reported by several researches with higher concentrations of Pb, Zn, Cu, Ni, Mn, Cd and Hg (Joseph*et al* 2005). Paints, lamp filaments, electrical items, batteries, ceramics, automotive parts and alloys are some of the sources of heavy metals in dumpsite(Fetter 1993). Heavy metals can accumulate within the food

chains through contaminated drinking water and cause several health problems such as cancers (Tunali *et al* 2006).

Characterization of adsorption of material may useful for the analysis of contaminant transport at the dump sites, since leaching of heavy metals depends on the adsorption characteristics of the waste. Adsorption/desorption is one of the key process controlling contaminant migration to the groundwater.

Contaminant transport models are used for identification of plume. These models have been developed with several geochemical parameters which are site specific. Distribution coefficient (K_d) and contaminant retardation factor (R) are some of the site specific geochemical constants used in contaminant transport models. The K_d is a function of aqueous and solid phase chemistry (Kaplan and Serne, 1995).This research was conducted to determine the K_d and Rof *Pb, Cu, and Ni* with decomposed waste in order to characterize the adsorption. This can be used to model the contaminant transport in landfill sites in Sri Lanka.

2. MATERIALS AND METHODS

2.1 Waste Sample Collection and Preparation

Decomposed waste samples were collected from a dumpsite, located in the right bank of the Mahaweli river in Udapalatha (070 80' 30.1" N and 80^0 34' 43.2 E), Wet Zone, Central Province of Sri Lanka. The site was used to dump municipal solid wastes for seven years of period. It has two sections, old and new with the age of 7 and 0.5 years, respectively. The dumpsite was abandoned in 2011.

Waste samples were collected on 2nd December 2014 from old and new sections, separately up to 60 cm using Auger. Samples were mixed, air dried, and sieved with 2 mm sieves for adsorption studies. Core sampler was used to collect undisturbed waste sample.

2.2 Sample Characterization

Electrical conductivity (EC) and pH of waste samples were measured in 1:10 suspensions of samples-to-water. The suspension was agitated for 4 h at 100 rpm, filtered through Whatman 42 (capability in capturing particles more than 2.5 microns) and measured EC and pH using the EC meter (EC5 HACH) and pH meter (pH1 HACH), respectively. Bulk density and moisture content of the samples were determined using core samples.

In addition, Total Organic Carbon (TOC)and Total Soluble Nitrogen (TN) were measured for filtrate using TOC analyzer (TOC-LCSH/TNM-L Shimadzu Corporation). Total Carbon (TC) was measured for sieved waste samples using organic elemental analyzer (FLASH 2000N+MAS200R Thermo fisher).

2.3 Characterization of Adsorption

2.3.1 Kinetic studies

Kinetic studies were carried out, in order to determine the time required for sorption equilibrium. The stock solutions of Pb, Cu, and Ni were prepared using $Pb(NO_3)_2$, $Cu(NO_3)_2$, and $Ni(SO_4)_2$. The waste samples were homogenized with distilled water at the ratio of 0.2 g for 100 ml. Heavy metal stock solutions(Pb and Cu) were spiked into homogenized solutions to make the initial concentration to 100 ppm. Low initial concentration of Ni (6 ppm) solution was prepared since the reported concentration of Ni is low in leachate (Esakku *et al.,* 2003).Solution were drawn at known time intervals (5 min – 120 min) and the supernatant was transferred for membrane filtration (0.45 µm), analyzed for heavy metals (Pb, Cu, and Ni) with Atomic Absorbance Spectrophotometer (AAS- AA-7000 Shimadzu corporation).

2.3.2 Isotherm studies

The study was done separately for each heavy metal with the initial concentrations between 10- 100 ppm for Pb and Cu and 0.6 – 6 ppm for Ni. Batch experiments were carried out with 2 g L^{-1} waste material. Suspensions of 2 mm size waste materials were stirred for 3 h at a speed of 100 rpm to hydrate the surface of material. After centrifugation, the supernatant was transferred by membrane filtration (0.45 µm) for heavy metal analysis. Detection of Pb, Cu and Ni were carried out by AAS.

Three models were used for the calculation of K_d for different heavy metal. The best fitted models were selected with respect to minimum chi squired values and highest correlation coefficient using Origin software (version 6).

Linear Model(EPA 1999)

$$Q_{(ads)} = K_d * C_{(e)}$$

Where; $Q_{(ads)}$ = amount adsorb per g of adsorbent (mg g^{-1})
$C_{(e)}$ = heavy metal concentration in aqueous(mg L^{-1}) at equlibirium
K_d = distribution coefficient(Lg^{-1})

Freundlich Isotherm model(Ulmanu *et al* 2003)

$$Q_e = K_f\, Ce^{1/n}$$

Where, Q_e = amount of metal ions adsorbed per gram of sorbents at equilibrium (mg g^{-1})
C_e = heavy metal concentration in aqueous (mg L^{-1})at equilibrium
K_f = constant related to adsorptive capacity (mg g^{-1})
n = intensity of adsorption (L mg^{-1})

Langmuir Isotherm model(Ajmal *et al* 2003)

$$q_e = \underline{q_m b C_e}/(1 + bCe)$$

Where, q_e = amount of metal ions adsorbed per gram of sorbents at equilibrium (mg g^{-1})
C_e = heavy metal concentration in aqueous (mg L^{-1})at equilibrium
q_m = maximum adsorption capacity (mg g^{-1})
b = Langmuir constants related to the energy of adsorption (L mg^{-1}).

2.3.3 Contaminant retardation factor

The R is determined for each heavy metal, separately using the equations below with respect to the model. Each equilibrium concentration was used for Freundlich and Langmuir models to determine the R (EPA 1999).

Linear isotherm model $R = 1 + \underline{p_d k_d}/\Theta$

Freundlich isotherm model $R = 1 + \underline{p_d} n k_f C_e^{a-1}/\Theta$

Langmuir isotherm model $R = 1 + \underline{p_d [b q_m]}/\Theta\, [(1 + kC_e)^2]$

Where,
R = contaminant retardation factor
p_d = bulk density (kgm^{-3})
k_d = distribution coefficient (mlg^{-1})

Θ = moisture content (%)
n = intensity of adsorption (L mg^{-1})
C_e = equilibrium aqeuous métal ion concentration (mg L^{-1})
b = Langmuir constants related to the energy of adsorption (L mg^{-1})
K_f = freundlich constant related to adsorptive capacity (mg g^{-1})

3. RESULTS AND DISCUSSION

3.1 Characterization of the Waste Material.

The isotherm constants and retardation factor vary with wastes and site-specific conditions. The use of generic or default K_d values in models can result uncertainty in prediction of impacts of contaminant migration or site-remediation options. Table 1 shows the EC, pH, moisture content, bulk density, TOC, TC and TN of the waste material collected from the old and new sections. TOC, TN and TC are high at new section compared to old. Bulk density and EC were higher in the waste in old section. These variations may be due to the aging of the dump site.

Table 1Characterizationof waste in old and new sections.

Sampling point	EC (mS/m)	pH	Moisture content (wet basis %)	Bulk density (g/cm^3)	TOC (mgL^{-1})	TN (mgL^{-1})	TC (%)
Old section	54.84	5.93	39.8	0.77	4.02	102	11.8
New section	37.53	6.89	31.8	0.73	5.20	129	21.2

3.1 Evaluation of experimental data

3.1.1 Kinetic studies

Figure 1 a, b and c show the adsorbed Pb, Cu and Ni with time for both samples new section and old section. The capacity of adsorption is higher in sample in old section than new section for all three heavy metals, The Pb adsorption at 2 h was52.7% in old dump samples. The adsorption of Cu increased at the beginning and reduces afterwards. The maximum adsorption for Cu in new and old dump were 15.5% and 26.1%, respectively while it was 49.8 % for Ni in both samples. The kinetic experiment results were used for isotherms studies.

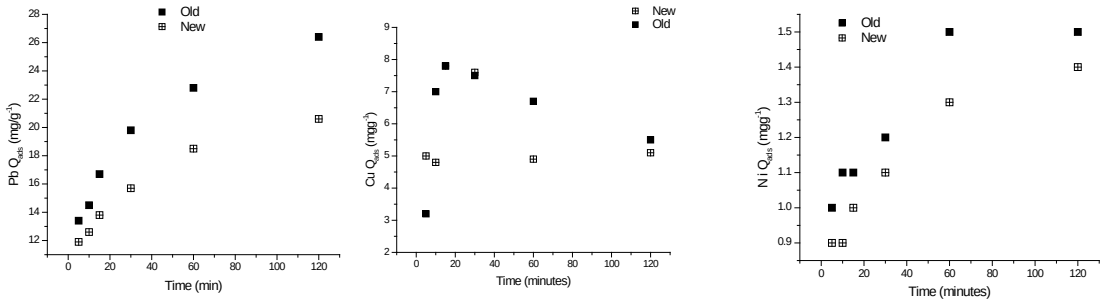


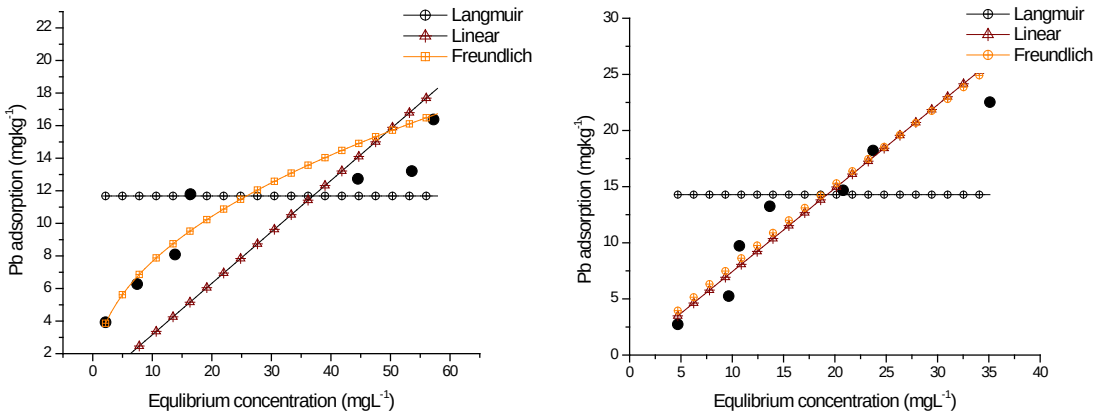
Figure 1 Adsorption of (a) Pb (b) Cu and (c) Ni with time for samples from old and new section

3.1.2 Isotherms

Linear, Freundlich and Langmuir isotherms model were fitted using the experimental results and shown in Figure 2, 3 and 4 for Pb, Cu and Ni, respectively. The adsorption isotherm parameters with the correlation coefficients and chi squared values for Pb, Cu and Ni for waste from old and new were shown in table 2.

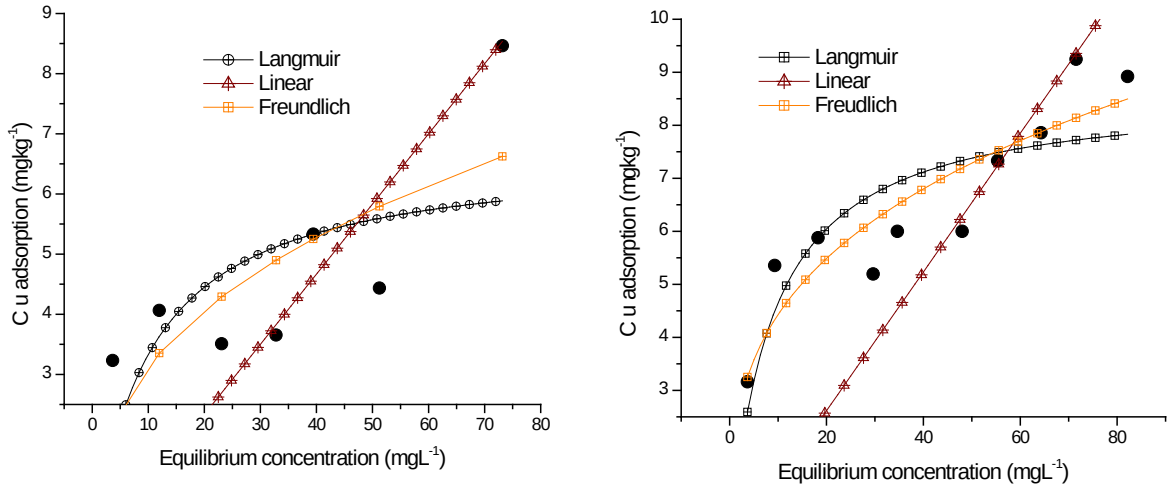
The best fitted model for all 3 heavy metal in this study was Freundlich model. According to Haghseresht(1998), the heterogeneous systems with high organic carbon and high interactive materials fit with the Freundlich isotherm model. The TC of both waste samples used in this study were 11.8 and 21.2 % in old and new sections, respectively. Thus Freundlich isotherm model can be used for characterization of adsorption of heavy metal with waste.

Adsorption mechanisms very with respect to heavy metal and waste. The adsorption mechanism of Ni in waste cannot be specified since all three isotherm models are fitted to the experimental results. The lowest correlation was found in Langmuir isotherm model with Pb thus may not suitable to explain Pb adsorption.



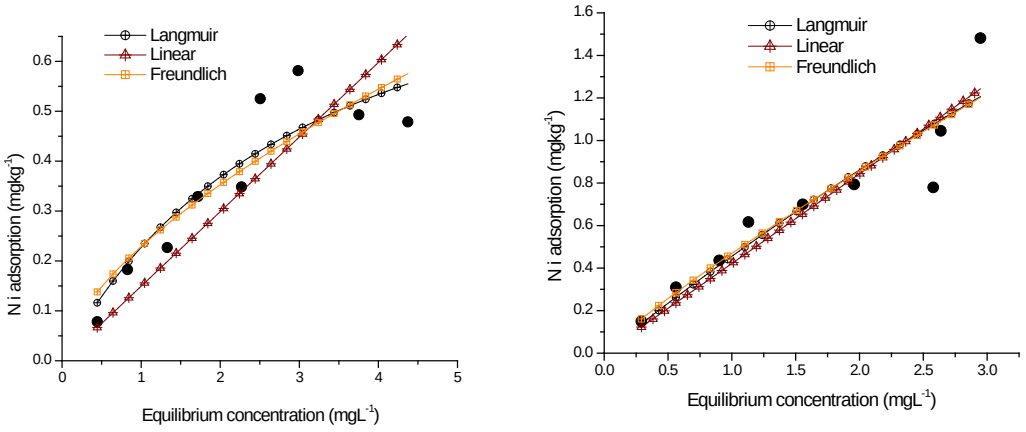
(a) **(b)**

Figure 2 Langmuir, Freundlich, Linear isotherm model (a) Old dump, (b) New dump for Pb adsorption



(a) **(b)**

Figure 3 Langmuir, Freundlich, Linear isotherm model (a) Old dump, (b) New dump for Cu adsorption



(a) **(b)**

Figure 4 Langmuir, Freundlich, Linear isotherm model (a) Old dump, (b) New dump for Ni adsorption

As shown in Table 2, the adsorption capacity, K_d, K_f and q_m varied with waste sample collected from old and new section. The waste characterization also shows that these waste samples are different with age.

The K_d varied from 0.11 to 0.74 for all the heavy metal and sites tested. The K_f for Pb in Freundlich model for old and new dump were 2.74 and 0.94 mgg^{-1}, respectively. The oldsamples showed higher adsorption capacity than that of new. As the Freundlich isotherm model explains, Pb may bind to waste material in heterogeneously and as multilayer.

The $K_f$for Cu for waste in old and new dump section were 1.31 and 2.17 mgg^{-1}, respectively. The Cu solubility in soil was mainly governed by Dissolved Organic Carbon (DOC) concentration rather than other physical parameters(Schmidt 2003). When the soil pH is 6.5–8, Cu forms the strongest complex with dissolved organic carbon, facilitating the transport of Cu in the soil (Cao*et al* 2002).However the lower pH of the old dumpsite may be a reason for the lower adsorption capacity in old section than the new.

For the Ni, the Langmuir constant was 0.3 Lmg^{-1} and the maximum adsorption is 0.96 mgg^{-1} in old dump section. According to the Freundlich model, the adsorptive capacity is 0.47 mgg^{-1} with the new dump section.

Table 2 Isotherm model parameters for Pb, Cu and Ni onto waste material

Metal	Sampling point	Langmuir				Freundlich			Linear			
		R^2	X^2	b (Lmg^{-1})	q_{max} (mgg^{-1})	R^2	X^2	Kf (mgg^{-1})	n	R^2	X^2	K_d
Pb	Old dump	-1.3	35.9	-7.5	11.6	0.82	6.31	2.74	2.20	0.49	15.6	0.31
	New dump	-6.6	72.5	3.56	14.3	0.93	5.92	0.94	1.07	0.92	5.3	0.74
Cu	Old dump	0.30	2.75	0.09	6.69	0.54	1.81	1.31	2.65	0.05	3.1	0.11
	New dump	0.69	1.19	0.12	8.65	0.82	0.68	2.17	3.23	-0.42	4.9	0.13
Ni	Old dump	0.83	0.00	0.30	0.96	0.79	0.00	0.23	1.59	0.63	0.01	0.14
	New dump	0.85	0.02	0.06	7.86	0.86	0.02	0.47	1.14	0.85	0.02	0.42

4. CONCLUSIONS

Waste samples from old and new section vary with TOC, TC, TN, EC and pH. Higher organic carbon was measured in old section. The adsorption of heavy metals was higher in waste from old section. The time required for maximum adsorption varied from 20 – 60 min by the kinetic study. Freundlich model was the best fitted for all three heavy metal tested as the waste material has higher organic content. The adsorptive capacities of Pb and Cu onto old and new dump were, 2.74 and 0.94 $mgg^{-1}$and 1.31 and 2.17 mgg^{-1}, respectively. The adsorptive capacity of Ni onto new dump and old were 0.47 and 0.23mgg^{-1} with freundlich model. The calculated isotherm constants can be used for the prediction of contaminant transport in similar dump sites.

5. ACKNOWLEDGEMENT

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Development of a Landfill Site Selection Model Using GIS Integrated with Analytic Hierarchy Process (AHP): A Case Study for the Hambantota District of Sri Lanka

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Abstract: *At present, a considerable amount of solid waste is disposed in landfills because of the accelerated increase of solid waste generation with the increase of the human population. A systematic process should be followed to ensure that an appropriate site is selected for landfilling. In this study, a model was developed using the geographical information system (GIS) to identify the potential landfilling sites considering seventeen sub-criteria under environmental, physical and social, and economic main criteria. Distance from water supply sources, surface water courses, natural reserves, coastal belts and wetlands; soil type; rainfall pattern; land use pattern; terrain and slope were sub-criteria under environmental main-criteria. Distance from important buildings, culturally, historically and archeologically interested places, tourist areas, town and village centers, air ports and roads are physical and social sub-criteria. Distance from roads and slope are economic sub-criteria. A scale was assigned to each of main and sub-criteria based on the relative importance among criteria for siting landfills. This scale eases the pairwise comparison used to obtain weights for sub and main criteria according to the Analytic Hierarchy Process (AHP). A GIS model was developed using the model builder tool of Arc GIS 10 incorporating the weight assigned for each sub/main criterion. The model can be applied to any area for landfill siting if the feature maps of sub criteria are available. Using the model, a map of Hambantota district illustrating the unsuitable, least and less suitable, and suitable landfilling sites was developed. Suitable areas for landfill siting mainly lie within Lunugamwehera, Tissamaharama, Hambantota and Suriyawewa divisional secretariats.*

Keywords: *Land filling, GIS, AHP, Sub-criteria, Main -criteria*

1. INTRODUCTION

Shortage of lands for waste disposal and inappropriate landfill siting are major issues faced by many urban areas all over the world. Most of the landfill sites in the world are selected randomly, and waste is burnt without treating. This affects adversely on the biophysical environment and the ecology of the surrounding area [Siddiqui et al (1996); Kontos et al (2003); and Erkut & Moran (1991)]. Though landfilling is the least preferable approach in the solid waste management hierarchy, it is unavoidable because zero waste generation is an ideal situation.Therefore selecting appropriate sites for land filling is a major issue in both developed and developing countries. Not only that, development of a model for identifying suitable landfilling areas is timely and topical. Selecting appropriate landfill sites as well as evaluating the locations of existing landfill sites are imperative to prevent negative impacts on the public as well as the environment. Landfill siting is a complex process involving the processing of massive amount of spatial data. It is evident that many factors must be incorporated into landfill siting decisions. Geographic information systems (GIS) are ideal for this kind of preliminary studies due to their ability to manage large volumes of spatial data from variety of sources (Sener et al, 2006).

GIS is an efficient and a latest tool that is integrated with computerized software tools, mathematical and statistical methods for efficient and effective site analyses. GIS has the capability to handle and simulate the necessary economic, environmental, social, technical, and political constraints.

Selection of appropriate landfill sites depends on several factors such as land use pattern, transportation network, environment, hydrology etc. A landfill must be situated and designed so as to meet the necessary conditions to prevent soil, groundwater and surface water pollution and to ensure efficient collection of leachate. A landfill site should also be kept as far as possible away from residential areas. On the other hand, the landfill site should be placed as close as possible to existing roads to reduce collection expenses. Furthermore a site with a slope either too steep or too flat is not appropriate for constructing a landfill from economic viewpoint (Nas et al, 2010). The degree of contribution of each factor to the landfill site selection is different. The Analytic Hierarchy Process (AHP) is a multi-criteria decision making technique that helps decision makers to set the priorities and make the best decision by reducing complex evaluations to a series of pair wise comparisons. It is a theory of measurement through pairwise comparisons, and it relies on the judgments of experts to derive priority scales (Saaty, 2008).Therefore a methodology that integrates GIS and AHP will be a powerful tool to solve the landfill site selection problem, because GIS provides an efficient manipulating and presenting method of data and AHP provides weights for the landfill siting criteria based on the relative importance of the criteria.

The aim of this study was to develop a GIS model for selecting solid waste landfilling sites, which would comply with local and international environmental protection regulations. The study included the identification of the factors, constraints and parameters that impact on the suitability of landfill sites, followed by an initial screening of suitable sites using the GIS software; development of a GIS model by excluding areas unsuitable and weighting residual areas; and selection of appropriate sites for land filling in the Hambantota district using the developed model.

2. MATERIALS AND METHODS

2.1. Overall Procedure

Feature maps to develop the maps for each criterion were obtained from the Survey Department. ArcGIS 10.0 software was used for this study. In developing the GIS model, initially main and sub criteria for landfill site selection were identified. Then a ‘feature map’ was developed for each identified sub-criterion. Each ‘sub-criterion feature map’ was classified into four zones as ‘suitable’, ‘less suitable’, ‘least suitable’ and ‘unsuitable’ using the ‘Multiple Ring Buffering’ tool. ‘Raster’ is suitable for the representation of continuously transitional phenomena. Therefore each ‘buffered sub-criterion feature map’ was converted into the raster format. Different number systems cannot directly be combined effectively. Because of the potential different ranges of values and the different types of numbering systems each input layer (criterion) may have, each must be reclassified to a common ratio scale prior to the multiple factors being combined for the analysis. Therefore all ‘sub-criterion raster maps’ were brought into a common ratio scale using ‘Reclassify’ tool. Weights were assigned based on the relative importance of all selected criteria for landfill siting. Using the ‘Raster Calculator’ tool and weights assigned for sub-criteria, all ‘reclassified sub-criterion raster maps’ under each main criterion were overlaid to derive an ‘overlaid map’ for each main criterion. Similarly the final maps of suitable, less suitable, least suitable and unsuitable areas for landfilling were derived overlaying all main criterion maps. Finally suitable sites for landfilling in the Hambantota district were found using the developed model.

2.2. Identification of Landfill Site Selection Criteria

The landfill site selection criteria were in accordance with the national and international guidelines and research articles on the same nature. When selecting landfilling criteria, guidelines published by the Environmental Protection Authority (EPA) (EPA, 2006), technical guidelines on solid waste management published by the CEA and research articles on the same nature [Jayawickrama &

Weerasinghe (2012); Monriya et al (2010); Babalola and Basu (2011); and citations mentioned in Table 1] were referred.Environmental, social and physical, and economical criteria were the selected main-criteria. In this study, seventeen sub-criteria were selected under three main criteria. Distance from water supply sources, surface water courses, natural reserves, wetlands and coastal areas; soil type; rainfall pattern; land use pattern; terrain and slope were environmental sub-criteria. Distance from important buildings, culturally, historically and archeologically interested places, tourist areas, town and village centers, airports and roads are sub-criteria under physical and social aspects. Distance from roads and slope are economic sub-criteria. Table 1 depicts the suitability and unsuitability buffering criteria for the selected sub-criteria.

Table 1 Suitability and unsuitability buffering criteria

Main and Sub Criteria	Unsuitability Criteria (Exclusionary Areas)	Suitability Criteria		
		Least-Suitable Areas	Less Suitable Areas	Suitable Areas
Environmental criteria				
Distance from surface water courses	< 500 m (Josimovik & Maric,2012)	500 -1000	1000-1500	> 1500
Distance from water supply sources	< 500 m (Josimovik & Maric,2012)	500-750	750-1000	> 1000
Distance from natural reserves	< 500 m (Kara & Dorati,2012)	500-750	750-1000	> 1000
Soil type	Karstified limestone, Sandy soil; Alluvial soil; Loess(collapsing soils);Incoherent rock mass; Alluvial plains and Karst terrains [Monriya et al (2010); Jayawickrama & Weerasinghe (2012); Josimovik & Maric (2012)]	Complex of incoherent and semi-cohorent rock ass (Deluvial sediments);Semi-cohorent rocks	Loamy soil; Cohorent rock, Slightly stoned rocks	Clayey soil; Solid rocks
Land use pattern	Parks and scenic areas, Agricultural lands;Orchads olives, citrus and carob trees),Forest (needles), Forest (eucalyptus); Cultivated agricultural land (ploughland,orchards),Sport grounds; Quality tall forests (Joisovic & Maric,2012)	Forest scrub; Irrigated agriculture	Forests; Dry-pasture, Ag-bare agriculture; Pastures, Shrub woods	Grass land; Uncultivated lands, Thickets, Barren land, Areas degraded by past quarry activities and excavations
Distance from the coastal belt	< 300 m (Jayawickrama & Weerasinghe, 2012)	300-600	600-1000	> 1000
Distance from wetlands	< 1000 m (Kara & Dorati,2012)	1000-1250	1250-1500	> 1500
Terrain	> 300 m(height)	200-300 m	100-200 m	0-100 m
Slope	> 20%	12-20 %	0-12%	8-12%
Rainfall	> 650 mm (Babalola & Basu,2011)	500 -650 mm	250-500 mm	0-250 mm
Physical and social criteria				
Distance from important buildings	< 500 m [Josimovik & Maric (2012);Lin & Kao (2005);and Nas et al (2010]	500-750	750-1000	> 1000
Distance from culturally, historically and archeologically interested places	< 500 m (Monriya et al,2010)	500-750	750-1000	> 1000
Distance from tourist areas	< 500 m (Monriya et al,2010)	500-750	750-1000	> 1000
Distance from air ports	< 3000 m (Alfy et al,2010)	3000-3500	3500-4000	> 4000
Distance from town and village centers	< 500 m	500-750	750-1000	> 1000
Distance from roads	< 1000 m (Monriya et al,2010),	500-750	750-1000	> 1000
Economic criteria				
Distance from roads	> 10 000 m (Nas et al,2010)	5000-10 000	1000-5000	250 - 1000
Slope	> 20 % (Kara & Dorati,2012)	12-20 %	0-12%	8-12%

2.3. Weights Calculation

Initially each main/sub criterion was given a scale according to how much important each criterion for siting landfills is over the other criterion/criteria. The scaling method used in this study was similar to the nine point scaling method described in Saaty (2008). The nine-point scale means scales of 1 to 9 which means its level of importance varies from equal importance to the extreme importance, respectively. According to the given scales, all sub/main criteria could be arranged in a hierarchical order of importance for landfill siting. Then each main/sub criterion was given a weight using the pairwise comparison method described in the Analytic Hierarchy Process (AHP). The pairwise comparison was aided by the scale assigned for each main/sub criterion. The weights were calculated after constructing a pairwise comparison matrix. Generally a pairwise comparison matrix is constructed based on comparisons decided by a decision maker. This matrix indicates the relative importance of each criterion over the other criteria toward landfilling. After constructing the pairwise comparison matrix, weight estimation was done using the AHP and it was used to derive a consistency ratio (CR) of the pairwise comparisons. If CR > 0.10, then some pairwise values need to be reconsidered and the process is repeated until the desired value of CR < 0.10 is reached. Table 2 exhibits the scales and weights assigned for main and sub-criteria.

Table 2 Weights for main and sub-criteria

Main and Sub Criteria	Scale	Weights
Environmental criteria	**1**	**0.54**
Distance from water supply sources	7	0.17
Distance from surface water courses	6	0.26
Distance from natural reserves	4	0.05
Soil type	5	0.09
Distance from wetlands	4	0.05
Land use pattern	4	0.05
Distance from coastal belt	4	0.05
Terrain	5	0.09
Rainfall	5	0.09
Slope	5	0.09
Social and physical criteria	**2**	**0.30**
Distance from important building	3	0.25
Distance from culturally, historically and archeologically interested places	3	0.26
Distance from air ports	3	0.25
Distance from town and village centers	2	0.09
Distance from tourist areas	2	0.09
Distance from roads	1	0.06
Economic criteria	**3**	**0.16**
Distance from roads	1	0.20
Slope	2	0.80

3. RESULTS AND DISCUSSION

Brito et al (2003) stated that future landfill site selection procedures need to be conducted within a framework designed to achieve the following objectives: to ensure that the most environmentally and economically suitable site is selected, and to engender a public consensus on the necessity of the landfill and that the site selection process has taken into account all relevant considerations. In order to ensure the environmental feasibility, technical criteria such as impacts on humans, flora, fauna, soil, water, air, climate and landscape should be considered (Brito et al, 2003). All these objectives can be achieved by the developed model because environmental, socio and cultural, and economic are its main criteria. The selected 10 sub-criteria under the environmental main-criterion address the most of the impacts likely to arise on above mentioned animate and inanimate features of the environment. In the developed model, the highest and lowest weights of the main criteria are owned by the environmental and economic criteria, respectively. Among the sub-criteria, the first and second highest weights belong to the criteria, ‘distance from water supply sources’ and ‘distance from surface water bodies’, respectively. The allocation of weights for sub/main criteria was done systematically so as to

be quite justifiable from the viewpoints of all stakeholders involving in the landfill site selection activity. The weight represents the degree of importance of the criterion in the landfill site selection. The importance of these criteria toward landfilling is based on the adverse impacts likely to be caused by violating the regulations associated with the criterion concerned. The importance is also based on how much the cost of construction, maintenance and operation of a landfill varies by altering the criterion concerned. The relative importance of each criterion to another was derived from a scale assigned for each sub-criterion. To make comparisons, it is essential to have a number scale that indicates how many times more important or dominant one element is over another element with respect to the criterion or property with respect to which they are compared (Saaty, 2008). When each sub/main criterion was assigned the number scale, the number scales assigned for the same criterion in the literature as well as the importance of the criterion toward landfilling based on the literature, were considered.

Figure 1, 2 and 3 show the maps of Hambantota district illustrating the unsuitable, less and least suitable and suitable sites for landfilling based only on environmental, socio and cultural and economic criteria, respectively. The distance from the waste generators is one of the important economic criteria. However the buffering distance for exclusionary areas for landfilling in terms of this criterion is about 10 000 km (Nas et al, 2010). Since the distance is a quite high value compared to the other buffering distances, the value was not compatible with the model. Hence the above criterion was not included in the developed model. This indicates the limitation of the GIS application. Figure 4 illustrates the map of Hambantota district illustrating the unsuitable, less suitable and least suitable and suitable areas for landfill siting. Unsuitable areas for landfilling lie in Tangalle, Katuwana, Okewela and Ambalantota divisional secretariats.Weeraketiya divisional secretariat has the least suitable areas while Angunukolapelessa and Beliatta divisional secretariats have less suitable areas. Lunugamwehera, Tissamaharama, Hambantota and Suriyawewa divisional secretariats have the suitable areas.

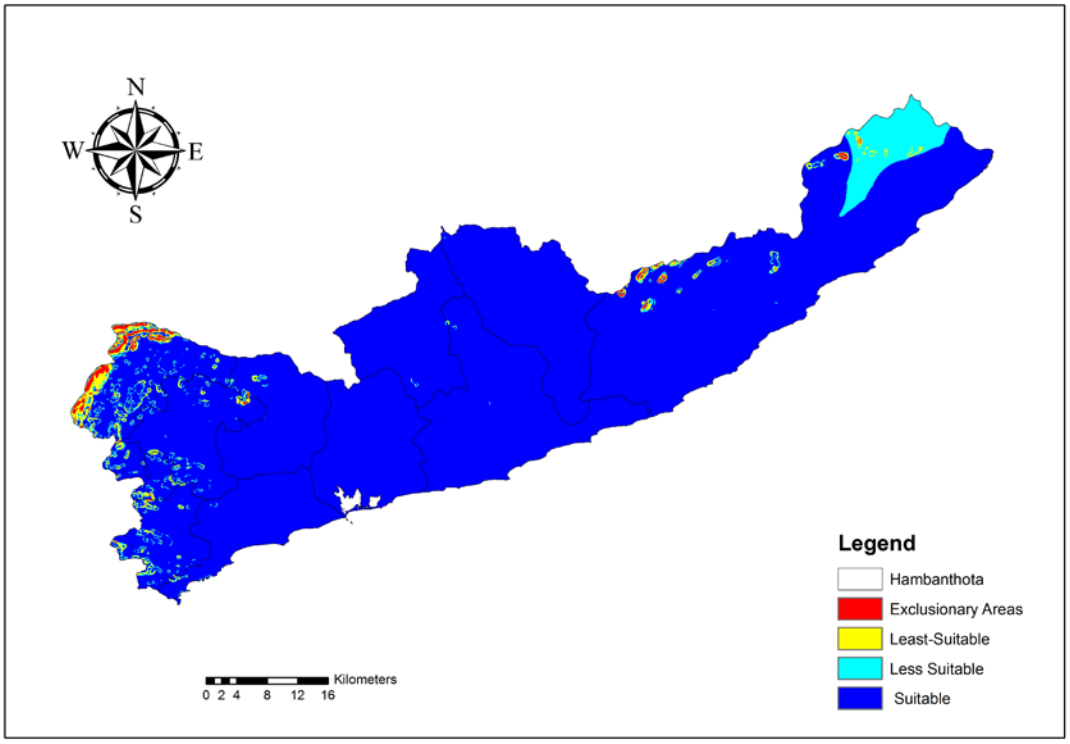

Figure 1 Map of Hambantota district illustrating the unsuitable, less and least suitable and suitable areas for landfill siting based on environmental criteria

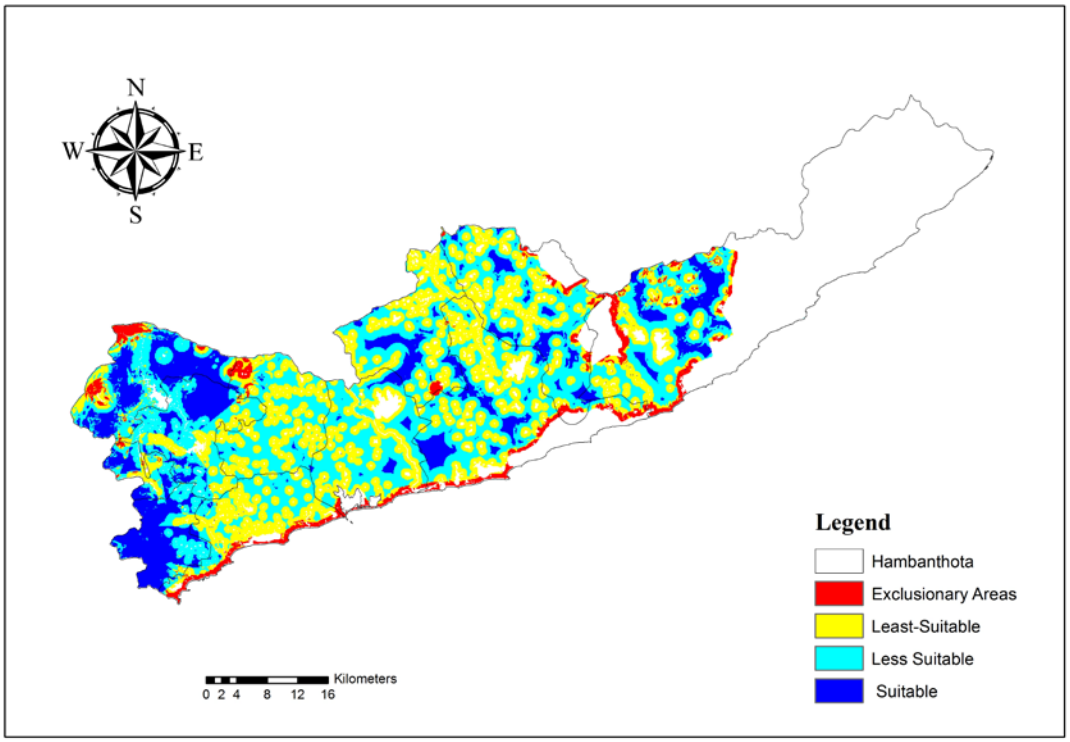

Figure 2 Map of Hambantota district illustrating the unsuitable, less and least suitable and suitable areas for landfill siting based on socio and cultural criteria

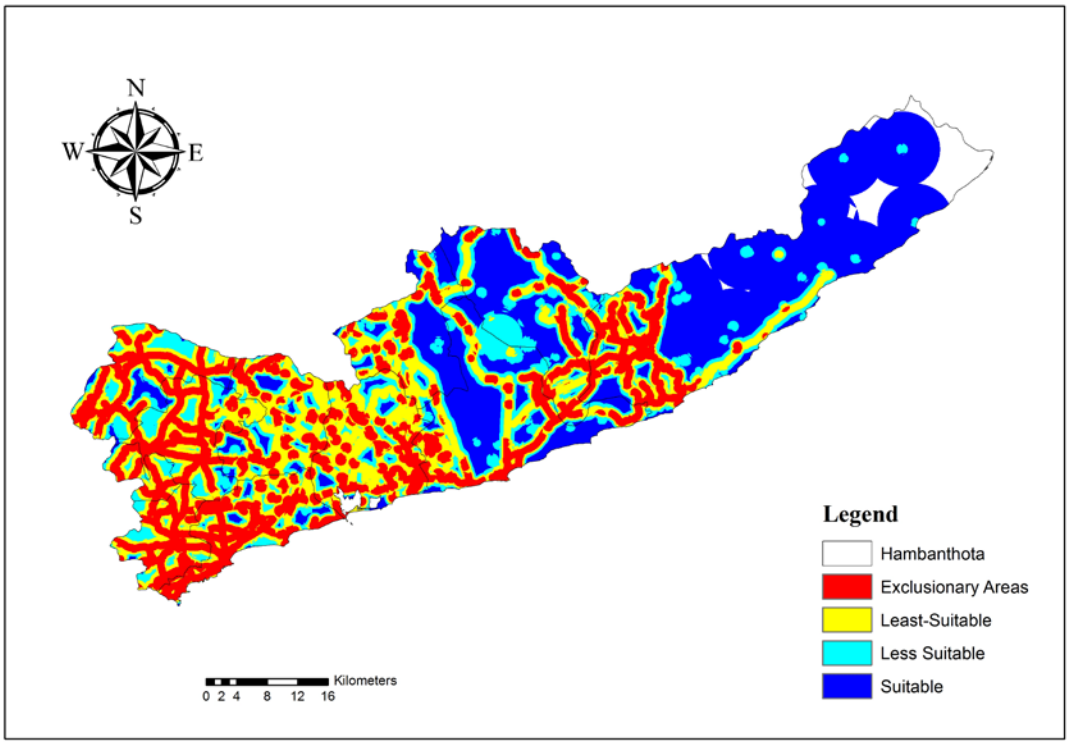

Figure 3 Map of Hambantota district illustrating the unsuitable, less and least suitable and suitable areas for landfill siting based on economic criteria

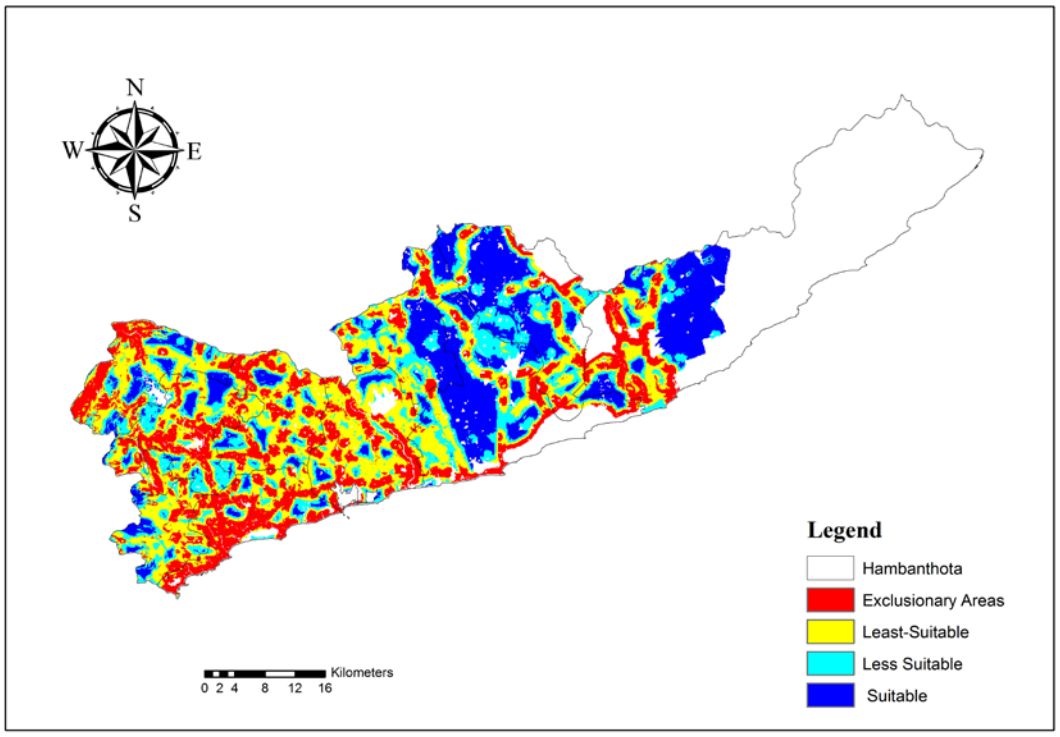

Figure 4 Map of Hambantota district illustrating the unsuitable, less and least suitable and suitable areas for landfill siting

4. CONCLUSIONS

The landfill siting process requires evaluating many criteria because different criteria effect on land filling in different ways. It is rather a complex task to consider everything at once. GIS is an ideal tool for this due to its ability to manage large volumes of spatial information from various sources. The methodology described in this paper is a sound approach for landfill siting. This study shows how to integrate the GIS technique with the analytic hierarchy process (AHP) to incorporate the true contribution of criteria affecting landfill siting into the GIS model. The model can be applied for any area if the feature maps are available. The importance of this model in landfill site selection lies in the features that the model can be repeatedly run and it can encounter errors easily. The site selection could be made more environmentally friendly by incorporating additional criteria according to the available guidelines. Additional criteria such as land ownership, electricity supply, flood plain layers and ground water table should be considered for a better output. The study also confirms the ability of the integrated methodology of GIS and AHP as a viable tool for decision making in the field of Environmental Science and Engineering.

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Use of Principal Component Analysis to Evaluate Stormwater Washoff Quality of Road Surfaces in Different Urban Land Uses in Galle

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Abstract: *Both univariate and multivariate data analysis were used to evaluate the stormwater washoff quality of road surfaces in different urban land uses and determine correlation between washoff quality and land use type namely residential, commercial and industrial. Stormwater washoff samples collected from three selected road surfaces in each land use in Galle city, maintaining minimum antecedent dry period of five days between each event while ensuring not to mix with other wastewater were tested to determine physical, chemical and biological characteristics of stormwater according to standard method of water and wastewater analysis published by American Public Health Association. Significant pollutants for each land use were identified based on the univariate data analysis. COD, BOD_5, turbidity, alkalinity and fecal coliform are the parameters exceeded the standards for inland waters of Sri Lanka, CLASS 1 Waters: Drinking water with simple treatment published by Central Environment Authority in 2001. Principal Component Analysis (PCA) is the multivariate data analysis technique further used to determine the correlations between water quality parameters and land use type. Residential and industrial land use have discriminated water quality characteristics each other according to the PCA. Thus, PCA is a tool can be used to identify the strong relationships between land use type and washoff quality parameters.*

Keywords: *Principal Component Analysis, Stormwater Washoff, Urban Land Uses, Road Surfaces, Galle*

1. INTRODUCTION

Urban stormwater washoff has been recognized as a non-point pollution source of deterioration of water quality in surface water bodies (Yufen et al., 2008). Stormwater washoff is originated when rain water flows over the impervious surfaces such as streets, parking lots and rooftops which prevent the infiltration (Barrios, 2000). A significant increase of impervious surfaces are resulted with the uncontrolled urbanization. Various types of pollutants are deposited and accumulated on these impervious surfaces and receives to the water bodies (Gromaire-Mertz et al, 1999, Barrios, 2000, Gobel, Dierkes and Coldewey, 2006). Quantification of pollutant loads in urban runoff was begun based on the results of studies in the 1950s and those analyses depicted that the shock load to a receiving water from urban stormwater washoff could be 100 to 1000 times greater than for sanitary wastewater, and having significance dominating effect on the quality of receiving water (Barrios, 2000). Attention was then focused upon quantifying the source of these pollutants in micro scale through the 1960s (Bradford, 1977) and further efforts have been focused upon identifying and quantifying the individual sources to form of water pollution. Consequently, land use has been become the major variable since it affects the types and levels of pollutant generating activities of man and nature (Bradford, 1977, Gromaire-Mertz et al, 1999 Goonetillekea et al., 2005, Huang, 2007, Yufen et al., 2008, Kothalawala and Kumara, 2013).Furthermore, researches have been carried out to identify the different impervious surfaces for the contamination of urban stormwater runoff classifying as road surfaces, roof surfaces and parking lots. Gobel et al. (2006) showed that runoff pollutant concentrations of impermeable roads and parking surfaces are significant compared to other

impervious surfaces and vary with traffic density, wind drift, duration and intensity of stormwater events.

Principal component analysis (PCA) has been used as a major analytical tool in numerous water quality studies (Thielemans and Massart, 1985, Huang et al., 2007, Bahar and Ohmori, 2008, Miguntanna, 2009). PCA is a well- known pattern recognition technique of multivariate statistical data analysis which reduces raw data into multiple components while retaining the most variance within the original data in order to identify possible patterns between objects and variables (Huang et al., 2007, Goonetilleke et al., 2005). Goonetilleke et al. (2005) has used PCA to recognize the set of water quality data from six catchments in Queensland State, Australia. PCA has been used by Zeng and Rasmussen (2005) for the evaluation of spatial and temporal variations of water quality of in Lake Laier, USA. Zhou et al. (2007) has employed PCA to analysis the data of marine water quality. Similarly, PCA has been used by Huang et al. (2007) to identify the stormwater quality in different urban surface types in Macuau.

There is very limited monitoring of stormwater quality in study area, Galle. Researches on evaluating stormwater quality in different impervious surfaces have been carried out (Kothalawala and Kumara 2013). Thus, this research was further conducted to determine the quality of stormwater washoff on road surfaces in different urban land uses namely residential, commercial and industrial while identifying the correlation between pollutants and different urban land uses with the means of PCA.

2. MATERIALS AND METHOD

2.1. Sampling

Three roads surfaces having same characteristics of each different urban land uses; residential, commercial and industrial were selected in Galle city as shown in Figure 1. Sampling of stormwater washoff from three roads in each land use was carried out into polyethylene bottles manually maintaining minimum antecedent dry period of five days between each event while ensuring not to mix with other wastewater. 1.5 L of stormwater washoff in each location was collected within 10 – 20 min for each event during the period from May to November in 2015.



Figure 1 Sampling Locations

2.2. Lab Testing

Collected samples were preserved and tested to determine physical, chemical and biological characteristics of stormwater according to standard method of water and wastewater analysis

published by American Public Health Association. Temperature, turbidity, total solids (TS), total suspended solids are the physical parameters measured and chemical parameters tested were the BOD_5, COD, alkalinity, chlorides, electrical conductivity and pH, hardness. Fecal coliform and total coliform are the biological parameters measured.

2.3. Data Analysis

Both univariate and multivariate data analysis techniques were used for statistical analysis (Huang et al., 2007). Univariate statistical analysis was used to compare the two parameters measured each other which is the basic and the simplest form of statistical analysis, carried out prior to other advanced data analysis methods. Mean and standard deviation (SD) is used for statistical univariate measurements to describe the characteristics of a single variable data set. Parameters exceeded the standards for inland waters of Sri Lanka, CLASS 1 Waters: Drinking water with simple treatment published by Central Environment Authority in 2001 was identified based on the univariate analysis results.

Correlations between water quality parameters and land use type were determined using multivariate data analysis technique, Principal Component Analysis (PCA). PCA were carried out for all stormwater quality parameters measured with three different land use types separately.

3. RESULTS AND DISCUSSION

Parameters exceeded the standards were identified based on the mean of the univariate data analysis are shown in Table 1.

Table 1 Summary of parameters exceeded the standards

Land use	Turbidity (NTU)	BOD_5 (mg/l)	COD (mg/l)	Alkalinity (mg/l)	Fecal Coliform
Residential					
Mean	65.13	38.67	198.59	133.33	1186.67
SD	54.83	4.51	173.36	75.06	125.83
Commercial					
Mean	277.30	82.33	266.50	253.33	206.67
SD	224.55	33.61	156.06	159.48	179.54
Industrial					
Mean	178.33	65.67	177.15	170.00	116.67
SD	36.95	10.69	116.70	72.11	41.63
Standard [a]	≤ 5	≤ 3	≤ 15	≤ 200	≤ 600

[a] Standards for inland waters of Sri Lanka, CLASS 1 Waters: Drinking water with simple treatment published by Central Environment Authority in 2001

Turbidity, COD, BOD_5 are the most significant parameters which have been exceeded the standards drastically for all three land uses. It reveals that all three land use has a significant contribution of organic matter into the stormwater washoff. The mean concentration of turbidity from the commercial land use is 1-4 magnitude higher than other land uses. Not only mean concentration of turbidity but also COD and BOD_5 are higher with respective to the residential and industrial land uses. Therefore, some measures should be taken to treat this washoff before enters them into the receiving water body.

The mean concentration of alkalinity and fecal coliform are only exceeded in commercial land use. It depicts that commercial land is the dominant land use for the contamination of stormwater washoff. Therefore, stormwater washoff generated in commercial land use should be treated separately to

enhance the efficiency of wastewater treatment in cost effective manner. Interestingly, mean concentration of alkalinity and fecal coliform of stormwater washoff from other land uses are in low level.

The SD of Turbidity, alkalinity, fecal coliform and BOD_5 from different land uses reflects great uncertainties of stormwater quality which makes more difficult to control the stormwater quality.

Multivariate data analysis techniques were further used to explore and generate hypothesis between variables (Bandara and Bandara, 2012) by interpretation of large volume of data while investigating the data under the effect of several conditions due to limitations of univariate statistical methods. These techniques are known as pattern recognition techniques.

XLSTAT version 2014.4.08 has been used for PCA of physical, chemical and biological parameters in three different urban land uses; residential, commercial and industrial in this research. XLSTAT provides outputs of the Eigen vectors along with mean and standard deviations of each predictive variable. Thus, maximum of five principal components (PCs) were calculated and first and second PCs (PC1 and PC2) were plotted in a common plot because first two components are usually responsible for the most of the variance. PCA provides scree plot, bi-plot and correlation matrix which are useful for the identifying correlation and patterns of the data. The appropriate number of PCs is determined by interpreting the slope of scree plot visually. The bi- plot shows the both loadings and scores for the two selected PCs in parallel and is displayed the best relation of them. The angle between the lines (loadings) or to be more precise, the cosine of the angle between the lines, approximates the correlation between the variables they represent. The angles closer to 90^0 or 270^0 shows a very less correlation (Bandara and Bandara, 2012, Kothalawala and Kumara, 2013). The PCA bi-plot is alone inadequate to identify the relations between parameters in the case of low total data variance exists. Moreover, degree of relationship of two variables that is parameters measured can be determined from the correlation matrix. Correlation coefficients in correlation matrix vary from -1 to +1. If correlation coefficient is closed proximity to ± 1, it reveals a perfect correlation between those two parameters; hence the best correlated parameters can be identified using the correlation matrix.

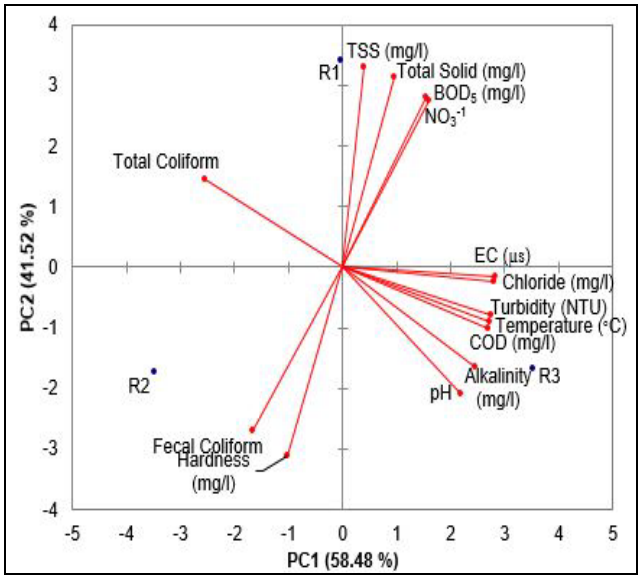

Figure 2 Bi-plot for the Residential Land Uses

Bi-plot for the residential land uses which is shown in Figure 2, PC1 was concerned with alkalinity, pH, EC, COD chloride, turbidity and temperature have closer angles among each parameters, and thus, it shows a strong relationship between physical and chemical parameters in residential land use. These variables are higher in R3. There is correlation coefficient of 0.99 for pH and alkalinity. Alkalinity is the capacity of water to neutralise acid added to it. The pH of water is a measure of the acid–base equilibrium. Thus, neutralising the acidity and H^+ concentration have a higher degree of relation. EC and chloride also has an angle closed to zero degree since EC is caused by the ions in the stormwater. TSS, total solids, BOD_5 and NO_3^{-1} are also another set of correlated parameters which consist of both physical and chemical parameters. BOD_5 is a measure of biodegradable organic matter

exist in the stormwater washoff as well as dissolved oxygen concentration required for microorganisms and NO_3^{-1} is nitrogenous compound that provides nutrients for microorganisms' growth. Microorganisms are in the form of TSS which is a sub group of total solid. Thus, there are acute angles between each parameter with relatively has same loadings.

Bi- plot for the stormwater washoff quality parameters measured in commercial land use is illustrated in Figure 3. Except COD and pH those chemical parameters, other all parameters are in positive region of PC1. There is a correlation coefficient of 0.971 which shows a clear relation with acute angle between loadings of COD and pH. TSS, total solid, chloride, hardness, NO_3^{-1} and total coliform forms a set of closely related variables of all three chemical, biological and physical water quality parameters with almost same size of loadings. Angle between TSS and NO_3^{-1} are very small closed to zero because particles containing NO_3^{-1} may be attached to the TSS in stormwater. Chloride and total coliform have zero angles between the loadings illustrating the strongest correlation in commercial land use which is commonly found in C1. Fecal coliform and BOD_5 also have acute angle between loadings. Microorganisms indicated maybe group of fecal coliform found in commercial land use. Turbidity and pH are most the significant variables in this land use as they have the largest size of loadings.

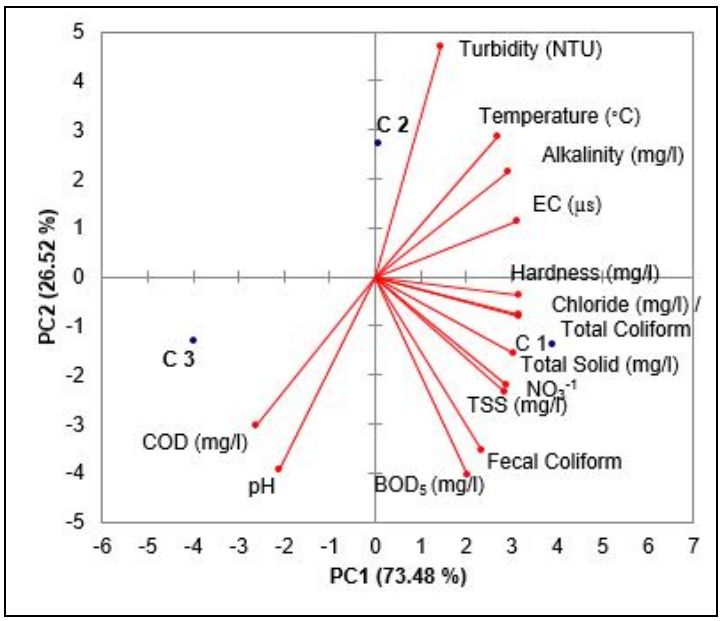

Figure 3 Bi-plot for the Commercial Land Uses

Bi- plot for the industrial land uses are shown in Figure 4. COD and temperature shows a strong correlation in the industrial land use. And TSS and alkalinity have the zero degree angle indicating the highest degree of correlation. It reveals that particles cause to alkalinity is attached to the suspended solids. BOD_5 and hardness also show the strong correlation in industrial land use. Total coliform, total solids, hardness, BOD_5, turbidity and NO_3^{-1} forms a cluster of chemical, physical and biological parameters which is commonly found in P2. Total solids are formed of microorganisms indicating by BOD_5 and total coliform. Furthermore, these solids are a transporter of the chemicals and pollutants cause for the hardness, $NO_3^{-1}$and turbidity.EC has the largest size of loading in industrial land use as a whole; however it can be noticed at P1.

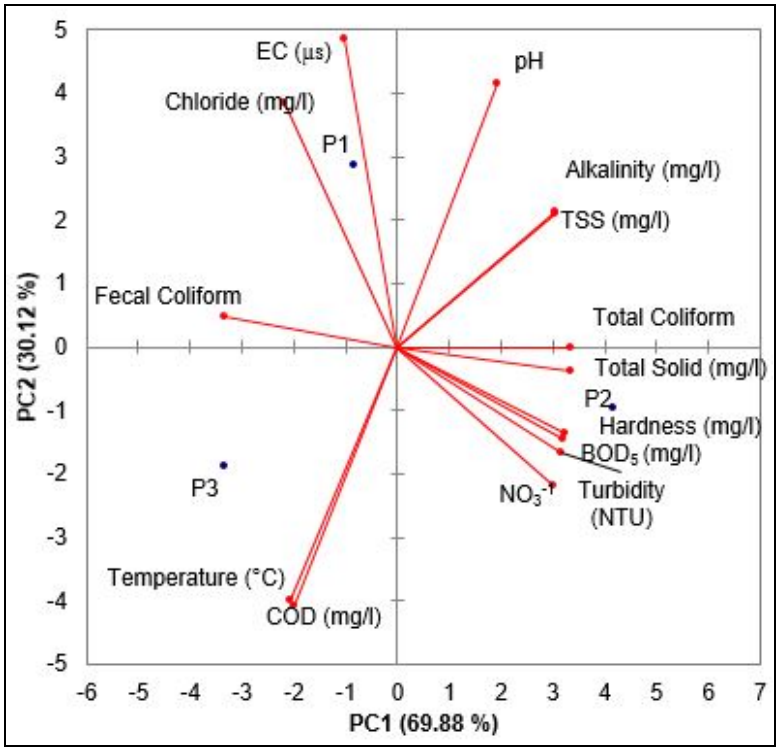

Figure 4 Bi- plot for the Industrial Land Uses

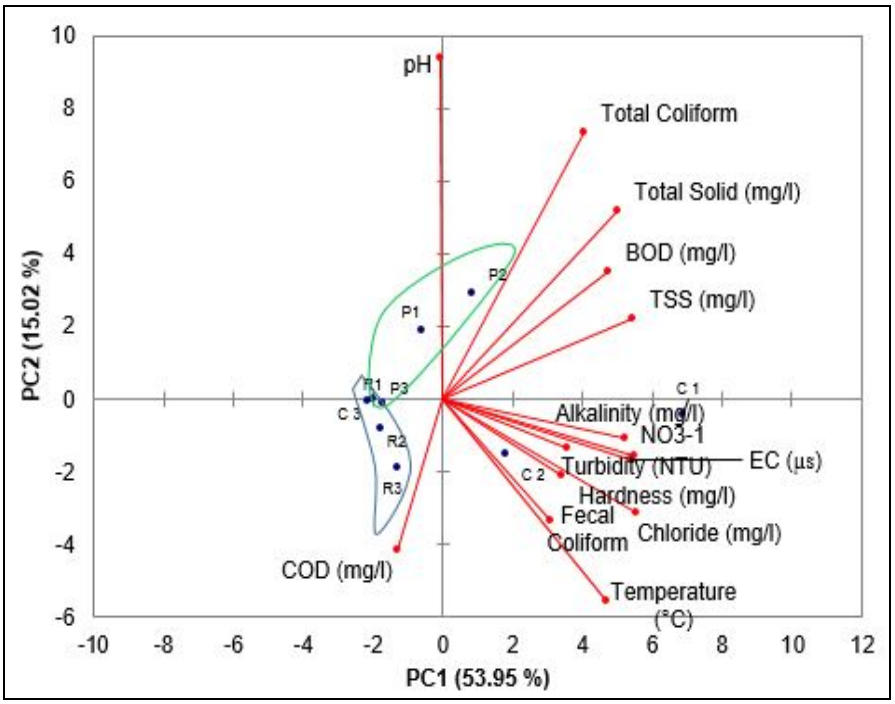

Figure 5 Bi- plot for the All Land Uses

There are two clusters of residential and industrial land uses shown in Figure 5 reveals each land use has different characteristics of pollutants are generated in stormwater washoff. According to the Principal Component Analysis, strongly correlated parameter were also identified. Alkalinity, chloride, hardness, electrical conductivity (EC), turbidity, nitrate (NO_3^{-1}), temperature are in one quadrant shows a strong correlation. Hardness is a measure of the quantity of divalent ions such as Calcium, Magnesium, and or ion in the water. In addition, Calcium Carbonate ($CaCO_3$) which is dissolved in water flow through geology that has limestone and marble is one source of alkalinity. Furthermore, most of the sampling locations are located in Galle that are near to the sea, thus high concentration of

$CaCO_3$ can be recorded. Hence, hardness and alkalinity of the stormwater washoff are proportionate each other. Besides, EC in stormwater is also measure based on existing ion concentration of hardness, alkalinity and chloride etc. Therefore, EC, alkalinity, chloride and hardness are strongly correlated parameters which are also proportionate each other. Total solids (TS) and total suspended solids (TSS) are another strongly correlated parameters since TSS is sub group of TS which describe the particulates suspended in a moving body of water.

4. CONCLUSION

According to the results of stormwater washoff collected from road surfaces in different urban land uses have different types of characteristics as physical, chemical and biological with different degree of pollution. BOD_5, fecal coliform, COD, alkalinity and turbidity have been exceeded required standards, thus, stormwater washoff should be treated to the acceptable limits prior to release the natural water bodies according to standards of inland water quality published by Central Environment Authority.

Commercial land use is the most important in the perspective of pollution source of stormwater washoff according to the results of univariate and PC analysis in all three land uses. This has become dominant with exceed of standard of COD, BOD_5, turbidity, alkalinity and fecal coliform. Organic pollutants discharge is referred to the COD, BOD_5 and turbidity. Emission of smoke, leakages of oil and grease on road surfaces are also cause to the turbidity. For the organic pollutants which demand the oxygen, structural stormwater improvement measures such as detention basins or sediment traps would be effective.

Since there is strong correlation between land use type and pollutants, zoning can be done in future infrastructure development. Then it is convenient to treat the stormwater washoff in economical and efficient way.

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Fluoride and Hardness Removal from Groundwater by Electrolysis System with Carbon Electrodes

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Abstract: *Groundwater is the most commonly found source of freshwater on Earth. Therefore, groundwater quality is vital in public health point of view, where excessive fluoride and hardness can cause negative health effects on consumers. Researchers state that in certain areas of Sri Lanka, fluoride and hardness levels exceed Sri Lankan standards for drinking water. Hence, it is important to remove fluoride and hardness before consumption. Most of existing methods for fluoride and hardness removal require high cost and sophisticated technology. Electrolysis is one low cost and effective removal method that can replace most existing methods, and highly applicable to Sri Lanka. Although, currently aluminium electrolysis is commonly applied in electrolysis systems, some researches reported high Al content in treated water than raw water. Therefore, this study aims to replace aluminium electrodes with non sacrificial carbon electrodes and evaluate its function to simultaneously remove fluoride and hardness. Results showed that simultaneous removal is possible using carbon electrolysis up to 87% and 99% highest, for fluoride and hardness respectively. Removal was highly dependent on the current and it was identified that current should be maintained at a constant throughout the electrolysis process. Initial fluoride concentration had no significant effect on removal, but, higher the initial total hardness, higher was the removal. Higher conductivity showed better fluoride removal of fluoride and hardness. According to results, initial conductivity should be maintained at, at least 1.2 mS/cm to achieve Sri Lankan standard for fluoride (1 mg/l). Carbon electrolysis method can be further improved into a continuous flow system with modifications.*

Keywords: *Groundwater, fluoride, hardness, electrolysis, carbon electrodes*

1. INTRODUCTION

Groundwater is the most commonly found source of freshwater on Earth, accounting for approximately 97% of non-frozen fresh water in the world (Margat and van der Gun, 2013). Further, groundwater supplies for nearly half of all drinking water requirements of the world population (World Water Assessment Programme, 2009) and about 43% of the global consumptive use in irrigation is also fulfilled by groundwater (Siebert et al., 2010). Hence, quality of groundwater is considered to play an important role in health and well being of the world population.

Presence of fluoride and hardness excessively in ground water is considered a serious problem in public health point of view. Consumption of fluoride concentrations between 0.8 mg/l and 1.0 mg/l, is beneficial to prevent tooth decay (British Columbia Groundwater Association, 2007). However, excessive consumption can cause dental fluorosis leading to mottling of teeth, skeletal fluorosis causing softening of bones, ossification of tendons and ligaments and also neurologic damage (Reardon and Wang, 2000). On the other hand, hardness in water has not been identified as a cause for serious health issues. However, it has been identified as a risk factor that could exacerbate

eczema and also contact irritation caused by soap salt residues while causing an unpleasant taste in drinking water and scaling in equipment and containers used for boiling water (WHO, 2011).

In Sri Lankan context, excessive fluoride and hardness in groundwater has proven to be a serious issue, especially since nearly 80% of rural domestic water requirements is fulfilled by groundwater (Panabokke and Perera, 2005). Recent researches confirm that groundwater in districts such as Anuradhapura and Moneragala contained fluoride exceeding 1.0 mg/l which is the maximum desirable limit according to Sri Lankan standards. Hardness levels were found to be high in Jaffna, Hambanthota, Vavuniya and Anuradhapura exceeding 180 mg/l (Kawakami et al., 2014).

Groundwater quality in Sri Lanka has gained much attention in recent years due to the chronic kidney disease of unknown etiology (CKDu). Latest research reveals that areas with groundwater containing high fluoride concentrations and very hard water overlap with CKDu affected areas (Kawakami et al. 2014). Hence, excessive fluoride consumption is assumed to be an etiology for CKDu by many researchers, however possibly not the only reason. It is further mentioned that CKDu prevalent areas and groundwater hardness has a strong correlation with each other (Jayasumana et al., 2012). Therefore, it is essential that the public is provided with treated water to ensure their health and safety.

There are numerous methodologies employed for removal of fluoride and hardness. Common fluoride removal methods are chemical additive methods, contact precipitation and adsorption/ion exchange methods (Lyengar, n.d). Hardness is removed by chemical precipitation and ion exchange methods which involve chemicals and reverse osmosis, electrodialysis, nano-filtration, crystallization, distillation and evaporation which do not involve chemicals (Malakootian, Mansoorian and Moosazadeh, 2010). But, most of these methods are of high cost and sophisticated technology. Use of bone char, activated alumina and calcined clay for adsorption/ion exchange method, require regeneration which can be expensive (Iyengar, n.d). Use of material like bone charcoal also has a psychological effect on the consumers depending on their religious and cultural backgrounds. Some techniques carry problems such as increased sludge production, high annual operation costs and also sediment formation on membrane (Malakootian, Mansoorian and Moosazadeh, 2010). In reverse osmosis, approximately two thirds of water fed in for treatment is disposed as reject water. Considering these issues, above mentioned sophisticated methodologies may not be suitable for community or domestic level treatment for hardness, especially in Sri Lanka.

One modern and low cost electrochemical technology that can be highly applicable in community level in Sri Lanka is electrolysis. The method has a high removal efficiency of many contaminants with no/less chemical addition, minimum waste production, low power requirement and least manpower requirements. Most popular and commonly used electrodes in this method are aluminium electrodes. Around ten aluminium electrolysis plants have been established in Sri Lanka, for treatment of groundwater in community level. However when used for removing fluoride and hardness, aluminium anode sacrifices itself during the electrolysis reaction. Thus, the amount of aluminium ions is greater in the effluent than that of the influent (Emamjomeh, Sivakumar and Schafer, n.d.).

Therefore, in this research, aluminium electrodes were substituted by carbon electrodes which is both conductive and almost non-sacrificial unlike aluminium. The aim of the study was to evaluate fluoride and hardness removal in contaminated groundwater by electrolysis using carbon electrodes and the effect of initial concentrations and other water quality parameters such as conductivity on the removal efficiency.

2. METHODOLOGY

A batch electro-coagulation reactor was prepared with a carbon (C) cathode and a platinum (Pt) anode, separated by an unglazed clay diaphragm (Figure 1). For each experiment, a direct current was passed for 120 minutes using a DC power supply unit, after filling anode and cathode baths with 550ml and 410ml of raw water respectively. Water levels at the anode bath and the cathode bath were maintained the same.

The electrolyte was prepared by spiking fluoride and magnesium standards in to water. Raw ground water collected from a dug well in Hapugala, Galle was used for the electrolysis experiments in which fluoride and total hardness are 0.03 mg/l and 10 mg/l as $CaCO_3$ respectively. Fluoride concentration of

raw water was adjusted using 1000 mgF^-/l stock solution prepared using Sodium fluoride (NaF) and total hardness of raw water was controlled using 5000 mgMg^{2+}/l stock solution prepared using $MgCl_2.6H_2O$.

Fluoride concentration was measured at 60 minute intervals using a Fluoride Ion Selective Electrode (ISE) (Thermo Fisher Scientific). Initial total hardness and final total hardness at 120 min were measured by titration with 0.01M EDTA (Ethylenediaminetetraacetic acid). The fluoride ISE was calibrated in advance, using standard fluoride solutions (diluted from 1000 mgF^-/l stock solution) of 0.1, 1.0 and 10.0 mg/l. TISAB III concentrate with CDTA (Orion 940911 - Thermo Scientific) was used as a buffer solution, when calibrating the ISE and measuring fluoride content in collected water samples.

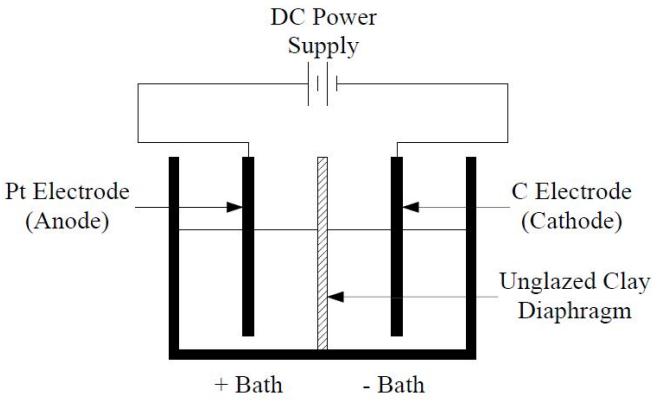

Figure 1 Schematic diagram of the batch electrolysis apparatus

An initial experiment was carried out to evaluate the effect of supplied DC voltage on removal. Initial current through the electrolysis system was measured under different DC voltages (31.7V, 45.1V, 60.2V, 76.4V, 91.6V). However, the current significantly fluctuated throughout the 120 minutes of electrolysis. Supplied initial electrical charge (Current x Time) was calculated using measured initial current. Initial fluoride concentration and total hardness were maintained at 10 mg/l and 208 mg/l as $CaCO_3$ respectively, for this set of experiments.

The second set of experiments was conducted to evaluate the effect of initial concentrations on fluoride and hardness. To evaluate the effect of initial fluoride concentration, initial total hardness was maintained at 200 mg/l as $CaCO_3$ for all cases with initial fluoride concentrations of 0.91, 2.68, 4.95 and 7.22 mg/l. Then, to evaluate the effect of initial total hardness on removal, the initial fluoride concentration was maintained at 5.0 mg/l for all cases with initial total hardness values of 202, 411, 988 and 1490 mg/l as $CaCO_3$.

3. RESULTS AND DISCUSSION

In all experiments, fluoride concentration and total hardness decreased with time. Hence, simultaneous removal of fluoride and hardness using carbon electrolysis was proven to be possible. Through the initial experiment, a direct relationship between initial electrical charge/voltage and removal could not be established due to significant current fluctuations. However, it could be confirmed that the governing factor for removal is the supplied current and not the DC voltage.

For initial fluoride concentrations below 3 mg/l, Sri Lankan standard for fluoride, 1 mg/l was achieved after 120 minutes (Figure 2).

Results showed that the initial fluoride concentration does not have a significant effect on total hardness removal. However, according to results, higher the initial total hardness, higher was fluoride removal. For initial total hardness values of 988 and 1490 mg/l as $CaCO_3$, fluoride concentration decreased below 1 mg/l (Figure 3).

Highest fluoride removal of 87% was obtained for 5 mg/l initial fluoride and 988 mg/l as $CaCO_3$ initial total hardness. According to results (Figure 4), the initial total hardness does not have a significant effect on total hardness removal. Highest total hardness removal of 99% was obtained for 5 mg/l initial fluoride and 1490 mg/l as $CaCO_3$ initial total hardness.

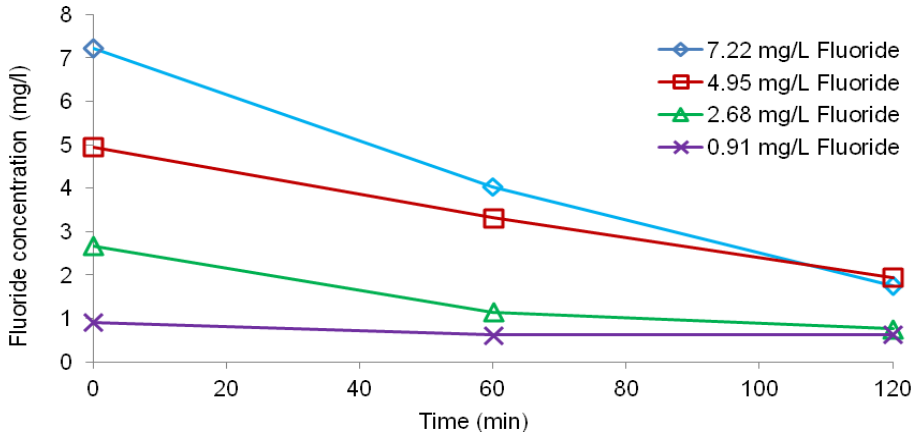

Figure 2 Variation of fluoride concentration with time under different initial fluoride concentrations

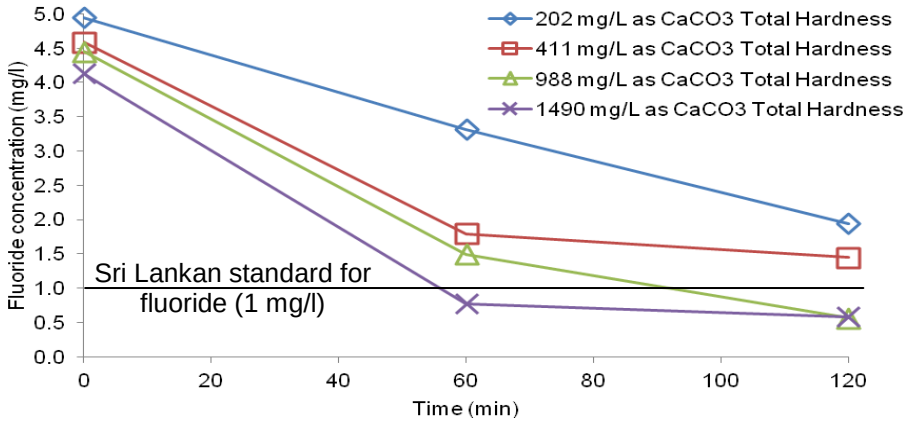

Figure 3 Variation of fluoride concentration with time, under different initial total hardness values

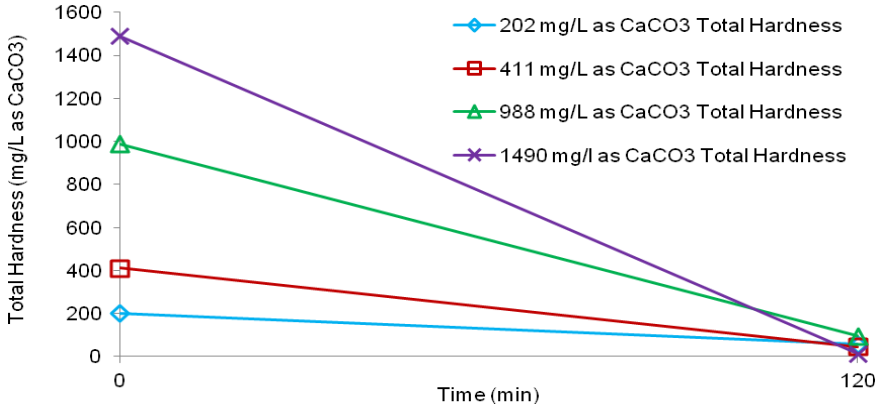

Figure 4 Variation of total hardness with time under different initial total hardness values

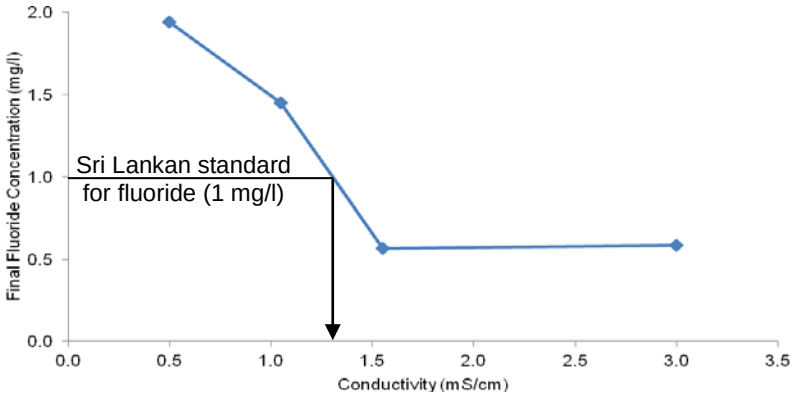

Figure 5 Final fluoride concentration vs initial conductivity

According to the relationship between initial conductivity (where initial fluoride concentration was 5 mg/l) and final fluoride concentration, a larger removal could be obtained when the initial conductivity was higher (Figure 5 above). According to Figure 5, to achieve the Sri Lankan standard for fluoride, initial conductivity has to be maintained at, at least 1.2 mS/cm.

4. CONCLUSIONS AND FUTURE DIRECTIONS

According to the results obtained during experiments, it is possible to remove fluoride and hardness simultaneously using electrolysis with carbon electrodes. However, removal efficiency is highly dependent on the current provided for electrolysis rather than the voltage. In fact, the electrical charge which is measured in Coulomb (Current x Time) is the governing factor for removal of fluoride and hardness through electrolysis. Therefore, it is important that the current is maintained at a steady state throughout the electrolysis process to ensure that expected removal is achieved. Required amount of electrical charge can be either provided by electrolysing under a large current for a short period or by electrolysing under a small current for a longer period.

Highest removal efficiencies of 87% and 99% were achieved for fluoride and hardness respectively. According to the variation of final fluoride concentration with initial electrical conductivity, a higher removal was obtained at higher initial electrical conductivities. According to the variation, to achieve the Sri Lankan standard for fluoride, it is important to maintain an initial electrical conductivity larger than 1.2 mS/cm.

According to obtained results, carbon electrolysis is recommendable to areas with high hardness and high fluoride contents due to high conductivity in such groundwater. But, complete treatment should include filtration to remove produced precipitates and a pH adjustment unit to decrease the final pH up to desirable level, after electrolysis. This batch unit can be further improved into a continuous flow system which can be applied to domestic or community level usage.

5. ACKNOWLEDGMENTS

The Department of Civil and Environmental Engineering, University of Ruhuna and Toyama Prefectural University, Japan are thankfully appreciated, for providing the research fund for the study.

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Computational Study on the Mechanism and Formation Pathway of Tri halo Methane During Chlorination from Carbonyl Group

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Abstract: *The mechanisms for the formation of tri halo methane (THM) by reacting $CCOCH_3$ with HOCl in gaseous phase were studied using computational chemistry methods to prove the possible pathways .The result shows that the title reactions could take place on the singlet potential energy surfaces (PES) and $CHCl_3$ formation occurs via single barrier process. By studying the energetic reaction at the B3LYP level, formation mechanism of THM from $COCH_3$ was proposed.*

Keywords: *THM (Tri Halo Methane), potential energy surfaces (PES)*

1. INTRODUCTION

Trihalomethanes (THMs) which exhibit a potentially carcinogenic activity are certainly the class of compounds that has been investigated most thoroughly during the last 20 years. Dissolved humic substances in water consisting, Humic acid (HA) and fulvic acid (FA) fractions reacts with chlorine during water treatment, thereby producing carcinogenic disinfection byproducts such as THMs was repoeted by Zhao *et* al (2008) and Wang *et* al (2008: 2010b). Classical THMs consist of chloroform ($CHCl_3$), dichlorobromoform ($CHCl_2Br$), dibromo-chloroform ($CHBr_2Cl$) and bromoform ($CHBr_3$).

Generally, HA and FA differ in molecular weight, elemental and functional group contents. HA is higher in molecular weight and contains less oxygen-containing functional groups compared with FA was shown in (Güngör & Bekbölet, 2010) and Weng et al (2006) studies. Despite the differences, their common functional groups such as carboxyl, phenol, hydroxyl, amine and quinine groups make it possible to form Disinfection by Products (DBPs).

Even though THMs are investigated highly there were few studies have been conducted to find the formation mechanism of THMs. In some cases such as compounds including a methylene group between two carbonyl functions, a high chlorine reactivity was shown by de Laat *et* al (1982), Folkes *et* al (1995), (Pattison &Davies, 2001) and Tachikawa *et* al (2002).

Reaction with carbonyl functional groups, Aldehyde or ketone chlorination, generally results from initial substitution reactions on the carbon to the carbonyl group was proposed by (Roberts & Caserio, 1968), (Morries & Baum, 1978), Dore (1989) and Larson and Weber (1994) studies. They proposed a reaction pattern for the reaction of chlorine with aldehydes and ketones. But, still no any theoretical investigation was reported to reveal the pathways of formation THMs. Therefore this study was conducted based on computational chemistry to obtain more accurate data meanwhile referring others studies. Most reactive carbonyl group C=O involved in methyl ketone ($COCH_3$) was used to propose new reaction mechanism pathway which was adopted from (Morries & Baum, 1978) proposed THM reaction pathway.

2. COMPUTATIONAL METHOD

All calculations are performed using Gaussian 03 program package(Frisch et al., 2009).The geometries of the reactant, Products, Intermediates and Transition states were optimized using

B3YLP method with 6-311++G (p, d) basis set in gaseous phase. The B3YLP method is proven to be an economic and accurate computational model for predicting electronic structures, and has been employed widely (Hou and El-taher 2006; Zheng et al., 2005; Zha, 2004) Compared with other levels of theory, the B3LYP method was found to be sufficiently accurate for predicting reliable geometries of the stationary points, at the same time, it is not expensive computationally for scanning potential energy surface. All the stationary points had been positively identified for minimum number of imaginary frequencies (NIMAG = 0) or transition state (NIMAG =1). The transition states were verified by the intrinsic reaction coordinate (IRC) calculations to connect the designated reactants and products

3. RESULTS AND DISCUSSION

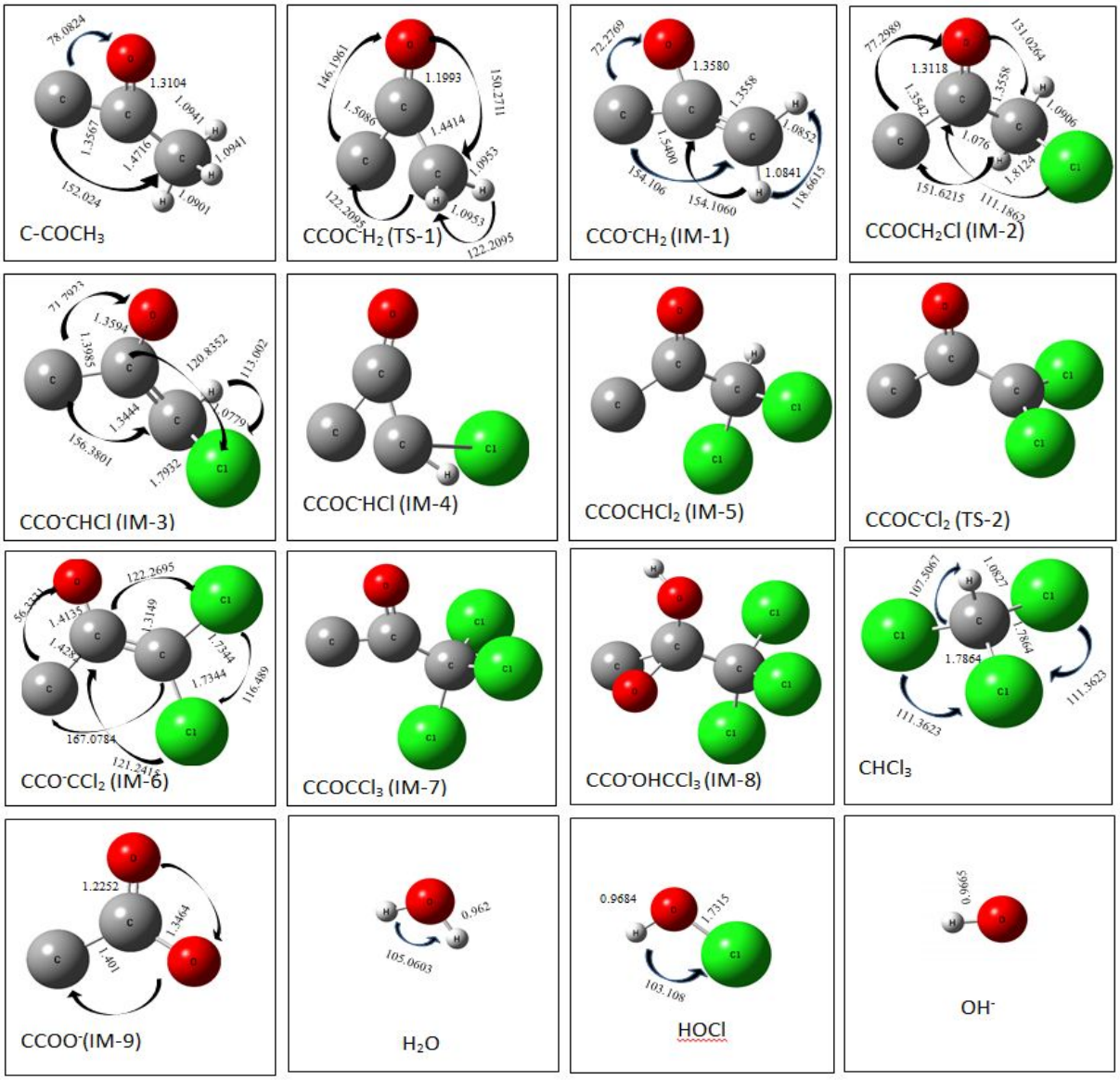

Figure 1: The optimized geometries for the C-COCH$_3$ +OH$^-$ +3HOCl reaction with B3YLP method. Bond distances are in angstrom A^0 and bond angles are in degrees ^{0} were calculated using 6-311 ++ G (d, p) basis set.

The geometries of all possible stationary points involved in the titled reaction optimized at the B3LYP/6-311++G (d, p) are shown in Fig. 1. The profile of the singlet potential energy surface (PES) for the C-COCH$_3$ +OH$^-$ +3HOCl reaction is depicted in Fig. 2. Table 1 exhibits the relative energy (ΔE) and reaction enthalpy (ΔH) at the B3LYP/6-311++G (d, p) level. In order to emphasize the important

pathways for the reactions, bold thick lined is used in Fig. 2 to give a direct and clear indication.

Table 1: relative energy (ΔE) and reaction enthalpy (ΔH) (in KJ/ mol) of all species for the C-$COCH_3$ + OH^- +3HOCl reaction on the singlet potential energy surfaces.

Species	B3YLP ΔE	B3YLP ΔG
C-$COCH_3$ + OH^- +3HOCl	0	0
TS-1	10.83207	11.752
IM-1	-42.4535	-42.53
IM-2	-38.7412	-37.7
IM-4	-93.4274	-92.63
IM-3	-59.486	-57.83
IM-5	-78.0183	-75.69
TS-2	-113.431	-111.4
IM-6	-99.924	-97
IM-7	-115.313	-111.4
IM-8	-162.108	-146.1
IM-9 + $CHCl_3$	-199.246	-196.1

With carbonyl group C=O involved in ketone ($COCH_3$) functional group, both O and C atoms are reactive centres when OH^- atom adds to C-$COCH_3$, the nature of the R group (bound to the carbonyl of acetyl function) influences the reaction rate. Electron-withdrawing groups make the hydrogen atoms of the methyl group more acidic, thus increasing chlorine substitution, whereas electron- donor groups decrease methyl acidity and therefore chlorine substitution (de Laat, 1981).

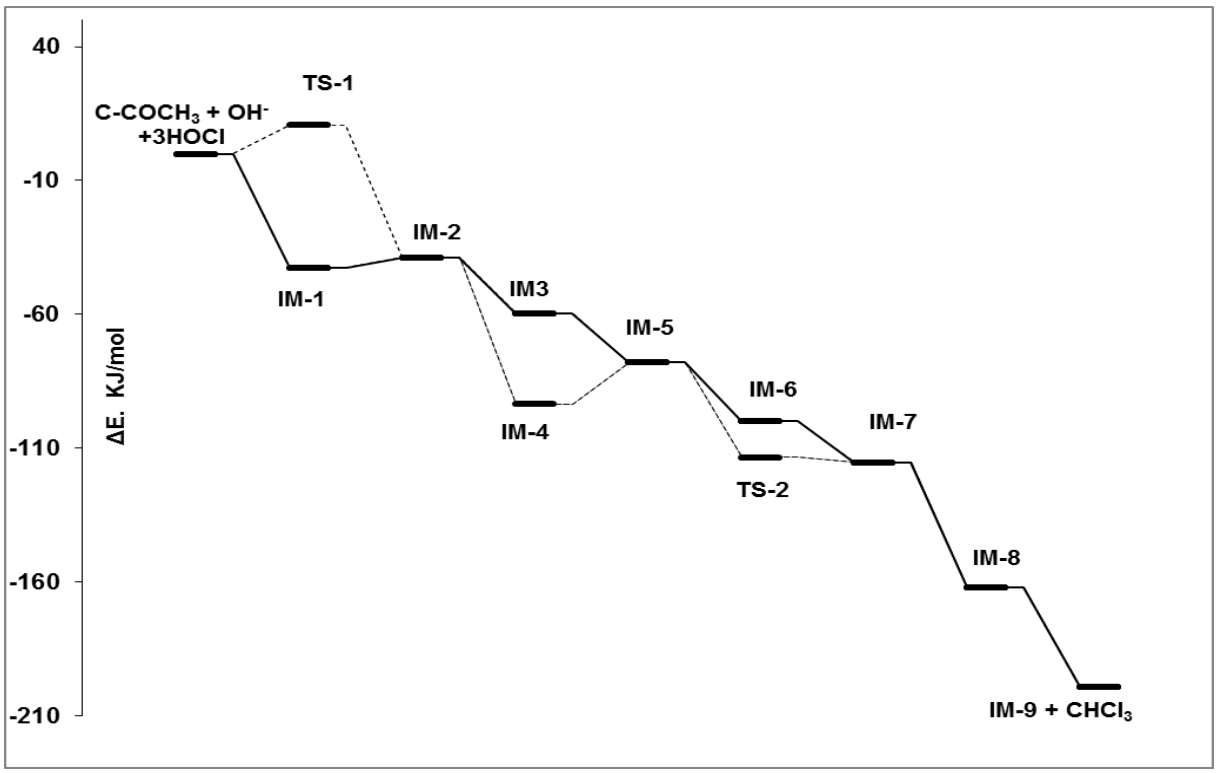

Figure 2: The energetic reaction routes of the C-$COCH_3$ + OH^- + 3HOCl reaction on the singlet Potential Energy Surface (PES) at the B3LYP level

According to theoretical data, formation of acidic intermediate IM-1 take place via barrier less process during TS-1 acidic intermediate form via barrier process as shown in figure 02. Intermediate IM-1 was more stable with about 42.45 KJ/ mol. lower than initial reactants. TS-1 is unstable having higher energy level of 10.83 KJ/mol. Therefore IM-1 is stable and reaction occurs through IM-1 rather than TS-1.

Formation of IM-2 by reacting TS-1 and HOCl is a barrier less process. Even though it is barrier less process formation of IM-2 is most favourable by reacting IM-1 with HOCl by passing small energy barrier of 3.71 KJ/mol.

Hydroxyl ion removes hydrogen ion from chloro methyl ketone (IM-2) by forming water molecule. According to Morries and Baum, 1978 proposed reaction sequence IM-3 forms resonance structure of IM-4 , IM-6 forms TS-2 and IM-1 forms TS-1. According to the calculation results formation of resonance structures are possible becaus IM-3 relative energy (-59.486 KJ /mol) is higher than IM-4 relative energy (-93.4274KJ/mol) value and IM-6 relative energy (--99.92 KJ /mol) is higher than TS-2 relative energy (-113.431 KJ/mol) value. But formation of dichloride methyl ketone (IM-5) is most viable through IM-3 because of no energy barrier. Hence formation probability of IM-4 resonance structure is very law.

IM-7 form via IM-6 Because of no energy barrier found via IM-6 pathway for IM-7 formation. Then TS-2 formation probability was decreased due to IM-7 formation via IM-6 .Formation of IM-7, IM-8 and IM-9 intermediates found no any other energy barrier according to the calculations. By consideration energy pathway most suitable mechanism of $CHCl_3$ formation pathway can be proposed as Fig. 3

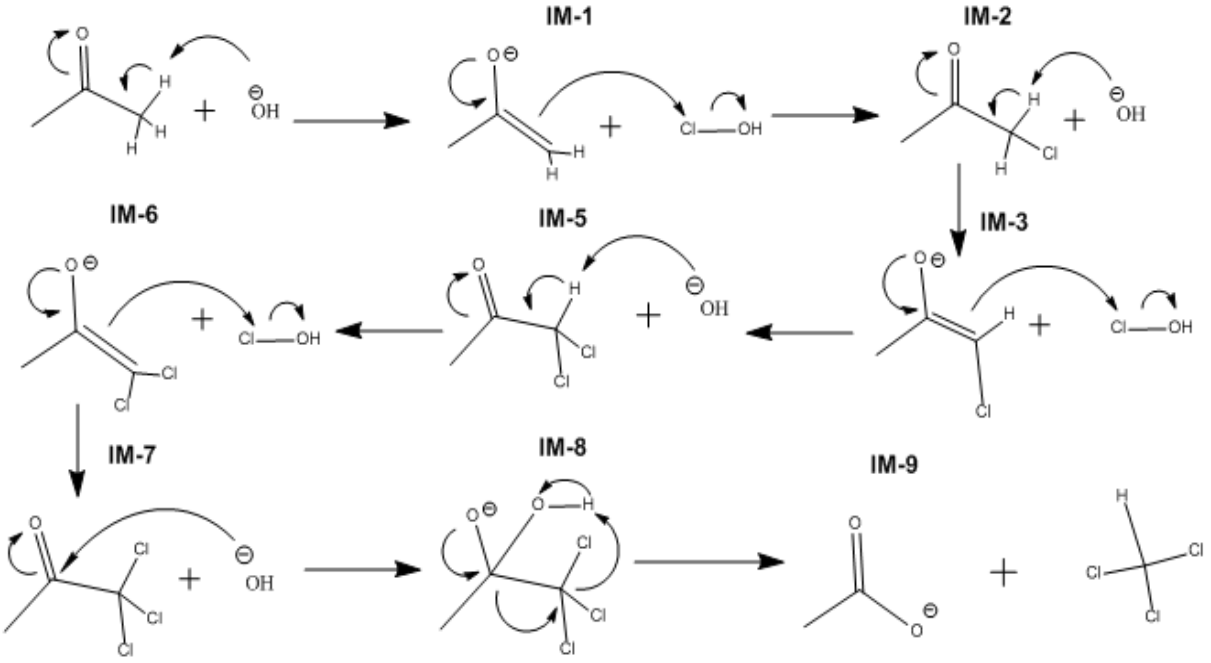

Figure 3: Proposed reaction mechanism for Tri halo methane formation

4. CONCLUSION

Proposed THM formation mechanism from C-$COCH_3$ was different from adopted method of Morries and Baum, 1978. The possibility of forming resonance structures (TS-1, IM-4, TS-2) was very low and negligible when excess HOCl presence in the water. However data show that 199.246 kJ/mol energy decrement by starting from C-$COCH_3$ + OH^- +3HOCl to form IM-9 + $CHCl_3$.

5. ACKNOWLEDGMENTS

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Waste Amount and Composition Survey (WACS) in Galle and Hambantota Municipal Councils

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Abstract: *The Solid Waste Management (SWM) is becoming an enormous problem for the Local Authorities (LAs) in Sri Lanka. As a developing country, it is unbearable to allocate huge funds for SWM and hence waste related problems have been aggravated. Knowledge of waste generation sources, amount and compositions will be useful to optimize waste management system within allocated money. It also makes a platform to reduce environmental and social issues related to SWM in Sri Lanka. This paper will be discussed waste generation amounts, waste compositions, waste management capacities and limitations of Galle Municipal Council (GMC) and Hambantota Municipal Council (HMC) based on the information gathered by Waste Amount and Composition Survey (WACS). Since demographic, socio-economic and climatic conditions very much different in Galle and Hambantota, influence of those factors on waste generating amount and waste composition clearly revealed by this survey.*

Keywords: *Galle, Hambantota, WACS*

1. INTRODUCTION

As the world hurtles towards industrialization, urbanization and higher growth of population, amount of Municipal Solid Waste (MSW) generation increases day by day. Besides that, waste composition also becomes complex due to above mentioned facts. According to World Bank records (2012), current global waste generation is 1.3 billion tons per year at the rate of 1.2 kg/capita/day. Waste generation rate will be doubled over next twenty years and expected volume of waste generation is 2.2 billion tons by the 2025. Further, Global Solid Waste Management cost will increase from today annual $205.4 billion to about $ 375.5 billion in 2025 making severe financial difficulties in low income and middle income countries in the world (World Bank, 2012).

According to the reported data in different studies, solid waste generation in Sri Lanka is over 6400tons/day and the waste generating potential is 0.25 to 0.5 kg/capita/day (Bandara, 2011; Wijethunaga, 2012; Karunasena and Wickramasundara, 2012). In year 2006 and 2007, Ministry of Local Government and Provincial Councils (MOLGPC) spent 8.5 million rupees and 11.4 million rupees respectively to raise the awareness of solid waste management and assist waste management projects. Besides that some other fund raising methods were exist at the time such as loans and donations (NSWMSC, 2007).

In Sri Lanka, there are several regulatory institutions involving waste management such that National Solid Waste Management Support Centre (NSWMSC) and Central Environmental Authority (CEA) other than the Ministry of Environment and Natural Resources (MOENR) and Ministry of Local Governments and Provincial Councils (MOLGPC) (NSWMSC, 2007). These organizations execute various projects and programs to manage MSW in the country. Those projects are including people

awareness; improve the waste collection and disposal systems, and environmental restoration of Local Authorities (LAs). Other than that, gathering SWM related data is one of the most important actions that have considered enhancing the waste management system. Waste Amount and Composition Survey (WACS) is one of such program conducted to gather information about amount and composition of waste generating in the LAs.

The WACS was conducted on several LAs in Sri Lanka in order to plan, design and operate solid waste management system thorough knowledge of the generated quantities and the compositions of wastes. This paper will be discussed only two Municipal Councils (MCs) named Galle and Hambantota located in southern province of Sri Lanka. Mainly these two cities have different backgrounds in demography, socio-economic conditions and culture. Also Galle and Hambantota MCs represent the wet and dry zone in southern coastal zone of Sri Lanka.

2. MATERIALS AND METHODLOGY

2.1. Determination of Sample Size

In order to collect basic information such as geographical size, demography and Socio- Economic pattern of survey area, pre investigation survey was conducted. Based on the pre investigation survey results, sample categories and sample sizes were determined. Then total number of sample locations selected for Galle and Hambantota MCs were 164 and 156 respectively. Table 01 and table 02 are representing the sample information and general information of the survey areas.

Table 1 Sampling procedure

Category	Target Sample	Sample Size (Galle MC)	Sample size (Hambantota)
Residential (Collection Area)	High income level	30 samples	30 samples
	Middle income level	30 samples	30 samples
	Low income level	30 samples	30 samples
Residential (Non-collection Area)	All	30 samples	30 samples
Commercial	Restaurants (small scale)	2 samples	2 samples
	Questionnaire survey (high and middle scale)	2 samples	2 samples
	Shops (middle scale)	4 sample	3 samples
	Questionnaire survey(high and middle scale)	5 samples	6 samples
	Hotel (small scale)	2 sample	1 sample
	Questionnaire survey(high and middle scale)	2 sample	2 samples
	Junk shop etc. (Questionnaire survey)	10 sample	4 sample
Institution	School (Questionnaire survey)	2 sample	2 samples
	Govt. office (Questionnaire survey)	1 sample	2 samples
	Hospital (Questionnaire survey)	3 sample	1 sample
	Religious place (Questionnaire survey)	4 sample	4 sample
Industry	Factory (Questionnaire survey)	3 samples	3 samples
Others	Questionnaire survey	4 samples	3 samples
Total		164 samples	156 samples

Table 2 Basic information of survey areas

Local Authority	Galle MC	Hambantota MC
Population	91,204	22,691
Area (km^2)	16.5	79.61
Population density (p/km^2)	5,528	285
Total number of household	19,806	7,422
Total number of commercial	4903	338
Total number of institution	894	66

2.2. WACS Execution

Waste samples were collected from randomly selected sample locations over eight days. In order to minimize errors during the sampling first day was allocated as trial day. People were advised to collect kitchen waste and other waste in to separate bags provided by survey team. Weighted waste samples sent to the waste handling location for further analysis. Other than the waste sample collection questionnaire survey was conducted for institutions, industries, high and middle scale hotels and shops. Figure 1 shows the how waste collected from residential.



Figure 1 Waste collection



Figure 2 Sorting of waste

To determine the bulk density and other parameters such as daily waste generating rate and composition percentage of waste, all the waste which was collected sorted in to different groups such as kitchen waste, garden waste, paper and cardboard etc as illustrated in Figure 2.

In order to find out daily waste disposal amount at dump site, waste fill weight and dimensions of collecting tractors were measured. In addition numbers of waste fill tractors come to the site were counted and took photos of tractors in order to count the total amount.

Bulk density of waste, is defined as the ratio of grand total weight of waste (kg) to the volume of the waste (m^3). The grand total weight means the sum of daily total weight of waste. The daily waste generating rate is vital result that can be attainment by this survey. It is defined as the ratio of sum of daily waste generation to the total population of the sample number of dates which sample are collected. Apart from this two parameters, composition percentage of each and every constituents were determined dividing the values, daily total weights of each constituents of waste by grand total weight of waste. In addition total waste generation was determined by using values of waste generation rates from households, total population of the MCs, and amount calculated by questionnaire survey.

3. RESULTS AND DISCUSSIONS

3.1. Waste Generation and Management in Galle MC and Hambantota MC

Generally households, commercials, institutions, and industries are generating huge amount of waste daily in both Galle and Hambantota MCs. The average waste generation rates of Galle and Hambantota MCs are 0.28kg/capita/day and 0.4kg/capita/day. Even though socio-economic condition high in Galle Municipal area, waste generating rate is lower than that of Hambantotata Municipal area. Main reasons for low waste generating rate in Galle MC is high employment and low land availability. Due to high employment of peoples in Galle MC area, people spend day time at work places and most of the residents have small gardens. Therefore amount of kitchen waste and garden waste is very low in Galle MC area.

In Galle Municipal area, daily waste generation is around 50.9t/d and Out of that; 40.8 tons dispose to open dump site daily. The daily disposal amount of Hambantota Municipal area was 9.9 tons and out of 20.4 tons of generation. It is very clear that majority fraction of waste is disposed to open dump sites by both Municipal Councils. The residual amount of generating waste is subjected to on site disposal, recycling and composting. The recycling percentage of Galle and Hambantota MCs were 12.97% and 14.7% respectively. It can be seen 36.7% of waste was disposed on sites due to low population density and high land availability in HMC. The on-site disposal of waste in Galle MC is very low (1.96%) compared with HMC area.

Figure 3 represents the current waste streams of Galle MC Area. Due to higher population and industrialization, waste generation in Galle MC area is very high. Waste generation contribution by waste generators very much high in households, commercial and institutions. According to the final disposal amount calculated at the landfill site using tractor loadings, it was 40.8 tons per day. However there is a shortage of 2.5 tons of waste for the dump site can be observed. The reason behind this shortage could be the illegal dumping of waste.

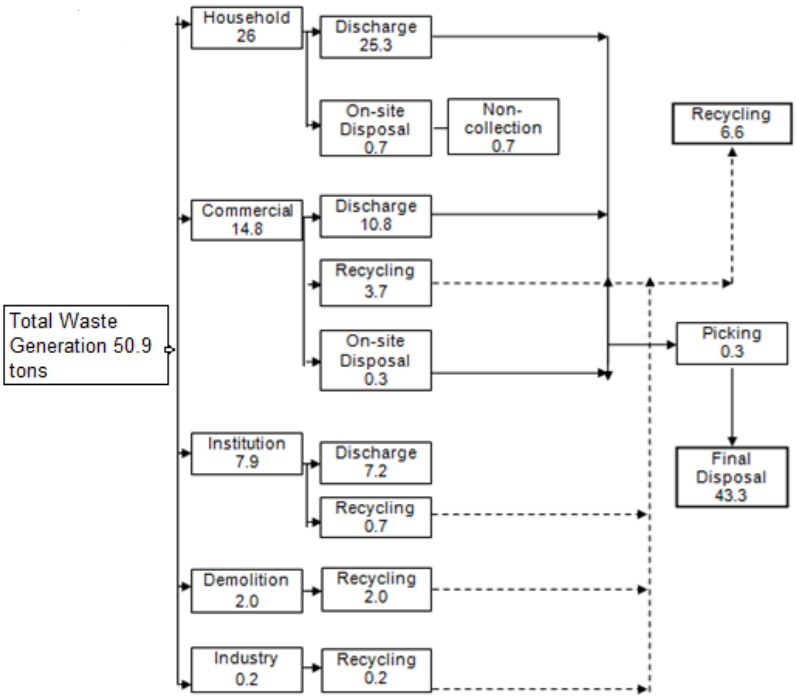

Figure 1 Current waste streams in Galle MC (Weight by tons)

In Galle MC Key recycling items are paper and cardboard, metal, plastic and construction demolitions. Local buyers collect metal and plastic, glass bottles and some e-wastes. Paper and cardboard and construction demolition recycling is doing with assistant of MC as shown in Figure 4. In addition, it can

be observed that household waste separation at dump site and plastic, glass and tin cans sell to large scale vendors in Colombo. The biodegradable fraction of house hold waste is used for composting.



Figure 2 Paper and cardboard recycling

Figure 5 illustrates the current waste streams of Hambantota MC area. Due to low population and industrialization, waste generation in Hambantota MC area is much lower than that of Galle MC area. Similar to the Galle MC Area, household waste generation contribution is the highest among all other waste streams in Hambantota MC. Even though final disposal amount is 9.6 tons, actual waste disposal at the site is 8.8 tons. Therefore, there is a shortage of 0.8 tons of waste to the dump site daily due to illegal dumping.

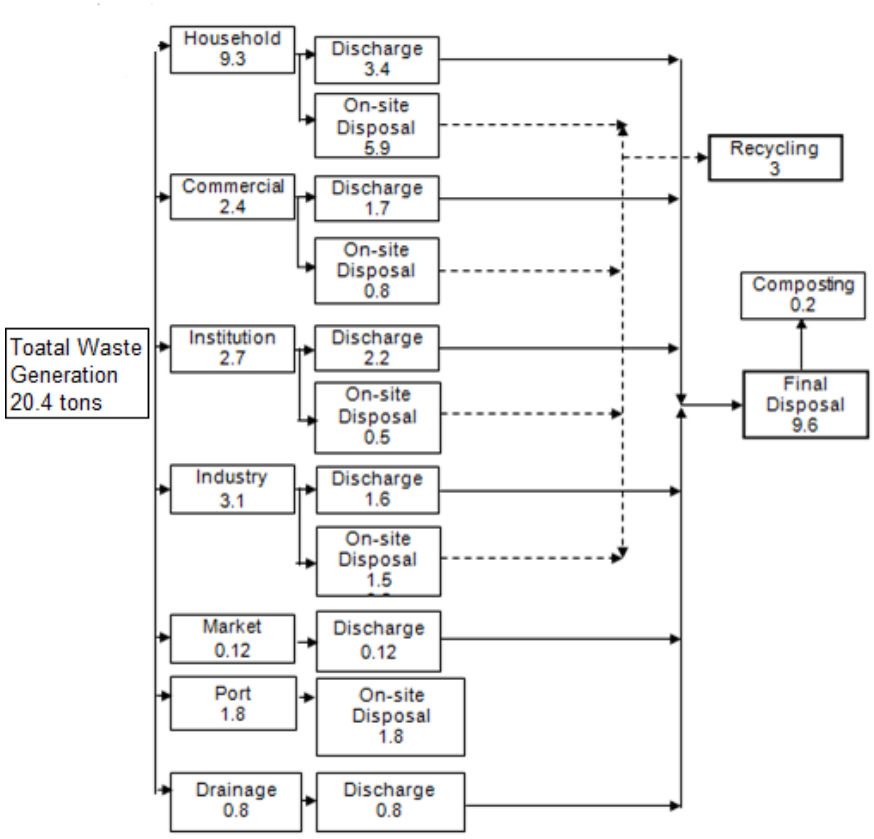

Figure 3 Current waste streams in Hambantota MC (Weight by tons)

According to the survey results In Hambantota MC Key recycling items are paper and cardboard, metal, plastic and glass bottles. Local buyers collect metal and plastic, glass bottles and some e wastes. There are no any recycling plants within the area and collected items transport to Colombo. At the waste disposal site, there is small scale composting unit operates using biodegradable waste of

selected tractors of household waste. Moreover, glass bottles, tin cans and plastic bottles collect at disposal site.

Based on the questioner survey, awareness of people about waste management in Hambantota MC area is considerably lower than Galle MC area. However, waste disposal is not a big problem due to availability of space for on-site disposal in Hambantota MC. Due to scarcity of resources waste management in Hambantota MC is becoming worst with the rapid development. In both MCs people do not separate wastes within the house hold and in Galle MC there are some locations (public places and institutions) separate wastes. Further, both MCs dispose waste on open dump sites making serious situation on surrounding environment.

3.2. The composition of waste in Galle MC and Hambantota MC

Municipal Solid Waste (MSW) is mishmash of organic waste such as kitchen, garden and paper & cardboard and non-organics such as plastic, textiles, rubber & leather, metal, glass etc. Waste composition is vital parameter that can use to optimize whole waste management system.

Waste composition analysis was conducted on residential and commercial waste collected in both MCs. Residential and Commercial waste compositions in Galle MC is shown in Table 3 & Table 4 respectively. According to the waste composition analysis, residential solid waste mainly consists of biodegradable waste (kitchen waste, garden waste, paper and cardboard). The amount of biodegradable waste was 85.9% and 89.4% in Galle MC and Hambantota MC respectively. The quantities of non-organic materials such as paper, soft and hard plastic, metal and glass are relatively small. Further organic wastes are dominant in commercial waste in both MCs (84.65% and 83.3% in Galle MC & Hambantota MC respectively) even though a significant proportion of non-organic waste presence.

Table 3 Residential waste composition in Galle MC (% by wet basis)

Waste Type	*Residential Waste*				
	High Income	Middle Income	Low Income	Non-collection Area	Average*
Kitchen	71.3	61.8	76.7	75.0	68.1
Garden	8.6	17.2	3.8	7.6	11.6
Paper, Cardboard	7.7	5.4	6.5	4.1	6.2
Soft Plastic	4.6	4.2	5.6	4.5	4.6
Hard Plastic	1.8	1.8	1.5	1.9	1.7
Textiles	0.2	1.2	1.3	1.5	1.0
Rubber, Leather	0.4	0.9	0.1	0.8	0.6
Metal	1.2	1.0	0.3	0.5	0.9
Glass	0.8	2.6	1.0	1.9	1.8
Ceramics	0.0	1.1	0.3	0.3	0.6
Hazardous	0.1	0.4	0.1	0.2	0.2
E Wastes	0.0	0.1	0.0	0.1	0.1
Miscellaneous	3.4	2.3	2.9	1.5	2.7

Table 4 Commercial waste composition in Galle MC (% by wet basis)

Waste Type	Commercial			
	Hotel	Restaurant	Organic Shop	Non-organic shop
Kitchen	69.3	76.6	96.5	44.5
Garden	0.0	0.0	0.0	0.0
Paper & Cardboard	8.2	12.3	2.1	29.2
Soft Plastic	9.8	10.7	1.3	19.3
Hard Plastic	0.9	0.0	0.0	5.0
Textiles	0.5	0.0	0.1	1.1
Rubber & Leather	0.0	0.0	0.0	0.0
Metal	1.4	0.3	0.0	0.7
Glass	2.4	0.0	0.0	0.2
Ceramics	6.7	0.0	0.0	0.0
Hazardous	0.3	0.0	0.0	0.0
E Wastes	0.0	0.0	0.0	0.0
Miscellaneous	0.4	0.0	0.0	0.0

It can be observed a little increment of non-organic materials in Galle MC than the Hambantota MC area. That is due to difference of living and consumption conditions in two MCs. It is believed that in higher socio- economic levels, people consume more non-biodegradable items than that of low socio - economic conditions.

4. CONCLUSIONS

MSW generation in Galle MC area (50.9t/d) is much higher than that of Hambantota MC area (20.4t/d) due to high population and industrialization. Nevertheless waste quantity is high in Galle MC, waste generating rate is lower (0.28kg/capita/day) than the Hambantota MC (0.4kg/capita/day) area due to influence of socio- economic conditions of two MCs. Both MCs major portion of generated waste dispose on open dump sites as primary disposal method. When the land availability is high, final waste disposal amount can be reduced by on-site waste disposal. As a percentage Hambantota MC collects more recyclable materials than the Galle MC even though Galle MC has recycling facilities within the area.

Both MCs solid waste consist of large amount of biodegradable materials such as kitchen waste, garden waste and paper products. Even though MSW consist of huge amount of organic matters that can easily degrade, the amount of composting is very low as a result of poor waste management. To get the maximum amount biodegradable portion for composting, the waste should be separate at the households and MC must collect it separately. In Galle MC area, non-organic waste content is little bit higher than Hambantota MC due to effect of socio- economic condition.

Galle MC has comparatively somewhat systematic waste management system than the Hambantota MC due to few facts. The industrialization and population are the main facts that Government pay high attention on waste management in Galle MC than the Hambantota MC. Also tourism makes great influence among all other facts. With the rapid development activities currently Hambantota MC solid waste generation is increasing significantly.

5. ACKNOWLEDGMENTS

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Infrastructure for Changing Climate and Sustainable Built Environment

Fractal Dimensions for Streetscape Visual Complexity Analysis

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Abstract: *Streetscape visual complexity can be measured in various ways including subjective and objective methods. In this paper an attempt was undertaken to compare the subjective judgments of streetscape visual complexity with the fractal dimension calculations of the streetscapes. The commercial streetscapes around Kita Urawa Station at Saitama Prefecture, Japan were selected for the study. The perspective views of the streetscapes were displayed to a group of subjects and their judgments on the streetscape visual complexity was measured using a Likert scale. Grayscale perspective images of the same streetscapes were used for the fractal dimension calculations. Both subjective and objective experiment results demonstrated a good correlation depicting the use of fractal dimensions for streetscape visual complexity analysis is a prolific task.*

Keywords: *visual complexity, Subjective judgments, fractal dimension, commercial streetscapes*

1. INTRODUCTION

Streetscape is a key element of townscape because of its substance that creates the city's attractiveness and affects the pedestrians' comfort (Ashihara, 1983). Streetscape is a combination of several profiles such as the sky, ground, buildings, vegetation, and pedestrian behaviors on the street. Therefore all these profiles are contributing to the streetscape visual complexity. Physical complexity of streetscapes could be defined and measured by analyzing physical attributes of the streetscape. However, streetscape visual complexity is a complex phenomenon and defining visual complexity is somewhat difficult task since it is basically a subjective experience. Perceiving visual complexity is different person to person based on their personal views and expectations. Thus defining and measuring visual complexity becomes difficult. Most of the time visual complexity measurements were undertaken subjectively using surveying techniques. In this paper, an attempt was carried out to compare the subjective judgments on visual complexity with an objective measurement such as fractal dimension analysis of streetscapes.

1.1 Streetscape visual complexity

Visual complexity is a direct consequence of the spatial distribution of the streetscape elements and is concerned with perception and cognition of the information. In environmental psychology, complexity is related to the involvement component, which means: "how much there is to see in a visual array?", and to the concept of affordance that refers to what a perceived scene has to offer as far as the perceiver is concerned (Kaplan, 1988). There are lots of ways to explain visual complexity; however, there is no fixed definition for the visual complexity since it is a subjective phenomenon.

1.2 Fractal dimension analysis

Many researchers attempted to apply different kinds of objective analysis methods to measure visual complexity in cities and streetscapes. Such methods include Shannon's entropy, similarity index, space syntax method, viewshed analysis and fractal dimension analysis. In this paper an attempt was undertaken to measure the visual complexity in commercial streetscapes using fractal dimensions and to compare the results with a survey analysis using a group of subjects.

The term fractal dimension itself was brought to the fore by Benoit Mandelbrot based on his 1967 paper on self-similarity in which he discussed fractional dimensions. A fractal dimension is a ratio providing a statistical index of complexity comparing how detail in a pattern (strictly speaking, a fractal pattern) changes with the scale at which it is measured. Fractal dimensions are used to characterize a broad spectrum of objects ranging from the abstract to practical phenomena, including turbulence, river networks, urban growth, human physiology, medicine, and market trends (Wikipedia, 2014).

1.3 Past studies on visual complexity

Lynch (1960) researched the use of experiences of pedestrians to design the streetscape environments to have a pleasant experience to the travelers while travelling along the street. In his research, Lynch developed the concept of imageability; an ability for the shape, color and arrangement of elements within an urban environment to evoke a strong image for an observer.

Rapport (1990) said that the perceived number of elements within a streetscape, and particularly the "noticeable differences" between them, provides a measure of visual complexity. Further many researchers independently explained in their researches that the perceived visual complexity is very important for the pedestrians to feel the streetscape as a comfortable place to walk (Stamps III 2003, Berlyne 1974).

Hillier (1984) used space syntax method to identify the organized complexities within a city. Hillier proposes that configurations of building facades may be viewed as an arrangement of shapes which are orientated "to and away from the ground on which they stand" (Hillier 1996). He represents a building's facade as both a "metric tessellation" and as a diagram of "the dominant elements in the facade, as a pattern of convex elements" (Hillier 1996).

The viewshed or isovist analysis method was used by many scholars to measure the spatial openness within three dimensional views (Hillier 1996, Batty 2001). Using various software packages, it was easy to apply isovist or viewshed analysis to urban or streetscape analysis in a virtual environment.

Visual ambiguity, which is associated with complexity of meaning, results from the juxtaposition of the physical reality of an image and what it appears to be (Venturi 1966, Rapoport 1990). Further Rapport (1990) said the perceived number of elements within a streetscape, and particularly the noticeable differences between them, provides a measure of visual complexity.

Various scholars attempted to use fractal dimension to measure city or streetscape complexity, for example, Chen Wei (2006) has made a quantitive comparison between traditional commercial streets and modern commercial streets with the help of fractal geometry quantitative methods and entropy value calculation. Further, Wang Jianfeng (2004) has analyzed urban heterogeneity, streetscape and skyline with information entropy and spatial scale quantitative indicators which is innovative to some extent. Oku (1990) and Cooper (2003) separately studied the use of fractal dimensions to measure the geometric qualities within streetscape environments.

Thus, there are lots of past researches on streetscape and city visual complexity analysis using different kinds of methods. In this research an attempt was undertaken to analyze the visual complexity prevailing within commercial streets using fractal dimensions and to compare the results with the survey techniques.

2. METHOD

2.1 The study area

The study was conducted in the city area around the Kita Urawa station at Saitama city, Japan. Figure 01 displays the selected study area.

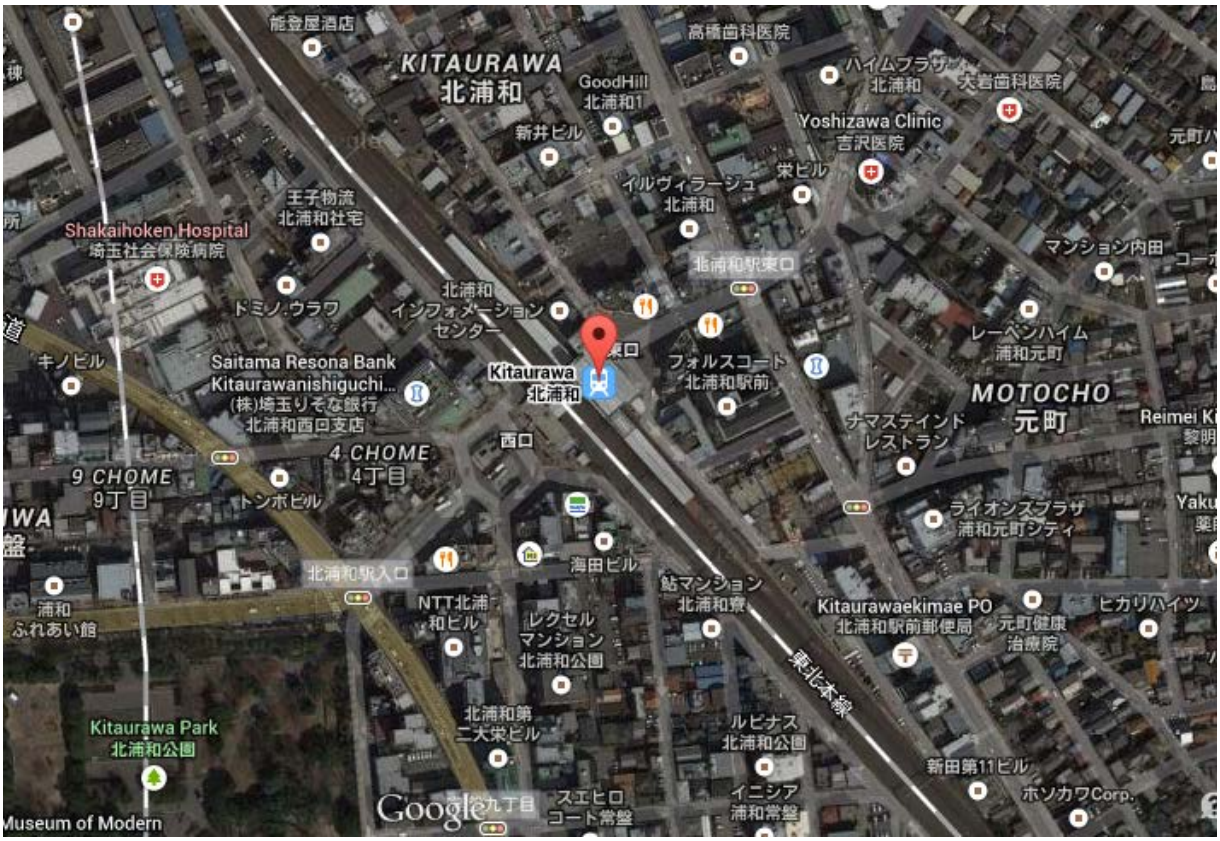


Figure 01: The Study Area

There are commercial and residential streets spread haphazard manner around the Kita Urawa station. Only commercial streets were selected for the study. At either side of commercial streets, various kinds of retail shops, restaurants, saloons and other commercial activities could be observed. Every commercial building has different kinds of billboards and other eye catching elements to attract pedestrians to the commercial building. All these buildings and their appearances and the advertising techniques contributed to the streetscape complexity.

2.2 Method

Twenty commercial streetscapes around Kita Urawa station were selected for the study. These streetscapes included low to high building densities. Gray scale and color perspective photographs were taken along the streetscape. Every streetscape has 6 serial vision photographs. Some of the views of the commercial streetscapes were displayed in the figure 02.



Figure 02: Perspective Views of the Selected Commercial Streets

2.2.1 Subjective analysis of streetscape views

The obtained perspective views along 20 residential streetscapes were displayed to 20 subjects. The subjects were selected from the Saitama University. The subjects were asked to mark the complexity level of each streetscape in a Likert Scale after observing 6 perspective views obtained from each streetscape. Likert Scale had 5 levels.

5. Highly Complex | **4**. Complex | **3**. Neither complex nor uniform | **2**. Uniform | **1**. Highly Uniform

1. Highly uniform: the commercial units and their surroundings are similar to each other. The variability along the streetscape at a minimum level
2. Uniform: the commercial units and surroundings are by and large show similar characteristics.
3. Neither complex nor uniform: the commercial units and the surrounding at the streetscape do not show a significant variability in both ends.
4. Complex: the commercial units and the surroundings show some differences in their visual characteristics.
5. Highly complex: the commercial units and the surroundings are different to each other significantly.

Based on the subjects' view and their individual preferences, the 20 residential streetscapes were categorized into one of the levels of the Likert Scale.

2.2.2 Fractal dimension analysis

The gray scale photographs were analyzed using freely downloadable "Fractalyse" software and the fractal dimension value for each photograph was obtained. Thus each streetscape had 6 fractal dimension values for 6 perspective views. The mean of these 6 values was taken as the fractal dimension value for one streetscape. Following this procedure 20 fractal dimension values were obtained for the selected 20 commercial streetscapes.

3. RESULTS AND DISCUSSION

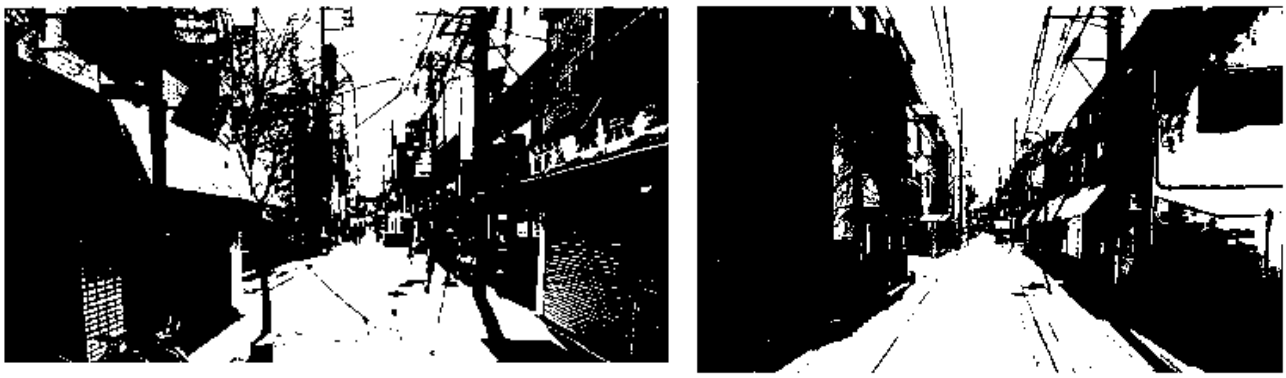
3.1 Fractal dimension analysis

Table 01 displays the mean of the 6 fractal dimension values of 20 streetscapes.

Table 01: Fractal dimension values of 20 streetscapes

Streetscape	Fractal Dimension	Streetscape	Fractal Dimension
1	1.98	11	1.96
2	1.90	12	1.74
3	1.75	13	1.85
4	1.65	14	1.43
5	1.98	15	1.97
6	1.55	16	1.98
7	1.97	17	1.85
8	1.35	18	1.88
9	1.87	19	1.94
10	1.68	20	1.87

The mean values of fractal dimensions of 6 photographs along streetscapes ranged between 1.35 and 1.98. Most of the streetscapes had values over 1.7. It implies most of the streetscapes have a high visual complexity. When an image has a high number of irregular shapes with sharp edges, the fractal dimension value becomes high. Thus, the results indicated that the selected streetscapes have a large number of sharp edges. It implies that the streetscapes have large amount of buildings and other elements with sharp edges. Figure 03 displays a view with a high fractal dimension streetscape and a view with a low fractal dimension streetscape.



A view with high fractal dimension A view with low fractal dimension

Figure 03: photographs used for Fractal Dimension Analysis

3.2 Subjective analysis of the streetscapes

Perspective views of twenty residential streetscapes were displayed to twenty subjects and they were asked to rank the streetscapes based on the given Likert Scale. Afterward the Likert ranks were counted for each streetscape and the percentage of marking each streetscape under each rank in the Likert Scale was calculated. If the percentage was over 50% under a specific Likert rank, subsequently that streetscape was grouped under that category. Table 02 shows the number of streetscapes under each category in the Likert Scale.

Table 02: Likert Ranking of Streetscapes

Likert Scale Category	No. of streetscapes under each category	Percentage
Highly complex	9	45
Complex	6	30
Neither complex or uniform	4	20
Uniform	1	5
Highly uniform	0	0

Survey results illustrated that there were 9 streetscapes with a high complexity. Similar output was obtained through the fractal dimensions resulting 8 high complexity streetscapes with fractal dimensions higher than 1.9. These commercial streetscapes had plenty of buildings and billboards. The streets showed mixed commercial activities and the buildings and other commercial advertisements are located in very haphazard manner. As a result fractal dimensions of these streetscapes become higher. Supplementary, more than 50% of subjects ranked one streetscape as uniform. Figure 04 shows two views of the highly complex and uniform streetscapes.



Highly Complex



Uniform

Figure 04: Subjects' Ranking of Highly Complex and Uniform Streetscapes

According to the survey results most of the streetscapes ranked under highly complex category. Thus the survey results were tally with the fractal dimension calculations.

4 CONCLUSIONS

The objective analysis of visual complexity on commercial streetscapes using fractal dimensions is a promising effort. The fractal dimension values ranged between 1.35 and 1.98 along with the visual density of the streetscapes the fractal dimension values changed steadily. When the building density is higher and the surrounding of the buildings displayed visual variations, that streetscape had high number of visual elements resulting high fractal dimension values. Thus the fractal dimensions could be successfully used for the visual complexity measurement on commercial streetscapes.

The commercial streets around Kita Urawa station display a haphazard development of commercial activities. The commercial activities are not properly organized; all most all streets show mixed commercial activities such as retail shops, restaurants, flower shops and convenient shops are all in the same street. Consequently the complexity of the streetscape became higher.

Because of this irregular arrangement of commercial activities, most of the streetscapes were categorized under highly complex during the survey with subjects. Accordingly both the fractal dimension results and the survey results pointed out that the commercial streets around Kita Urawa station have highly complex visual characters.

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Proper Project Planning in avoiding Construction Project Delays

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Abstract: *A construction project is commonly acknowledged as a successful project when the aim of the project is achieved in terms of predetermined objectives of completing the project on time, within budget and to the required quality standard. Delay in the completion of a construction project can be a major problem for contractors, consultants as well as for clients. These delays lead to costly disputes and adverse relationships amongst project participants. Projects can be delayed due to large number of reasons. The reasons are related to various types of uncertainties associated with activities during the construction process or during the planning and design stages. Project delays in general are due to delays caused by the client, delays caused by the contractors/consultants and delays due to equipment/materials & environmental factors. The objective of this research was to identify the major causes of construction project delays in the construction industry and find out how proper project planning would avoid / minimize their impacts. This study was carried out through questionnaire surveys and interviews conducted within the construction industry professionals. It is expected that this study would identify project planning deficiencies in the construction industry and propose recommendations to rectify identified issues and thereby avoid project delays which would contribute towards sustainable construction.*

Keywords: *Construction Delays, Construction Planning, Delays, Planning.*

1 INTRODUCTION

A construction project is a high risk activity which must be managed effectively in all stages. A project represents a unique set of activities that need to be completed to produce a unique product. The success of a product is judged by meeting the criteria of cost, time, quality, safety.One of the most important problems that may occur in a construction project is delay and the significance of these delays varies considerably from project to project. Any disruption to a project's objectives will certainly contribute to project delays with its specified adverse effects on project objectives.

The causes for construction project delays are client related causes, contractor related causes, consultant related causes, material related causes, labour related causes, and equipment related causes and external causes. The effects of construction delays are acceleration of work, schedule change, delayed project completion, increased cost, liquidated damages etc.

Planning is the heart of construction project management. The planner must weigh the cost and the reliability of different options, at the same time ensuring technical feasibility. Construction planning is more difficult in same ways since the construction process is dynamic as the site and the physical facility change over time as construction proceed.

1.1 Objective of the Study

i. Study the causes of construction project delays
ii. Identify methods to minimize construction project delays
iii. Propose proper project planning methods to avoid construction project delays

1.2 Scope of the study

The scope of the study was limited to civil engineering projects such as Building, Roads/Highway, Irrigation, Water supply etc.

2 METHODOLOGY

To understand the current status of project delays and planning solutions in construction industry, the data collection was carried out through a questionnaire survey and a series of interviews. Preliminary survey was carried out through interviews and discussions to finalize the questionnaire.

The questionnaire survey included 107 nos. Sri Lankan construction related organisations and 33 nos. foreign organisations (Singapore, Hong Kong, U.A.E, Qatar, and Iraq etc.). Among them 69 nos. responses were obtained via online questionnaire.
The collected data was analysed using the software SPSS and MS Excel. This data analysis was used to determine the relative importance of the various factors that contribute to causes of construction project delays, minimizing methods of construction project delays and to identify proper planning methods to avoid construction project delays. The following 3 steps were followed in the analysis of data:

i. Calculate the Relative Importance Index (RII)

$$RII = \frac{\sum Wi\,Xi}{\sum xi}$$

Where: i = Response category index
Wi = Weight assigned to ith response = 1, 2, 3, 4, 5, respectively
Xi = Frequency of the ith response given as percentage of the total responses for each factor

ii. Ranked the factors in each category based on the Relative Importance Index (RII)
iii. Factor Analysis using IBM SPSS software

3 ANALYSIS AND RESULTS

The data analysis was carried out separately for Sri Lankan construction industry and foreign construction industry.

3.1 Sri Lankan Construction Projects

There were 107 responses from Sri Lankan construction projects. These covered projects related to Buildings, Road/Highway, Irrigation, Water Supply etc.

3.1.1 Factors that contribute to construction project delays

All causes of delays were ranked based on their Relative Importance Index as shown below.

Table 1 Ranking of Causes of Delay in Sri Lankan Construction Projects

Causes of delay	RII	#
Conflicts in sub-contractor's schedule during execution of project	3.27	1
Delay in progress payments	3.27	1
Weather effect on construction activities	3.22	3
Difficulties in financing project	3.21	4
Shortage of labour	3.20	5
Frequent change of sub-contractors	3.18	6

Low productivity level of labour	3.14	7
Delays in sub-contractor's work	3.13	8
Rework due to errors during construction	3.13	8
Effects of subsurface and ground conditions.	3.08	10
Poor communication and coordination	3.08	10
Delay in material delivery	3.07	12
Delay in approving major changes in the scope of work	3.03	13
Personal conflicts among labour	3.02	14
Ineffective planning and scheduling of project	3.01	15
Change orders by owner during construction	3.01	15
Insufficient data collection and surveying before design	3.00	17
Slowness in the decision making process	3.00	17
Suspension of work by owner	2.98	19
Lack of high-technology	2.97	20
Inadequate contractor's work	2.94	21
Traffic control and restriction at job site	2.94	21
Delay to furnish and deliver the site	2.93	23
Shortage of equipment	2.92	24
Delays in producing design documents	2.92	24
Non use of advanced engineering design software	2.92	24
Equipment breakdowns	2.91	27
Delay in obtaining permits from municipality	2.91	27
Poor communication and coordination	2.91	27
Implementation of Improper construction methods	2.90	30
Delay in approving shop drawing and sample materials	2.90	30
Late procurement of materials	2.89	32
Delay in manufacturing special building materials	2.88	33
Conflicts between contractor and other parties	2.87	34
Unclear and inadequate details in drawings	2.83	35
Delay in revising and approving design documents	2.83	35
Shortage of construction materials in market	2.82	37
Low productivity and efficiency of equipment	2.81	38
Work permit of labour	2.79	39
Poor communication and coordination	2.78	40
Changes in material types during construction	2.77	41
Poor qualification of the contractor's technical staff	2.75	42
Delay in providing services from utilities	2.73	43
Damage of sorted material while they are needed urgently	2.73	43
Mistakes and discrepancies in design documents	2.73	43
Delay in performing final inspection and certification	2.72	46
Inadequate experience of consultant	2.71	47
Low level of equipment-operator's skill	2.71	47
Changes in government regulations and laws	2.64	49
Conflicts between joint-ownership of the project	2.59	50
Delays in site mobilization	2.59	50
Accident during construction	2.40	52

- Rank for Relative Importance Index

3.1.2 Factors affecting construction project delays

Delay affecting factors are ranked based on their Relative Importance Index shown below.

Table 2 Ranking of factors affecting delays in Sri Lankan construction projects

Causes of delay	RII	#
Labour Related	3.04	1
Contractor Related	2.99	2
Owner Related	2.92	3
Consultant Related	2.90	4
Equipment Related	2.86	5
Material Related	2.86	6
External	2.83	7

- Rank for Relative Importance Index

3.1.3 Methods of minimizing delays in Sri Lankan construction projects

Methods of minimizing delays are ranked based on their Relative Importance Index shown below.

Table 3 Ranking of methods of minimizing delays in Sri Lankan construction projects

Minimizing methods	RII	#
Effective strategic planning	3.97	1
Proper project planning and scheduling	3.93	2
Collaborative working in construction	3.77	3
Site management and supervision	3.75	4
Frequent coordination between the parties involved	3.71	5
Accurate initial cost estimates	3.70	6
Frequent progress meetings	3.68	7
Use up-to-date technology utilization	3.62	8
Complete and proper design at the right time	3.59	9
Use proper and modern construction equipment	3.58	10
Use appropriate construction methods	3.58	11
Proper emphasis on past experience	3.58	12
Proper material procurement	3.54	13
Clear information and communication channels	3.54	14
Compressing construction durations	3.40	15

- Rank for Relative Importance Index

3.1.4 Ranking of proper planning activities to avoid construction project delays in Sri Lankan construction industries

Proper planning activities are ranked based on their Relative Importance Index shown below.

Table 4 Ranking of proper planning methods to avoid delays in Sri Lankan construction projects

Proper planning	RII	#
Continuous monitoring	4.16	1
Financial controlling	3.88	2
Labour management	3.75	3
Revising schedule	3.64	4
Material/Equipment controlling	3.64	5
Inspection and certification	3.59	6
Complete site study	3.51	7
Services from utilities	3.47	8
Staff controlling	3.44	9
Legal Permissions	3.44	10
Considering weather	3.41	11
Usage of planning software	3.23	12

- Rank for Relative Importance Index

3.2 Foreign Construction Projects Results

There were 33 responses from foreign (Singapore, Qatar, U.A.E, Hong Kong etc) construction projects considered in the analysis. These covered projects were related to Buildings and Roads/Highways.

3.2.1 Factors that contribute to construction delays

All causes of delays are ranked based on their Relative Importance Index shown below.

Table 5 Ranking of causes of delay in foreign construction projects

Causes of delay	RII	#
Conflicts in sub-contractors schedule in execution of project	3.15	1
Shortage of labours	3.09	2
Delays in sub-contractors work	2.97	3
Delay in approving major changes in the scope of work	2.97	3
Suspension of work by owner	2.88	5
Delay to furnish and deliver the site	2.82	6
Delay in progress payments	2.82	6
Difficulties in financing project	2.73	8
Personal conflicts among labour	2.70	9
Frequent change of sub-contractors	2.70	9
Low productivity level of labour	2.61	11
Work permit of labour	2.61	11
Effects of subsurface and ground conditions	2.55	13
Rework due to errors during construction	2.55	13
Ineffective planning and scheduling of project	2.55	13
Delay in obtaining permits from governed authorities	2.55	13
Delay in material delivery	2.52	17
Changes in material types during construction	2.48	18
Damage of sorted material while they are needed urgently	2.48	18
Conflicts between contractor and other parties	2.45	20
Traffic control and restriction at job site	2.42	21
Implementation of improper construction methods	2.39	22
Weather effect on construction activities	2.36	23
Late procurement of materials	2.33	24

Delay in approving shop drawing and sample materials	2.33	24
Poor communication and coordination	2.33	24
Inadequate contractor's work	2.33	24
Change orders by owner during construction	2.30	28
Delay in revising and approving design documents	2.30	28
Poor communication and coordination	2.30	28
Slowness in decision making process	2.30	28
Poor communication and coordination	2.30	28
Delay in manufacturing special building materials	2.27	33
Delay in performing final inspection and certification	2.24	34
Changes in government regulations and laws	2.13	35
Unclear and inadequate details in drawings	2.12	36
Non-use of advanced engineering design software	2.12	36
Lack of high-technology	2.09	38
Conflicts between joint-ownership of the project	2.06	39
Equipment breakdowns	2.06	39
Shortage of equipment	2.06	39
Delays in site mobilization	2.03	42
Shortage of construction materials in market	2.03	42
Insufficient data collection and survey before design	2.00	44
Delay in providing services from utilities	1.97	45
Mistakes and discrepancies in design documents	1.94	46
Low productivity and efficiency of equipment	1.94	46
Poor qualification of the contractor's technical staff	1.94	46
Low level of equipment-operator's skill	1.91	49
Delays in producing design documents	1.85	50
Inadequate experience of consultant	1.58	51
Accident during construction	1.48	52

- Rank for Relative Importance Index

3.2.2 Factors affecting construction project delays

Delay affecting factors are ranked based on their Relative Importance Index shown below.

Table 6: Ranking of factors affecting delays in foreign construction projects

Causes of delay	**RII**	**#**
Labour Related	2.75	1
Contractor Related	2.51	2
Owner Related	2.46	3
Material Related	2.35	4
External	2.21	5
Consultant Related	2.11	6
Equipment Related	2.01	7

- Rank for Relative Importance Index

3.2.3 Methods of minimizing delays in foreign construction projects

Methods of minimizing delays are ranked based on their Relative Importance Index shown below.

Table 7 Ranking of methods of minimizing delays in foreign construction projects

Minimizing methods	RII	#
Proper project planning and scheduling	3.78	1
Effective strategic planning	3.59	2
Collaborative working in construction	3.56	3
Frequent coordination between the parties involved	3.72	4
Proper material procurement	4.19	5
Accurate initial cost estimates	3.88	6
Site management and supervision	3.81	7
Frequent progress meetings	3.66	8
Use of appropriate construction methods	4.00	9
Proper emphasis on past experience	3.69	10
Clear information and communication channels	4.31	11
Utilization of up-to-date technology	3.56	12
Use proper and modern construction equipment	3.78	13
Complete and proper design at the right time	4.00	14
Compressing construction durations	3.06	15

 # - Rank for Relative Importance Index

3.2.4 Ranking of proper planning activities to avoid construction project delays in foreign construction industries

Proper planning activities are ranked based on their Relative Importance Index shown below.

Table 8 Ranking of proper planning methods to avoid delays in Sri Lankan construction Projects

Proper planning	RII	#
Continuous monitoring	4.27	1
Financial controlling	4.06	2
Material/Equipment controlling	3.81	3
Revising schedule	3.70	4
Legal Permissions	3.67	5
Labour management	3.64	6
Inspection and certification	3.64	7
Services from utilities	3.55	8
Complete site study	3.48	9
Considering weather	3.25	10
Usage of planning software	3.15	11
Staff controlling	3.06	12

 # - Rank for Relative Importance Index

4 CONCLUSIONS

As the final result of this project the factors that contribute to construction delays, mitigation methods to minimize construction delays and proper planning activities to avoid construction delays were categorized and interconnected. This categorization was made on the basis of the results of Relative Importance Index and Factor Analysis. Through this categorization a new guideline named **DMP guideline** was introduced. DMP (D:Delay, M:Mitigation, P:Planning) guideline contains 3 main categories called Category D, Category M and Category P.

Category D (D - Delay factors)
This category contains 8 groups of factors that contribute to construction delays. The groups are named as D1, D2, D3, D4, D5, D6, D7 and D8. All the factors in a group have inter- relationships

among them. The causes of delays were grouped by the result of Factor Analysis and Relative Importance Index using IBM SPSS software.

Table 9: Factors that contribute to construction delays – Category D

D1	Delays in sub-contractors work
	Delay in approving major changes in the scope of work
	Suspension of work by owner
	Delay to furnish and deliver the site
	Delay in progress payments
	Work permit of labour
	Rework due to errors during construction
	Ineffective planning and scheduling of project
	Changes in material types during construction
	Inadequate contractor's work
	Change orders by owner during construction
	Late in revising and approving design documents
	Delay in providing services from utilities
	Low productivity and efficiency of equipment
D2	Poor qualification of the contractor's technical staff
	Low level of equipment-operator's skill
	Delays in producing design documents
	Inadequate experience of consultant
	Accident during construction
	Conflicts between joint-ownership of the project
	Difficulties in financing project
	Mistakes and discrepancies in design documents
D3	Conflicts between contractor and other parties
	Traffic control and restriction at job site
	Poor communication and coordination
	Slowness in decision making process
	Poor communication and coordination
	Delay in performing final inspection and certification
D4	Shortage of labour
	Changes in government regulations and laws
	Unclear and inadequate details in drawings
	Non-use of advanced engineering design software
	Lack of high-technology
	Shortage of construction materials in market
D5	Damage of sorted material while they are needed urgently
	Implementation of Improper construction methods
	Weather effect on construction activities
	Late procurement of materials
	Delay in approving shop drawing and sample materials
	Poor communication and coordination
	Delay in manufacturing special building materials
D6	Equipment breakdowns
	Shortage of equipment
	Delays in site mobilization
	Insufficient data collection and survey before design
	Delay in obtaining permits from municipality
	Low productivity level of labour
D7	Personal conflicts among labour
	Frequent change of sub-contractors

D8	Effects of subsurface and ground conditions (E.g. soil, high water table, etc.)
	Delay in material delivery

Category M (M – Mitigation methods)

This category contains 4 groups of mitigation methods to minimize construction delays. The groups are named as M1, M2, M3 and M4. All the methods in a group have inter-relationships among them. The mitigation methods were grouped by the result of Factor Analysis and Relative Importance Index using IBM SPSS software.

Table 10: Mitigation methods to minimize construction delays – Category M

M1	Use appropriate construction methods
	Proper emphasis on past experience
	Clear information and communication channels
	Use up-to-date technology utilization
	Use proper and modern construction equipment
	Complete and proper design at the right time
M2	Frequent coordination between the parties involved
	Proper material procurement
	Accurate initial cost estimates
	Site management and supervision
	Frequent progress meeting
M3	Proper project planning and scheduling
	Compressing construction durations
M4	Effective strategic planning
	Collaborative working in construction

Category P (P – Planning activities)

This category contains 5 groups of proper planning activities to avoid construction delays. The groups are named as P1, P2, P3, P4 and P5. All the planning activities in a group have inter- relationships among them. The planning activities were grouped by the result of Factor Analysis and Relative Importance Index using IBM SPSS software.

Table 11 Proper planning activities to avoid construction delays – Category P

P1	Revising schedule
	Labour management
	Inspection and certification
	Services from utilities
P2	Continuous monitoring
	Financial controlling
	Usage of planning software
P3	Legal Permissions
	Considering weather
P4	Complete site study
P5	Material/Equipment controlling

Above all categories have interconnections. In the category D, each group was inter connected to the groups of category M and category P. Conclusively, for each cause of construction delay has identified mitigation methods and proper planning activities.

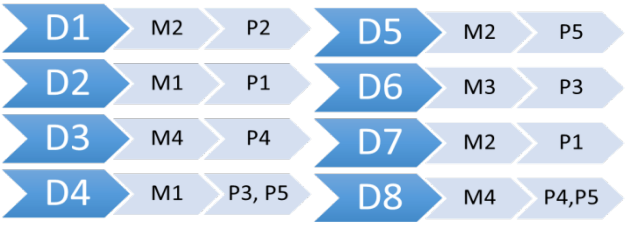

Figure 1 Inter connection among the categories

By identifying specific delay factors, it was able to get the mitigation methods and proper planning activities through the above figure.

5 ACKNOWLEDGMENTS

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Risk Analysis of Building Projects

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Abstract: *The Various types of risks are occurred in any type of construction projects. Risks are categorized as cost related, time related, quality related, environmental related and safety related risks. Risk analysis is an important to overcome various types of risks and achieve the project objectives in terms of cost, time, quality, environmental and safety. This research aims to identify highly affected risk factors in building project. Then Solutions are given to overcome those risks. Questionnaire survey was used to collect data from project participants based on likelihood occurrence and affected level. This research identifies thirty two risk factors which are affected to building constructions in Sri Lanka. Tight project schedule is the most affected risk factor in cost related risk. Most affected risk factors related to time, quality, environmental and safety have been mentioned in the research. Possible reasons and solution are given for those high affected risks. Also risk factors are categorized according to project stakeholders. This research also found that these risks spread through the whole project life cycle and many risks occur at more than one stage, with the construction stage.*

Keywords: *risk, risk analysis, construction projects, project objectives, life cycle, stakeholders.*

1. INTRODUCTION

Numbers of risks are occurred in construction industry. Those risks should be identified and addressed suitably. Risk analysis may be described as “a systematic way of looking at areas of risks and consciously determine how each should be treated. It is a management tool that aims identifying sources of risks and uncertainty, determining their impact, and developing appropriate management responses”. A systematic process of risk analysis has been divided in to risk classification, risk identification, risk analysis. There are many risk classifications. In this research risks are classified as cost, time, quality, environmental and safety.

Previous research has mainly focused on examining the impacts of risks on one aspect of project strategies with respect to cost, time, and safety. Some research has mainly investigated risk analysis in the context of a particular projects phase, such as conceptual /feasibility phase, design phase construction phase, rather than from the perspective of a project life cycle.

1.1. Literature Review

A myriad of risk and risk-related definitions are applied to construction projects, and no standard definitions or procedures exist for what constitutes a risk assessment. In the construction industry there are many definitions. The institute of civil engineers and the institute of actuaries in the United Kingdom (Ramp 1998) define the risk as the presence of potential or actual treats or opportunities that influence the objectives of a project during construction, commissioning, or at time of use Risk Analysis and Management of Projects (RAMP).

Al- Bahar (1990) defines the risk as the exposure to the chance of occurrences of events adversely or favorably affecting project objectives as a consequence of uncertainty.
Plenty of similar definitions can be found from literatures.

Also risk classifications are highly addressed in literatures. Richard D. Land classified risk as in Figure 1.

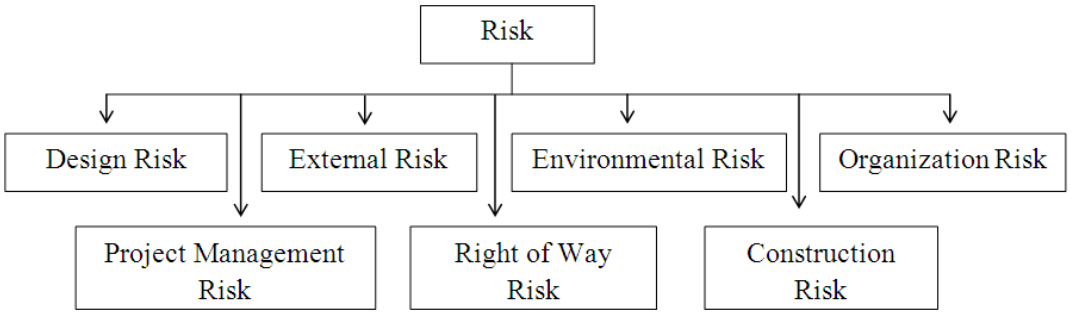

Figure 1 Richard D. Land's Risk Classification

NafisahBinti Abdul Rahiman has classified risk in to seven categories as shown in Figure 2.

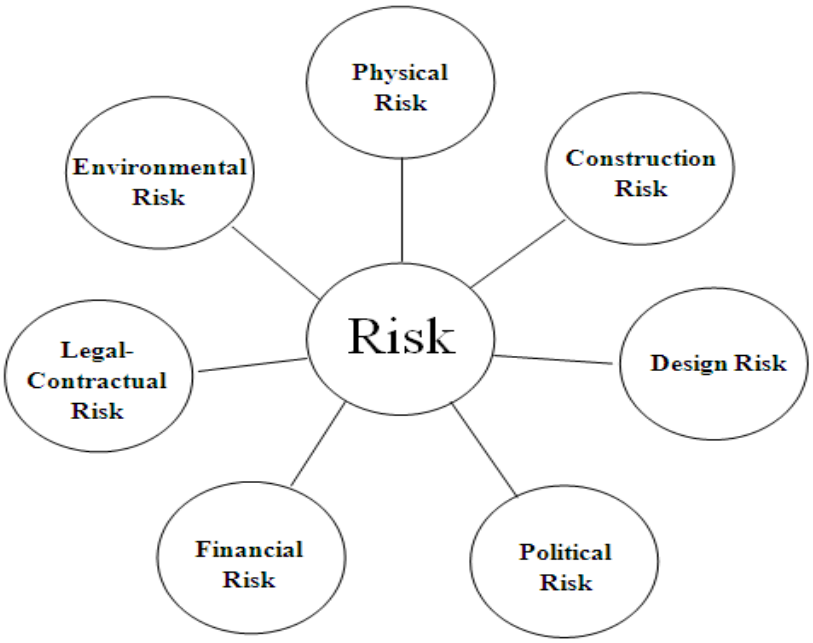

Figure 2 NafisahBinti Abdul Rahiman's classification

Different researchers have identified risk under different categories but there are more or less similarities.

2. METHODOLOGY

Methodology is started mainly from literature survey. It consists of questionnaire preparation, Collecting data, data analysis.

2.1. Questionnaire Preparation

Preliminary questionnaires were prepared for likelihood occurrence and affected level based on cost, time, environmental and safety. For getting information, pilot survey was done for getting ideas from industrial people. A final questionnaire consists of 32 risk factors. Table 1 shows part of the questionnaire of level of impact. Also questionnaire was prepared for likelihood occurrence.

Table 1 Part of the Questionnaire

Risk Type	Risk factors	Affected Level				
Cost related risk	Tight project	1	2	3	4	5
	Occurrence of	1	2	3	4	5
	Location of the	1	2	3	4	5

1- Very Highly
2- Very Highly
3- Medium Affected
4- Lowly affected
5- Very lowly affected

2.2. Data Collection

Professionals in construction industry like project managers, site engineers participated to fill questionnaires. Twenty two questionnaires were filled for both likelihood occurrence and affected level.

2.3. Data Analysis

Significant index method was used to data analysis with five point scale. Equation (1) shows the significant index equation.

$$R_i^k = \frac{1}{n}\sum_{j=1}^{n} \alpha_{ij}\,\beta_i^k \qquad (1)$$

R_i^k = Significance Index Score. (S.I.S)
α_{ij} = Likelihood occurrence of risk.
β_i^k = Level of impact of risk.
n = Total number of valid feedback of risk.

According to scale “very highly” takes a value of 1, “highly” takes a value of 0.8 “medium” takes a value of 0.5, “lowly” takes a value of 0.3 and “very lowly” takes a value of 0.1.Those are valid for both likelihood and level of impact.
Finally possible reasons and solutions were given for highly affected risk factors.

2.4. Sample Calculation

Unsuitable construction program planning is considered for sample calculation under cost related risk.

Calculations are shown in Table 2.

Table 2 Calculation of significant index score

`Form Number	Affected Level 1 2 3 4 5	α	Likelihood occurrence 1 2 3 4 5	β	$r^k_{ij} = \alpha\beta$
1	■	0.8	■	0.3	0.24
2	■	0.5	■	0.1	0.05
3	■	0.5	■	0.3	0.15
4	■	0.5	■	0.5	0.25
5	■	0.8	■	0.5	0.40
6	■	0.3	■	0.1	0.03
7	■	0.3	■	0.5	0.15
8	■	0.5	■	0.1	0.05
9	■	0.8	■	0.5	0.40
10	■	0.5	■	0.8	0.40
11	■	0.8	■	0.5	0.40
12	■	0.5	■	0.5	0.25
13	■	0.5	■	0.3	0.15
14	■	0.8	■	0.3	0.24
15	■	0.5	■	0.8	0.40
16	■	0.5	■	0.3	0.15
17	■	0.8	■	0.5	0.40
18	■	0.5	■	1.0	0.50
19	■	0.3	■	1.0	0.30
20	■	0.8	■	0.5	0.40
21	■	0.5	■	0.3	0.15
22	■	0.5	■	0.3	0.15
					∑αβ= 5.16

$$R_i^k = \frac{1}{n}\sum_{j=1}^{n} \alpha_{ij}\,\beta_{ij}^k$$

$$= \frac{5.16}{22}$$

=0.1984

3. RESULTS AND DISCUSSION

Results were taken from significance index calculations. Calculations were done for all risk factors and then they were sorted from high priority to lower priority. Highly affected risk factors are shown in Table 3, Table 4, Table 5, Table 6 and Table 7 corresponding to cost, time, quality, environmental and safety related risks.

Table 3 Survey Results for Cost Related Risks

	Risk Factors	S.I.S
1	Tight project schedule	0.66231
2	Less optimization usage of materials	0.59500
3	Price inflation of construction materials	0.54692

Tight project schedule is the most affected risk factor in cost related risks. When taking more days to completion of the work, Cost will be increased. Buffer time and extra cost allocation can be given as solution. Material wastage can be minimized to reduce cost.

Table 4 Survey Results for Time Related Risks

	Risk Factors	S.I.S
1	Bad weather condition	0.58346
2	Lack of materials	0.51385
3	Unavailability of sufficient amount of skilled labor	0.46692

Bad weather condition is the most affected risk factor in time related risk. Weather condition of the area should be considered before preparing working schedule. Enough materials are not in construction, construction will be late. Therefore proper plan should be follow in transporting material.

Table 5 Survey Results for Quality Related Risks

	Risk Factors	S.I.S
1	Lack of usage new technology	0.46462
2	Lack of materials	0.44885
3	Unavailability of sufficient amount of skilled labor	0.39846

Lack of usage of new technology is the most affected risk factor in quality related risk. If old machines and equipment are used, required efficiency can't be taken. Therefore required quality can't be expected in construction. When lacking of suitable material, Alternative materials can be used in construction. It may be a reason to reduce required quality.

Table 6 Survey Results for Environmental Related Risks

	Risk Factors	S.I.S
1	Usage of unsuitable machines	0.60269
2	Usage of unsuitable construction methods	0.54846
3	Release of toxic matters	0.38231

Most affected risk factor in environmental related risks is usage of unsuitable machines. When using unsuitable machines in construction, toxic materials toxic gasses can be released as Sulfur dioxide and Nitrogen dioxide. It may cause to environmental damage. This risk can be minimized by using new machines and equipment. To overcome unsuitable construction methods, well experienced person can be appointed to work.

Table 7 Survey Results for Safety Related Risks

	Risk Factors	S.I.S
1	Less attention for safety procedures	0.60692
2	General safety accident occurrence	0.56038
3	Usage of unsuitable and damaged equipment	0.47731

Less attention of safety procedure is most affected risk factor in safety related risks. Safety officer can be appointed to work for overcome less attention of safety procedure. Labours have not enough knowledge about safety procedure. Therefore labours should be informed about safety procedure. General safety accident occurrence can be overcome

4. CONCLUSIONS

While most research has focused on some aspects of construction risk management, this research try to identify key risks associated with the achievement of all project objectives in terms of cost, time, quality, environment and safety. On the basis of a survey with industry practitioners owning robust experience and knowledge of construction projects, 32 key risks were highlighted on a comprehensive assessment of their likelihood of occurrence and level of impacts on project objectives. An innovative attempt to analyze these key risks from the perspectives of project stakeholders and project life cycle presented the following insights.

Clients, designers and government bodies should work cooperatively from the feasibility phase onwards to address potential risks effectively and in time; contractors and subcontractors with robust construction and management knowledge must be employed early to make sound preparation for carrying out safe, efficient and quality construction activities.

Finally we provide the following recommendations.

(1) Clients should know what kind of product they want and clearly define it in the brief.
.

(2) The client, early involved designers and contractors should help clients produce an appropriate project schedule, form reasonable expectation on the product quality, prepare for financial fluctuation such as price inflation of construction material and get documents approved by government agencies in time.

(3) Designers (including consulting engineers) should carry out in-depth investigation of site conditions at first, articulate the clients' needs in a technically competent way within the limitation of the clients' resource and work collaboratively to minimize the design and cost variations.

(4) Government bodies should avoid bureaucracy and create a swift environment to support the project development while the project team should always maintain close relationship with the government officers to shorten the time for approvals.

Although some risks in project feasibility and design stages also extend their occurrence and influence to the post-design stage, most risks associated with the construction are more likely to root in contractors and subcontractors. In this phase, the design is fixed, the project progress no longer depends on creating a realistic schedule but on sticking to it, and budgetary risk is no longer a matter of pricing but that of cost control. To keep the construction work on track, experienced contractors need to be involved in the project as early as possible to make sound preparations for developing valid construction programs. On the other hand, contractors need to establish a highly cooperative construction team in which competent specialist contractors and skilled labours are staffed, and communication, trust, commitment and integration is expected to bridge the physical and knowledge gap between different project participants. With maximum team efforts, construction programs can be well executed, and negative issues associated with construction such as friction, inefficiency, duplication of effort, accident and pollution can be significantly minimized.

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Comparing Health Risks among Vehicle Operators in Construction Sites exposed to Whole-Body Vibration Using EU directive 2002/44/EC and ISO 2631-1

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Abstract: *In Sri Lanka, although workers in construction sites face with several risk factors, they are less aware about risk factors such as exposing Whole-Body Vibration (WBV). Two of main legislations, ISO 2631-1 and EU directive 2002/44/EC, use average weighted RMS acceleration and daily vibration exposure value A(8), respectively, to predict the health risk and discomfort of workers. The purpose of this study was to evaluate health risks and discomfort among vehicle operators exposed to Whole-Body-Vibration (WBV) based on the criteria given in EU directive 2002/44/EC and ISO 2631-1. Many of operators selected in this study are exposed to vibrations that exceed the Exposure Action Value (EAV). Among the operators, only the Soil roller compactor operator was in the Health Caution Zone as specified in ISO 2631-1. Nevertheless all the operators were in extremely uncomfortable level besides Front end loader operator and the Dozer operator. Therefore, interventions, such as seat redesign, introduce a policy for buying low vibration vehicles and machinery, choose the right vehicle for the ground surface and task, introduce several rest breaks while operating the machineries, effective training on operating the large vehicles can positively affect WBV exposure and improve operators comfort.*

Key words*:* *EU directive 2002/44/EC, Exposure Action Value (EAV), Health Risks, ISO 2631-1, Operators in Construction Sites, Whole-Body Vibration (WBV)*

1 INTRODUCTION

Site safety was an increasingly interesting topic in the past decade. Workers in construction sites face with several risk factors. It has been found that risk factors such as noise and vibration exposures affect the workers, although the workers are less aware about these risk factors than the other risk factors such as working in higher places (Vitharana *et al,* 2013). Vibration exposure significantly affects the workers, although the workers are less aware about vibration.

Although discomfort can be subjectively evaluated, legislations and standards predict discomfort based on vibration magnitude and durations. Two of main legislations, ISO 2631-1,1997 and EU directive 2002/44/EC, use average weighted RMS acceleration and daily vibration exposure value A(8), respectively, to predict the health risk and discomfort of workers. In 2002, directive 2002/44/EC was adopted. This is known as the physical agents (vibration) directive. It applies the principles of the Framework Directive to risks arising from hand- arm vibration and Whole-Body Vibration (WBV).

The Root Mean Square value (RMS value) is obtained by taking the square root of the mean of the sum of the squares of the instantaneous accelerations measured during the total measurement time, T. If the RMS value is calculated over a relatively long time interval, e.g. 1 minute or 1 hour, the RMS value gives an acceleration value, which is related to the energy content of the vibration.

$$a_w\ RMS = \frac{1}{T}\left(\int_0^T a^2(t)dt\right)^{1/2} \quad (1)$$

However, if the RMS value is calculated over a much shorter time interval e.g. 1 second, the RMS value is known as the "instantaneous" or "running" RMS value. The crest factor defines the ratio between the peak value and RMS value for the measurement time, T.

The peak value is the maximum instantaneous acceleration measured during the measurement time, T. It is a useful indicator of the magnitude of short duration shocks (Mansfield, 2005).

International Standard ISO 2631-1, 1997 includes revised frequency weightings, calculation of a single overall RMS acceleration and the introduction of methods for assessment of WBV containing shocks (Monaghana and Twest, 2004). In order to assess a vibration which takes place in more than one direction simultaneously, the ISO 2631-1 suggests that the effect of such a vibration can be calculated by taking the vector sum, a, of the three weighted acceleration values, a_x and a_y and a_z as follows:(International organization for standardization, 1997)

$$a = \sqrt{[(1.4a_x)^2 + (1.4a_y)^2 + a_z{}^2]} \quad (2)$$

The daily exposure action and limit values in the directive are all specified as an eight-hour equivalent frequency weighted acceleration (known as the A(8) value), although Vibration Dose Value (VDV) alternatives are given for WBV(Nelson and Brereton, 2005). The assessment of the level of exposure of vibration is based on the calculation of daily exposure A(8) expressed as equivalent continuous acceleration over an eight- hour work period. For the determination of A(8) it is not necessary to measure over eight hours. It is sufficient to make short- term measurements during representative work steps. These measurements are normalised to eight hours. Daily exposure is calculated as follows:

$$A\ (8) = a_{we}\sqrt{\frac{T_e}{T_0}} \quad (3)$$

Where a_{we} is the energy equivalent, mean value of the frequency weighted acceleration during the exposure.

T_e is the total duration of exposures during one-week day.
T_0 is the reference duration of 8 hours.

The present study predicts the health risk and discomfort of the vehicle operators in the construction sites, based on criteria established in ISO 2631-1,1997 and EU directive 2002/44/EC. Vibration exposure level of 22 construction workers (i.e., seven Soil roller compactor operators, five Motor grader operators, four Excavator operators, two Skid steer loader operators, two Pile machine operators, one Front-end loader operator and a Dozer operator) were measured by using a Tri Axial Seat Pad Accelerometer.

2 METHODOLOGY

Building and road construction sites, in Southern and Western provinces, were selected. Contractors of these construction sites had registered under grading M1 by the Institute for Construction Training and Development (ICATAD).Vibration exposure levels of three Excavator operators; three Soil roller compactor operators, three Motor grader operators, two Skid steer loader operators and a Pile driving machine operator were measured at the driver-seat interface during actual operating conditions, according to the recommendations of the International Standard ISO 2631-1(1997).

Tri-axial vibration meter and a seat pad accelerometer were used to measure the vibration exposure level of operator. The seat pad accelerometer was placed on the seat of the operator and connected to the tri-axial vibration meter as shown in Figure1. Then, the operator was instructed to sit on the seat pad. In the tri-axial vibration meter, frequency weighting of W_d , W_k and measurement range of 10 ms^{-2} were selected. As the operator was in seated position, frequency weighting, W_d, was used to weight the vibration in X and Y directions. Frequency weighting, W_k, was used to weight the vibration in Z direction. Figure 2 shows the basic centric axes (X, Y and Z-axis) at seat back, seat surface and feet of the human body in seated position.

Directional factors k_x, k_y , k_z were set as 1.4, 1.4 ,1.0, respectively. Measured vibration exposure levels were assessed by limit boundaries of vibration exposures as defined by the ISO 2631-1 (1997) (i.e. mentioned in Table 1).

A questionnaire form was developed for the equipment operators in construction sites to record subjective responses. Vibration exposing hours per day, working experience and vibration related symptoms of the worker were found through the questionnaire survey.

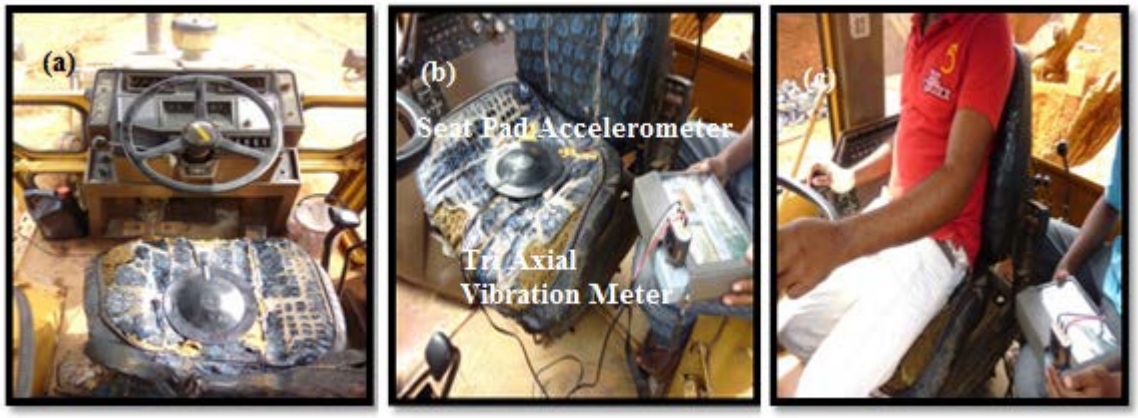

Figure 1 Procedure of measuring vibration exposure level of an operator (a) Seat pad accelerometer on the seat of the operators, (b) Seat pad accelerometer with the tri-axial vibration meter, (c) Operator sitting on the seat pad accelerometer

Table 1 Approximate Indications of Likely Reactions to various Magnitudes of Overall Vibration Total Value in Public Transport as stated in ISO 2631-1 (1997)

Frequency – weighted Vibration Magnitude (Root-Sum- of- Squares)	**Likely Reaction in Public Transport**
Less than 0.325 ms^{-2}	Not uncomfortable
0.315ms^{-2} to 0.63ms^{-2}	A little uncomfortable
0.5ms^{-2} to 1ms^{-2}	Fairly uncomfortable
0.8ms^{-2} to 1.6ms^{-2}	Uncomfortable
1.25 $ms^{-2}$to 2.0ms^{-2}	Very Uncomfortable
Greater than 2ms^{-2}	Extremely Uncomfortable

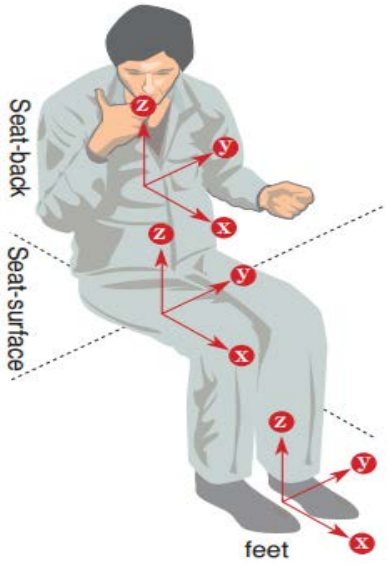

Figure 2 Basic centric axes of the human body in seated position

3 RESULTS AND DISCUSSION

Table 2 presents the average weighted acceleration, crest factor, daily vibration exposure A(8), likely reaction and health risk according to the ISO 2631-1. WBV daily Exposure Action Value (EAV) is 0.5 ms^{-2} and WBV daily Exposure Limit Value (ELV) is 1.15 ms^{-2} according to the EU directive 2002/44/EC.

Table 2 Average vibration exposure of operators of construction machineries

Operator	Average Value(RMS) m/s^2	Crest Factor		Daily Vibration Exposure A(8) m/s^2	Risk Level According to the EU directives	Likely Reaction according to the Public Transport as stated in ISO 2631-1	Health Risk according ISO 2631-1
Soil Roller Compactor Operator	2.19 to 4.69	X axis	5.45 to 14.73	1.10 to 2.48	Above ELV	Extremely Uncomfortable	Caution Zone
		Y axis	5.22 to 22.27				
		Z axis	3.7 to 12.52				
Motor Grader Operator	0.88 to 2.06	X axis	16.05 to 6.52	0.44 to 1.03	Above EAV and below ELV	Extremely Uncomfortable	Below the Caution Zone
		Y axis	20.58 to 6.61				
		Z axis	13.02 to 5.66				
Excavator Operator	1.70 to 2.20	X axis	27.22 to 7.00	0.85 to 1.10	Above EAV and below ELV	Extremely Uncomfortable	Below the Caution Zone
		Y axis	17.21 to 10.04				
		Z axis	11.31 to 22.11				
Skid steer	2.01 to	X	10.35	1.00 to	Above	Extremely	Below the

loader Operator	2.18	axis	to 34.91	1.09	EAV and below ELV	Uncomfortable	Caution Zone
		Y axis	6.99 to 18.26				
		Z axis	11.10 to 16.02				
Pile driving Machine Operator	1.82 to 2.20	X axis	7.39 to 11.58	0.91 to 1.10	Above EAV and below ELV	Extremely Uncomfortable	Below the Caution Zone
		Y axis	7.21 to 11.74				
		Z axis	4.20 to 5.09				
Front end loader Operator	1.09	X axis	6.95	0.54	Above EAV and below ELV	Uncomfortable	Below the Caution Zone
		Y axis	5.12				
		Z axis	11.11				
Dozer Operator	0.94	X axis	9.84	0.47	Below EAV	Uncomfortable	Below the Caution Zone
		Y axis	9.44				
		Z axis	8.30				

Many of operators selected in this study were exposed to vibrations that exceed the EAV (Table 2). Only the Dozer Operator does not reach the level EAV. However, the workers do not reach the vibration exposure level of ELV besides Soil Roller Operators.

If the EAV is exceeded, an appropriate formalized preventative program must be set up by the employer to reduce a minimum exposure to a vibration and the attendant risks. Operators must not be exposed above the ELV and the directive includes a requirement for immediate corrective action by the employer if the ELV is exceeded. Nevertheless, vibration directive suggested the duties of employers to carry out a risk assessment competently and identify the measures required to control or manage the risk. It requires the daily exposure to vibration to be assessed and compared with the EAV and ELV, but the employer must also consider other risk factors, such as intermittent exposure, repeated shocks, workers who may be particularly sensitive to vibration (Nelson and Brereton, 2005). It is fair to consider the repeated shocks.

Among the operators, only the Soil roller compactor operator was in the Health Caution Zone as specified in ISO 2631-1 (Table 2) . Nevertheless all the operators were in extremely uncomfortable level besides the Front end loader Operator and the Dozer Operator. Both the Front end loader Operator and the Dozer operator were in uncomfortable level too; despite being the crest factor of the exposing level of vibration of operators are high.

Therefore, WBV is prevalent in Sri Lankan construction industry, with many workers are exposed and with the absence of mandatory standards; effort should be made to increase knowledge and awareness of the subject through professional education (Vitharana *et al*, 2014).

4 CONCLUSIONS

It has been found that risk factors such as noise and vibration exposures affect the worker, although the workers are less aware about these risk factors than the other risk factors such as working in higher places.

The present study predicts the health risk and discomfort of the vehicle operators in the construction sites, based on the criteria established in ISO 2631:1997 and EU directive 2002/44/EC. Vibration exposure level of 22 construction operators (i.e., seven Soil roller compactor operators, five Motor grader operators, four Excavator operators, two Skid steer loader operators, two Pile driving machine operators, one Front-end loader operator and a Dozer operator) were measured by using a Tri Axial Seat Pad Accelerometer

Many of operators selected in this study were exposed to vibrations that exceed the Exposure Action Value. Among the operators, only the Soil roller compactor operator was in the Health Caution Zone as specified in ISO 2631-1. Nevertheless all the operators were in extremely uncomfortable level besides Front end loader operator and the Dozer operator. Both the Front end loader operator and the Dozer operator were in an uncomfortable level too, despite being the crest factor of the exposing level of vibration of workers are high.

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Development of a Walkability Evaluation Instrument to Evaluate Walkability in Streets of Developing Countries

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Abstract: *Walkability Evaluation Instruments (WEIs), evaluating influence of the physical environment on walkability are important for creating walkable streets. This paper presents on development of a WEI named WETDC (Walkability Evaluation Tool for Developing Countries) for streets of developing countries. Three separate instruments were developed for three identified road categories namely Local, Collector & Distributor and Arterial roads. The instrument was developed based on a three-level hierarchical model in which level 1 parameters represented walking needs, level 2 parameters represented physical elements influencing respecting walking needs and third level representing different categorical data for physical elements. Based on previous research work, six walking needs and weights explaining their influences were identified. By getting the contribution of a panel of 50 local and international transportation professionals, influential physical parameters and weights to represent their influence were developed. Reliability test revealed the instrument to be reliable. Further the validity of developed instrument was substantial for arterial and, collector & distributor road while being moderate for local road. Thus for a given road if the relevant road category and influential physical parameters are known, it is possible to quantify the walkability level.*

Keywords: *Walkability Evaluation Instrument, Pedestrian, Local road, Collector and Distributor road, Arterial roads, Physical Environment.*

1. INTRODUCTION

According to 2005 figures (IEA, 2009) transportation sector is responsible for 23% of world carbon emissions. Walking for transportation is being promoted as one way to overcome the above issue. With physical environment being identified as a factor governing a walking decision, many measures are taken to encourage walking by improving the physical environment (Pikora, 2003). Such improvements should start with the detailed objective evaluation of the existing condition of the physical environment that may support or hinder walking. This has led to development of many Walkability Evaluation Instruments (WEIs). Investigation about instrument developed to-date reveals that, most are based on pedestrian friendly physical environments in developed countries. Usage of such instruments has a limited applicability in poor pedestrian environments of the developing world, This research attempted to develop a such instrument, WETDC (Walkability Evaluation Tool for Developing Countries) to evaluate the walking environment in developing countries.

2. LITERATURE REVIEW

2.1. Considerations in developing a WEI

In developing a WEI, due consideration should be given to the approach of evaluating the walkability, conceptual model used for WEI, selection of significant walkability correlates to be used as input parameters, and the methods of establishing construction validity and reliability of

the instruments. In evaluating walkability some instruments (SPACES- Systematic Pedestrian and Cycling Environment Scale, PEDS- Pedestrian Environment Data Scan, IM – Ivrine – Minnesota Tool) simply attempt to check the presence of some pre-identified influential physical features. Such instruments will not quantify the overall walkability of the street making those less useful. As an example these will inform whether the pedestrian friendly features such as sidewalk, shade, smooth walking area are present or not. But these will not quantify the ultimate walkability such as whether the place is highly walkable, average walkable or poorly walkable. A good WEI should not only judge the presence of influential physical elements, but also produce the ultimate walkability level based on the presence of such elements. Ewing et al., (2006) evaluating the walkability of USA streets based on urban design qualities and Samarasekara, (2007) evaluating the walkability of tourists in Japanese streets have produced such WEIs. Presented here is an attempt to develope a WEI, capable of evaluating the ultimate walkability by judging the presence of influential physical environmental correlates.

2.2. Conceptual model for WETDC

According to Ewing et al., (2006), a walking decision is mediated through perceptions of different physical environment features instead of the direct influence such features. The WEIs developed by Ewing et al., (2006), and Samarasekara, (2010)]are based on this concept of walking decisions mediated. Balasooriya et al. (2011), have developed such a conceptual model consisting of a two level hierarchy to describe how the influence of physical environment on the walking decision of a Sri Lankan pedestrian would be mediated through such perceptions. Figure 1 depicts the conceptual model used in this research. As shown the figure, the model of Balasooriya et al., (2011), described that the decision to walk is influenced by the level 1 walking needs which in turn is influenced by the level 2 physical features. The level 1 of their model consisted of six such perceptions named as walking needs which were, Traffic safety in walking area, Environmental Appearance, Sense of Space, Design comfort of walking area, Illegal invasion of pedestrian area and Shade. For each of the aforementioned walking need, several level 2 detailed parameters representing physical environmental features which would create such a walking need were also introduced by the authors. As an example the level 1 walking need of “Traffic Safety” was represented at level 2 by the parameter “Presence of sidewalk separation”. Since the above model is developed in Sri Lanka which is a developing country, it was used as the base model in creating the WEI for developing country.

In the above model, the authors have comprehensively identified the level 1 walking needs and have estimated a set of weights to describe their relative influence. But their work related to level 2 parameters had limitations. The research has not produced weights to describe the relative influence of the parameters. Further these parameters consisted of both objective parameters and subjective parameters and had not been transformed in to measurable parameters that are used in a WEI.

Thus the development of the WEI was done by expanding the model of Balasooriya et al. (2011), beyond the level 1 parameters of walking needs.

2.3. Separate WEIs for separate road classes

The road infrastructure and the road side infrastructure can vary significantly based on road class. Thus any such differences would again lead to differences in walking decision. In an arterial road, several pedestrian safety measures like comfortable pedestrian walking surfaces or pedestrian separation devices can be observed. But in a local road pedestrian safety may only be ensured by the white line demarcation. Thus in order to incorporate such differences at physical features level, it is important to devise different WEIs for different road types. But when the process of taking a walking decision is considered, irrespective of differences among physical features which would generate the walking need, the walking decision is taken based on the six perceptions which would not differ based on road type. Thus at the level 1 of the instrument, the same set of walking needs and their weights as proposed by Balasooriya et al. (2011), were used. But in extending beyond that level, three different instruments were devised for the three commonly observed road classes, which are Arterial roads, Collector and Distributor roads and Local roads.

3. METHODOLOGY

The WETDC was developed based on the three level hierarchical model depicted in Figure 1. The work was conducted in below mentioned four stages.
Stage 1: Identification of the level 2 detailed parameters and develop weights for those
Stage 2: Identification of the level 3 variables and develop weights for those
Stage 3: Checking the reliability of WETDC using inter-rater agreement
Stage 4: Checking the predictive validity and calibration of the model using psychological experiment based on photo simulation

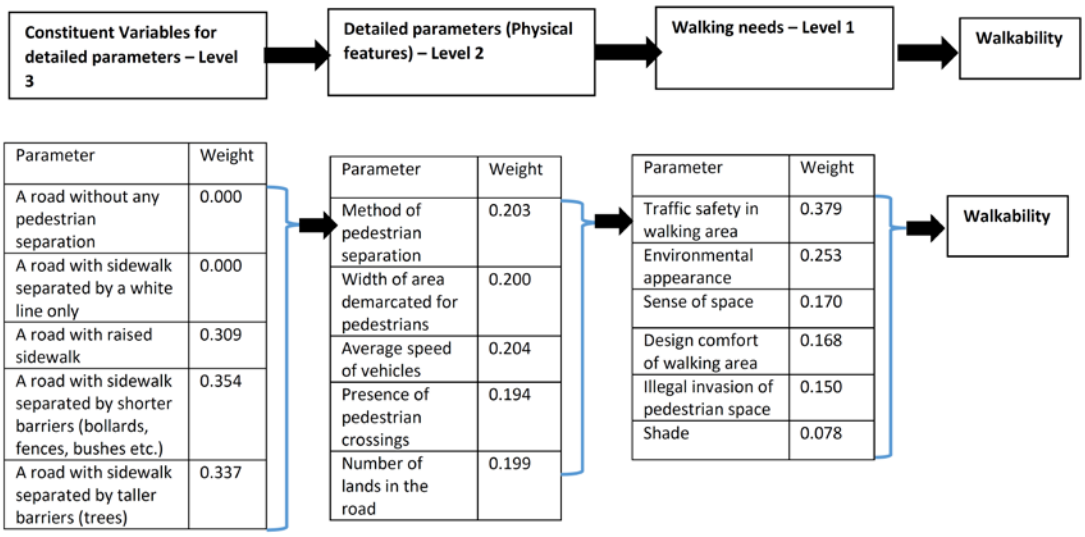

Figure 1 Hierarchical model for walkability evaluation

3.1. Stage 1: Identifying detailed parameters and developing weights

Approach: In identifying the relevant parameters and developing weights for the SPACES, Pikora et al., (2003), conducted a Delphy study by getting the participation of a set of international experts. Based on previous literature and experience, the authors there, initially proposed a set of potential parameters. Through the opinion of the international experts, this list was then modified through additions and deletions to finalise the set of relevant parameters. Thereafter, based on their perception of the relative influence of the selected parameters on the particular walking need, the set of relative weights were devised. In this research also a simular approach has been used.

Participants: Fifty national and international professionals whose work involves or has involved road planning/design/construction/ maintenance and management of developing countries were selected for the questionnaire. Participants were selected through requests made personally and through professional networks. Among them 43 participants completed the questionnaire in stages 1 and 2. The participants either answered the questionnaire hosted on the web or emailed.

3.1.1. Methodology

Initial proposal of level 2 parameters was done based on the level 2 parameters identified by Balasooriya et al. (2011). Since some of their parameters suffered from the issues of subjectivity and measurability, only those parameters which were objective and measurable were selected. In order cover any gaps, additional parameters were introduced from previous research work of Pikora et al., (2003), McCormack, (2006), Samarasekara (2010) and Ewing et al. , (2006).

This list was then transformed in to a questionnaire, consisting of potential level 2 parameters for each walking need. The participants were requested to rate the degree of influence of each

of the proposed parameter on the respective walking need by selecting whether they are Very Important, Important and Not Important. Three separate questionnaires were prepared for the three road classes. Further they were given a chance to propose new level 2 parameters which they believed to be important.

Data Analysis: The answers were transformed in to a Likert scale of Importance which was a 3-point scale of 1 to 3. The values 1, 2 & 3 reflected the conditions of Not Important, Important, Very Important. The parameters qualified to be level 2 parameters were selected based on the median values and inter-quartile range. Those had a median value more than 2.0 and an inter quartile range of less than 0.5 were qualified to be level 2 parameters. For the selected parameters the Importance Value was calculated by taking the average value across participants. In order to calculate the ultimate walkability, the calculation of each level 1 walking need will have to be done by taking the weighted average of the level 2 parameters relevant to that walking need. For each walking need, the applicable level 2 parameters were available present with their Importance Values. These Importance Values reflected how strongly each parameter would contribute to respective walking needs. Thus the weights for the level 2 parameters relevant to a particular level 1 walking need, were obtained by transforming the ratio of Importance Values of the level 2 parameters, in to weights. The weights are found in the developed walkability index available with the first author.

3.2. Stage 2: Identifying level 3 parameters (variables) and developing weights

3.2.1. Approach

Participants & Data Analysis: Each level 2 parameter reflected a feature of the environment which could exist in the form of a categorical data. As an example the Slope of the road could belong to one of the categories a flat slope, a mild slope or a steep slope. Thus the work of this stage was intended to identify those categories (level 3 parameters) and to develop the relevant weights. The level 3 parameters and weights were obtained only for those level 2 parameters selected in stage 1. The parameter identification was done as adopted in stage 1. A similar questionnaire was distributed among those who participated in stage 1. The parameter selection and the calculation of weights were also done using a similar method to stage 1. The weights are available with the first author.

3.3. Stage 3: Checking the reliability of WETDC using inter-rater agreement

3.3.1. Approach

One important characteristic of WEI is its reliability. A reliable instrument will ensure that the same result is produced when a street is evaluated by two independent evaluators. Thus the reliability WETDC was checked based on the evaluation of two trained independent evaluators.

3.3.2. Method

A pool of 30 streetscape images were created by selecting ten images from each of the three road classes. The two trained evaluators independently evaluated the images using the WETDC.
Data Analysis: Agreement between the two evaluators was calculated using Kappa statistics for categorical data. The results are shown in Table 1.

3.4. Stage 4 Checking the predictive validity and calibration of the model

3.4.1. Approach

The predictive validity of a WEI would ensure whether the instrument will be able to predict

accurately what it intends to predict. Thus for a set of selected streets, by understanding the correlation between the walkability level as perceived by a set of road users and by the walkability values predicted by WETDC, validity was checked. Also using same data, the threshold levels of WETDC were obtained.

3.4.2. Method

For each of the three road types a set of 50 representative images taken under standard condition were selected from a pool of images. The walkability levels of each street depicted in the image was calculated using WETDC. In a psychological experiment, these images were shown to a set of 80 undergraduate students. They evaluated the walkability of the given images on a 5 point likert scale of 1 to 5 where 1 reflected "Very bad for walking" and 5 reflected "Very good for walking".
Data Analysis: The perceived walkability level of a given street was calculated by taking the average of the 80 participants. The correlation between such average and the walkability value from WETDC was studied. The results for three street types are shown in Figure 2, 3 and 4.

4. RESULTS

4.1. Stage 1: Level 2 parameters and weights

As depicted in the developed index, 23 level 2 parameters were proposed to represent the six walking needs. Only two new parameters were suggested for inclusion with both having a frequency of one. Since these reflected a marginal case, those were not incorporated for further stages.

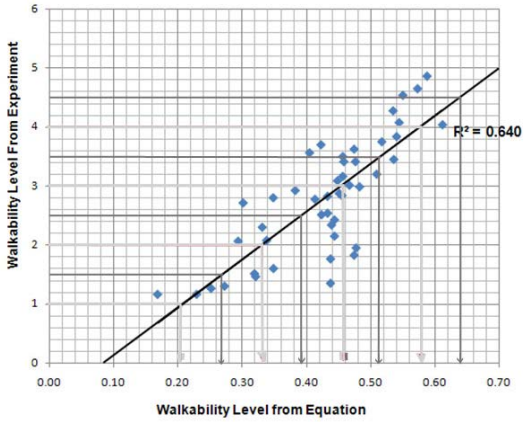

Figure 3 Correlation between Walkability level from Experiment and from WETDC for Arterial Roads and establishment of thresholds

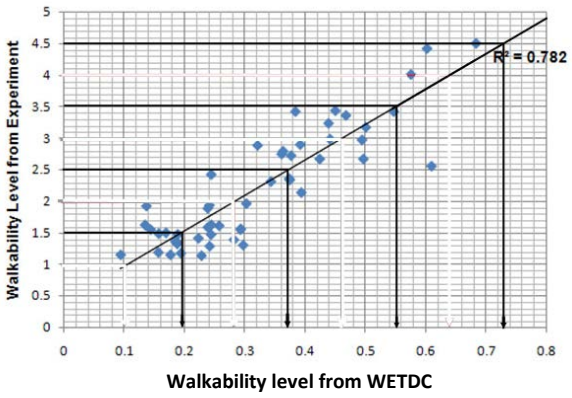

Figure 4 Correlation between Walkability level from Experiment and from WETDC for Collector Distributor Roads and establishment of thresholds

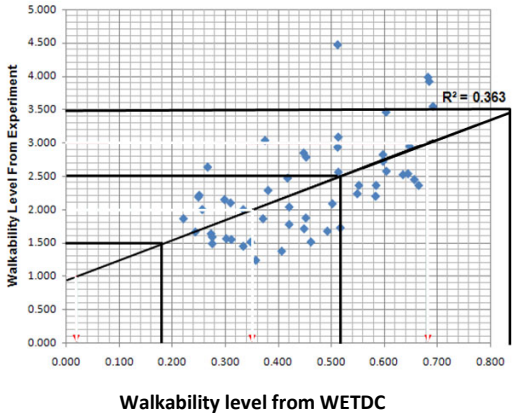

Figure 5 Correlation between Walkability level from Experiment and from WETDC for Local Roads and establishment of thresholds

Based on the results of this stage the influential level 2 parameters were selected. In the case of Arterial roads all 23 proposed parameters qualified to be influential parameters. But in the case of Collector Distributor roads only two parameters namely, Presence of attractive built elements/architectural elements and Presence of attractive street furniture were excluded. In the case of Local roads 14 parameters were rejected. Parameters such as Method of pedestrian separation or Number of lanes in the road were among the rejected. A local road generally will have neither means of pedestrian separation nor multiple lanes per direction. Therefore the above parameters will have no relevance in the context of a local road. Thus this confirms the sensitivity of the developed walkability index consists of only the parameters relevant to the context.

Table 1 Results of Reliability testing

Main Factors	Detailed Parameter		Kappa's Coefficient		
			Arterial Road	Collector Road	Local Road
Traffic Safety	1	Method of pedestrian separation	0.583	0.545	-
	2	Width of area demarcated for pedestrians	0.400	0.600	-
	3	Average speed of vehicles	0.523	0.744	0.600
	4	Presence of pedestrian crossings	0.444	0.655	-
	5	Number of lanes in the road	0.583	0.667	-
Environmental appearance	1	Presence of well-maintained/attractive trees	0.583	0.615	0.730
	2	Presence of well maintained / attractive flowers/shrubs/ grass	0.444	0.545	-
	3	Presence of attractive built elements/architectural elements	0.444	0.600	-
	4	Presence of attractive street furniture	0.545	0.583	-
	5	Presence of litter/ garbage/graffiti	0.474	0.600	0.600
	6	Street maintenance	0.800	0.600	0.615
Sense of space	1	Width of pedestrian area	0.655	0.524	-
	2	Width of the road	0.600	0.615	0.600
	3	Presence of obstructions in pedestrian area	0.520	0.524	-
Design comfort of the walking area	1	Condition of the walking area	0.545	0.800	0.545
	2	Paving material of walking area	0.600	0.545	-
	3	Gradient of the road	0.615	0.737	0.782
	4	Evenness of walking area	0.615	0.545	0.583
Illegal invasion of pedestrian space	1	Presence/ level of street side activities using pedestrian space	0.583	0.615	-
	2	Presence/ level of illegal parking on the pedestrian space	0.583	0.600	0.524
	3	Usage/level of pedestrian area by drivers	0.782	0.524	-
Shade	1	Canopy Shape	0.583	0.545	-
	2	Percentage of shaded pedestrian area during noon	0.600	0.545	-

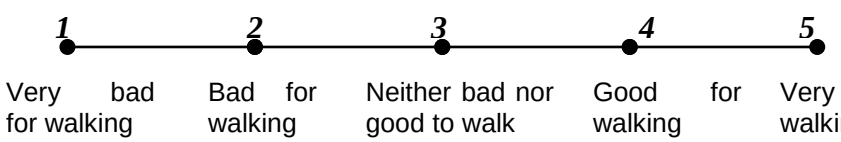

Figure 2 Walkability scale used for psychological experiment and adopted for WETDC

4.2. Stage 2: Level 3 parameters and weights

Similar to stage 1, in this stage a set of level 3 parameters were finalised along with weights to describe their influence. As observed in the previous stage some of the irrelevant parameters, especially in the context of local road were rejected.

4.3. Stage 3: Reliability Check

The agreement levels of Kappa coefficients would be almost perfect if the value is more than 0.80, substantial of the values lie between 0.60 and 0.80 and moderate if the value lie between 0.40 and 0.60. (Landis & Koch., 1977). As revealed by Kappa coefficients, the instrument holds a moderate degree of reliability for most parameters and substantial or almost perfect reliability for the other elements.

4.4. Checking the predictive validity and calibration of the model

The correlation coefficients reveal the predictive validity of the three instruments developed. Accordingly the instruments for Arterial roads and Collector Distributor roads show strong correlation whereas the instrument for Local roads showed moderate correlation.

Also based on the outcomes, the thresholds for walkability index were also developed. The values are shown in Table 2.

Table 2 Thresholds for WETDC

Walkability Level from Rating Scale		Thresholds for WETDC		
Implication	**Thresholds in Rating Scale**	**Arterial Road**	**Collector Distributor Roads**	**Local Roads**
Very bad	< 1.5	< 0.27	< 0.2	< 0.2
Bad	1.5 – 2.5	0.27 - 0.39	0.2 - 0.37	0.2 - 0.52
Neither bad nor good	2.5 – 3.5	0.39 - 0.51	0.37 - 0.55	0.52 - 0.82
Good	3.5 – 4.5	0.51 – 0.64	0.55 – 0.73	>0.82
Very good	>4.5	>0.64	>0.73	

5. DISCUSSION

This paper presents the development of WETDC, a WEI to be used for streets of developing countries. The two main challenges faced during the development of a WEI is the parameters selection and the development of weights to represent their relative influence. Once the WEI is developed additional work has to be done to be check for reliability and for validity. The cumbersome nature of completing all four above mentioned aspects have compelled many researches to develop instruments which addresses only a few of those. But the authors here have developed the WETDC by incorporating all four above mentioned considerations.

For most instruments, the parameter selection is initially done by starting from a set of parameters found to have a significant correlation to walkability. By considering the other physical environment features, some more parameters would be added to this set, ensuring the comprehensiveness. Although a comprehensive set would be resulted in, if the process is not followed by a systematic or experimental selection, the resultant instrument will consist of many unnecessary parameters.

The systematic development of weights is another important yet cumbersome task. Due to the heavy work load involved many researches forgo this step by assuming equal weights. But in this work the authors with the aid of a questionnaire survey adopted a systematic process to select parameters and develop weights. With the predictive validity being confirmed, this instrument thus has been successful

in terms of parameters selection and weight development.

Almost all the instruments developed to date have been tested for reliability. But only a few a tested for validity. This instrument has confirmed its reliability through a inter-rater agreement. Also its predictive validity has been confirmed for all three street types using a psychological experiment.
WEIs are expected to serve two functions. The instrument should be capable of calculating the walkability level and meaningful associations should be established to the calculated walkability values As an example once the walkability level is calculated to be value “n”, it should meaningfully express how walkable it is by stating whether it is too bad for walking or very good for walking. This will enable a meaningful comparison of the walkability status across different streets.

The instrument can also be utilized to propose improvements to increase walkability of existing streets. While the walking needs at the level 1 can suggest overall strategies of improvement, the second and third level parameters can suggest corresponding physical elements for improvement. Further, the instrument can objectively be used to judge the effectiveness of potential alternatives to improve walkability. The road authorities can use the relevant instrument to judge the walkability levels of existing streets to identify the streets requiring improvement. Once the improvement need is identified they can further utilize the instrument to identify effective improvement strategies.

In developed countries neighbourhood walkability level is one parameter which governs the sales of real estate sector. Thus the buyers seeking potential residences can use this instrument to judge the walkability associated with such streets. This instrument is simple enough for a average person to conduct the calculation by himself.

The predictive validity checking and calibration of the instrument was done by getting the perceptions of a student sample who presented a bias sample especially in terms of demographic characteristics. Pedestrians from special populations such as disabled people, elderly people or children could require additional support from the physical environment facilities beyond what has been found in this study. Future work should attempt to generalize these results to other user groups. In a previous research work comparing the performance of students and non- students, Stamps 2013, however, found a high correlation between the two groups in relation to the preference for scenes. The desire to walk is closely related to preferences. Thus the possibility of generalizing these outcomes to other groups cannot be ruled out.

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Exposure Level of Noise and Vibration among Construction Workers and Their Health Effects

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Abstract: *Noise and vibration are some of the most highly found physical contaminants in the construction sector. The most probably the negative effects caused by noise and vibration exposure are hearing loss and low back pain, respectively. As these affect on people's health and well-being, it is necessary to assess the noise and vibration exposure levels and to develop mechanisms for studying and proposing necessary preventive solutions.*

This investigation has studied the noise and vibration levels, in which the workers of the construction sites are exposed to. A number of noise and vibration measurements have been performed using a sound level meter and a tri-axial vibration meter, respectively. A-weighted decibel values (dBA) and Vibration total values (ms^{-2}) were determined and they were compared with the limits imposed by the different current regulations such as Occupational Safety and Health Administration (OSHA), International Organization for Standardization (ISO), National Institute of Occupational Safety and Health (NIOSH) and World Health Organization (WHO).

Key words: *Noise, Vibration, Construction sites, Back pain, Hearing loss*

1. INTRODUCTION

Noise is the most persistent physical contaminant in human environment. The technological development and the growth of population are key factors in the increase of noise pollution. It is difficult to define what noise means as a physical contaminant. Normally, it is kind of undesired sound. That concept noise is bond with a subjective perception, and therefore, a sound can be pleasant for some people but, at the same time, can also be annoying for some other people, or even the cause of physical or psychic illnesses. Even more, some sounds that can be acceptable in a certain period of time can change into annoying in other periods (Smith and Wellens, 2007).Unlike other contaminant agents, the effects of noise might be unnoticed, instantaneously, and its accumulation can lead to an obvious physical, psychic and social deterioration (Tewari *et al*, 2005). The best studied effect of the overexposure to noise is the loss of hearing. The problem is that the exposed people are scarcely aware of the cause-effect relation given that it is produced slowly but progressively (Leather *et al*, 2003).

For several decades, there has been a strong body of opinion that mechanical vibration and shock because a significant damage to the health of people exposed to them. This was recognized by, and incorporated in, the first International Standard on human Whole-Body Vibration (ISO 2631-1974), development of which began in the mid-1960s.

In Sri Lankan construction sites, safety against vibration is one of the most important concerns of relevant parities of projects. Whole Body Vibration (WBV) is one of hidden hazards which should have more attention, probably because vibration hazards have chronic effects on health of construction machine operators (Bo¨ rje Rehna *et al*, 2005). There are no immediate effects of exposure to vibration on human health. Exposure of WBV for operator should be cared if the level of exposure

exceeds the industrial occupational safety limits ISO 2631-1, 1997 (ISO). Exceeding of these standard levels may cause different level of health issues.

Objective of the current study is to investigate the noise and vibration exposure levels of the operators, who are operating heavy machineries in the road development sites. In addition, measured exposure levels were compared with the limits imposed by the different current regulations such as Occupational Safety and Health Administration (OSHA), International Organization for Standardization (ISO), National Institute of Occupational Safety and Health (NIOSH) and World Health Organization (WHO), in order to identify the possible health effects.

2. METHODS

2.1 Instrumentation

Measurements of noise were performed in road development sites. For the measuring procedure it was included only operators who are working in excavators (JCB machines), soil rollers, loaders, motor graders etc. Noise exposure levels (dBA) of 21 operators at a distance about 1 foot from the operator's ear were measured using a sound level meter, SL-1350 (CUSTOM).

A tri-axial vibration meter (VM-54) together with a seat pad accelerometer (PV-62) was used to measure WBV exposure levels. Seat pad accelerometer was connected to the tri- axial vibration meter and placed on operators' seat. The operator was instructed to sit on the seat pad accelerometer as his whole weight distributed through the seat pad.



(a) **(b)**

Figure 1 (a) Sound level meter, (b) Tri-axial vibration meter and seat pad accelerometer

2.2 Analysis

2.2.1 Noise

Sound level (L_i) was measured for 20 seconds and equivalent sound value (L_{eq}) was obtained by using Equation (1).

$$L_{eq} = \log_{10}\left(\sum_{i=1}^{n} 10^{L_i/10}/n\right) \quad (1)$$

All of operators were working different number of hours per day with different noise exposure levels. With the aim of comparing with the standard limits, noise dose was calculated using Equation (2) (OSHA Regulations Standards 29CFR, 2012).

$$Noise\ Dose = \frac{C_1}{T_1} + \frac{C_2}{T_2} + \frac{C_3}{T_3} + \cdots \frac{C_n}{T_n} \quad (2)$$

Where
C_n is the time spent during each work task; T_n is the permitted time of exposure of that level calculated using Equation (3).

$$T_n = \frac{8}{2^{(L_{eq}-90)/5}} \quad (3)$$

Then Time Weighted Average (TWA) values for 8 hour duration of noise were obtained as given in Equation (4) (OSHA Regulations Standards 29CFR, 2012).

$$TWA = 16.61 \log_{10}(Dose/100) + 90 \quad (4)$$

However, due to noise exposure there may be number of health issues. Among them investigation about hearing loss is vital. However, in a fixed sample population, hearing loss could be happened due to different reasons including age and noise exposure levels. World Health Organization (WHO) has recognized relative risk factors for hearing loss problem as shown in Table 1(Concha-Barrientos *et al*, 2004).

Table 1 Relative risks for hearing loss by gender, age group and level of noise exposure

Gender	Exposure level(dBA)	15-29yrs	30-44yrs	45-59yrs	60-69yrs	70-79yrs	80+yrs
Male	<85	1.00	1.00	1.00	1.00	1.00	1.00
Male	85-90	1.96	2.24	1.91	1.66	1.66	1.66
Male	>90	7.96	5.62	3.83	2.82	2.82	2.82
Female	<85	1.00	1.00	1.00	1.00	1.00	1.00
Female	85-90	1.96	2.24	1.91	1.66	1.66	1.66
Female	>90	7.96	5.62	3.83	2.82	2.82	2.82

The first step in the calculation is to determine the fraction of the total burden of Noise Induced Hearing Loss (NIHL) in the study population that is attributable to the risk factor, by combining the exposure distribution and relative risk information. The attributable fraction (AF), also called the impact fraction in this context, is given by Equation (5) (Marcos et al, 2005).

$$AF = \frac{\sum P_i RR_i - 1}{\sum P_i RR_i} \quad (5)$$

Where P_i is the proportion of the population in each exposure category, *i* (i.e. $P_{unexposed}$, $P_{low\ exposure}$, $P_{high\ exposure}$) RR_i is the relative risk at exposure category, *i*, compared to the reference level (RR_i equals 1.0 for an unexposed population).

2.2.2 Vibration

The vibration total value (a_v), which measures the degree of vibration, was calculated using Equation (6) (ISO 2631-1, 1997).

$$a_v = \left[(k_x a_{wx})^2 + (k_y a_{wy})^2 + (k_z a_{wz})^2\right]^{0.5} \quad (6)$$

Where, k_x, k_y and k_z are the scaling factors, which equal to 1.4, 1.4 and 1.0, respectively .a_{wx}, a_{wy}, a_{wz} are the frequency weighted r.m.s. acceleration in the X-axis, Y-axis and Z-axis, respectively.

With the aim of identifying the influence of noise and vibration into human life and health issues a questionnaire and discussion about safety practices, attitudes and health issues were conducted among 21 operators.

3. RESULTS AND DISCUSSION

3.1 Questionnaire Survey

By the questionnaire survey, it was hard to identify the people who are suffering from noise, although some understanding on vibration related health problems were evident. As a general, all operators were exposed to high and hazardous noise levels. However, operators were not aware that they are in high noisy environment. Most of them are smoking and having alcohol as a habit. None of them were recorded with suffering from noise related hearing loss. Percentages of operators, who are suffering from vibration related health issues, are shown in Figure 2.

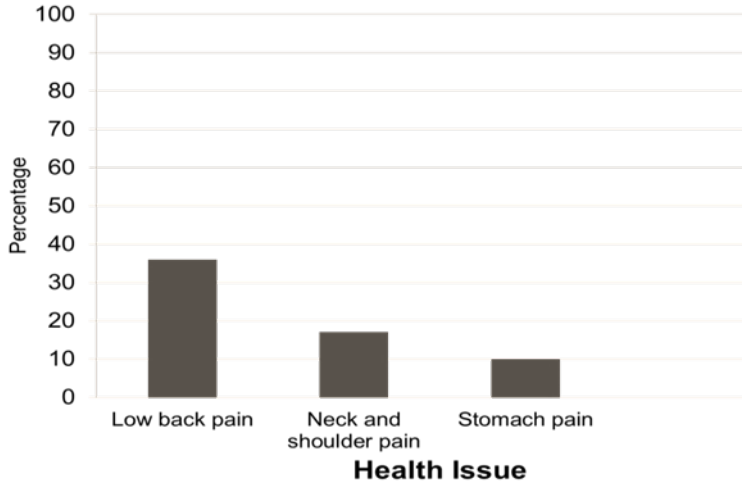

Figure 2 Distribution of different types of vibration related health issues among operators

3.2 Noise

Figure 3 shows time weighted average (TWA) values for noise exposures. It can be seen that many operators (20 operators out of 21 operators, i.e., 95%) were exposed to noise level greater than the permissible limit of 75dBA introduced by ISO. Nine operators out of 21 operators (i.e., 43%) exceed the permissible limit of 85dBA specified by NIOSH. Five operators out of 21 operators (i.e., 24%) were at a risk of having health issues as they were exposed to noise level greater than the permissible noise level of 90dBA, according to OSHA.

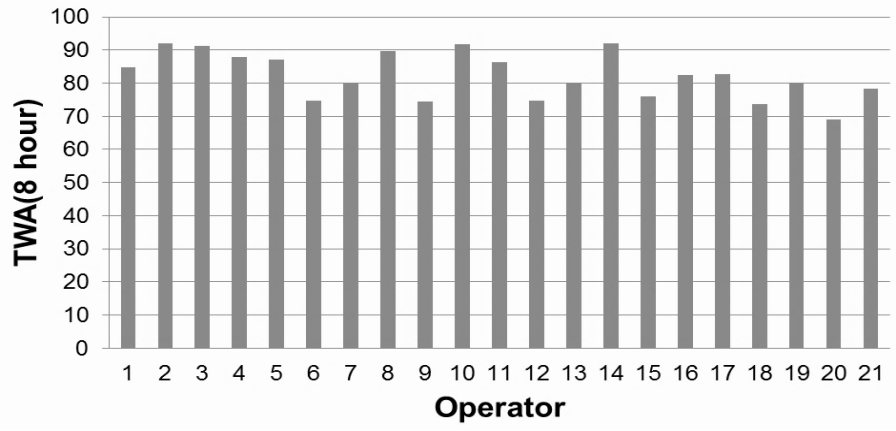

Figure 3 Time weighted average noise exposure values of operators

Figure 4 compares the noise exposure levels (Time weighted average values) of operators in different construction machineries. It can be seen that most of soil roller, soil grader and pile driving machine operators were exposed to a noise level greater than 80 dBA implying that they were in a risk of having noise related health issues. Open cabin position may be the key factor for high noise exposure for these machineries. Small size machineries (i.e., Soil rollers) are having less distance between the operator and the engine, consequently high sound pressure level is inevitable.

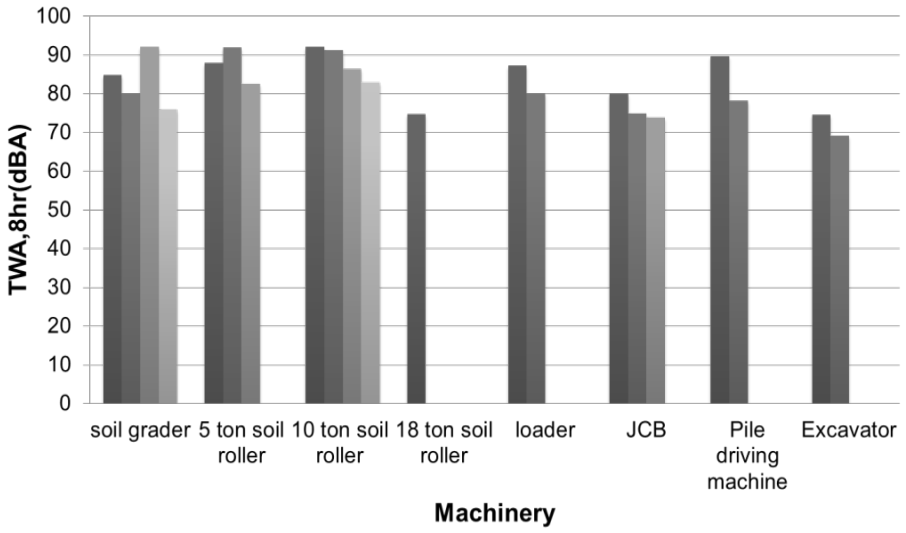

Figure 4 Time weighted average noise exposure of operators in different construction machineries

For the health issue of hearing loss, number of reasons may be attributed. However, attributable fractions of impact of noise exposure in different age ranges were obtained. By declining the noise exposure for this sample, issue of hearing loss could be avoided by 50% , 56%, 23% and 64% for the age ranges of 15-29 years, 30-44 years, 45-59 years and 60-69 years, respectively.

Age category of 60-69 years is the most impacted sample. Except 45-59 years age range, burden of hearing loss due to noise exposure is more than 50%.

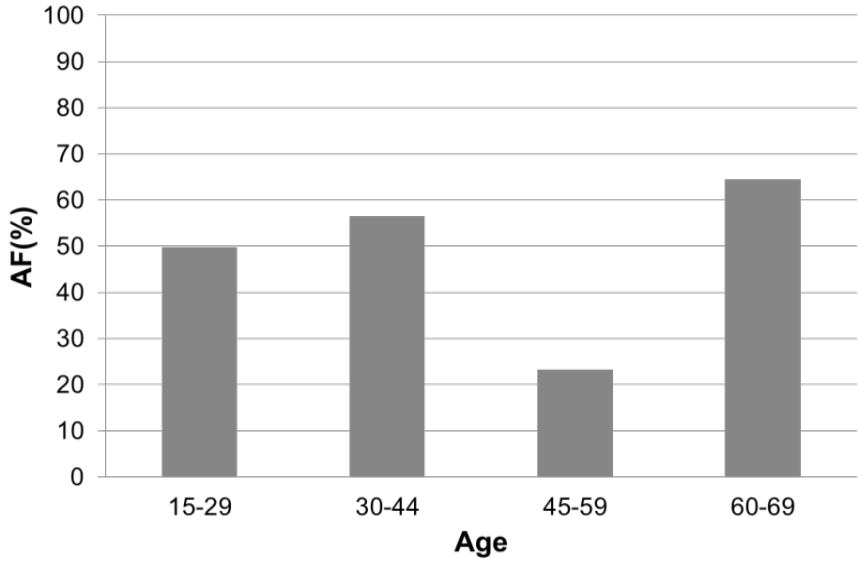

Figure 5 Attributable Fraction for different age groups

3.3 Vibration

Figure 6 shows vibration total value (a_v) for operator of each type of equipment used in road development sites. It can be seen that large magnitude of vibrations were generated by pile driving machines and soil rollers.

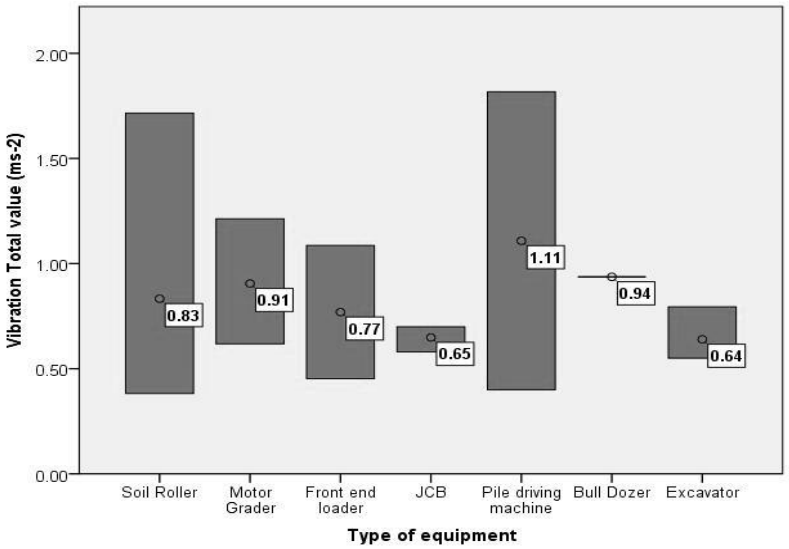

Figure 6 Variation of vibration total value, a_v for operators in different construction machineries

Most of operators were exposed to vibration level greater than 0.5 ms^{-2} (Figure 6), which is the limit of fairly uncomfortable level according to the ISO 2631-1. For example, all operators except a few operators of soil rollers and front end loader were exposed to a vibration level greater than 0.5 ms^{-2}.It was found that, considering the exposure level of each operator, 81% of operators were exposed to vibration exposure level greater than 0.5 ms^{-2}.

This group, who exceeded the exposure level of 0.5 ms^{-2}, may have high health risk. Above fact was verified by a questionnaire survey, which was conducted for operators. Figure 7 shows that variation of vibration total value measured at the operator seat-interface together with the current health condition of the operators.

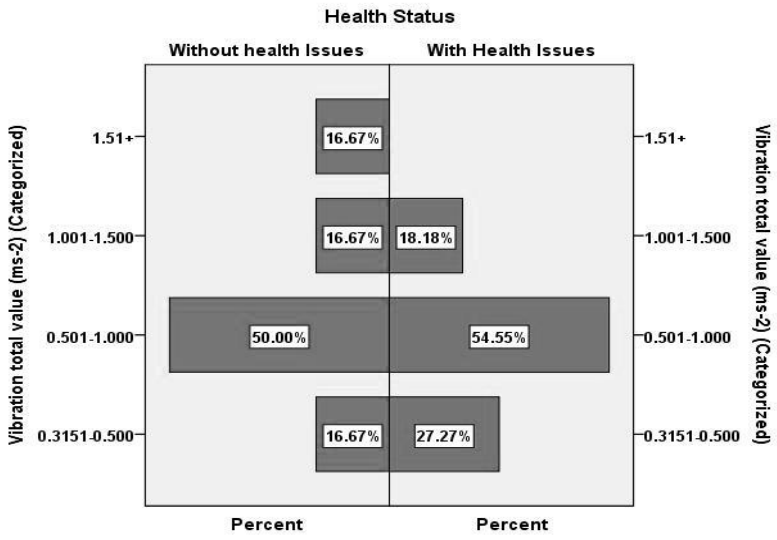

Figure 7 Vibration total values of operators with different health conditions

Among the operators, who were exposed to fairly uncomfortable level of vibration (i.e., 81% of operators) 73% of them reported that they have health issues due to vibration exposures. In addition, it was found that, health issues due to vibration exposures were affected by the age of operator. Fifty three percent of total operators, who participated in the questionnaire survey, were below 30 years. And 73% of operators, who have reported health issues due to vibration exposure, were in the age above 30 years.

4. CONCLUSIONS

Noise and vibration exposure levels of twenty one operators in road development sites were measured and were compared with the limit values specified in current standards. In addition, a questionnaire survey was conducted to identify probable health effects due to long-term exposure to noise and vibration. Among the operators participated in the study, 95%, 43% and 24% of operators are exceeding permissible noise exposure limits introduced by ISO, NIOSH and OSHA, respectively. They are in a risk of having hearing loss. They had better to be protected by wearing ear plugs, ear muffs during the working period. Operators, who are operating soil rollers, soil graders, loaders and pile driving machines, must be protected against noise by wearing ear muffs. This is a must if the machinery is without a closed cabin.

It was found that 81% of operators are exposed to vibration, having vibration total value greater than 0.5 ms^{-2}, implying that these operators are in or above fairly uncomfortable level. These operators have a risk of having health issues due to prolong exposure of vibration. Low back pain was identified as the most common health issue due to prolong exposure of vibration: 36% of operators suffer from low back pain. In addition to vibration total value, its' exposure time has great influence on operators health. It was observed that age may also affect on operators' health.

In Sri Lanka, although less attention has been paid for chronic health effects of workers in construction industry, operators in heavy construction machineries and equipments are in risk of having health issues due to prolong exposure of vibration prevailing in construction sites. Therefore, amount of continuous working time can be reduced and more comfortable seats that have been provided in the original condition of the machinery should be kept and maintained.

5. ACKNOWLEDGEMENTS

Supports provided by TURIS (Transforming University of Ruhuna into International Status) project are greatly appreciated. Authors wish to extend their appreciation for the assistances given by site engineers and the operators of heavy construction machineries in construction sites, which were selected for this research.

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Modelling Noise Pollution Levels along Expressway in Rural Southern Sri Lanka

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Abstract: *This paper summarises the results of a study conducted in the Galle district with the aims to map the noise levels near the Southern expressway in Galle district and develop a traffic noise attenuation prediction model for the rural areas along Southern Expressway, in an uncontrolled traffic flow condition along the southern expressway. The maximum traffic noise levels prevailed during a 20 minute period was recorded at 205 location along the southern expressway along with location data and land use data. Polynomial model was developed to predict traffic noise attenuation with respect to the distance from the centreline of the expressway. Dense tree cover area was identified as the most suitable area for attenuate the noise coming from the Southern Expressway. It was concluded that 40 m of thick tree cover would attenuate a maximum noise level of 87 dB (A) to a permissible 55 dB (A).*

Keywords: *Traffic noise pollution, GIS model, natural noise attenuates*

1. INTRODUCTION

Noise pollution is one of the serious types of environmental pollution. (Birgitta et al., 1995, and Vinaykumar, 2007). Physically, there is no difference between sound and noise, however, according to World Health Organization guideline for community (Birgitta et al, 1995) and highway traffic noise analysis and abatement policy and guidance (1995) noise is defined as unwanted sound. Noise is generally divided in to two; community noise and industrial noise (Birgitta et al., 1995 and Diaz et al., 2001). Main sources of community noise include road traffic, rail and air traffic, and construction. Among the sources of noise road traffic noise is probably the most serious and pervasive type of noise pollution. Fast growing vehicle population in towns in the recent years, has resulted in considerable increase in traffic on roads causing alarming noise pollution and noise level increases with traffic volume in an exponential manner (Marathe 2012).

According to study by Keulen (2004) road traffic noise has become an important issue in western European countries. In the European Union about more than half of all citizens are estimated to be living zones that do not ensure acoustical comfort to residents (Birgitta et al. 1995). Road traffic noise pollution is also severe in cities of developing countries. In Bangladesh increasing use of motorized private vehicles and deterioration of mass transport facilities are leading to a large problem of noise pollution in urban and semi urban areas of Dhaka city (Tanvir & Rahman 2011). In India with urbanization and corresponding increase in number of vehicles in metropolitan cities, the pollution is increasing at an alarming rate (Marathe 2012). Kumar et al. (2011) has calculated more than 70% of total noise in their environment is due to vehicular noise. Noise levels are showing an alarming rise and in fact level exceeds the prescribed levels in most of the areas.

Recent studies have shown that in Sri Lanka too noise pollution has become a very big problem and people starting give attention to this (Sooriyaarachchi & Sonnadara 2006). According to study by Wickramasinghe (1990) there are many complaints in Sri Lanka concerning the noise produced by the road traffic, industries and pertaining to the use of loudspeakers. According to Nandika & Nagodawithana (2014), noise pollution is a public health problem in the city of Colombo Sri Lanka.

The maximum permissible noise levels at the boundary of the land in which any source of noise is located is regulated under section 23P, 23Q and 23R of the National Environmental Act, No.47 of 1980 read with section 32 of that act (National Environmental (Noise Control) Regulations No.1 1996). The Permissible noise levels in accordance with Sri Lanka noise control regulations are shown in Table 1.

Table 1 Permissible noise levels in accordance with Sri Lanka noise control regulations (National Environmental (Noise Control) Regulations No.1 1996)

Zone	Noise Level dB (A)	
	Day Time	Night Time
Low Noise Zone	55	45
Medium Noise Zone	63*	50
High Noise Zone	70	60
Silent Zone	63	45
Construction Zone	75	50

(*) The noise level should not exceed 60 dB (A) inside existing houses, during day time

Although the traffic noise is not usually loud enough to cause hearing problems, continuous exposure to unacceptable noise levels can create depression. Other health effects are, health issues, sleeping disorders trouble communicating and effect on wildlife (Birgitta et al. 1995). Already, traffic noise pollution has gained attention as a problem in metropolitan areas, however, with current development activities traffic noise pollution is slowly creeping into the rural areas. Especially with major portion of the expressways running through rural areas this problem will be felt more. As a first step, there is a need to identify and map the traffic noise levels along the expressways in the rural areas.

This study as two aims they are; (a) to develop a model for predicting traffic noise level attenuations in the rural areas along expressways, and (b) to identify features that might be useful in attenuating traffic noise levels.

2. METHODOLOGY

2.1. Data Collection

Rural locations along the southern expressway in Galle districts were selected as the study area for this study. In selection of sites consideration was given to include varying geographical and residential conditions. The total study area was about four kilometres in length. When collecting noise level data, “uncontrolled traffic approach” was adopted. Since under the uncontrolled traffic approach, the traffic flow is not disturbed and data is collected under real-life conditions. This results in more realistic model development will possible than using the controlled approach (Sooriyaarachchi & Sonnadara, 2008). Noise level measurements were taken at three locations situated approximately perpendicular to the centre line of the expressway. First point was selected near the expressway right-of-way line, other points were located approximately 20 m apart. But the distance was increased until the permissible noise level for humans is recorded. A graphical representation of the data collection locations are shown in Figure 1. “Twenty minute maximum traffic noise level” was recorded GPS location of the point is also recorded with land use type. Measurements were limited to day time from 09.00 am to 05.00 pm at 205 locations. Descriptive statistics of the traffic noise levels are shown in Table 2.

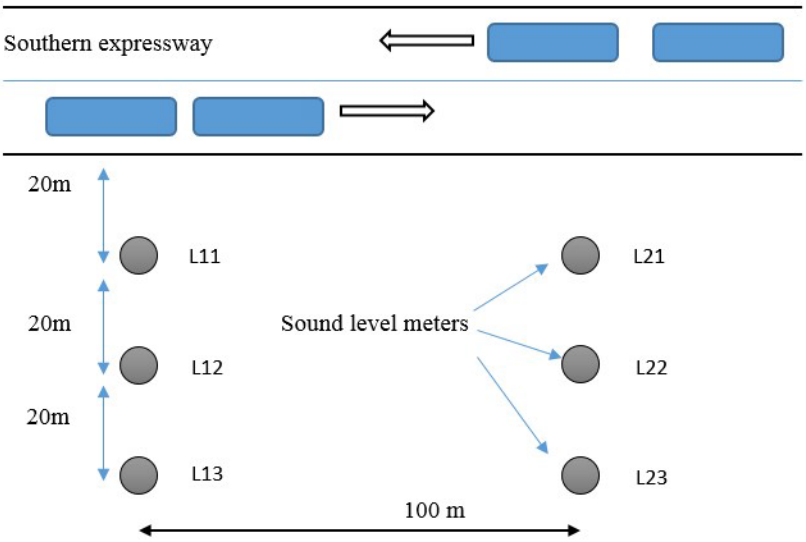

Figure 1 Data collection setup

2.2. Noise Level Mapping

Collected noise level data along with other relevant data was inserted in to Geographical Information System based data base. ArcGIS was used for this purpose. Data were mapped using the SLD 99 datum.

Table 2 Descriptive statistics of the traffic noise levels

Statistical parameter	Noise level (dB A)
Minimum	48.00
Maximum	87.10
Mean	63.97
Standard deviation	8.12
Skew-ness	0.30
Median	63.20

2.3. Noise Level Modelling

Noise level attenuation (measured as the difference between the sound levels at the ROW boundary and the current location) is modelled against the distance. Polynomial models were developed for each type of land-use.

3. RESULTS

3.1. Models

Noise level attenuation models developed for the different land-use types are shown in Table 3. According to the polynomial models shown in Table 3, noise attenuation can be calculated with the distance. This done by substituting value of distance in the polynomial model. Resulting noise attenuation with distance is shown in Table 4. The same information is presented in the graphical format in the Figure 2. SPSS statistical software package was used to develop the models

Table 3 Noise attenuation models for different land use types

No	Land-use	Model
1	Dense Tree Cover	$Y = 0.0107x^2 - 0.4452x + 4.3664$
2	Thin Tree Cover	$Y = 0.0006x^2 - 0.0022x - 0.666$
3	Tea Plantation	$Y = 0.0015x^2 - 0.0143x - 0.8606$
4	Open area	$Y = 0.0007x^2 - 0.0785x + 2.0198$

Table 4 Noise attenuation with distance for different land use types

Distance (m)	Dense tree cover	Thin tree cover	Tea estate	Open area
40	3.6784	0.206	0.9674	-
60	16.1744	1.362	3.6814	-
80	37.2304	2.998	7.5954	0.2198
100	66.8464	5.114	12.709	1.1698
120	105.0224	7.71	19.0234	2.6798
140	151.7584	10.786	26.5374	4.7498
160	207.0544	14.342	35.2514	7.3798
180	270.9104	18.378	45.1654	10.5698
200	343.3264	22.894	56.2794	14.3198

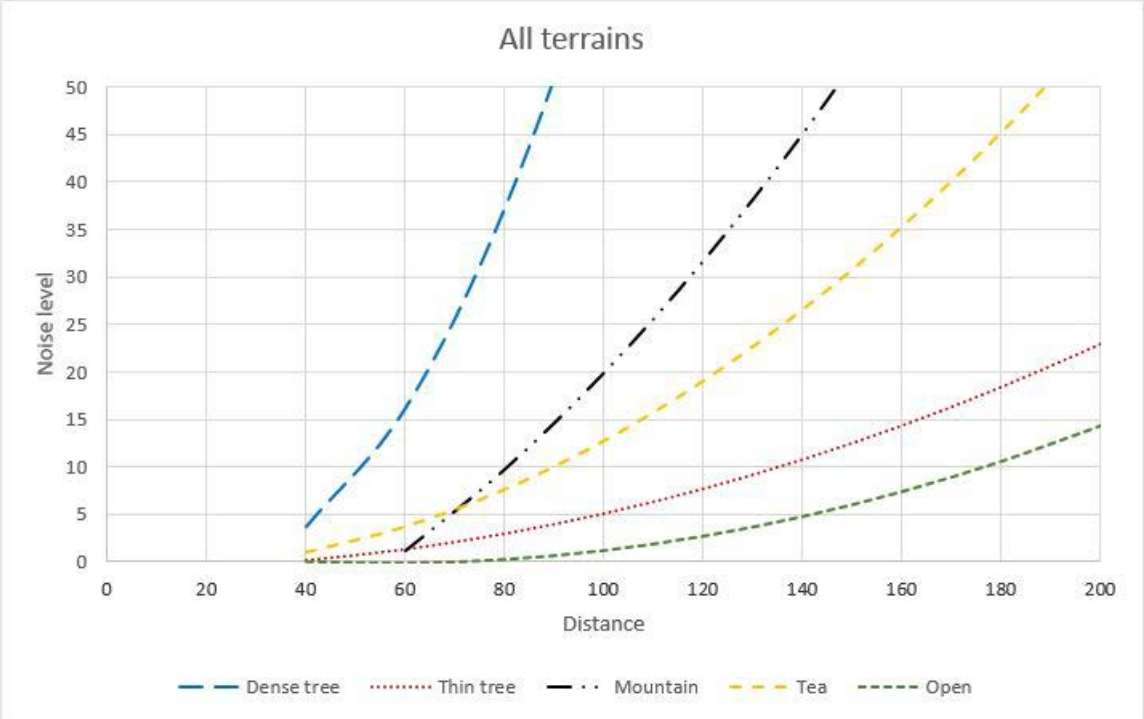

Figure 2 Noise attenuation with distance for different land use types

4. CONCLUSIONS AND DISCUSSIONS

Noise prediction is an integral part of environmental impact assessment of expressway projects. However it is very essential that the noise levels are measured and mapped once project gets implemented. This study traffic noise levels were modelled with respect to distance for different land-use types. All types of land-uses showed noise attenuations greater than the theoretical attenuation given by the inverse square rule. However it was clear that the thick tree cover followed by tea plantations achieve a high rate of noise attenuations.

According to the noise level readings it was found that the existing noise levels near the Southern expressway are exceeding the Sri Lankan standards in most of the areas while some areas are safe due to the existing tree cover between the location and the expressway. 20 m of thick tree cover can achieve an average of 16 dB (A) attenuation while to get the same attenuation 110 m of thin tree cover or 70 m of tea plantation is needed. In this study maximum recorded noise level was 87 dB (A), to get this to permissible level of 55 dB (A) about 40 m of thick tree cover or 120 m of tea plantation cover is needed.

Considering the land required and the difficulty in having tea plantations everywhere it is recommended that the dense tree cover as a suitable noise attenuation barrier. Suitable types of plant for plantation in road sides has to be decided considering the expressway aesthetics, soil type and climate of the area.

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River Engineering and Hydroinformatics

Model Study of Sediment Flushing in Rantambe Reservoir

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Abstract:*About 1% of the total storage capacity in the world's reservoirs is lost annually due to sedimentation. Sediments can also block intakes in reservoirs and damage tunnels or turbines. One of the most effective techniques to remove these sediments is flushing, whereby water level is lowered sufficiently to re-erode deposits and flush them through the intakes. Outflow sediment discharge may well be related to the parameters such as the sediment characteristics in the reservoir, during flushing. The purpose of this research is to investigate sediment removal efficiency during flushing of sediment through low level sluice in Rantambe Reservoir and increasing the flushing efficiency by changing different discharges and water level combination. In this project laboratory experiments were performed on a physical model. Similitude and scaling principles were used in order to create the model. Based on the physical properties of the media and data collected the coal is used to model sand media. The combination of 0.1 & 0.2 l/s flow releases through low level sluice is the most effective to flush the sediment. The time ratio between model and its prototype is equal to 1:16.*

Keywords:*Rantambe reservoir, Sediment flushing, Flushing efficiency, Physical modelling.*

1. INTRODUCTION

Reservoir sedimentation not only decreases the storage capacity, but also increases the probability of flood inundation in the upstream reaches due to heightening of the bed elevations at the upstream end of the reservoir. In order to reduce reservoir sedimentation and remove sediment, many approaches such as flushing, sluicing, dredging of the reservoir, and water & soil conservation in the catchment are used. Among these approaches, flushing is considered an economical approach to swiftly restore the storage capacity of the reservoir with severe deposition.

The purpose of this research is to investigate sediment removal efficiency during flushing of sediment through the low level sluices in Rantambe Reservoirunder different combination of discharges and reservoir water surface level. Laboratory experiments were performed on a physical model of the reservoir using coal as the sediment.

2. LITERATURE REVIEW

Substantial amount of sediment could be flushed out and reservoir capacity could be restored to maximum if periodical flushing is conducted (NeenaIsaacab, Eldho, T & Gupta, I 2013).

For a given reservoir, the sediment flushing efficiency changes widely by various factors such as configuration of reservoir, volume and grain size of deposited sediment, discharge rate during sediment flushing, duration time from the start of draw down flushing etc.(Shen, H1999).

In a physical model study, the sediment diameter should not be less than 0.7 mm lest the bed of the model forms ripples (Jean & Rene, T 2011).

3. METHODOLOGY

3.1. Experimental Set-up

Scaled down model of Rantambe reservoir was constructed with use of timber frames and concrete. The geometric scale ratio between the model and the prototype is equal to 1:250.

Table 1 Dimensions of the prototype and the model

Item		Prototype length (m)	Model length (m) (1:250)
Dam length (m)		415	1.66
Reservoir length (Project area) (m)		700	2.8
Sluice gate	Width (m)	2.5	0.010
	Height (m)	3.8	0.016
Water depth (m)		30	0.12



Figure 1 Constructed model

Coal was used as sediment in the model testing based on dimensional analysis for similarity condition. Sieve analysis was used to take out coal of the required sediment size of 0.8 mm. Sediment passing through 1 mm and retained on 0.6 mm were separated by sieves.

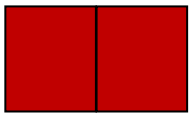
Initially a scale down model of the reservoir was constructed and the periphery of the reservoir was produced to the scale. Sediment was laid on the bed for a thickness of 5 cm and allowed to consolidate to simulate the reservoir bed. The time ratio between model and its prototype is equal to 1:16.

Sediment (coal) was placed in the model bed to form reservoir bed topography using grid lines marked to cover the model bed. After that sediments were allowed to consolidate and thereafter initial bed levels were measured.

3.2. Experiment Programme

Reservoir impounding was carefully done by sending water at a very small flow rate as high flow erodes sediment particles. Low level sluice gates were opened and loss of sediments in the reservoir bed was measured for different inflow and gate opening combinations (Table 2).

Table 2 Schedule of trials

	CASE No.	Gate opening	Discharges
 16mm 10 mm	CASE 1-i	One gate open	0.1 l/s
	CASE 1-ii		0.2 l/s
	CASE 1-iii		0.1 & 0.2 l/s combination
 2X10 mm	CASE 2-i	Two gates open	0.1 l/s
	CASE 2-ii		0.2 l/s
	CASE 2-iii		0.1 & 0.2 l/s combination

All experiments were conducted for 2 ½ hours. In the cases with combination flows, i.e. Case-iii, flow rate of 0.1 l/s was fed first for 30 minutes. Thereafter, flow rate of 0.2 l/s was fed for 5 minutes and flow rate of 0.1 l/s was again fed for 30 minutes.

4. RESULTS AND DISCUSSION

Case 1- i & Case 1-ii: No sediment was flushed out due to low flow rate and only ponding occurred.

Case 1-iii (0.1 l/s & 0.2 l/s combination): A little amount (108 g) of sediment was flushed through the low level sluice gate. There wasn't channel formation in the reservoir bed.

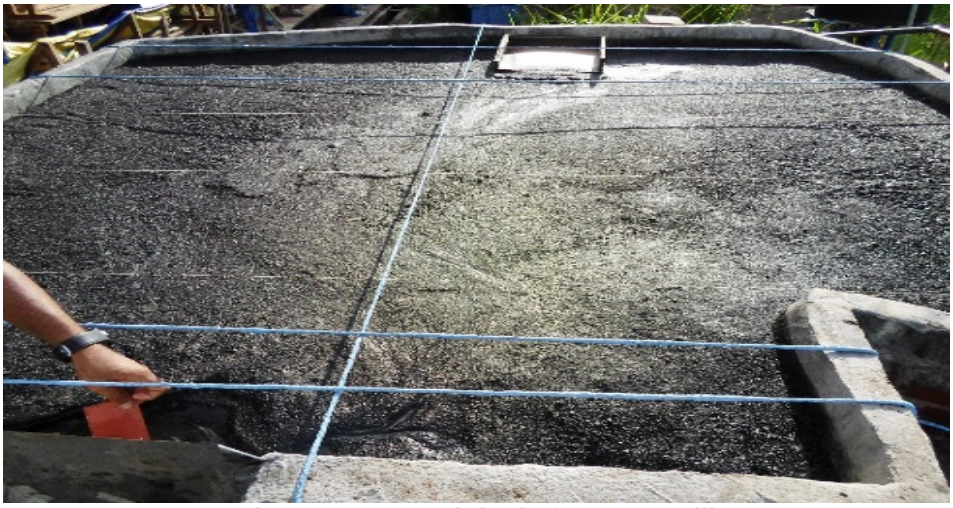


Figure 2 Reservoir bed after Case 1-iii

Case 2- i (0.1 l/s flow rate):

In this case 0.1 l/s flow was continuously fed from upstream for 2 ½ hours. Sediment was continuously flushed through the gate. A curved channel was formed in the reservoir. At a certain point inflow was equal to outflow, reservoir water level remained constant. In this case a substantial amount of sediment was flushed and measured as 6300 g in weight.

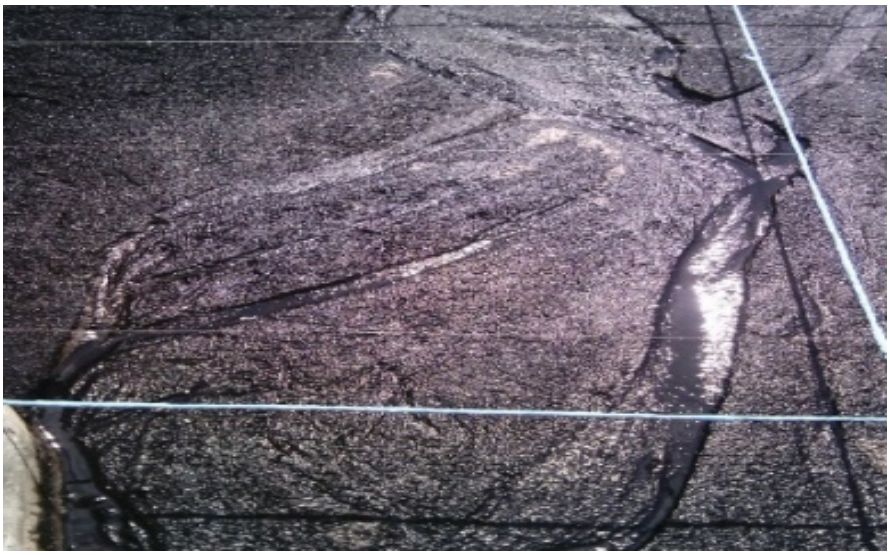


Figure 3 Reservoir bed after Case 2-i

CASE 2- ii (0.2 l/s flow rate):

In this case, 0.2 l/s of flow was continuously fed from the upstream. Initially upstream sediments migrated towards the gates and start to pass through the gates. Ponding occurred after sometime and there were not sediment flushed out through gates.

Initially sediment movement occurred in the direction indicated in fig4. After that erosion path have changed to the direction indicating in red colour. In this case Erosion rate is less than Case 2-i.



Figure 4 Reservoir bed after Case 2-ii

Case 2-iii (combination of 0.1 l/s & 0.2 l/s):

The quantity of flushed sediment is high. During 0.1 l/s flow rate, sediments eroded at a small rate in the vicinity of the gate. When flow rate was changed to 0.2 l/s at the upstream, sediment started to migrate towards the gate. This substantially increased the amount of sediment flushing. The flushed sediment in this case was measured as 9500 g.



Figure 5 Reservoir bed after Case 2-iii

Table 3 Summary of result in model

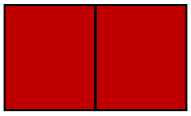
Gate size (mm)	Flow rate (l/s)		Weight of sediment collected /(g)
 16mm 10 mm	CASE 1-i	0.1 l/s	No sediment collected
	CASE 1-ii	0.2 l/s	No sediment collected
	CASE 1-iii	0.1 & 0.2 l/s combination	108
 2X10 mm	CASE 2-i	0.1 l/s	6300
			6000
	CASE 2-ii	0.2 l/s	3400
			3900
	CASE 2-iii	0.1 & 0.2 l/s combination	8100
			9800
			9500
			8900

Table 4 Summary of result in prototype

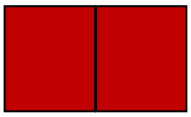
Gate size (mm)	Flow rate (m³/s)		Increased volume (MCM)
 16mm 10 mm	CASE 1-i	0.1 l/s	-
	CASE 1-ii	0.2 l/s	-
	CASE 1-iii	0.1 & 0.2 l/s combination	0.001
 2X10 mm	CASE 2-i	0.1 l/s	0.080
			0.076
	CASE 2-ii	0.2 l/s	0.043
			0.050
	CASE 2-iii	0.1 & 0.2 l/s combination	0.103
			0.124
			0.121
			0.113

Table 5 Feeding time duration of 0.2 l/s

Case No.	Flow Rate (m³/s)		Flushed sediment weight (g)
	0.1 l/s	0.2 l/s	
Case 2-iii	135	15	9800
	140	10	9500
	145	05	8900

4.1. Comparison of Experimental Results with Flushing of Rantambe Reservoir

Rantambe reservoir was flushed during June 2014 and he reservoir bed after flushing was compared with that of the model. There were some small sand bars formed in the model though there wasn't any bar formation in the prototype. This can be attributed to the fact that sediments in the prototype had been consolidated for a very long period compared to the sediments in the model. The channel paths observed in both prototype and model were similar. Sediment deposited in the sides of the channel was not eroded.Duration of feeding of water at a rate of 0.2 l/s has a prominent effect on the flushing sediment volumes.



Figure 6Comparison of channel formation between the model and the prototype

5. CONCLUSIONS

A scale model experiments were conducted to investigate the sediment flushing in Ranambe reservoir. Based on series of model test results, the flushing efficiency is highest at the combination of flow rates of 0.1 l/s & 0.2 l/s. Use of constant flow rates of 0.1 l/s or 0.2 l/s is less effective than using combinations of flow rates. When combination of flows is used, the time duration of inflow of 0.2 l/s has a prominent effect on flushing. Thus the combination flows of 100 m^3/s and 200 m^3/s in the prototype is recommended, and this is feasible by supplying discharge from the Randenigala releases together with flow of Uma Oya to improve the flushing efficiency. Flushing will recover about 0.124 MCM of reservoir storage.

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Flood Modelling in Waidina Tributary, Fiji Islands

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Abstract: *Rewa River is the widest river and the second longest river in Fiji Islands. It is in the islands of Viti Levu. The river is 145 km in length and starts from the highest peak of the island, Tomanivi. Rewa River experiences frequent floods. However, a detailed flood model for Rewa River is still to be tabled. This paper briefs the flood modeling of part of Waidina tributary of the Rewa River. US Army Corps Engineers HEC-RAS hydraulic model is being successfully applied to the Rewa River and initial results are drafted for the Waidina tributary. These results are promising; however, a completed flood model should be developed for sound conclusions. Inundation top widths due to a random flood were presented. These inundations can effectively be used to inform the residents in the vulnerable areas in a flood event. In addition, the final expected results can be used for the flood protection structural measures.*

Keywords: *Flood inundation, inundation top widths, HEC-RAS hydraulic model, Rewa River, Waidina tributary*

1. INTRODUCTION

River discharges and river water heights are usually used to warn the general public about the floods. However, the general public would not worry about the discharges, but they highly concern about the potential inundation areas. Flood analyses are usually carried out to find out the inundation areas. Not only these analyses help the general public, but also the engineered results will be highly used in urban planning. Rapid urbanization has increased the direct runoff. This is due to the increased impervious pavements, not only in cities, but also in rural areas (Ku *et al,* 1992). In addition to the rapid urbanization, climate change has some adverse effects on the runoff (Arnell, 2004). For some countries, unusual rainfall patterns can be distinguished. These unusual rainfall patterns have increased the number of floods per annum. However, authorities including civil engineers have to identify the potential solutions to overcome the disastrous damage from the floods.

These potential solutions can either be structural or non-structural. Construction of levees, revetments, flood control reservoirs are among the structural solutions. In addition, an efficient drainage network can be used to convey the additional flows. People's awareness, proper land-use, improving the infiltration characteristics in upper catchments and avoiding marshy lands for development activities are among the non-structural measures for floods. However, it is always better to carry out a detailed flood analysis before the implementation of structural or non-structural measures.

With advancements in computing and numerical analysis, hydraulics of flood modeling is not any more a difficult engineering problem to be solved. There are many commercial and non-commercial numerical solvers to compute the flood modeling. River roughness is one of the most important parameters to be determined in flood modeling (Timbadiya *et al*, 2011). False roughness can lead to inaccurate solutions and therefore, urban planning can be affected. Among many other commercial and non-commercial numerical solvers, Hydrological Engineering Centers River Analysis System (HEC RAS) by the United States Army Corps of Engineers is a widely used numerical solver for flood modeling. HEC RAS has successfully applied to many rivers around the world (Prata *et al*, 2011;

Timbadiya *et al*, 2011; Yuan and Qaiser, 2011; Rathnayake *et al*, 2007; Goodell and Warren, 2006; Tate, 1999). HEC-RAS is a powerful hydraulic model and it is capable of analyzing one-dimensional steady, unsteady flow hydraulics, sediment transport computations and water temperature modeling. It has a graphical user interface so that the users can get the benefit of graphical modeling (Brunner, 2010).

This paper briefs the flood modeling of part of Waidina tributary in Fiji Islands. Waidina tributary is a branch of the Rewa River in Fiji Islands and experienced severe floods in February 1965, October 1972, April 1980 and January 1993 (McGree *et al.,* 2010). HEC-RAS model is being successfully applied to the Rewa River and initial results are drafted for the Waidina tributary.

2. HYDRAULIC MODELING OF RIVERS

Flow continuity and the conservation of momentum are the two basic physical laws, which govern the flow in an open channel. River itself is an open channel; however, it usually does not have uniform (lined) banks and uniform bed profiles. However, these governing equations are still valid for the river flows.

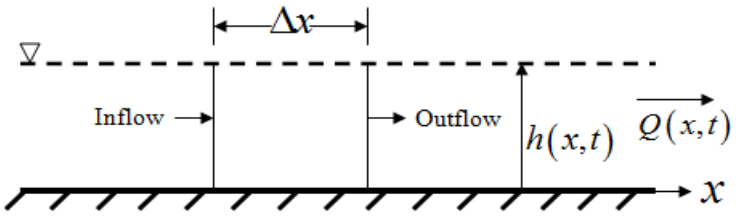


Figure 1 Schematic view of the river flow.

Figure 1 shows the schematic diagram of a river flow. Considering the control volume presented in Figure 1, the following equations (Equations 1 & 2) are derived. Equation 1 gives the unsteady continuity of the flow.

$$\frac{\partial A}{\partial t}+\frac{\partial Q}{\partial x}-q_1=0 \qquad (1)$$

where *A, Q, q_1, t* and *x* are the total flow cross sectional area, volume flow rate, the lateral inflow per unit length, time and longitudinal direction coordinate, respectively.

Equation 2 presents the unsteady momentum equation for the river flow.

$$\frac{\partial Q}{\partial t}+\frac{\partial QV}{\partial x}+gA\left(\frac{\partial z}{\partial x}+S_f\right)=0 \qquad (2)$$

where *V*, *∂z/∂x* and S_f are the river flow velocity, water surface profile variation and the friction slope, respectively. S_f is expressed using Manning's equation as shown in the following equation.

$$S_f=\frac{Q|Q|n^2}{2.208R^{4/3}A^2} \qquad (3)$$

where *n* and *R* are the Manning's friction coefficient and the hydraulic radius. However, the governing equations have to enhance when there is a flood. During a flood, the water flows not only along the channel direction, but also to the lateral direction.

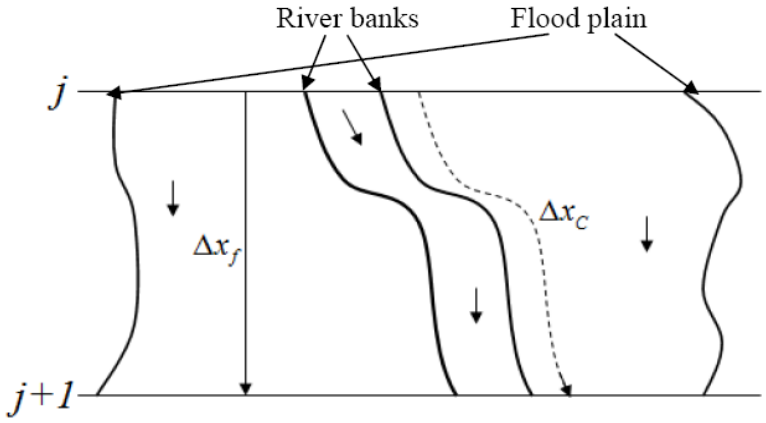


Figure 2 Schematic view of the flood plain.

Figure 2 illustrates the two dimensional characteristics of riverbanks and a flood plain. Water moves laterally away from the channel in an event of a flood. The excessive flood flow inundates the flood plain and temporally stores the water. The following equations govern the total flow in a flood event. Equation 4 expresses the continuity of a flood flow.

$$\frac{\partial A}{\partial t}+\frac{\partial \phi Q}{\partial x_c}+\frac{\partial\left[\left(1-\phi\right)Q\right]}{\partial x_f}=0 \tag{4}$$

where ϕ is the conveyance ratio and given in the following equation (Equation 5).

$$\phi=\frac{K_c}{K_c+K_f} \tag{5}$$

where K_C and K_f are the conveyance in the channel and the flood plain. The conservation of momentum of a flood flow can be expressed in the following equation (Equation 6).

$$\frac{\partial Q}{\partial t}+\frac{\partial\left(\phi^2 Q^2/A_c\right)}{\partial x_c}+\frac{\partial\left(\left(1-\phi^2\right)Q^2/A_f\right)}{\partial x_f}+gA_c\left(\frac{\partial z}{\partial x_c}+S_{fC}\right)+gA_f\left(\frac{\partial z}{\partial x_f}+S_{ff}\right)=0 \tag{6}$$

where A_C, A_f, S_{fC} and S_{ff} are the flow cross sectional area in channel and flood plain and friction slopes in channel and flood plain. These governing equations (Equations 4 & 6) can be approximated using implicit finite differences and solved numerically for flood flows using Newton-Raphson iteration method. Sections, *j* and *j+1*are in Figure 2, are two example sections used for finite difference scheme.

3. GEOMETRIC MODELING OF RIVERS

Topography information is a must for the flood modeling in rivers. Hydraulic models incorporate Geographic Information System (GIS) maps are often used to model the river flows or floods. HEC-GeoRAS is a GIS based freely available hydraulic model. If the GIS maps are readily available for the location or the river that is interested, then, the hydraulic model can be easily applied and the flood flows can then be analyzed. However, if these GIS maps are not readily available for the interested area, then, one has to go to the basic version of the hydraulic model, HEC-RAS. HEC-RAS requires the cross-sectional profiles for the river, including the surroundings. A better river profile can be achieved, if the distance between two cross-sections can be minimized. However, these cross-sections have to be fed manually.

4. WAIDINA TRIBUTARY FLOOD MODELING

Rewa River is the widest river and the second longest river in Fiji Islands. It is in the islands of Viti Levu. The river is 145 km in length and starts from the highest peak of island, Tomanivi. Rewa River drains approximately one thirds of island to Luacala bay. Wainibuka and Wainimala are the two major tributaries in Rewa River; however, there are many other tributaries, including Waimanu and Waidina rivers. Few research work was carried out for Waidina tributary (Brodie and Morrison, 1984, Greenbaum, 1979, Terry *et al.,* 2002); however, a flood model was never discussed.

Fiji Islands has a good spatial and temporal variation of precipitation through rainfall. Cyclones are very frequent for Fiji Islands; however, most of them are tropical. Tropical cyclone "Kina" tracked in December 1992 and intense rainfalls were found in Waidina tributary. Not only Kina, but also many other cyclones were happened throughout the recent history. Therefore, Waidina tributary is experiencing some variable rainfalls but they are intense.

The authors could not find the GIS maps of Rewa River. Even though the digitized maps are available at Fiji department of Lands and Survey, these maps could not use in this research work. This is due to the lack of support from the respective authorities. Therefore, the manual method was used to obtain the topographical view of the Waidina tributary. 1:50,000 topographical maps were obtained from the Fiji Lands and Survey department. Cross-sections, along the Waidina tributary, were taken at an interval of 500 m.

Figure 3 shows the Waidina tributary and its connection to the Rewa River. This research is an ongoing research and the highlighted section of the Waidina tributary is completed for its geometrical modeling.

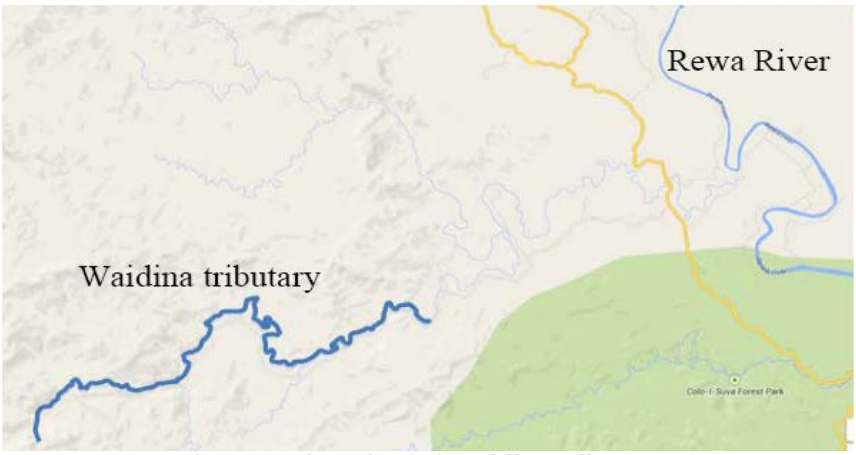


Figure 3 Plan view of Waidina tributary.

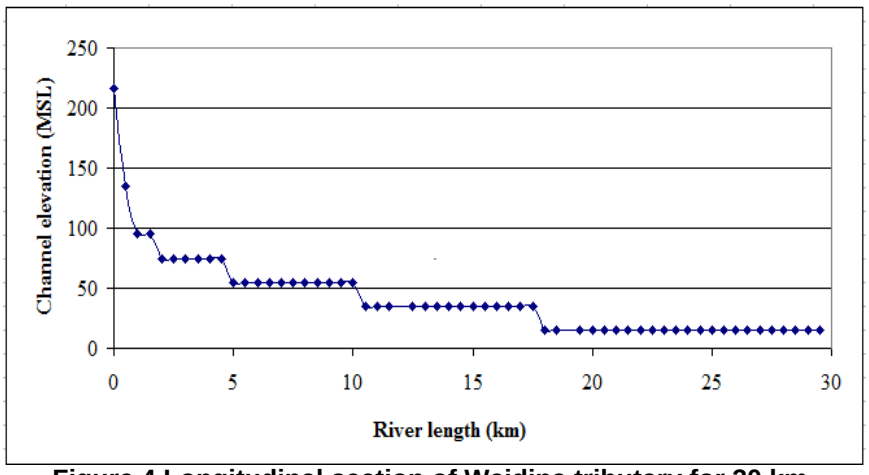
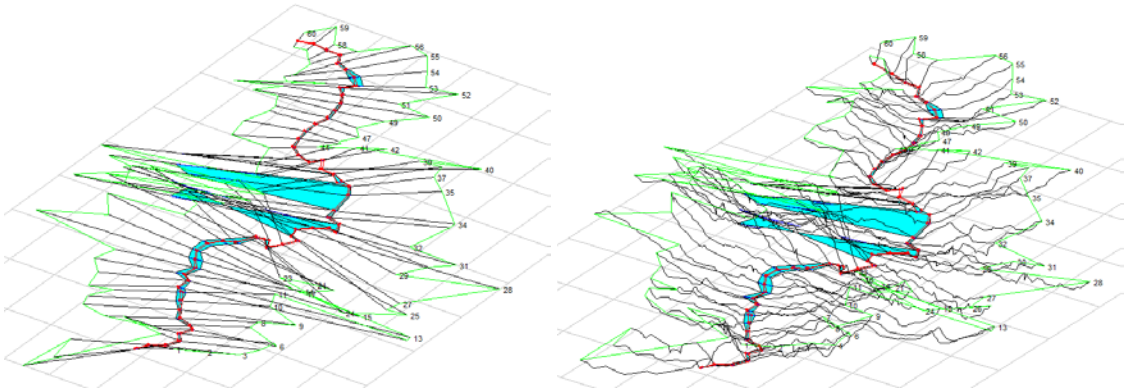


Figure 4 Longitudinal section of Waidina tributary for 30 km.

Figure 4 shows the longitudinal section of the section shows in Figure 3. Tributary has a very steep slope in its first few kilometers; however, it changes to a mild slope in downstream. About 200 m elevation drop can be seen along the 30 km length of the tributary (Figure 4). As it was stated above, the cross sections were obtained for this section of the tributary. The cross-sections were manually taken along the highlighted route of the tributary. On average eight to ten kilometers in width were considered. About 30 km along the tributary is being completed. These cross-sections were manually fed to the hydraulic model, HEC-RAS for the flood analysis.
Tributary is a mountainous stream, and assumed with no vegetation in channel. Banks were assumed to be steep with considerable vegetation. Bottom of the stream has cobbles with large boulders. Therefore, a slightly high Manning's roughness coefficient (0.05) was used for the flow calculations. In addition, the banks and the floodplain were assumed to be dense with vegetation. Therefore, 0.07 Manning's roughness coefficient was used for the flood calculations. Several random flood flows were used to visualize the flood inundation; however, the results are presented herein for a flood flow of 100 m^3/s.

5. RESULTS AND DISCUSSION

The simulation has taken 0.44 seconds of simulation time on an Intel® Core™ i3 desktop personal computer with a 3.40 GHz processor and 4 GB of RAM. The following figures (Figures 5a and 5b) show the inundation maps for the 100 m^3/s of flood. Figure 5a presents the plan view of the inundation whereas Figure 5b shows a 3-D view. Two red lines are the banks of the tributary. Perpendicular lines to the banks (Figure 5a) are the fed cross-sections. Flood inundations can be seen from 8 to 25 km. However, a considerable flood inundation can be observed from the 10th to 16th kilometer of the tributary. This result is further illustrated in Figure 6. Figure 6 shows the comparison of the top tributary width against the top inundation width.



a) Plan view b) 3-D view

Figure 5 Inundation maps of the tributary for 100 m^3/s flood

Tributary top widths are around 50 m. However, floods even have laterally spread to 2 km from the riverbanks in an event of 100 m^3/s of flood. This is because of the shallowness of the tributary. These inundation widths can be projected to the real map of the tributary area and the real inundation locations can be identified. Those identifications can effectively be used to inform the residents in the vulnerable areas. In addition, these results can be used for the flood protection structural measures.

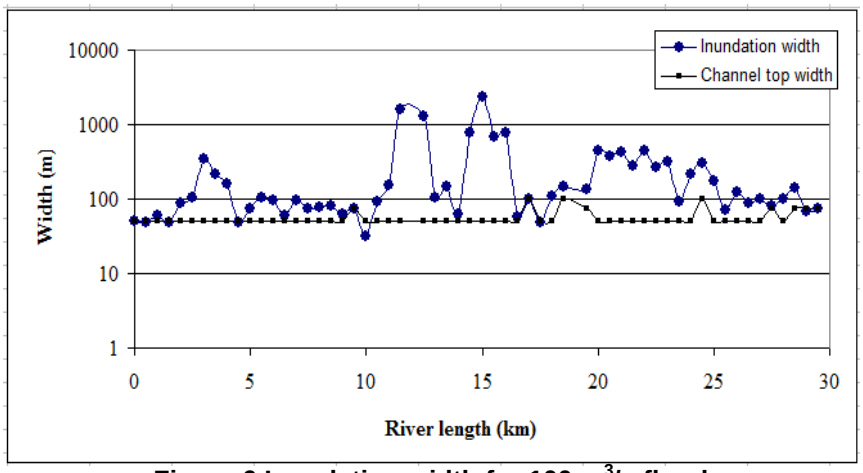


Figure 6 Inundation width for 100 m^3/s flood

Figure 7 shows the cross-section at the 14th km of the tributary. Circled section gives the existing Waidina tributary. However, a flood water profile (rectangular section) can be seen to the right hand side of the existing tributary. Even though, the tributary water cannot flow directly from the circled section to rectangular section, the lower land (to the right hand side) shows the flood inundation. This is due to the existing lower lands close to the tributary in near by cross-sections (cross-sections at 13, 13.5, 14.5 km). This can further be identified by analyzing the near by 3-D topography.

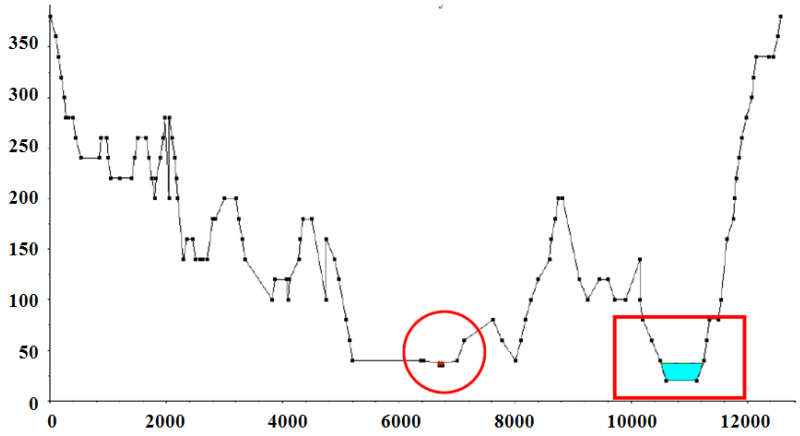


Figure 7 Cross-section at 14th km

6. CONCLUSIONS AND RECOMMENDATIONS

This is an ongoing research work to develop a flood model for the Rewa River in Fiji Islands. Initial flood modeling is being completed for a section of (30 km) of the Waidina tributary of the Rewa River. US Army Corps Engineers HEC-RAS hydraulic model was successfully applied to that section and initial results were obtained. They are promising; however, a completed flood model should be developed for sound conclusions.

The developed flood model has run for several random floods (including 100 m^3/s, 200 m^3/s, 500 m^3/s and 1000 m^3/s) and presented for a 100 m^3/s of flow. However, the flood model is still to be validated for a real world flood flow. Developing a detailed flood model, including cross structures (bridges, culverts and so on) is being carried out. Furthermore, usage of GIS maps is highly encouraged. The support from the authorities is highly required to compile a holistic flood model to the Rewa River.

7. ACKNOWLEDGMENTS

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Model Development for Assessment of Unmet Irrigation Water Demand in Left Bank Development Area of the Deduru Oya Reservoir Project

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Abstract*: Rainfall in Deduru Oya basin has a significant temporal variation and thus the Deduru Oya carry flash floods during rainy season and very low flows during dry season. The multi-purpose Deduru Oya reservoir project with a reservoir of a capacity of 75 Million Cubic Meters (MCM) augments water resources in 136 existing tank based irrigation systems in the Deduru Oya Left Bank through a Left Bank (LB) canal and also diverts water to the Iginimitiya tank in the Mee Oya basin through a Right Bank (RB) trans-basin canal. This study is focused on the setting up of WEAP21 version 3.43 (Water Evaluation And Planning) model to study the water management including diversion requirement from the Deduru Oya reservoir through the LB canal to supplement LB irrigation demand. Hydrological Engineering Center-Hydrological Modeling System (HEC-HMS) is used for runoff estimations and CROPWAT model is used to estimate crop water requirements. WEAP model is used for water balance simulations in Deduru Oya LB canal development area and to calculate water requirements from LB canal for the period of recent 10 years. The study reveals that the annual unmet demand varies from 26 MCM to 41 MCM without Deduru Oya reservoir project and from 0.44 MCM to 1.07 MCM with Deduru Oya reservoir project, for the study period from 2000 to 2010. The developed model can be used as a tool for irrigation water management and drought impact assessment in the LB development area of the Deduru Oya reservoir project.*

Keywords*: Deduru Oya, Hydrological Modeling, Irrigation, Unmet demand*

1. INTRODUCTION

The Deduru Oya carries flash floods during rainy season and very low flow during the dry season owing to the significant temporal variation of its basin rainfall. Also, rainfall is the only source of water and there are no trans-basin diversions into or out of the basin at present. It releases about 1600 MCM of water to the sea annually without being much used in the basin. Deduru Oya basin, which has an area of 2620 km^2 is the sixth largest river basin in Sri Lanka flows through Matale, Kurunegala, and Puttalam districts. There are several anicuts across it to divert water for irrigation, but there is no single reservoir intercepting the Deduru Oya except the reservoir at Thunmodara constructed under Deduru Oya project (Figure 1). There is strong requirement to regulate Deduru Oya flow for its optimum use especially for irrigation during dry season. Even though there are 3000 small tanks are scattered all over the basin there is a considerable issue with water scarcity. Figure 2 shows spatial distribution of current water scarcity in Irrigation systems of Deduru Oya basin (Somaratne, *et al*. 2003).

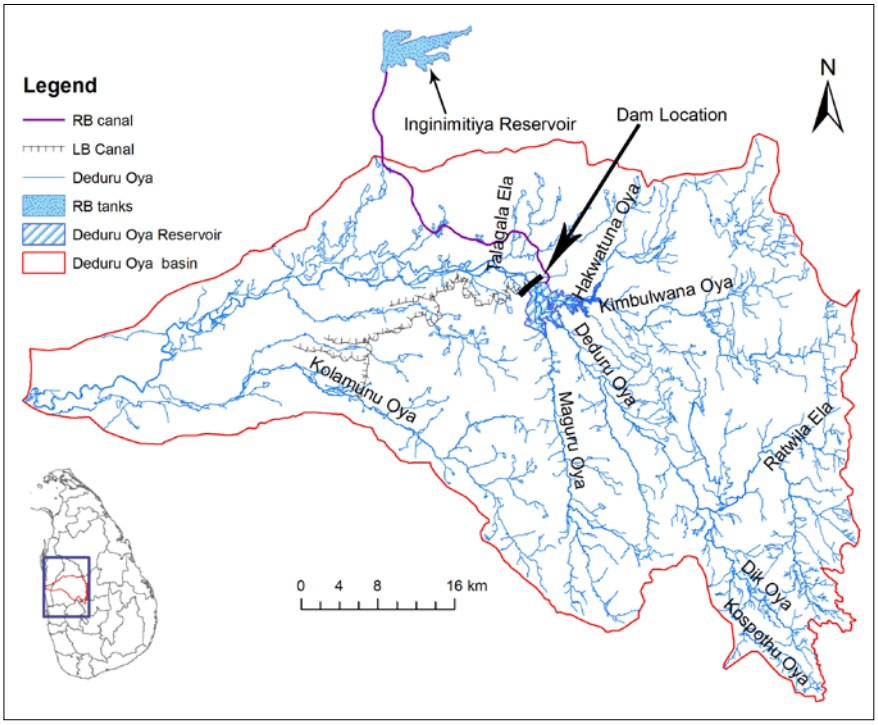

Figure 1 Study area

The Deduru Oya reservoir project is aimed to improve the livelihood of farmers in parts of the North-Western Province by increasing the productivity of its land and water resources by regulating and diverting the water by two main canals. The proposed Left Bank (LB) canal which is 44.1 km long will supply water to augment about 136 existing storage-based minor irrigation systems in the LB of the Deduru Oya (Figure 1). The Right Bank (RB) canal is a trans-basin canal to augment water supply to Iginimitiya reservoir which is located in Mee Oya basin (Figure 1). This study is focused on the setting up WEAP21 version 3.43 (Water Evaluation And Planning) model to study future drought impact on irrigation water availability under different water management scenarios with proposed reservoir project.

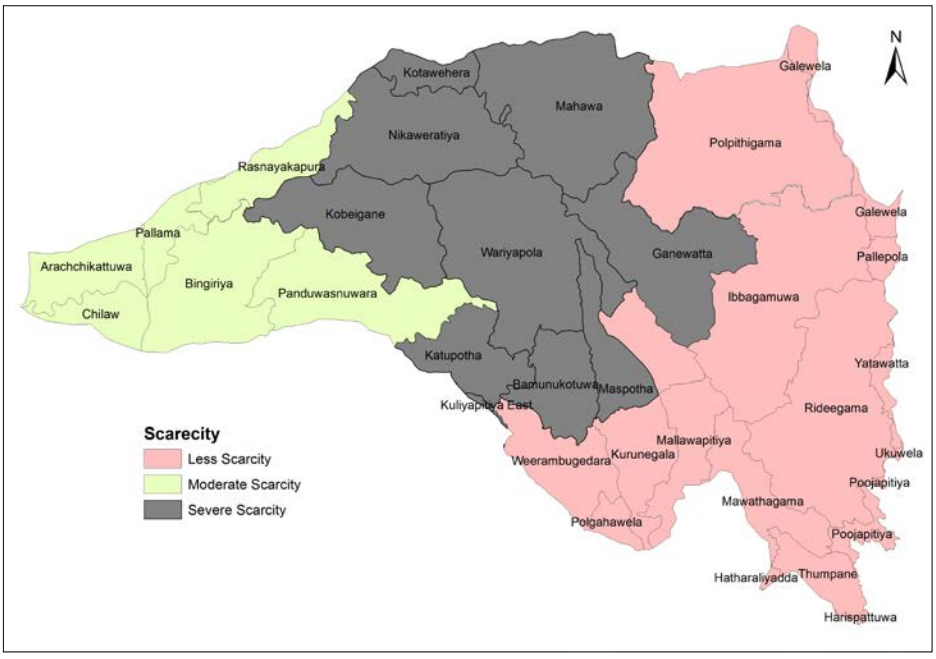

Figure 2 Variation of water scarcity in the basin (Somaratne et al. 2003)

2. METHODOLOGY

WEAP model was developed to Deduru Oya reservoir project and unmet demands of the LB for the period of 10 years from 2000 to 2010. Runoff data and crop water requirements are the model inputs to the WEAP model. As runoff data were not available during the study period, HEC-HMS rainfall runoff model was developed to obtain the runoff data at desired locations for the study period. Crop water requirements were estimated using CROPWAT model (Figure 3).

Hydrologic Engineering Center – Hydrologic Modeling System (HEC-HMS) version 3.0.1 developed by US Army Corps of Engineers in USA (Scharffenberg, *at al*, 2006) was used as rainfall runoff model for each of the catchment. HEC-HMS model was calibrated and verified for the Tittawella Tank in Kurunegala, because of rainfall or runoff data are not available for the rain-fed tanks in LB region. (Sampath, *et al*, 2014). Calibrated HEC-HMS model for the Tittawella Tank and rainfall data at Nikaweratiya and Ridi Bendi Ella stations were used to develop inflows to the minor tanks. Also HEC-HMS model was used to flow simulation of Deduru Oya. Thirty years daily rainfall data from 6 rain gauge stations in the Deduru Oya basin and runoff data at Moragaswewa from 1984 to 1989 together with monthly evaporation data at Mahawa agro-meteorological station was used in the simulation. Diversions, reservoir storages and losses were also accounted in the study (Sampath, *et al*, 2015). Calibrated HEC-HMS model of Deduru Oya and rainfall data at Millawa, Kurunegala, Ridi Bendi Ella, and Wariyapola stations were used to develop inflows to Deduru Oya reservoir.

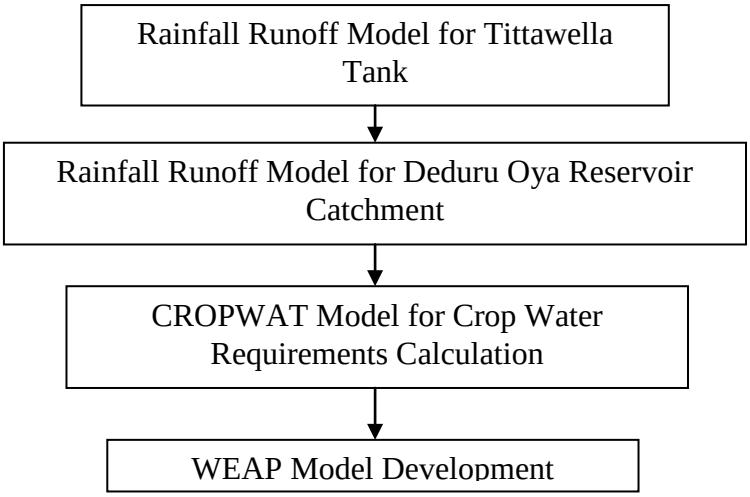

Figure 3 Methodology

The relevant catchment areas, storage areas, natural streams, land use patterns and cascades were identified for each of the 136 rain-fed minor tanks that will be supplied water by the LB canal for modeling the system. Also 12 directly feeding demand sites were identified. Topographic, geology and land use details were collected from the digital data of the Survey Department of Sri Lanka and Arc GIS 9.2 was used as a tool for spatial analysis. In the WEAP model, LB canal was modeled as a “Diversion links” and the minor tanks were connected by “Diversion elements” to LB canal. Demand sites and minor tanks were connected using 188 “Transmission links”. Minor tanks were connected to LB canal with 136 diversions. Four types of data viz: Area cultivated annually, Annual water use rate, Monthly variation and Consumption are required at each of the demand sites for WEAP model application. Crop water requirement was calculated using CROPWAT model by assuming 105 day low land paddy will be cultivated. The crop water requirement was calculated on a monthly basis. Rainfall data at Nikaweratiya and Ridi Bendi Ella station in year 2000 to 2010, Mahailuppallama reference crop evapotranspiration rates and crop factors for each growth stage were used for the calculation of crop water requirements. Computations of irrigation water requirements were made using 60% application efficiency and 75% conveyance efficiency. Land soaking and tiling requirement were also taken into account (Pre-feasibility study report, 2000). Calculated crop water requirements and irrigable area data was used to develop demand site ‘Annual water use rate’ and ‘Monthly variation’. Different ‘Supply preference’ and ‘Priorities’ were used to model the diversion link and transmission links.

In WEAP model two Flow Requirement elements were used to model RB trans-basin diversion and downstream environmental and irrigation requirements. Annual RB trans-basin diversion water requirement was fixed as 90 MCM. Hydropower requirement was not taken into account in this analysis. The minimum value environmental requirements were used as 3.2 m^3/s (Pre-feasibility Report 2000). Total downstream water requirement was calculated by considering both Ridi Bendi Ella scheme and mandatory release and maximum value was used as downstream irrigation flow requirement.

3. RESULTS AND DISCUSSION

With detailed irrigation demands analyses in LB region, unmet demands in LB region have been investigated. Figure 4 shows the total annual unmet demand from 2000 to 2010 in the LB development area without project. Figure 5 shows annual unmet demand for each demand site with the same scenario. Figure 6 shows that annual total unmet demand with project for each demand sites from 2001 to 2010.

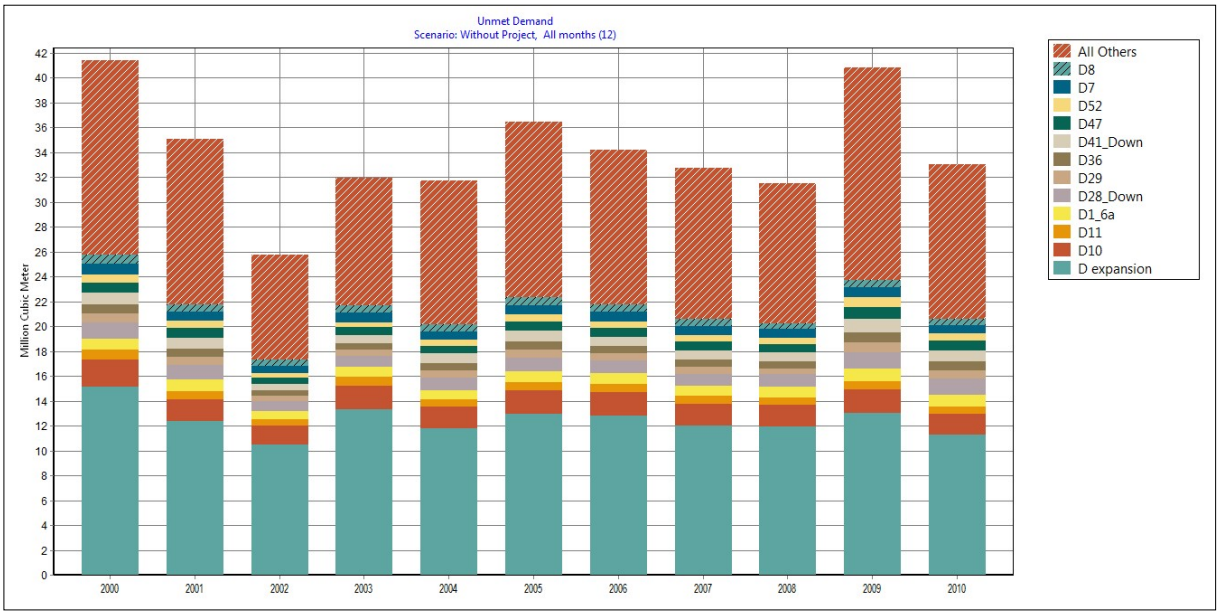

Figure 4 Annual unmet demand without project

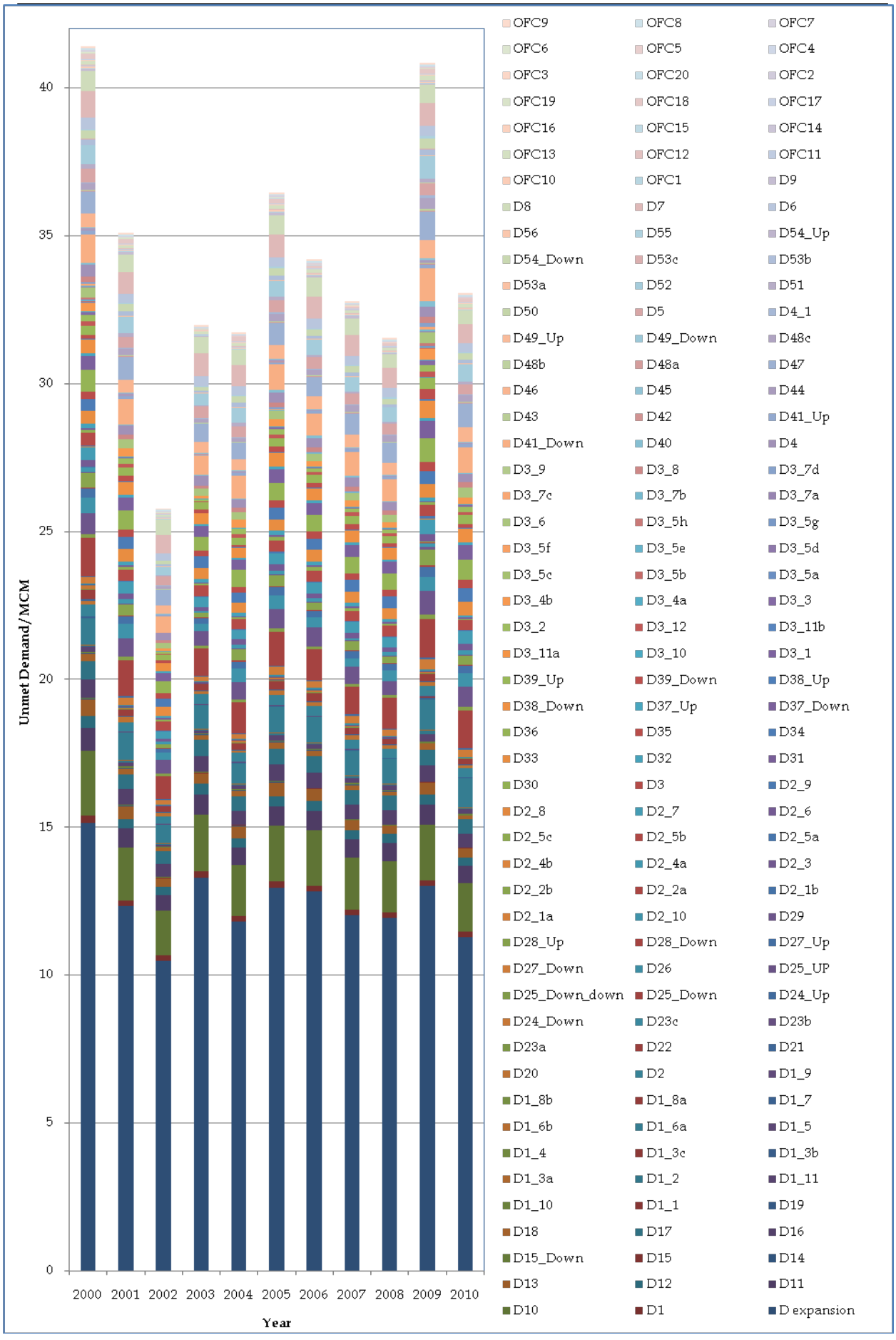

Figure 5 Detailed annual unmet demands during 2000 to 2010

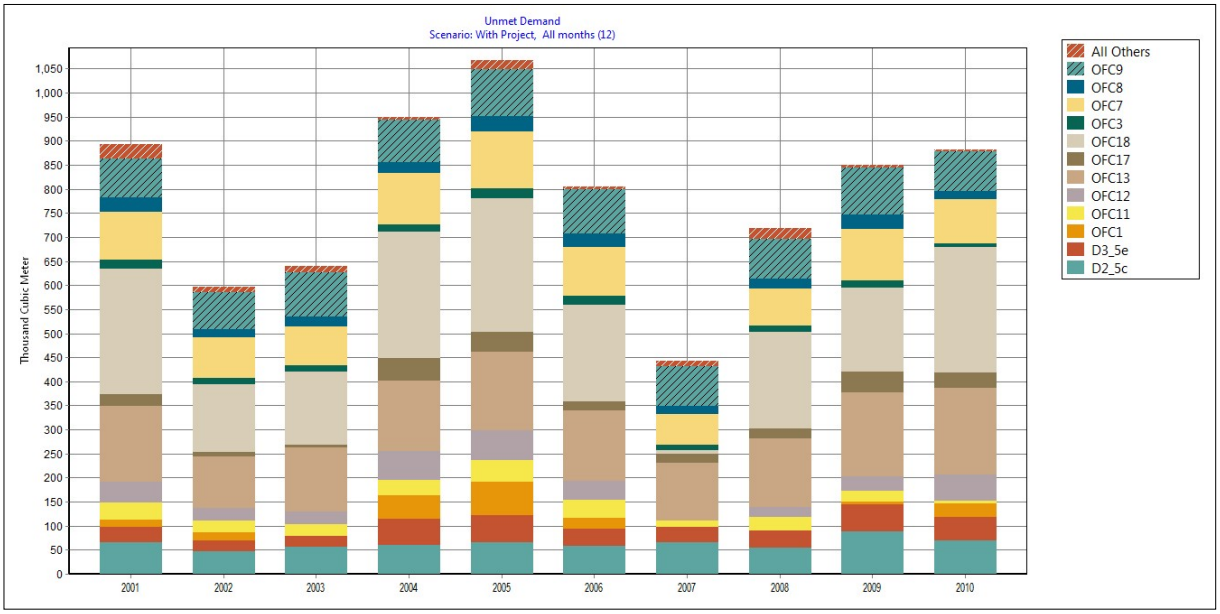

Figure 6 Annual unmet demand from with project scenario

As shown in Figure 4, Annual unmet demand varies 26 MCM to 41 MCM from 2000 – 2010 for 'without project' scenario. Figure 6 shows that annual unmet demand varies 0.44 MCM to 1.07 MCM from 2000 to 2010 for 'with project' scenario. Unmet demand in 'with project' scenario is comparatively less than unmet demand in 'without project' scenario.

The present study uses only recent 10 years for investigating unmet demand from 2000 to 2010 due to changing climate conditions and to demonstrate the model capability. With long term forecast rainfall data, the model with the calibrated parameters can be used for long-term projections of LB unmet demand of the Deduru Oya reservoir project. The model predictions will be useful for irrigation water management and drought impact assessment studies in the LB canal development area of the Deduru Oya reservoir project.

4. CONCLUSION

Analysis of results revealed that annual unmet demand in the LB development area of the Deduru Oya reservoir project varies from 26 MCM to 41 MCM without Deduru Oya reservoir project for the study period, 2000 to 2010. Also annual unmet demand varies from 0.44 MCM to 1.07 MCM with Deduru Oya reservoir project for the same period. The results of the analysis show the important role of the Deduru Oya reservoir project to reduce water scarcity of irrigation systems in the LB canal development area. The developed model can be used as a tool for irrigation water management and for drought impact assessment studies with forecasted rainfall in the LB canal development area.

5. ACKNOWLEDGEMENTS

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Structures and Building Materials

Effect of Gradation on Vertical Deformation and Bearing Index of Rail Track Ballast

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Abstract: *Crushed, angular rock fragments derived from quality stones are universally regarded as the most suitable for rail track substructure. The primary component of a rail track foundation, in which maintenance is concentrated, is the ballast. The ballast layer is frequently subjected to cyclic stresses from the moving train and exposed to extreme weather conditions. The prime cause for unsatisfactory performance of a rail track is the excessive deflection and vertical shortening at the passage of train and is often associated with the support condition of the track. Traditional method of characterising the vertical support condition of a rail track structure is the Track Modulus. Track modulus is defined as the supporting force per unit length of rail per unit vertical displacement of the rail and is a measure of the vertical stiffness of the track foundation. Track modulus represents the equivalent series stiffness of rails, fasteners, sleepers, ballast, sub ballast and subgrade. As a bulk material ballast is relatively of high stiffness which is significantly affected by the degree of compaction. In this study, the stiffness of ballast is represented by "Ballast bearing index" and degree of compaction is represented by "Degree of packing". This paper presents the results of plate load testing on prototype ballasts under laboratory conditions to investigate the influence of ballast particle size distribution on the stiffness and thus effects on the track modulus. Experimental results revealed the impact of smaller particles on the compaction behaviour and confirmed that the stiffness is influenced by the aggregate gradation of the assemblage.*

Keywords: *Ballast, Degree of packing, track modulus, ballast bearing index, track deflection.*

1. INTRODUCTION

Excessive deflections and the loss of vertical alignment of rail tracks often invoke for costly track maintenance and concerned with the safety and comfort of passengers. From the aspect of track design and maintenance, the track modulus is an important parameter since it indicates the structural state of the support system of the track structure.

Fundamental analysis of the track is based on the idealized beam on elastic foundation initially proposed by Zimmermann (Esveld, 2001). This approach introduces the foundation modulus which represents the combined effect of stiffness of rails, fasteners, sleepers, ballast and underlying layers (Selig & Li, 1994). The track modulus, which is defined as the supporting force per unit length of rail per unit vertical displacement of the rail is a measure of the vertical stiffness of the track foundation (Kerr, 1998).

Both low and large values of track modulus are undesirable. A high value for the foundation modulus distributes loads over fewer sleepers and leads to high stress concentration in the ballast layer and sleeper fasteners. On the other way, low track modulus has seen to cause differential settlement and fast deterioration of the track geometry due to the excessive abrasion of ballast that leads to frequent maintenance needs.

Conventionally a rail track foundation is a platform of compacted uniformly graded aggregates placed over a natural or improved subgrade (Goldbeck, 1936). Researchers (Selig & Waters, 1994) have found that the track modulus is greatly influenced by the stiffness of the ballasted layer. Over the

World, railway authorities specify aggregate size distribution limits as one of the selection criteria for track ballast though there is no universal agreement. Generally ballast particle sizes vary from medium to coarse gravel sized particles with small percentage of cobble sized particles. External cyclic loads applied during the passage of a train continuously change in-place physical state of ballast bed. The in-place state is defined by the density and is influenced by the degree of packing and void ratio of an aggregate skeleton (Selig & Waters, 1994). Degree of packing is expressed in terms of the amount of solid aggregate minerals per unit volume and is mathematically express as unity minus porosity.

Conventional method of testing large grained materials to study the strength and deformation behaviour is scaling down the particle size and testing in available apparatus. This disparity could greatly alter the behaviour of modelled material to the behaviour of actual material. Therefore to overcome size dependent problems and obtain more realistic results, large scale testing is preferred (Indraratna *et al*, 2000). This paper presents an experimental study of effects of the particle size distribution on degree of packing and ballast bed stiffness. Ballast stiffness was measured using a series of plate load tests conducted on deferent gradations of prototype ballast in laboratory conditions. The stiffness of ballast was represented by ballast bearing index (Selig & Waters, 1994) and in this study, the stiffness of rails, fasteners, sleepers and subgrade were assumed to be constant to study the effect of different ballast gradations on the track modulus. The knowledge gained would support selecting ballast materials having optimum track stiffness.

2. MATERIAL AND METHOD

In order to compare the effect of ballast gradation on the stiffness, six different samples of ballast gradations were prepared. All the aggregate samples were prepared from the same parent rock to maintain the material properties consistent. Physical properties of ballast used in the study are given in Table 1. The ballast gradations for the study were selected to confirm with the upper and lower limits of ballast specifications which are currently used in three countries. The particle size distribution curves of ballast specimens used in the study are illustrated in Figure 1.

Table 1 Properties of fresh Ballast

Property	Test value
Aggregate Crushing Value	28.5%
Aggregate Impact Value	35%
Water Absorption	0.194%
Bulk Specific Gravity	2.68
Flakiness (Shape)	15%
Elongation	27%

The degree of packing of ballast samples was examined according to ASTM C29-97,Standard test method for bulk density (“unit weight”) and voids in aggregate.

Standard method of investigating ballast stiffness is conduct plate load tests (PLT) in the field. Due to varying field conditions, it is not practical to make a comparison of different ballast gradations in-situ to obtain the effect on ballast bearing index. Therefore a test box arrangement was designed and fabricated using steel and timber to simulate the test portion of the track as shown in Figure 2. Using this setup it was possible to execute controlled full scale ballast tests to study the load- deformation response of ballast in the laboratory simulating field conditions.

The test box was capable of accommodating a 900x900x400 mm prototype ballast sample. Three sides of the apparatus were fixed to simulate the restraints provided by the surrounding ballast and one side was free to move laterally simulating the partial restrain provided by shoulder ballast.

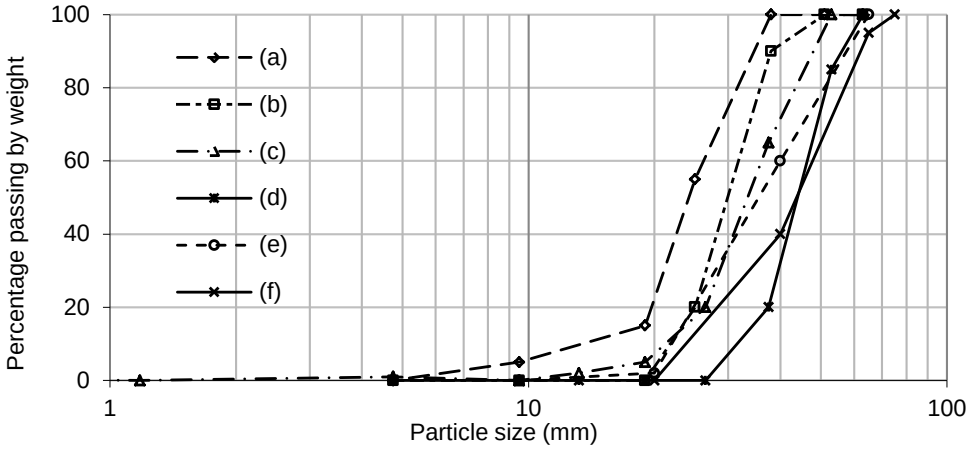

Figure 1 Particle size distributions of ballast samples

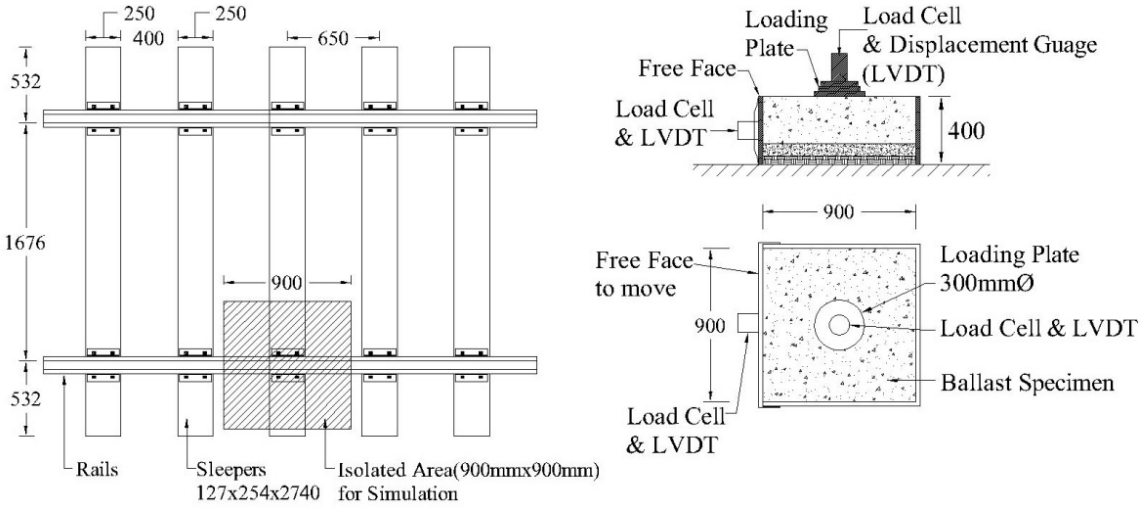

Figure 2 Schematic view of simulated area and test box for plate load test

Ballast samples were placed in uniform 75 mm thick layers and compacted to form 275 mm thick layer (15.6~17.2 kN/m^3) on top of a compacted 75 mm thick sub ballast layer (14.9 kN/m^3) underlain by 50 mm thick compacted soil layer (19.4 kN/m^3) to simulate the sub grade. Each ballast layer was compacted using the same effort for all the tests. Axial loading was applied on top of a 300 mm diameter rigid steel plate by a hydraulic jack. Loads and settlement data were acquired by using a set of linear variable displacement transducers (LVDT) and load cells connected to a computerized data acquisition system. The tests were conducted for all the samples in constant load increments until maximum pressure on the ballast surface reached 0.5 N/mm^2 which well exceeds 0.3 N/mm^2, the maximum pressure would be exerted on ballast surface underneath the sleeper by a train having 200 kN axel load (Esveld, 2001).

3. RESULTS AND DISCUSSION

Recorded compaction properties of ballast samples are given in Table 2. The pressure-settlement response of ballast bed is illustrated in Figure 3. It is seen that the gradations (e) and (f) demonstrated lowest deformation while the gradation (b) demonstrated the highest deformation with loading. Ballast bearing index is defined as the plate load per unit area required to displace the plate by a specified amount, which is usually 7.6mm divided by this displacement (Selig & Waters, 1994). Variation of ballast bearing index (C_{sb}) against degree of packing (k) is shown in Figure 4.

Table 2 Compaction properties of ballast samples

Gradation	C_u	C_c	Void ratio (e)	Degree of packing (k)
a	1.89	1.25	0.598	0.626
b	1.48	1.02	0.682	0.594
c	1.68	1.08	0.530	0.654
d	1.47	1.06	0.660	0.602
e	1.96	1.00	0.577	0.638
f	1.75	0.95	0.568	0.634

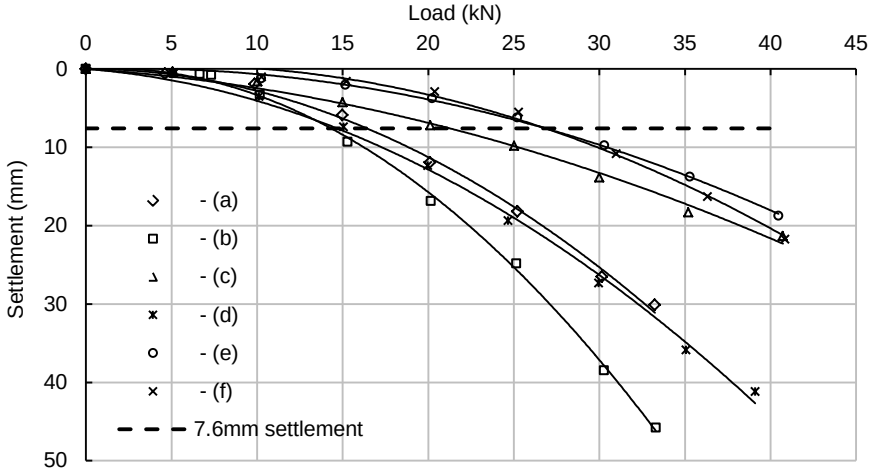

Figure 3 Pressure – settlement response of selected ballast gradations

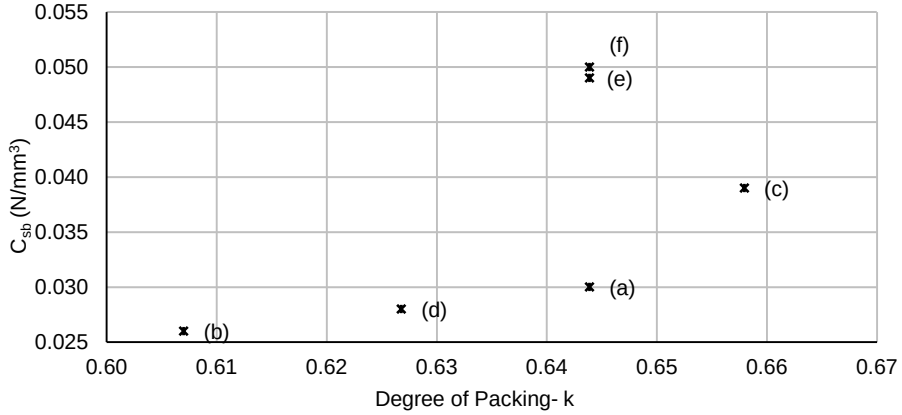

Figure 4 Variation of ballast bearing index(C_{sb}) against degree of packing (k)

Fuller and Thompson (1907) initially introduced an expression of the particle size distribution line which yields the maximum density for a mixture of aggregate. The expression describes the maximum density gradation for a specific aggregate mixture. The expression is as follows.

$$p = \left(\frac{d}{D}\right)^n \quad (1)$$

where: p = percentage finer than size;
d = aggregate size being considered;
D = maximum aggregate size of the mixture;
n = 0.5 according to Fuller & Thompson.

Using this expression, in the early 1960s, the Federal Highway Administration (FHWA) introduced a standard gradation chart to use in highway industry with n = 0.45. This chart is different to the conventional gradation charts since it uses the sieve size raised to n^{th} power (usually 0.45) as the x axis (Tutumluer *et al*, 2009). Thus, for a selected aggregate mixture, the maximum density line is a straight line passing through the origin and the maximum size of aggregate on 0.45 power gradation plot. On this curve, if one eliminates all the particles smaller than a particular size, the gradation curve of the rest of the particles will be again a straight line on 0.45 power plot as described in equation 1 but with different slope according to the selected minimum particle size. Using this concept, Tutumluer *et al*. (2009) introduced another curve called "characteristic maximum gradation line" for a selected aggregate assemblage. The new curve will be the maximum possible density gradation curve for the new mixture of particles and when slope increases the gradation becomes more uniformly graded and when the slope decreases it becomes more well graded.

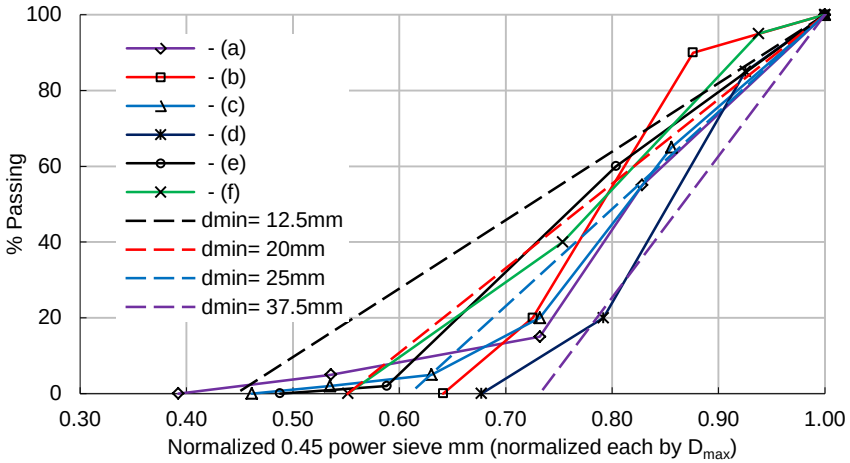

Figure 5 Characteristic gradation curvs on normalized 0.45 power plot;Tutumluer et al. (2009)

Figure 4 shows the calculated track ballast bearing index values against degree of packing for each sample, where samples (e) and (f) demonstrated higher stiffness while samples (b) and (d) produced lower values for ballast bed stiffness. Figure 5 shows gradations, (e) and (f) are well close to d_{min}=20 mm characteristic gradation curve and well-graded compared to other gradations. Therefore it could be suggested that this character of gradation derived a stable aggregate skeleton with efficient load transferring through effective inter-particle contacts to produce high foundation stiffness. Further, gradations (b) and (d) highly deviate from any of the characteristic gradation curves and of more uniform gradation compared to other samples, resulting in lower ballast bed stiffness. This is also consistent with the compaction parameters shown in Table 2. From Table 2 and Figure 4, gradations (a) and (c) demonstrate higher compaction behaviour yet yielding more settlement resulting lower ballast stiffness. This can be explained by analysing corresponding gradations in Figure 5. It is seen that both gradations have noticeable smaller particle content. It could be suggested that the unstable aggregate structure has been generated by the presence of smaller particles separating larger particles apart and acting as bearings in between larger particles. This intervention has generated high void spaces which can be seen in Table 2 and Figure 4. Further, this phenomenon is more evident in behaviour of gradation (a) which has the highest amount of smaller particles. In gradation (c) the smaller particles failed to make ballast skeleton more unstable by separating larger particles apart instead it has filled the voids contributing to increase in the degree of compaction.

4. CONCLUSION

This paper sought to explore the influence of particle size distribution on track modulus incorporating the particle packing behaviour. Results of compaction parameters demonstrate the influence of particle size distribution and content of smaller particle on track modulus. It was reconfirmed that well gradations of aggregate are efficient in load spreading characteristics than more uniformly graded ballast. When particle size distribution is closer to the characteristic gradation curve, the ballast structure seemed more stable and efficient in load spreading with minimal deformation and high stiffness. It is emphasized that the presence of smaller particles in high percentages could affect the track stiffness adversely.

5. ACKNOWLEDGMENTS

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Site Investigation on Rail Track Modulus of Ballasted Rail Track

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Abstract: *With the demand for increasing capacity and efficiency of mass transportation system, railways provide a decent solution being an artery in a country's economy. Compared to the advancement of locomotive, the development of rail track structure lags behind due to the varying nature of the track components compromising the essential safety and efficiency of the whole system. Thus the continuous understanding of the behaviour of rail track is necessary for engineers to design and maintain the rail tracks to meet the expectations. The vertical deflection of the track under the exerted forces of moving train is a good performance indicator of a rail track structure. This deflection is depending upon the collective properties of fasteners, sleepers, ballast and underlying layers. The amount of deflection the track experience under the load reflects the stability of the track and its quality of maintenance. In rail track design, this force and the vertical deformation are related by the track modulus or modulus of elasticity of the track. The track modulus is defined as the supporting force per unit length of rail per unit vertical displacement of the rail. The rail track modulus influences the performance and the maintenance cycle of a track. Higher the track modulus, higher the stresses transferred on to the track components causing more fatigue and wear. On the other hand low track modulus cause high differential settlements and making the track need to be frequently maintained. Different methods have been proposed and being used by various agencies for calculation and measurement of track modulus theoretically and experimentally. In this paper, Track modulus and deflection charts were generated using the method proposed based on continuous beam on elastic foundation approach. Field investigation was conducted using a four axel Class M7 locomotive with weight 66 ton and measured the vertical deflection of sleepers at the passage of locomotive. The measured deflection of the track was used to obtain the track modulus from the deflection and track modulus charts. The calculated track modulus value suggested the track segment is moderately fouled.*

Keywords: *Track modulus, Track deflection, Ballast, Subgrade.*

1. INTRODUCTION

There are the two types of deformations occurring in a rail track structure and which are identified as resilient and permanent deformations. Both these deformations represent two most important aspects of design and performance of track structure. The recoverable deformation of the structure at the passage of train is known as the resilient deformation. The majority of the deformation caused by the imposed load from wheels is recovered after the train has passed. Permanent deformation which is not recovered will be contributed to the total track settlement over the time under repeated loading. In this paper the concern is on the resilient deformation of the track and is an indication of the condition of the track foundation. The structural state of the track could be assessed by measuring the vertical deformation of the track under the wheel load and express in terms of track stiffness or track modulus.

Track modulus has been considered as one of the accepted indicators of the quality and safety of the track structure and defined as the supporting force per unit length of rail per unit vertical displacement of the rail (Selig & Waters, 1994). Track modulus is influenced by many factors such as the quality of rails, ties, rail joints, ballast, and sub-grade and is usually represented by the letter "u" or "k" in literature.

Even though the track modulus is considered as an important parameter it is seldom measured and

the magnitude is mostly unknown for the most of the sections of a track. For a given track there is an optimum track modulus and it should neither be too high nor too low. Higher track modulus value would lead to fatigue, fracture, wear and excessive vibrations of all the track components. On the other way too low track modulus value would cause excessive deflections of the track and even permanent deformations. Studies over the years have related the track modulus to the type of track structure and its subsequent behaviour. But, an upper limit for the track modulus has not been set and need more research and field experiments in defining a value. The track modulus values could range from 3 Mpa to over 100 Mpa (Selig & Li, 1994).

The most commonly known and recommended method of analysis of cross tie track is the beam-on-elastic foundation approach originally proposed by Zimmermann (Esveld, 2001). Based on the theoretical formulation of beam on elastic foundation model, several methods have been developed by various researchers for the determination of track modulus. The approach assumes that the rail responds like an elastic beam which is supported by closely spaced elastic springs attached to a continuous base as shown in Figure 1.

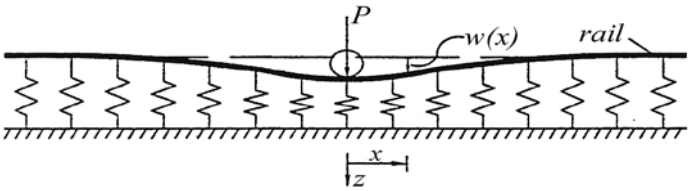

Figure 1 Beam on elastic foundation model, Kerr (1998)

The governing differential equation for this model is as follows,

$$EI\frac{d^4w}{dx^4} + uw(x) = p(x) \qquad (1)$$

in which, w(x) is the vertical deflection of the rail axis at *x, EI* is the vertical flexural stiffness of one rail, u is the track modulus (for one rail), and p(x) represents the vertical wheel loads.
Then, the vertical deflection *w*, for a given point load *P* as a function of the distance x,

$$\mathrm{w(x)} = \frac{P\beta}{2u}e^{-\beta x}\left(\cos\beta\mathrm{x} + \sin\beta\mathrm{x}\right) \qquad (2)$$

where,

$$\beta = \sqrt[4]{\frac{\mathrm{u}}{4EI}} \qquad (3)$$

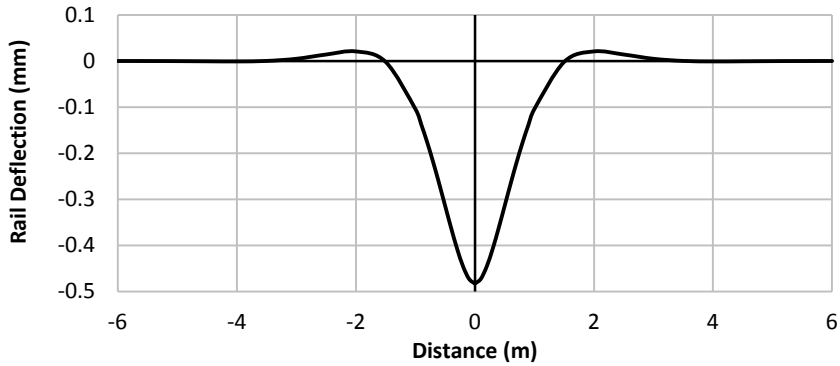

Figure 2 Typical influence line for deflection of rail under single point load

The expression 2 is for a single wheel load and typical influence line for rail deflection caused by a single wheel load is shown in Figure 2. For more than one wheel load, the rail deflections have to be obtained by superposing the effects of the various wheel loads. Using the approach of superposition, Kerr (1998) proposed expression 4 for the rail deflection caused by all four wheels of a two-axle truck based on beam on elastic foundation method.

$$w(0) = \frac{P\beta}{2u} + \frac{P\beta}{2u}e^{-\beta l_2}(\cos\beta l_2 + \sin\beta l_2) + \frac{nP\beta}{2u}e^{-\beta l_3}(\cos\beta l_3 + \sin\beta l_3) + \frac{nP\beta}{2u}e^{-\beta l_4}(\cos\beta l_4 + \sin\beta l_4) \quad (4)$$

In the expression, $P_1 = P_2 = P$ and $P_3 = P_4 = nP$, l_i is the distance to each wheel from left wheel of truck I shown in Figure 3 and the number n is obtained by weighing the two trucks in track scale.

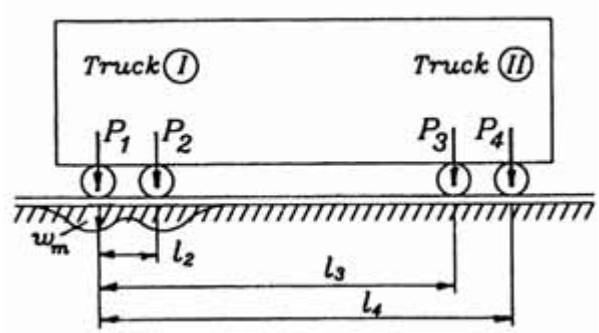

Figure 3 Railway wagon on two-axle bogies and track deflection (Kerr 1998)

From the above expression, the rail track modulus u to be obtained by equating the measured deflection $w_{(0)} = w_{measured}$ as below.

$$\frac{w_m}{p} = \frac{\beta}{2u}[1 + e^{-\beta l_2}(\cos\beta l_2 + \sin\beta l_2) + ne^{-\beta l_3}(\cos\beta l_3 + \sin\beta l_3) + ne^{-\beta l_4}(\cos\beta l_4 + \sin\beta l_4)] \quad (5)$$

For a given field test, u is the only unknown in equation 5. But it is seen that the expression cannot be solved explicitly for u. Therefore Kerr (1998) proposed w_m to be evaluated numerically for a given set of E, I and P with a known range of u and produce a graphical representation of deflection against track modulus. Then the measured deflection is known, the track modulus could be directly obtained using the graph. According to equation 5, it is clear that there will be separate graphs for different rails and loading vehicles due to the influence of E, I and l. In this research, deflection - moduli charts for rail sections 36.2 kg/m (80 lb/yd), 40.8 kg/m (90 lb/yd) and 60 kg/m were generated and used to obtain the track modulus of the test segment of the track by measuring the sleeper deflection.

2. FIELD MEASUREMENT METHODOLOGY

Tests were conducted in Kelanivelley railway line between 6.5th and 7th kilometre posts. Two class M7 locomotives each weight 66 ton were provided by Sri Lanka Railways. Class M7 locomotive has four axels and therefore the axel load is 16.5 ton and wheel load is 8.25 ton. The locomotives were run at low speeds to minimize the dynamic effects. The schematic representation of the locomotive with the axel loading arrangement is shown in Figure 4 and locomotive at site is shown in Figure 5.

In this field investigation, linear variable differential transducers (LVDT) were used for accurate measurement of linear deflection of the each sleeper at the passage of train. LVDT equipment were mounted on firm supports and fixed at the end of selected sleepers. The time history data of vertical deflection of sleeper were acquired by connected data logger and recorded by a computer. LVDT instruments are capable of measuring least 0.005 mm deflection and maximum 50 mm. The used data logger is capable of recording up to 1000 data per second. A load cell with a capacity 200 kN was planted beneath the sleeper directly under the rail to read the load at sleeper ballast interface and connected to the computer via data logger. The sleepers at site were standard timber sleepers with cross section dimensions 127 x 254 mm and 2740 mm long. Line was broad gauge track with 1676 mm gauge. The rail section was 36.2 kg/m (80 lb/yd). At site, the sleeper intervals were different to each other and the detailed schematic diagram of test setup is shown in Figure 6.

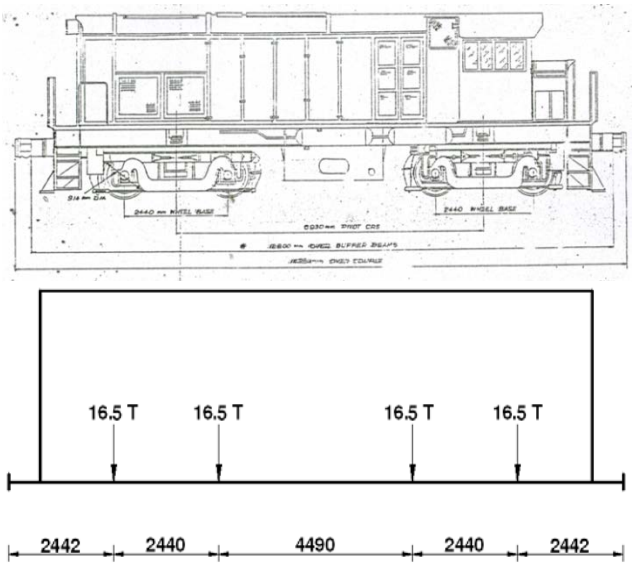

Figure 4 Class M7 Locomotive used for the field investigation with axel loads



Figure 5 Image of test set up & Class M7 Locomotive

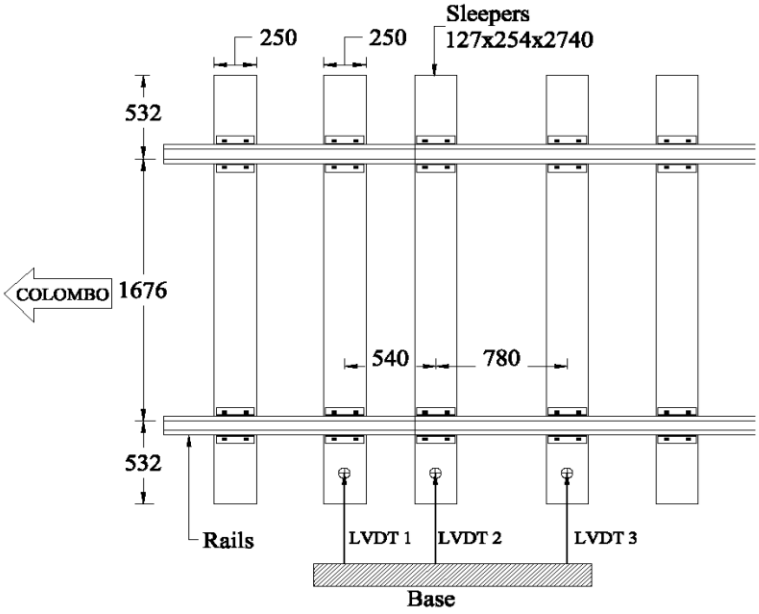

Figure 6 Schematic representation of the test set up

Track deflection and modulus chart was developed using equation 5 proposed by Kerr, 1998 for 36.2 kg/m (80 lb/yd), 40.8 kg/m (90 lb/yd) and 60 kg/m rail sections which are currently available in Sri Lanka. Class M7 locomotive was used as the loading vehicle in generating the curves. At the test segment in Kelanivelley line, the rail section was 36.2 kg/m (80 lb/yd). Produced chart is shown in Figure 7.

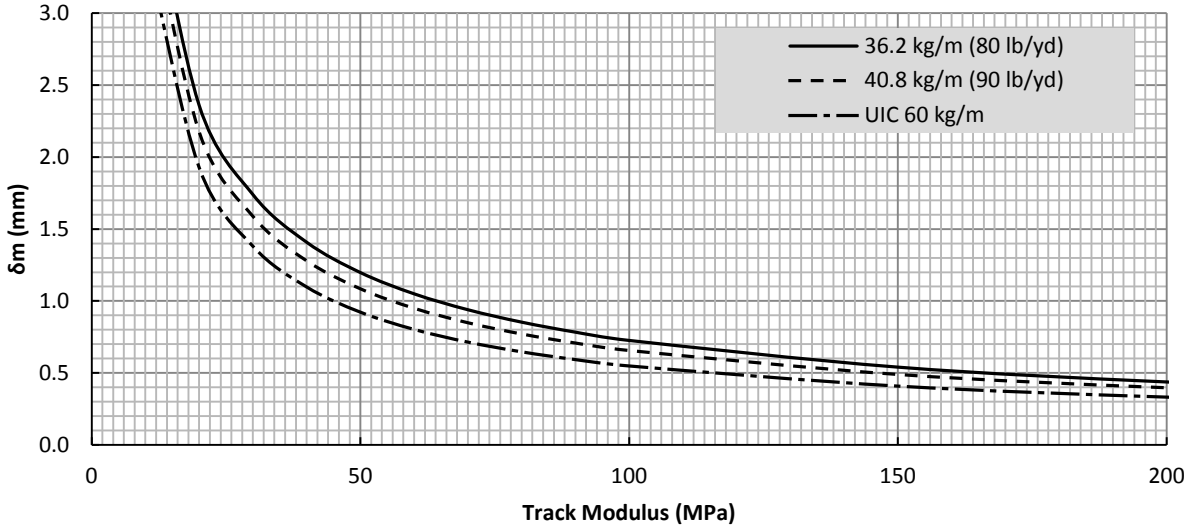

Figure 7: Track deflection – Track Modulus charts for 80 lb/yd and 90 lb/yd rail

Results obtained after the pssage of train from LVDTs were further processed and presented graphycally to compare the deflections at the stations. When the loading vehicle (the M7 locomotive) passed over the sleeper which LVDTs were installed, the data logger records the sleeper difflection as a time history and graph for Station D1 is shown in Figure 8. From the data it is possible to obtain the track diffleciton of the sleeper at any moment.

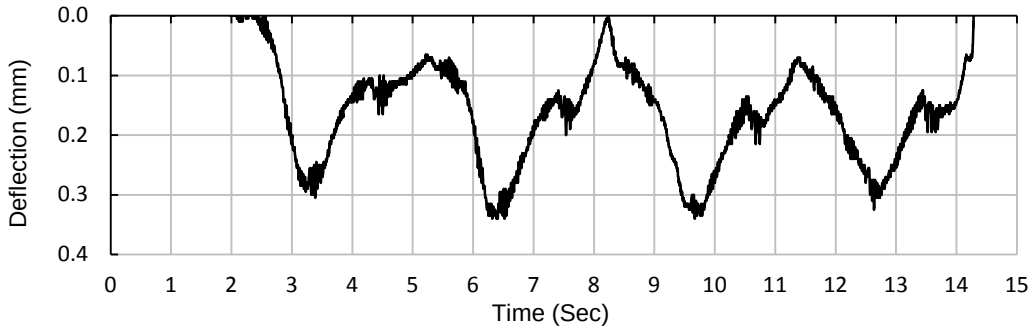

Figure 8 Time history graphs of measured sleeper deflections at stations D1

In this study, the mass of the locomotive was asssumed to be evenly distributed over the axels and therefore the average of difflection caused by the wheels were considered for the calculation of track foundation modulus. The dynamic factors were assumed to be neglegible since the speed of the train was very low for this particular test. It is possible to have a opening between the sleeper and the ballast bed due to the permenent deformaiton of foundation layers or a gap between the rail and the sleepr due to fastening defects where there is no contact in between. This gap is usually refered in the literature as “slack”. The deflection caused by the slack excluded in the track fundation modulus calculaiton described above. Therefor, the measured deflection by LVDT shall be corrceted for an existing slack at the measuring station. In this test, for stations D2 and D3, slack was observed and considered as 15% from the maximum deflection caused by wheel load (Louis M. and Hannes G, 2013) and no slack was observed at station D1. For this test, the recording stations were close and therefore average of the maximum deflectons directly underneath the wheel from all three stations were used to obtain more realistic track foundation modulus for the track segment.

After corrected for the slack, the average deflection (δm) was 1.06 mm for wheel load 80.9 kN (8.25 ton). Then using the curve for 36.2 kg/m (80 lb/yd) rail seciton in Figure 7, the track foundaiton

modulus was obtained as 62 MPa.

The obtained value of track foundation modulus 62 Mpa is exceeding the recommended value 30 Mpa to 53 Mpa, set by Ministry of Railways, india for designing broad gauge traks. The increase of track modulus indicates the state of faouling of the ballast. The ballast bed of the test location was not replaced since the major rehabilitation carried out in 1996. The effect of fouling on the foundation modulus was demonstrated by Zakeri *et al*. (2012). The findings indicated that the foundation modulus tends to increase at higher fouling degrees. For 25% fouling the foundation modulus was at 90 Mpa and at 62% it was around 110 Mpa. Considering the above values, It could be suggested that the degree of faouling at the test locaiton was moderate.

3. CONCLUSION

Track deflection and modulus charts for 36.2 kg/m (80 lb/yd), 40.8 kg/m (90 lb/yd) and 60 kg/m rail section was generated. Track deflection was measured at the passage of class M7 locomotive and the measured deflection was used to obtain the track modulus of the test location using the track deflection and modulus chart. The obtained track modulus value 62 Mpa for the track segment tested between 6.5th and 7th km posts in Kelanivelley track is high compared to the Ministry of Railways, India. This high value of track modulus is an indication of ballast bed fouling. The produced curve for Class M7 Locomotive and for 36.2 kg/m (80 lb/yd), 40.8 kg/m (90 lb/yd) and 60 kg/m rails allows for a rapid and economical determination of assessing the the track modulus u at various track locations and hence assess the structural state of local tracks in the aspect of rehabilitation or upgrading. It should be noted that the curves were generated neglecting the slack and when using the chart the deflection to be corrected for slack.

4. ACKNOWLEDGMENTS

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Comparison of Mesh Type Seismic Retrofitting for Masonry Structures

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Abstract: *The tremendous loss of life that resulted in the aftermath of recent earthquakes in developing countries is mostly due to the collapse of non-engineered building structures. It has been observed that these buildings cannot withstand the lateral loads imposed by an earthquake and often fails, in a brittle manner. This underscores the urgency to find simple and economic solutions to reinforce these buildings. Different conventional retrofitting techniques are available to increase the strength and/or ductility of unreinforced masonry walls. Recent years, several researches work on mesh type retrofitting for masonry structures to delay or prevent the collapse of buildings and reduce the number of lives lost during devastating earthquake events. This paper reviews and discusses the state-of-the-art on seismic retrofitting of masonry walls with emphasis on the mesh type retrofitting techniques include retrofitting procedures, cost, improvement in structural performance and limitations.*

Keywords: *Masonry, retrofitting, meshes, seismic loading, in-plane tests.*

1. INTRODUCTION

The result of earthquake damage investigations and studies conducted in earthquake-prone regions have revealed that the masonry constructed type buildings would collapse within a few seconds during earthquake movement, and does become a major cause of human fatalities. Lack of structural integrity is one of the principal sources of weakness responsible for severe damage leading to collapse. Lack of interlocking units between external and internal wythes of the wall sections and the lack of connection between crossing walls give rise to the possibility of out-of-plane behavior, as their formation increases the net length of the walls. In-plane lateral loads induce shearing deformations in masonry walls. This deformation elongates one diagonal, including tension, and shortens the other, including compression perpendicular to the tension. Since masonry materials have much lower strength in tension than compression, in-plane forces typically induce diagonal cracking perpendicular to the tension axis. Out-of-plane wall collapse is another major cause of destruction of masonry buildings, particularly in buildings with flexible floors and roofs. The inadequacy of connections between the cross walls and long walls is one of the key factors influencing out-of-plane wall collapse. When the walls are not connected to the roof, collapse is often caused. Roof collapse can also be caused by the collapse of walls subjected to shear forces and gravity loads. Heavy roofs also contribute to the seismic vulnerability of masonry buildings.

This study focus on an experimental program of in-plane tests of masonry specimens retrofitted with different mesh types of steel, soft polymer, industrial geo-grid, PP-band and plastic carrier bags. These tests performed to investigate the effectiveness of different mesh types in preventing brittle failure of unreinforced masonry specimens, under in-plane loading.

2. MESH TYPE RETROFITTING

In order to reduce damage on these masonry buildings during earthquakes, which could happen anywhere in the world, it is important to examine at the early stage how to improve and upgrade the

earthquake resistance of an existing masonry construction and to propose a concrete countermeasure method. Researchers assessed the feasibility of applying the various strengthening methods for existing masonry structures in developing countries; it is difficult to make direct comparisons regarding the structural performance of the techniques. The existing methods of retrofitting un-reinforced masonry buildings currently under research or early stages of application, including:

- External reinforcement: bamboo, seismic wallpaper
- Post-tensioning using material such as rubber tyres
- Mesh reinforcement: steel mesh cage, polymer mesh, polypropylene band (PP-band) mesh, Plastic Carrier Bag Mesh

For the seismic safety of the structure, good connections between walls and floors or foundations, between adjacent walls and between walls and roof are essential. Integrity of masonry can prevent large pieces of debris to fall out and injure inhabitants. Considering these facts, recent years, several researches work on mesh type retrofitting for masonry structures to delay or prevent the collapse of buildings and reduce the number of lives lost during devastating earthquake events. The main objective of mesh retrofitting is to hold the masonry components into a single unit and to prevent the collapse of masonry structures. The mesh type retrofitting can be made of any ductile material, including: steel cage, industrial geo-grid, soft polymer, polypropylene band and plastic carrier bag as shown in Figure 1.

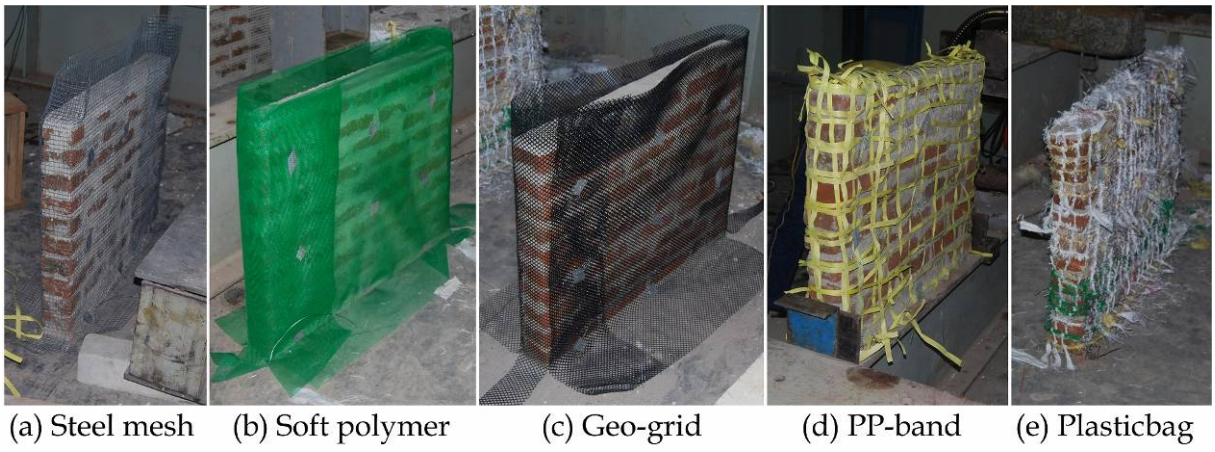

Figure 1. Various mesh types retrofitting techniques used for masonry structures.

Vertical bands help to tie the wall to the foundation and to the ring beam and restrains out-of-plane bending force and in-plane shear force. Horizontal bands help to transmit the bending and inertia forces in transverse walls (out-of-plane) to the supporting shear walls (in-plane), as well as restraining the shear force between adjoining walls and minimizing vertical crack propagation. The horizontal and vertical bands should be tied together and to the other structural elements (foundations, roof, etc.) by means of nylon string. This attachment provides a stable matrix, which is inherently stronger than the individual components. According to Sathiparan *et al* (2008), there is a trend toward a significant increase in cracking in masonry wall retrofitted by meshes, which is generated by the incompatibility of deformation between the masonry and meshes. Generally mesh distributed the stress through the cracks, transferring it to the undamaged portions of the structure. As a result, new cracks appeared. By allowing cracking without the loss of wall integrity, the meshes enhance structural ductility and energy dissipation capacity while holding disintegrated structural elements together, thus preventing collapse. The following are the major types of methods used for mesh type seismic retrofitting of masonry structures;

- Steel Cage: The use of external welded wire mesh has been studied by several researchers as a reinforcement system that could be applied both to new and existing earthen construction (San Bartolome *et al*, 2008; Tetley & Madabhushi, 2007). The mesh is placed in horizontal and vertical strips nailed with metal bottle caps to the adobe walls, and it is covered with a thick cement and sand mortar. Shaking table test results show that, the steel mesh retrofitted model suffered damage, but did not collapse. Clearly the use of reinforcement is successful in improving seismic performance of adobe buildings, but it has been recognized that the use of steel is problematic as

it is expensive and susceptible to corrosion problems.

- Polymer mesh (Industrial geo-grid and soft polymer): This technique uses polymer mesh (geomesh) commonly used in geotechnical applications. The advantage of this material lies in the compatibility with the earthen wall deformation and its ability to provide an adequate transmission of tensile strength to the walls up to the final state. The mesh is attached to adobe walls by plastic or nylon forming a confinement and consequently preventing the total collapse. The researchers found that it is possible for the walls to disintegrate into large blocks during severe ground shaking, however the mesh prevents the walls from falling apart, and collapse can be avoided (Blondet *et al*, 2006). Research performed in recent years also indicated that various polymer mesh retrofitting system such as fiber-reinforced polymer, polymer, textile and polymer carbon mesh are effective strengthening solutions for masonry structures.

- PP-band mesh: PP-band retrofitting is a simple and low-cost method that consists of confining all masonry walls with a mesh of PP-bands. PP-bands are an inexpensive, durable, strong, and widely available material, commonly used for packing. PP-band retrofitting technique is simple enough to be understood and applied by craftsmen and homeowners without any prior knowledge and special expertise, thus, it is expected to meet the very critical requirement of developing countries, the “easy-to-use” method by this retrofitting technique. The applicability of PP-band meshes to retrofit unreinforced masonry structures has been tested under static (Sathiparan *et al* 2005), cyclic (Mayorca & Meguro, 2003) and dynamic loading (Sathiparan *et al* 2012).

- Plastic Carrier Bag Mesh: Thus an innovative new method was tested, using ordinary plastic carrier bags made into a mesh to reinforce the model (Tetley & Madabhushi, 2007). Plastic bags were cut into 20mm strips, these were then plaited together to make “ropes”. These plaited ropes were then knotted together to make 50mm×50mm mesh. The mesh was wrapped around the wall in one continuous piece and fixed to the base using tacks to mimic pegging to the ground. The surface of the wall was plastered with the adobe mortar mixture; this was to hold it in place and for aesthetics. The addition of the carrier bags clearly made the model more ductile. The ductile failure mechanism would mean there would be advance warning of the collapse, which would improve the fatality rates in earthquakes.

3. COMPARISON OF MESH TYPE RETROFITTING METHODS

3.1. Retrofitting Material Strength

In order to obtain the tensile strength and deformation properties of the retrofitting materials, preliminary tensile testing was carried out by several researchers (Bischof & Suter, 2014; Blondet *et al*, 2006). Generally retrofitting material exhibited large tensile strength and deformation capacity. Table 1 summarized the properties of the material used for mesh type retrofitting.

Table 1 Retrofitting material properties comparison

Retrofitting material	Cross-section (mm)	Pitch (mm)	Tensile strength (MPa)	Cut-off strain (%)	Cost per m^2 (Rs.)
Steel mesh	d = 0.86	14	850	> 5.0	325
Soft polymer	1.26 × 1.26	6.8	58	1.5	390
Industrial geo-grid	d = 0.66	2.1	372	1.5	414
PP Band	15 × 0.28	80	171	14.0	180
Plastic bag	25.6 × 0.15	50	25	9.5	178

3.2. Retrofitting Material Cost

The cost of retrofitting material and strengthening cost are major constraints to implement this strengthening method. Strengthen by different type of meshes can be done with 5-15% cost compares with construction cost, in many parts of the world. Retrofitting cost comparison of different strengthening methods is summarized in Table 1.

3.3. Structural Performance - In-plane Behaviour

In order to assess the retrofitting by different meshes, six masonry walls as shown in Figure 1 were constructed. One wall without reinforcement and one each with steel, soft polymer, industrial geo-grid PP-band and plastic beg mesh were tested. The wall dimensions were 900×750×105 mm^3 and consisted of 10 brick rows of 4 bricks each. The panel was placed on a steel I-beam for testing, without any special connection provided between the panel and the steel beam. In order to prevent the wall from sliding, two welded L angle steel plate stoppers at the wall toes were provided. The specimens were first subjected to vertical pre-compression and then to horizontal racking. The concrete beam was cast in dimension of 920x180x160 mm^3 and weighted around 160 kg. The testing setup is shown in Figure 2. The horizontal load was applied by means of a hydraulic actuator. The actuator had hinges on both ends and hung from the loading frame to prevent that its weight induced any flexural moments on the specimen. The horizontal load was applied through a steel plate on the brick surface.

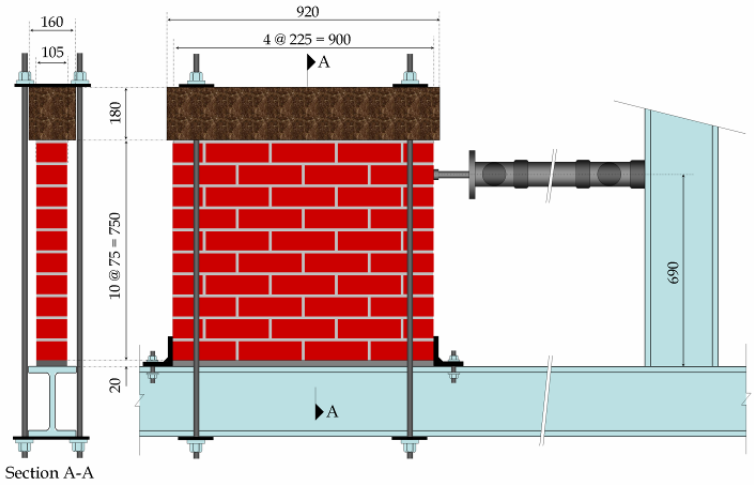

Figure 2. Test setup for in-plane test

Each panel was subjected to a static cyclic loading sequence of 11 successive motion steps (test phases) with increasing amplitude in the same direction, where in each phase wall becomes loaded and then unloaded. Half-triangle shape was adopted for one side cyclic loading up to 75 mm amplitude and ramp wave adopted for 100 mm amplitude.

3.3.1. General Response of the Walls

As observed during the experimental program; initially in the non-retrofitted panel, the first crack was appeared at the base of the panel near the second layer. Afterwards the panel was splited into two pieces subjected to sliding without propagating further cracks. In case of geo-grid retrofitting panel, initial crack was appeared in the eighth row of the panel and then sequentially, it was propagated to diagonal cracks in the downward direction. At the same time uniform diagonal crack was appeared along with small cracks in the loading side of the wall. The upper portion of the panel above eighth row remains as a rigid block while subjecting to sliding. In the polymer retrofitted wall, the first diagonal crack was propagated at an upper layer and then subsequently the cracks become gradually longer and wider as the horizontal load is increased. In the steel mesh retrofitting panel after having the initial cracking, when the horizontal load is increased few additional cracks were appearing progressively. In case of PP band retrofitted panel, the initial crack was propagated at the fourth row of the panel and then a larger diagonal crack appeared starting from the eighth row at the bottom of the wall while dividing the wall into two pieces. In the plastic bag mesh retrofitted panel, the first crack was progressed across the seventh layer and then it was not propagated towards the bottom of the panel. Crack patterns of non-retrofitted panel and each retrofitted panels were shown in Figure 3. Figure 4 shows the shear resistance - drift relationship for non-retrofitted and mesh strengthened masonry wall panels under in-plane loading. It shows that after the initial breaking point, mesh strengthened wall panels have comparatively higher residual strength values than non-retrofitted masonry panel.

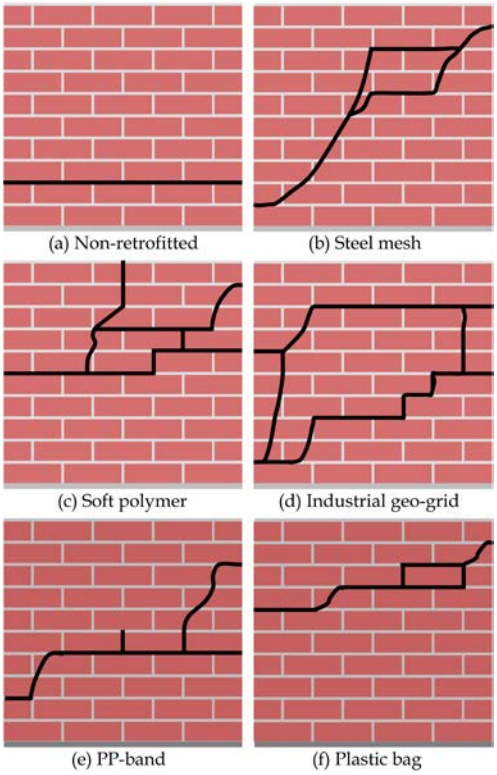

Figure 3. Crack pattern for different mesh type retrofitting

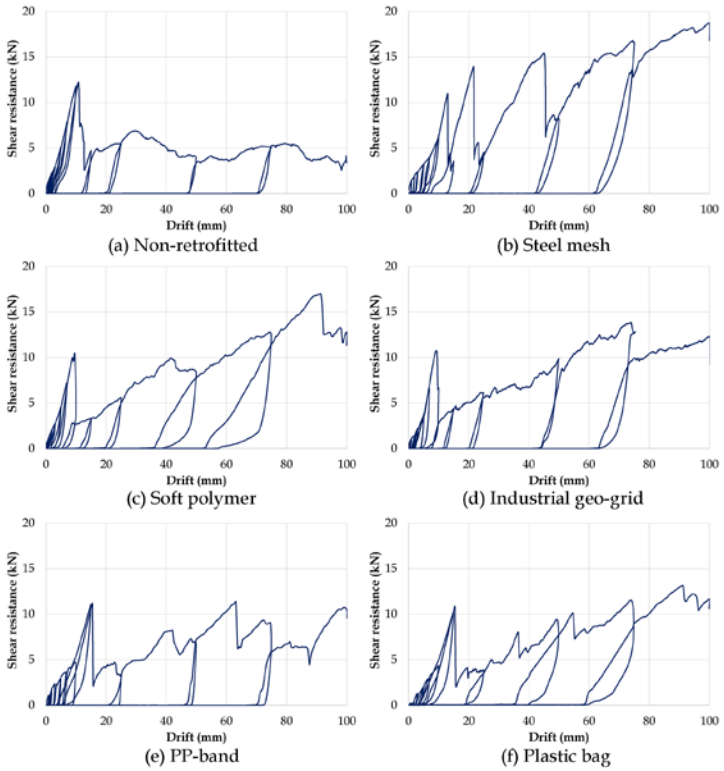

Figure 4. Shear resistance vs. deformation for different mesh type retrofitting

3.3.2. Basic Parameter Comparison

In order to compare the behavior of retrofitted masonry walls in a common base, the behavior of each panel was idealized as shown in Figure 5. Initial strength (Vo) and Initial stiffness (Ko) mainly depends on the masonry properties of brick, mortar and workmanship. Maximum residual strength after the initial crack (Vr) and residual stiffness (Kr) mainly depend on mesh strength and density.

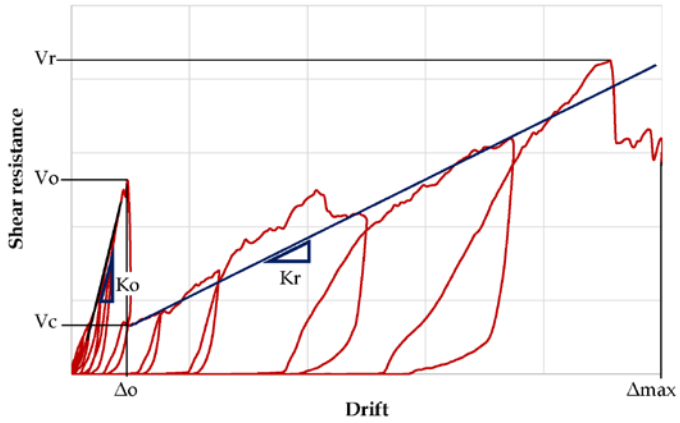

Figure 5. Real and ideal behavior of mesh retrofitted wall.

The effective stiffness of a wall is defined by the gradation of the force - deformation graph. According to the force deformation curves presented in Figure 4, residual stiffness values and strength values were obtained for each mesh type retrofitted panels under in-plane loads. The comparison of basic parameters for each mesh type panel is shown in Figure 6 and Figure 7. Those values show that the mesh retrofitted panels have relatively higher residual stiffness and strength than the non-retrofitted panel. Specifically steel mesh retrofitted panel has the highest residual strength among those different mesh types retrofitting materials. Subsequently residual strength is lesser for polymer, geo-grid, PP band and plastic bag mesh retrofitted panels. While, non-retrofitted masonry panel shows the lowest

residual strength value after the initial breaking point. In terms of residual stiffness also mesh type retrofitted panels show a similar behaviour where steel mesh retrofitted panel has the highest residual stiffness and subsequently residual stiffness is lesser for polymer, geo-grid, PP band and plastic bag mesh retrofitted panels.

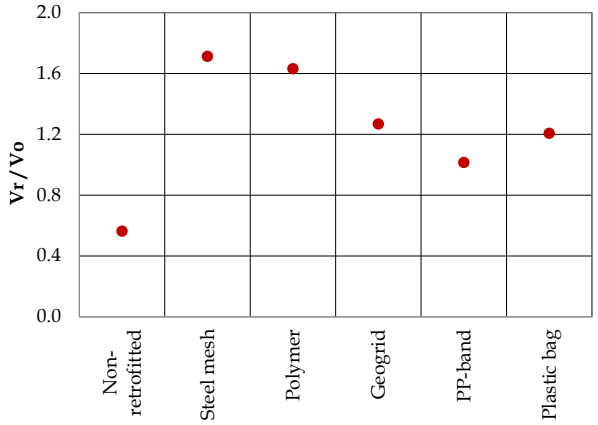

Figure 6. Residual strength/Initial strength ratios for various retrofitting methods

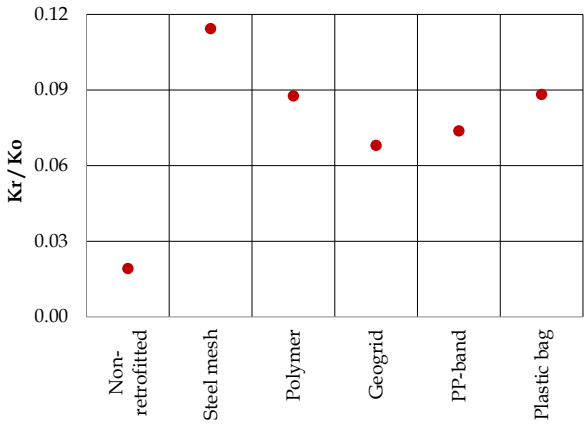

Figure 7. Residual stiffness/Initial stiffness ratio for various retrofitting methods

3.3.3. Stiffness Degradation

Stiffness is the ability of a material to deform beyond the elastic range without fracturing or breaking and is an important parameter in seismic design structures. When structures are subjected to reverse cyclic loading they exhibit a gradual loss of lateral stiffness normally referred as stiffness degradation due to the inelastic deformation. Therefore the determination of stiffness degradation of different mesh type retrofitted wall panels under the in-plane loads were analyzed and summarized in Figure 8. It shows that after the initial cracking point, even at 100mm loading cycle also residual stiffness of steel mesh retrofitted panel is comparatively higher than the geo-grid and polymer mesh retrofitted panels. Also, it shows that non retrofitted panel has the lowest stiffness at 100 mm loading circle.

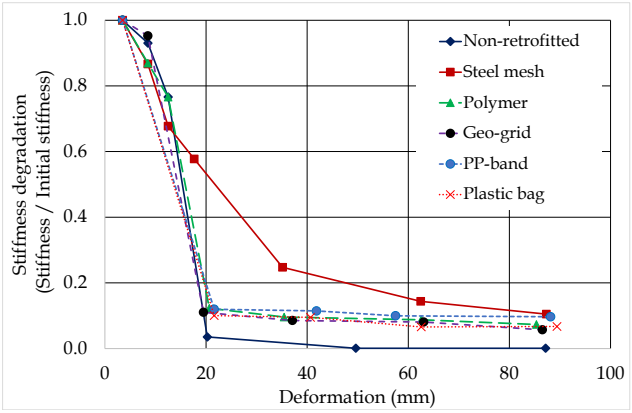

Figure 8. Stiffness degradation for various retrofitting methods

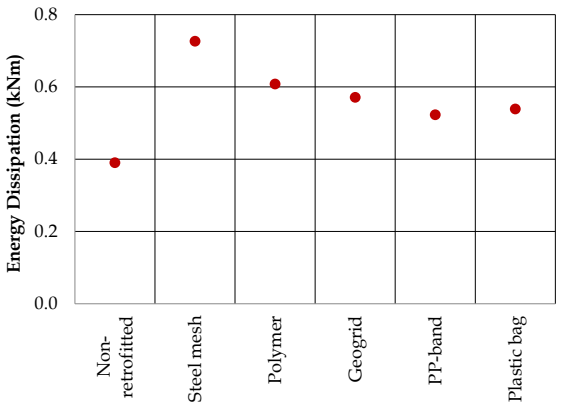

Figure 9. Cumulative energy dissipation capacities for various retrofitting methods

3.3.4. Energy Dissipation

Energy dissipation through hysteretic damping is an important aspect in seismic design since it reduces the amplitude of the seismic response and reduces the ductility and strength demands of the structure. The cumulative energy dissipated under the in-plane load is evaluated for each mesh type retrofitting panel on the basis of the measured shear resistance - drift hysteretic loops and summarized in Figure 9. It shows that steel mesh has the highest cumulative energy dissipation under in-plane loads and subsequently polymer, geo-grid, PP band and plastic bags have lower energy dissipation capacities. Non-retrofitted panel has the lowest energy dissipation capacity.

3.4. Construction Complexity and Seismic Safety

Table 2 presents a construction complexity and seismic safety comparison of various retrofitting method. The first parameter is related to the complexity of construction, ranging from simple to complex based on house owners or masons experience. Complexity mainly depended on retrofitting material flexibility, preparation of the mesh, amount of surface finishing required and knowledge required to retrofit the structures. The second parameter ranks the reinforcement systems, according to the seismic safety they provide. The objective of providing seismic retrofitting of the building is to protect the lives of the building occupants. Based on that, suitability of polymer mesh and PP-band mesh retrofitting method are considered as strong option; steel mesh and plastic carrier bag retrofitting method are considered as moderate option for application (Smith & Redman, 2009).

Table 2 Construction complexity and Seismic safety of retrofitting methods.

Retrofitting Method	Construction Complexity	Seismic safety
Steel mesh	Simple	Moderate
Soft polymer mesh	Moderate	High
Industrial geo-grid	Moderate	High
PP-band mesh	Moderate	High
Plastic carrier bag	Moderate	Moderate

3.5. Advantages and Drawbacks of Retrofitting Methods

Table 3 summarized the advantages and drawbacks of various mesh type retrofitting methods (Sathiparan *et al*, 2012; Smith & Redman, 2009; Tetley & Madabhushi, 2007). Lack of flexibility of steel and industrial geo-grid makes its application difficult, but soft polymer, PP-band and plastic carrier bag can be easily deformed so application is easy. The major sustainability problem with these methods is demolished and disposal of retrofitting material at the end of the structural life.

Table 3 Advantages and drawbacks of different mesh type retrofitting methods.

Method	Advantages	Drawbacks
Steel mesh	• High increment in lateral resistance. • Improves ductility.	• Lack of flexibility of steel cage makes installation around opening and the corner is difficult. • Steel is susceptible to corrosion problems and disposal at the end of the design life will be difficult.
Soft Polymer	• Soft polymer mesh can be easily deformed so transportation, application and removal are easy.	• Polymer mesh requires use and processing of petrochemicals, which is not sustainable unless sourced from recycled or re-used units.
Industrial geo-grid	• High increment in lateral resistance. • Improves ductility.	• Tough nature of material and lack of flexibility makes Industrial geo-grid application and removal is difficult.
PP-band	• PP-band is water proof and chemically stable, it is possible to use with mud mortar. • PP-band is light weight and can be easily carried in the mountainous region and so on. • Retrofitting is simple enough for application without any prior knowledge.	• PP-band is not available as mesh, so preparation of mesh required special equipment and time.
Plastic carrier bag	• The materials used are light and flexible. • Carrier bags are incredibly cheap. • Carrier bags are normally sent to landfill or thrown away. Reusing these bags is therefore considered a sustainable solution.	• It takes a very long time to construct the mesh, which is a barrier for this method unless the mesh manufacture can be industrialized/streamlined in some way.

4. CONCLUSIONS

This paper discusses the results of a series of in-plane tests that were carried out for non-retrofitted and different mesh type retrofitted panels. The test results show that mesh type retrofitted panels obtained comparatively higher residual seismic characteristics rather than a non retrofitted panel as the post collapse strength. So considering the overall performance, it can be concluded these mesh types can effectively increase the residual seismic capacity. But, tests conducted only for in-plane behaviour, so it is difficult to make direct comparisons regarding the overall structural performance of the techniques. It is therefore recommended that further work is carried out on the some more tests on out-of-plane and dynamic tests, which can provide some sort of structural performance comparison of the methods discussed in this study. Generally, when selecting the particular mesh type, in addition to structural performance following characteristics should be considered;

- Available in local or in the market
- Low-cost when compared with others
- Masonry have a high water absorption capacity, therefore, meshes should be non-corrodible
- Mesh roughness, it is important that the mesh surface is unpolished to provide a good grip
- Thickness of the mesh, without making the plaster difficult to apply
- Flexible material, which can provide easy installation.

Even though above mention mesh types retrofitting have some drawbacks, still the mesh type material was considered for retrofitting of masonry houses because they are affordable and notably improve the structure seismic behavior.

5. ACKNOWLEDGMENTS

The authors express their sincere gratitude to the support given by Building Material and Construction laboratory, Department of Civil and Environmental Engineering, Faculty of Engineering, University of Ruhuna.

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Experimental and Numerical Investigation on Strength Degradation of Corroded Steel Bridge Members under Cyclic Loading

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Abstract: *Steel bridges are usually essential elements of road and railway infrastructures; consequently it plays a major role in any countries' economy. Damages in many old steel bridge structures occur due to age related deterioration of materials which causing unavoidable strength reduction which is becoming a serious problem in all over the world today. Many dynamic loadings like recent earthquakes demonstrated potential collapsing vulnerability of some steel bridges. Corrosion and its effects can prompt the damages caused by dynamic excitations on the structures. Therefore there is an intensified importance of careful evaluation of remaining load-carrying capacity of existing structures for understanding of the feasibility of those steel structures for the current usage and to evaluate the necessity of retrofitting of selected corroded members to strengthen the existing structure to ensure public safety. This paper proposes simple and reliable methodology to estimate remaining yield and ultimate cyclic strength by measuring only the minimum thickness of a corroded surface based on the results of many experimental coupon tests of actual corroded plates with different corrosion conditions.*

Keywords: *Residual strength, Cyclic loading, Steel plates, Finite element analysis, Corrosion, Bridge structures*

1. INTRODUCTION

Bridges are a major component of any infrastructure and the failure of a bridge will affect the economy of any country. When bridge structures expose to harsh environmental conditions result will be time-variant changes of their load-carrying capacity. Once the load carrying capacity reduced, the bridges' ability to safe service too violates (Appuhamy *et al,* 2009). In the future, it is evident that serious social problems will arise when the number of damaged bridges increases, as it is very difficult to retrofit or rebuild those aged bridges at the same time. Therefore, it is important to evaluate the remaining strength capacities of those bridges, in order to keep them in-service until they required necessary retrofit or rebuild in appropriate time.

Benefits of regular and proper inspections of older bridges cannot be disregarded. They not only help in planning the necessary work but also help in discovering and monitoring any problems, thereby reducing expensive maintenance, reducing operating hazards, preventing structural failures and preventing emergencies. Therefore, no negligence in inspections should be permitted as they form the essential source of information to carry out a comprehensive evaluation of its current capacity. Some

researchers have already done several experimental studies in terms of estimating the remaining strength capacities under monotonic loading, and developed some durability estimation techniques which were performed with detailed investigations on corroded surfaces (Matsumoto *et al,* 1989; Muranaka *et al*, 1998; Kariya *et al,* 2003; Appuhamy *et al,* 2009).

Recent severe earthquakes worldwide have shown that steel bridges can be vulnerable. In addition, there were many collapse, buckling, fracture and cracking occurred in many steel infrastructures due to earthquakes. It is evident that, even though most of the steel bridge structures perform well with their specific energy dissipation characteristics, the failure risk associated with severe corrosion under mega earthquake events could not be underestimated. Therefore this paper investigate the effect of corrosion damage on remaining dynamic behaviour of existing steel bridge infrastructures and presents a simple, accurate and reliable methodology to estimate residual strength capacities of corroded steel plates using both experimental and numerical analysis with the results of coupon tests and finite element modelling technique. Further validation of use of corrosion condition modelling parameters (CCM) as a more reliable methodology to model the corrosion surface in FEM modelling software in cyclic strength estimation is also considered.

2. METHODOLOGY

2.1. Experimental Analysis

2.1.1. Corroded Test Specimens

The test specimens for this experimental study were cut out from a steel girder of the Koggala Bridge in Sri Lanka on the shoreline of the Indian Ocean, which had been used for about hundred years. This bridge was constructed as a railway bridge in 1900s and this bridge was dismantled due to serious corrosion damage in year 2012.

The specimens for cyclic loading tests were taken, cutting out from the corroded members of the bridge. Four corrosion-free specimens, cut down smoothly from both sides of corroded steel plate in order to clarify the material properties. Test specimens prepared according to the JIS No.5 as shown in Figure 1 and Figure 2.

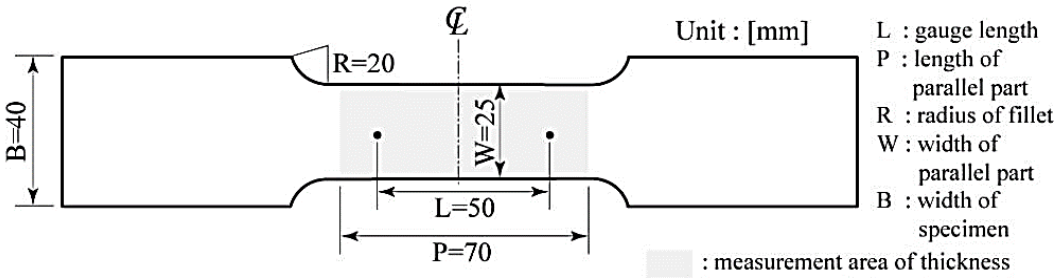


Figure 1 JIS No.5 Specimen for cyclic loading test



Figure 2 Prepared test specimens according to standard shape

2.1.2. Corroded Surface Measurements and Material Properties

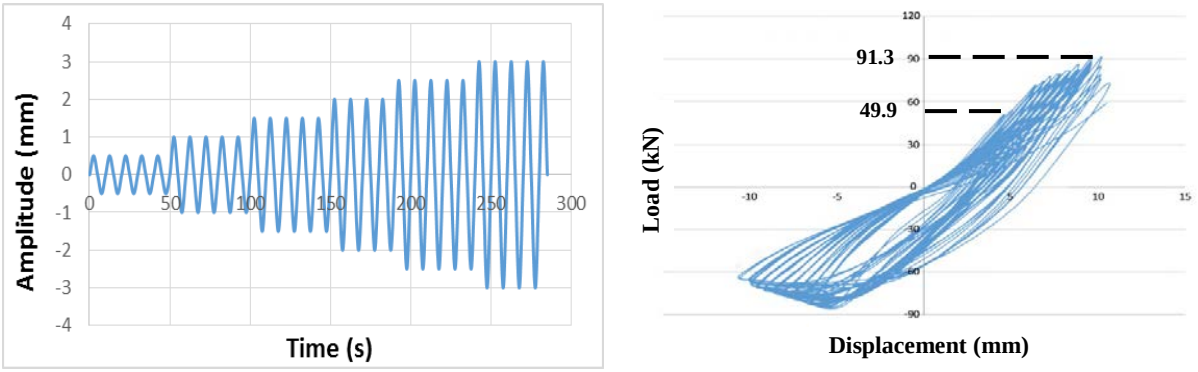
The rust and paint on the surface were removed by using a steel wire brush and then applying high pressure water carefully in order to not change the condition of the corrosion irregularity. Then the thicknesses of all scratched specimens were measured by using a 3D scanning of surface before the cyclic loading test and the intervals of measurement data are 2mm in X and Y directions respectively. The statistical thickness parameters such as average thickness (t_{avg}), minimum thickness (t_{min}), were calculated from the measurement results. Tensile testing was performed for the four corrosion-free specimens. The fundamental mechanical properties of the material were obtained and checked with the standard values by JIS as shown in Table 1.

Table 1 Material properties of test specimens

Non Corroded Specimen no.	Elastic Modulus (GPa)	Yield Stress (MPa)	Tensile Strength (MPa)	Elongation at Breaking (%)
Sample set 1	184.4	272	431	25.59
Sample set 2	180.1	246	418.5	20.01
SS400 JIS	200	245~	400~510	-

2.1.3 Cyclic Strength Determination by Cyclic Loading Test

Each specimen was tested under predetermined cyclic displacement as shown in Figure 3-a and applied load and the stroke displacement was recorded as shown in the Figure 3-b. Displacement histories were analysed to identify actual yield and ultimate cyclic strength of each specimen; shown in Table 2.



(a) (b)

Figure 3 (a) Graph of amplitude vs. time of applied cyclic displacement pattern, (b) Load vs. displacement curve of specimen (μ = 0.359) obtained by experimental analysis

Table 2 Yield and Ultimate cyclic strength values of test specimens

Minimum Thickness Ratio (t_{min}/ $t_{initial}$)	Yielding Load - P_b (kN)	Ultimate Load - P_y (kN)
1.000	51.25	95.10
1.000	38.70	61.92
0.375	42.90	71.90
0.378	31.45	47.99
0.456	33.73	54.02
0.743	37.06	58.89
0.691	48.45	87.8
0.359	49.89	91.30
0.871	50.38	92.67
0.689	34.67	57.07
0.785	49.00	90.53

2.2. Numerical Analysis

Numerical analysis was conducted based on finite element modelling to have more reliable strength data easily. Usually, the accurate predictions are based on how accurately statistical parameters are estimated and therefore mainly depend on experimental and field data. But, to develop a more reliable strength estimation technique, only experimental approach is not enough as actual corroded surfaces are different from each other. Further, due to economic constraints, it is not possible to conduct tests for each and every aged bridge structure within their bridge budgets. Therefore use of numerical analysis method could be considered to have a reliable estimation in bridge maintenance industry.

2.2.1. Numerical Procedure

Actual specimens were modeled as it is in finite element modeler and actual material properties obtained were assigned and the model was compiled by assigning the same loading condition given to actual experimental specimens at the experimental analysis. Obtained results for yield and ultimate strength capacity were compared with the actually obtained strength

2.2.2. Modelling of Corrosion Condition

Two corrosion condition modelling (CCM) parameters were defined to model a corroded surface considering the material loss due to corrosion and stress concentration effect (Ohga *et al,* 2011) and (Appuhamy *et al,* 2011).

$$D^* = 5.2\, t_{c,max}, \tag{1}$$

$$t^*_{avg} = t_0 - 0.2 t_{c,max}, \tag{2}$$

Where D^* and t^*_{avg} are the representative diameter of maximum corroded pit and representative average thickness respectively. ($t_{c,max}$ = maximum corroded depth, t_0 = initial thickness)

Actual specimens were modelled using CCM parameters and analysed using validated FEM modeller and cyclic strength values were obtained. Stress distributions of different analytical models using CCM parameters at yield load are shown in Figure 4.

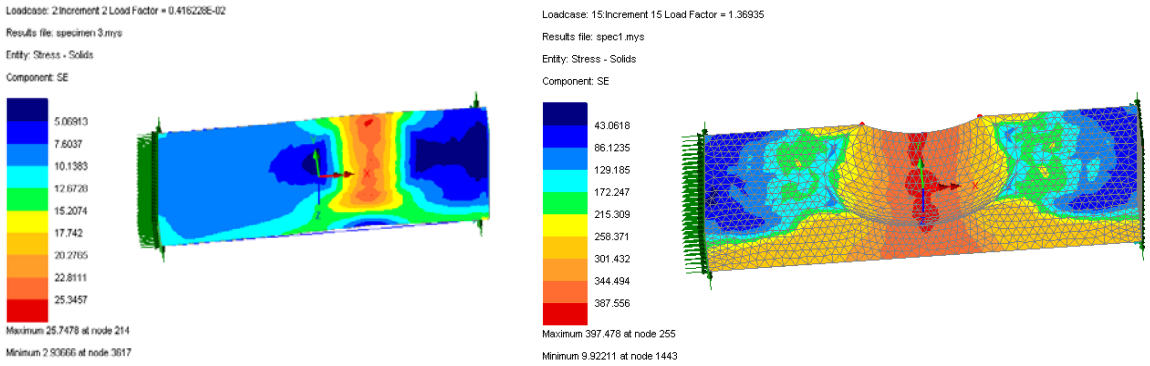


Figure 4 Stress distributions of different analytical models using CCM parameters at yield load

Comparison of experimental results with analytical results shows that there is a close relationship between strength results as shown in Table 3. The percentage error varies within negative four and positive six, which can be considered as a considerably small value. Therefore obviously the corrosion condition modelling parameters derived for tension can be effectively utilized under cyclic or seismic loading conditions.

Table 3 Ultimate cyclic strength values of test specimens under numerical analysis

Minimum Thickness Ratio (t_{min}/ $t_{initial}$)	Experimental ultimate load (kN)	Numerical ultimate load (kN)	Percentage Error (%)
1.000	95.10	94.50	0.63
1.000	61.92	63.01	-1.76
0.375	71.9	70.00	2.64
0.378	47.99	45.01	6.23
0.456	54.02	55.25	-2.27
0.743	58.89	56.97	3.26
0.691	87.80	84.00	4.32
0.359	91.30	94.70	-3.72
0.871	92.67	87.08	6.032
0.689	57.07	55.54	2.68
0.785	90.53	90.53	-0.58

3. RESULTS AND DISCUSSION

The percentage yield and ultimate strength reductions (%SR) were obtained by comparing the strength results of corroded specimens with non-corrosion specimen from same source according to the Equation 3. And the percentage strength reduction was plot against minimum thickness ratio of each specimen as shown in Figure 5 and Figure 6.

$$\%SR_{Yield/ultimate} = \left|\frac{P_{Y(Non-corroded\ specimen)} - P_{Y(Model\ with\ specimen)}}{P_{Y(Non-corroded\ specimen)}}\right| \times 100 \qquad (3)$$

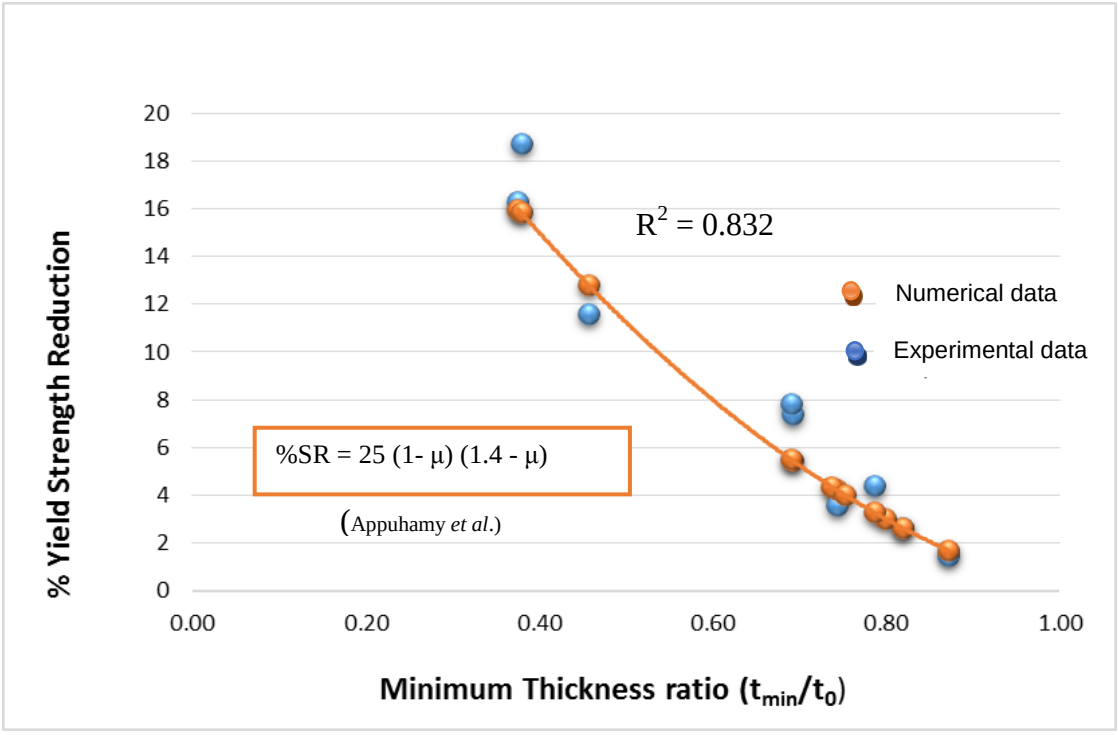


Figure 5 Relationship of percentage yield strength reduction vs. minimum thickness ratio (μ)

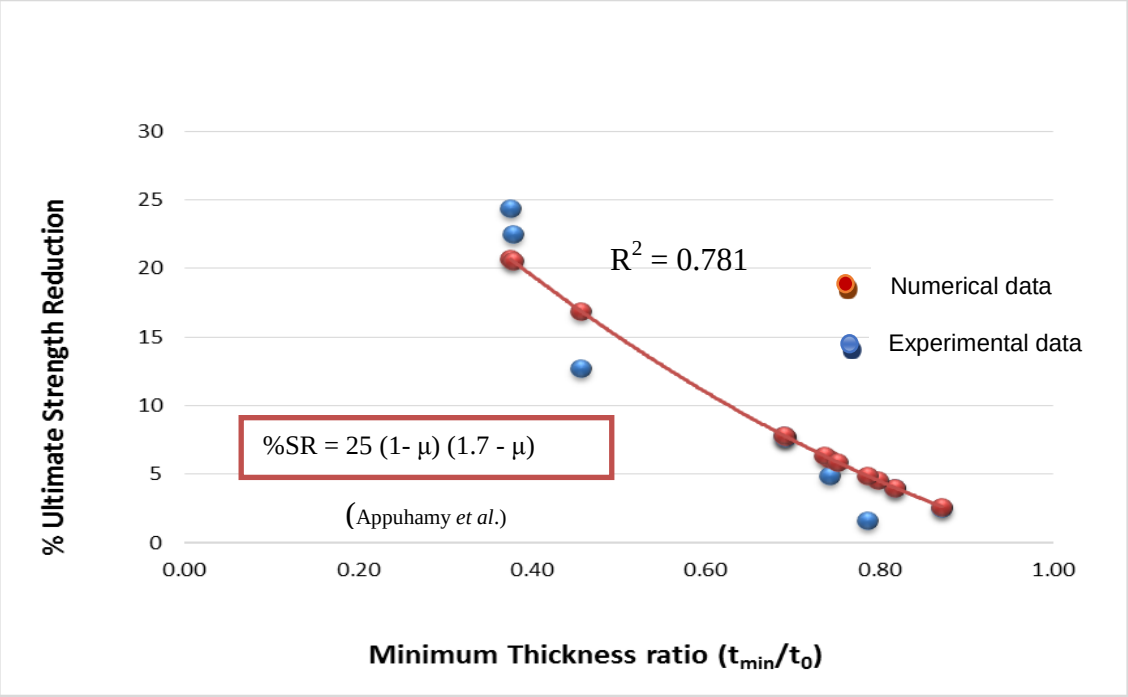


Figure 6 Relationship of percentage ultimate strength reduction vs. minimum thickness ratio (μ)

Obtained relationships between yield and ultimate percentage strength reduction values with minimum thickness ratio of corroded specimens were experimentally tested, having close relationship with the relationship equations derived by previous researcher using numerical investigation (Appuhamy *et al,* 2011). Therefore obviously the previously derived equations for finding out residual strength by using only measurable parameter is experimentally validated.

Thus $\%SR_{yield} = 25\ (1- \mu)\ (1.4 - \mu)$, $\%SR_{ultimate} = 25\ (1- \mu)\ (1.7 - \mu)$, can be used as a tool to estimate the residual strength of prevailing corroded bridge parts by only measuring minimum thickness.

4. CONCLUSIONS AND FUTURE DIRECTIONS

The yield, ultimate behaviors of steel bridge members with different corrosion conditions under cyclic loading were studied in this research. The main conclusions of this study can be summarized as follows.

It was revealed that the corrosion has a significant effect on the dynamic behavior of steel bridge infrastructures. As a reliable and efficient methodology to estimate the percentage reduction of yield/ultimate strengths due to corrosion under cyclic loading can be obtained using following Equation (4) and Equation (5).

$$\%SR_{Yield} = 25 \times (1 - \mu) \times (1.4 - \mu) \quad (4)$$

$$\%SR_{Ultimate} = 25 \times (1 - \mu) \times (1.7 - \mu) \quad (5)$$

As the proposed strength and energy reduction equations only requires the measurement of minimum thickness ratio μ, which is an easily measurable parameter through a quick and careful site investigation, this method can be used as a simple and reliable method to predict the cyclic, seismic behaviors of corroded steel members more easily and precisely. Furthermore, as the %SR charts give a good indication about the percentage strength reduction according to the severity of corrosion, bridge engineers would be able to decide whether the infrastructure requires any initial corrosion prevention precautions such as painting etc., retrofitting of some selected members or replacement of some critical members in order to assure the adequate safety of the existing structure.

In addition a very good agreement between experimental and nonlinear FEM results can be seen for all three classified corrosion types. Thus the adopted modelling technique can be used to predict the remaining strength capacities of actual corroded members accurately.

Usually, the accurate predictions are based on how accurately statistical parameters are estimated and therefore mainly depend on experimental and field data. But, to develop a more reliable strength estimation technique, only experimental approach is not enough as actual corroded surfaces are different from each other. Further, due to economic constraints, it is not possible to conduct tests for each and every aged bridge structure within their bridge budgets. Therefore use of numerical analysis method could be considered to have a reliable estimation in bridge maintenance industry.

Therefore adopted numerical modelling technique can be precisely used as a more reliable method to model run and retrieve the residual strength data of modelled actual bridge members.

Finally it can be concluded that this research findings have immense importance in bridge maintenance and management industry as well as the ultimate goal of this findings may safeguard human lives and property from accidental collapses in bridges all around the world.

Further experimental analysis using different cyclic loads and further use of finite element modelling method to identify stress variation due to interconnected corrosion pits for further retrofitting and bridge management decisions can be suggested as future directions.

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Post-Strengthening of Un-Reinforced Masonry Structures with Seismic Belts

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Abstract: *Un-Reinforced Masonry (URM) buildings constitute a significant part of existing building inventory in Sri Lanka. Past earthquakes have shown that these buildings are highly vulnerable to earthquakes. Application of seismic-belts is one of the commonly used techniques for seismic retrofitting of such buildings. The present experimental study performed to investigate the efficacy of seismic-belts (Ferro-cement belts) on retrofitting of masonry walls with openings. Two unreinforced panels: one without opening and one with opening, and three strengthened panels are subjected to the reversed cyclic lateral in-plane loading. Three different seismic-belts configurations: vertical, horizontal and combination of vertical and horizontal, are evaluated. Size of each specimen and opening was 640 x 600 x 215 mm and 200 x 225 mm, respectively. Under the in-plane loading, URM walls and all the retrofitted walls were subjected to a combination of shear and flexural mode of failure. All the retrofitting configurations improve the lateral resistance, initial stiffness, energy dissipation and ductility factor of the URM wall where combination of both horizontal and vertical seismic-belts causes a significant enhancement of those parameters.*

Keywords: *Unreinforced masonry, post strengthening, seismic belts, in-plane cyclic loads, retrofitting.*

1. INTRODUCTION

Masonry construction is widespread in all regions of the globe and Sri Lanka is a country of masonry construction by excellence; this method of construction is still currently the most often used in the country. Despite this wide usage, some points of the masonry behaviour remain to be investigated; the response of Un-Reinforced Masonry (URM) walls under dynamic loading is part of these active research issues. Particularly, the behaviour under seismic loading is a “complex and ill-understood” aspect in Sri Lanka. During earthquake, the masonry walls are subjected to both in-plane and out-of-plane action. URM walls are very weak in out-of plane bending action due to extremely low tensile strength of the mortar-brick interface and resist the lateral load primary in in-plane action. However, the door and window openings in the walls, are sources of weakness in in-plane action of walls. Openings result in reduction of effective cross-sectional area of walls resisting lateral loads. Further, in-plane failure is the global and a fundamental failure mode for URM.

Seismic improvement techniques are used to limit damage by increasing the strength and by prolonging the onset of failure by increasing the ductility of the URM building (Agarwal and Shrikhande, 2007). Various seismic strengthening methods have been evaluated and are available worldwide such as old tire strips, fibre reinforced polymer, poly-propylene band mesh, stitching and grout injection, re-pointing, bamboo reinforcement, post-tensioning using scrapped rubber and engineered cementitious composites, reinforced surface plastering, grouted centre core steel insertion and near surface mounted steel insertion. Some of these techniques are costly and others are less in seismic performance improvement in masonry. Strengthening of URM buildings with ferro-cement method is simple and economical. The ferro-cement strips consisting of welded Steel Wire-Mesh (SWM) reinforcement embedded in cement-sand mortar or micro-concrete have a composite action

with the URM resulting in significantly enhanced strength and ductility in shear, rocking and out-of-plane bending. The bandages provide the support to the walls in out-of-plane action, whereas the splints enhance the shear and rocking behaviour of the piers (Kadam, Singh and Li, 2014). The horizontal 'bandages' are applied continuously on all the walls at lintel, sill, floor, and roof levels, whereas the vertical 'splints' are applied at corners and junctions of walls and along the jambs of the openings. One of objectives of this study is to study dynamic in-plane behaviour of URM walls with and without openings/windows. Another is to introduce and evaluate different configuration of seismic belts technique to enhance lateral in-plane load resistance of masonry walls with openings. In the present paper, a comprehensive experimental research on full-scale brick masonry panels with openings strengthened with several configurations of seismic belts and subjected to reversed cyclic in-plane loading is presented.

2. METHODOLOGY

Total five URM walls: four panels with openings and one panel with no opening were cast to conduct the experimental study. The specifications of each wall unit are summarized in Table 1.

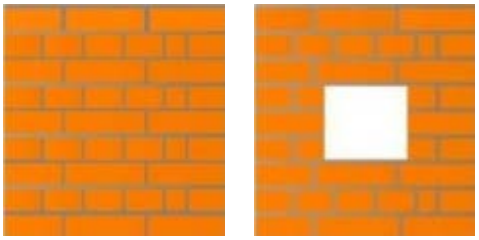
Table 1 Proposed retrofitting techniques and nature of the test masonry wall panels

Specimen	Retrofit Scheme
URMWNO	URM shear wall without opening – No retrofit
URMWOC	URM wall without opening, Control unit – No retrofit
RMWOHS	100 mm high two SWM belts – Horizontal application around opening
RMWOVS	100 mm high two SWM belts – Vertical application around opening
RMWHVS	100 mm high four SWM belts – Horizontal and Vertical application around opening

One wall panel (i.e., URMWNO – Un-Reinforced Masonry Wall with No Opening) was used to study performances of URM walls without openings or URM shear walls. Anonther specimen was used to study performances of URM walls with openings (i.e., URMWOC – Un-Reinforced Masonry Wall with Opening, Control specimen) and treated as control wall panel. Three wall panels with openings were used for each retrofitting configuration: horizontal bandages (i.e., RMWOHS – Retrofitted Masonry Wall with Only Horizontal Seismic bands), vertical splints (i.e., RMWOVS – Retrofitted Masonry Wall with Only Vertical Seismic bands) and combination of horizontal and vertical seismic bands (i.e., RMWHVS – Retrofitted Masonry Wall with combination of Horizontal and Vertical Seismic bands).

2.1. Test Wall Panels

Wall panels were cast using commercially available burned clay bricks of 215 x 102.5 x 60 mm. The size of a wall panel was 640 x 600 x 215 mm. The dimensions of test walls were decided according to the aspect ratio of 1.1. The opening was positioned at the centre and middle of the wall panel. Size of the opening was 200 x 225 mm. The geometry of two different wall panels is shown in Figure 1. The test walls were double wythe and English bond pattern was used in wall casting. Mortar joint thickness was 10 mm and 1:6 cement-sand mortar (i.e., in volume basis) was used. The specimens were allowed for curing of 28 days. At the end of curing process, retrofitting techniques were installed.



(a) (b)

Figure 1 Geometry of wall elements; (a) URMWNO and (b) URMWOC

2.2. Strengthening Procedure/Retrofitting of the Test Walls

Three wall panels with opeings were symetrically retrofitted using plain wave and electro galvanized steel wire mesh. The mesh pitch was 2 mm and wire diameter was 0.5 mm. Three different configurations were used as shown in Figure 2.



(a) (b) (c)

Figure 2 Reinforcement grid layout; (a) Horizontal layout, (b) Vertical layout and (c) Combination of horizontal and vertical layout

The upgrading of specimen NRMWOC to specimens RMWOHS, RMWOVS and RMWHVS consisted in the application of layers of SWM as a belt pattern on the masonry wall around the opening. This was a particularly easy operation. A layer of 10 mm thick rich cement-sand mortar (i.e., cement: sand = 1:3 in volume) is first applied on the masonry surface. The required reinforcement in the form of SWM is then applied over the mortar layer on both sides of the walls. To simulate the bond of continuously applied mesh with the masonry, SWM was bent on sides of walls. The mesh was also connected to the wall by steel bolts at regular intervals. Afterwards, second layer of 10 mm thick cement-sand mortar is applied. The mesh was also, bent against two wall surfaces at the wall bottom in the case of RMWOVS and RMWHVS specimens.

2.3. The Test Setup

The experimental setup with a typical wall for the cyclic loading is presented in Figure 3. The masonry wall panels were founded on a stiff concrete support beam of 150 x 750 x 325 mm and concrete beam of 100 x 600 x 325 mm was fixed on the top of the wall panel.



Figure 3 URMWOC at the test setup

The masonry wall panels were founded on a stiff concrete support beam of 150 x 750 x 325 mm and concrete beam of 100 x 600 x 325 mm was fixed on the top of the wall panel. The bottom concrete beam represented the foundation as in practical situation. The top beam represented axial load from roof and other structures acting on the wall. A layer of G20 concrete (i.e., 50 mm thick) was applied on the foundation around the wall and mesh with covering the mesh area laying on foundation. The specimen was fixed on the test steel floor/frame. The concrete foundation was pre stressed to the strong steel frame using 0.75 inch (19.05 mm) diameter six steel thread bars so that it would not slide relative to the strong floor. The boundary conditions at the top and bottom of a test specimen emulated a cantilevered element, with both vertical and horizontal loading applied at the top of the wall.

Horizontal forces were applied to the top concrete beam, which in turn, distributed the lateral forces to the masonry test walls evenly. The load was also applied at the centre line of test walls using a servo control hydraulic actuators that were reacted against a stiff steel reaction frame.

2.4. Loading History

The lateral reversed cyclic in-plane loading was applied on test specimens as a programmed sinusoidal wave with varying amplitudes and frequencies under the displacement control method. All the specimens were subjected to 34 test runs as summarized in Table 2.

Table 2 Loading history of dynamic in-plane shear load test

Main Step	Test Run	Amplitude (mm)	Frequency (Hz)	No. of repeating	Main Step	Test Run	Amplitude (mm)	Frequency (Hz)	No. of repeating
1	1	02	5	100	5	18	08	1	200
2	2	02	5	200		19	10	1	200
	3	02	5	300		20	12	1	200
	4	02	5	400		21	14	1	200
3	5	02	5	100	6	22	02	5	100
	6	04	1	200		23	04	1	200
	7	06	1	200		24	06	1	200
	8	08	1	200		25	08	1	200
	9	10	1	200		26	10	1	200
4	10	02	5	100		27	12	1	200
	11	04	1	200		28	14	1	200
	12	06	1	200	7	29	14	1	200
	13	08	1	200		30	25	0.6	200
	14	10	1	200		31	30	0.5	200
5	15	02	5	100	8	32	14	1	200
	16	04	1	200		33	40	0.4	200
	17	06	1	200		34	50	0.3	200

3. RESULTS AND DISCUSSION

The effect of strengthening on in-plane shear behaviour of URM walls, including failure modes, crack formation and propagation, lateral load carrying capacity, maximum lateral deflection, stiffness, energy dissipation and ductility, were investigated. The test results are shown in Table 3.

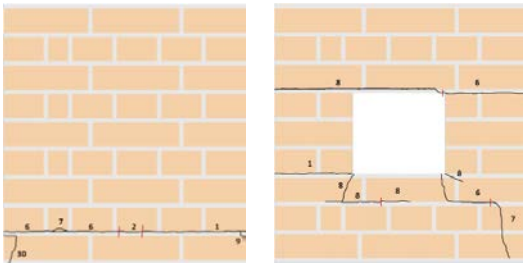
Table 3 Test Results and observations

Specimen	URMWNO	URMWOC	RMWOHS	RMWOVS	RMWHVS
Maximum Diplacement (mm)	4.11	4.87	15.32	20.78	44.10
Peak Load (kN)	2.64	2.25	2.48	5.36	8.69
Peak Load Increment (%) (Compared to URMWOC)	17.33	~	10.22	138.22	286.22
Initial Stiffness (kN/mm)	2.24	1.35	2.96	2.86	3.13
Stiffness at Peak Load (kN/mm)	1.09	0.55	0.77	1.05	0.58
Energy Dissipation (J)	8.49	8.29	29.18	85.65	274.30
Ductility	4.56	5.90	8.95	13.44	14.46
Failure Mode	R, TC, SL	R, SL	R, TC, SL	R, TC, SL	R, TC ,SL

R: Rocking, TC: Toe Crushing, SL: Sliding

3.1. Dynamic Behaviour of Walls

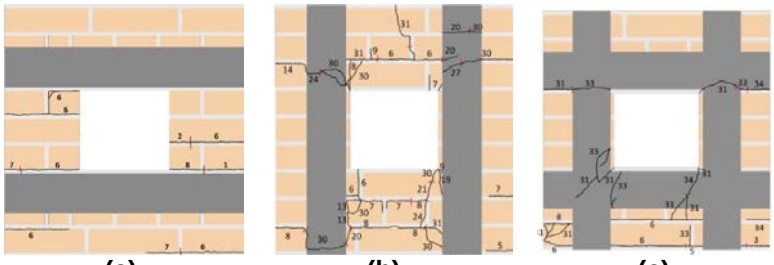
All the specimens showed both flexural and shear mode of failures. Behaviour of the specimen URMWNO was dominated by rocking mode of failure originated by a flexural cracking as shown in Figure 4(a) and of URMWOC was dominated by inclined shear cracking as shown in Figure 4(b).



(a) (b)

Figure 4 Cracking pattern for; (a) URMWNO and (b) URMWOC

The horizontal cracking in URMWNO was initiated at 1st test run in bed joint between 1st and 2nd brick coursers. Rocking developed on the wall at 5th test run. The upper wall part completely separated from the lowest brick course at 6th test run. The sliding could be observed at end of 6th test run. Despite the low strength of wall, its sliding friction and rocking noticeably dissipate the energy. Toe crushing could be observed in both end of the wall started at 6th test run. Heavy toe crushing was characterized at test end. The specimen URMWNO was able to withstand displacements several times greater than the cracking displacements without significant damage, even after numerous rocking cycles. It proved that rocking is a stable and favourable post cracking behaviour for URM walls without openings. The horizontal cracking in URMWOC was initiated at 1st test run in bed join between 3rd and 4th brick coursers along the sill level as shown in Figure 6(b). Rocking was started at 5th test run. The inclined shear cracking was initiated at 6th test run in sill masonry sub unit of the wall. The spandrel masonry sub unit was completely separated from bottom part of the wall at 8th test run along the lintel level. Inclined shear cracking or horizontal cracking couldn't be observed in this spandrel masonry area. The sliding could be observed at 8th test run. The inclined shear cracking is occurred due to stresses developed at corners of wall opening. Sliding friction capacity of the URMWNO was not sigificantly harmed after the sliding of the wall; whereas after formation of inclined cracks in URMWOC, load transfer mechanism along crack is significantly harmed. The cracking pattern observed in retrofitted walls is shown in Figure 5.



(a) (b) (c)

Figure 5 Cracking pattern for retrofitted walls; (a) RMWOHS, (b) RMWOVS and (c) RMWHVS

Flexural horizontal crack initiation in specimen RMWOHS, RMWOVS and RMWHVS was occurred at 2nd, 5th and 3rd test run, respectively. In RMWOHS, cracking was continued in bed joint between third and fourth brick courses up to 8th test run until the bed joint was completely cracked along sill band (Figure 5(a)). In addition to that, horizontal cracking was observed in bed fourth and fifth courses and fifth and sixth courses at 2nd and 6th test run, resulted in heavy sliding and rocking along those bed joint beside the opening at end of the 6th test run. Further, rotation could be observed in base line and sill level of the wall. The small sliding was occurred along the sill level of the wall. Any cracking couldn't be observed in spandrel masonry area and inclined shear cracks were not observed. In case of RMWOVS, cracking was initiated in bed joint between wall and foundation in 5th test run (Figure 5(b)). Further cracks was originated and propagated through whole length of the bed joint until 30th test run. The wall started to rock at the base at end of the 7th test run. Small sliding of wall started in 30th test run. Rocking was also campaigned with the crushing of wall corner in one direction. Any cracking couldn't be observed in wall pier masonry area.

In RMWHVS, cracking was initiated in bed joint between wall and foundation and first and second brick courses in 6th test run (Figure 5(a)). The wall was completely separated from wall base at 6th test run. The wall was also separated completely along the first and second brick courses at 34th test run. The rotation was started at 6th test run. A small sliding was observed during 7th and 25th test run and after 25th test run, sliding couldn't be observed. A heavy toe crushing could be observed. These observations confirm that, retrofitting delay the crack initiation, rotation, crushing and sliding of the URM walls. However, the specimen RMWHVS showed greater delay regarding these behaviours compared to specimen RMWOHS and RMWOVS. Also, compared with conventional reinforced concrete, ferrocement is reinforced in two directions; therefore, it may have homogenous isotropic properties. Ferrocement generally has a high tensile strength and high modules of rupture, because the specific surface of ferrocement reinforcement is higher than that of reinforced concrete. Therefore larger bond forces develop with matrix resulting in average crack spacing and crack width of smaller magnitude in retrofitted wall with ferrocement overlay than those of URM walls and even conventional reinforced concrete.

3.2. Force-Deformation Response

The hysteretic curves of the test specimens are presented in Figure 6. From these figures the effect of retrofitting on deformation capacity and ultimate lateral strength is readily seen. As shown the Figure 6, the improvement of lateral in-plane strength of retrofitted walls, RMWOHS, RMWOVS and RMWHVS is 1.10, 2.38 and 3.86 times the capacity of URMWOC.

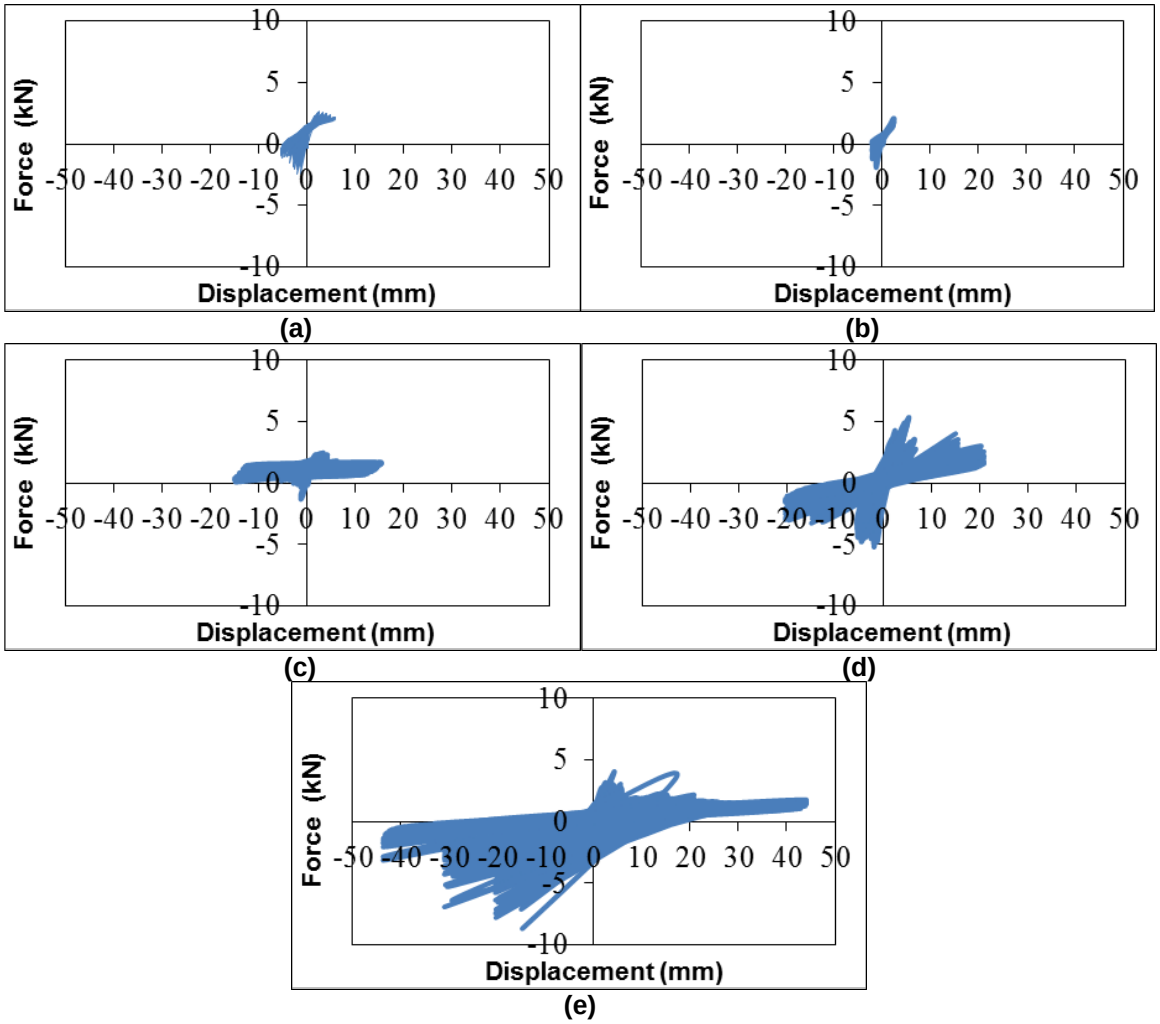


Figure 6 Hysteretic curves; (a) URMWNO, (b) URMWOC, (c) RMWOHS, (d) RMWOVS and (e) RMWHVS

The retrofitting enhances the initial stiffness of windowed URM walls. The initial stiffness of specimens was calculated as the secant stiffness at 40% peak load (Seaders, Gupta and Miller, 2009). The initial stiffness of RMWOHS, RMWOVS and RMWHVS is 2.19, 2.12 and 2.32 times greater than that of URMWOC, respectively. Hence, the retrofitting material of SWM has significance on improving lateral strength of the URM walls with opening. The stiffness of the retrofitted specimens RMWOHS and RMWHVS at the peak load was very similar to the reference specimen URMWOC. On the other hand, all the retrofitting specimens showed considerably lager peak load than that of URMWOC. This confirms that deformation capacity of those retrofitted walls improved in rapid increasing rate compared to URMWOC. Specimen, URMWNO showed 1.17 times lager load carrying capacity, 1.7 times lager initial stiffness and 2 times lager stiffness at peak load compared to those of URMWOC. Also, URMWNO showed comparatively less deformation than URMWOC (Table 2). These observations confirmed that URM shear wall behave in stiff manner than behaviour of windowed URM walls when subjected to lateral dynamic in-plane loads.

RMWOVS showed better load-deformation behaviour than RMWOHS, though both of walls retrofitted with same degree of reinforcement. This variation is caused by the difference in retrofitting pattern as show in Figure 2(a) and 2(b). Therefore, against lateral dynamic in-plane loads, the vertical layout is more suitable than the horizontal layout where RMWHVS is the best for load-deformation behaviour.

3.3. Energy Dissipation

The envelope curves for the hysteresis loops of test specimens are shown in Figure 7. The area enclosed by an envelope curve of a specimen for its hysteresis loops at given cycles represents the energy dissipated by the specimen during these cycles. All the three retrofitting methods improve the energy dissipation of URM walls with opening as shown in Figure 7. As expectedly, RMWHVS showed larger energy dissipation compared to URMWNO, URMWOC, RMWOHS and RMWOVS. The energy dissipated by the specimen was attributed (1) friction along the joints and existing cracks, (2) formation of new cracks, (3) crushing of bricks and (4) steel yielding and crushing. The retrofitting improved the energy dissipation by a factor of 3.52, 10.33 and 33.01, in the case of RMWOHS, RMWOVS and RMWHVS compared to URMWOC. It is observed that the energy dissipation of URMWNO were possessed an increment of 2.41% compared to URMWOC.

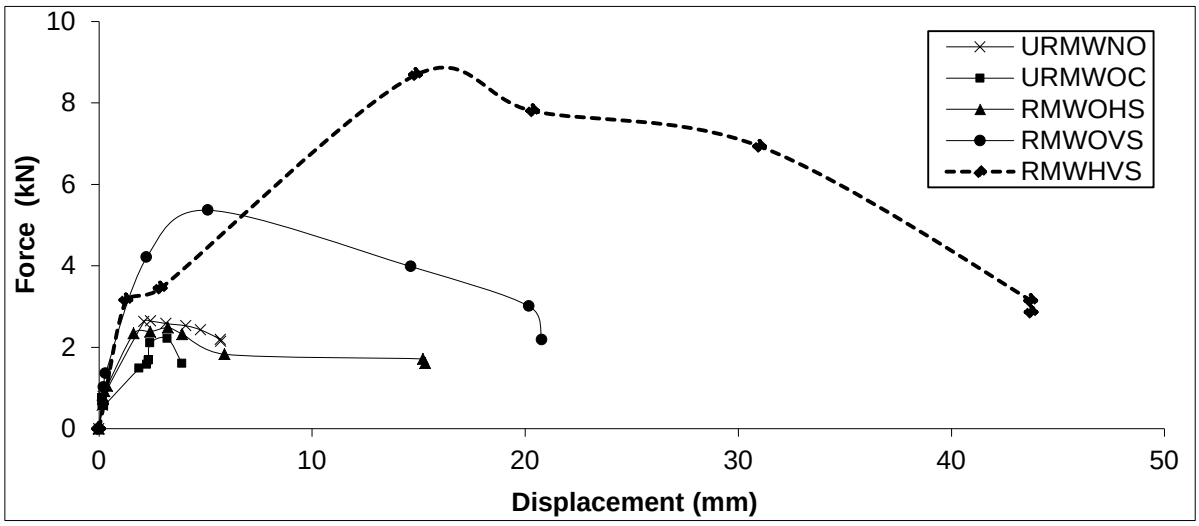


Figure 7 Envelope curves for the specimens

3.4. Ductility

The term “ductility” in seismic design is used to undergo large amplitude cyclic deformations in the inelastic range without a substantial reduction in strength. In order to assess the ductility of the specimens, the ductility factors (Vaughan, 2010) of each specimen were calculated for the pull loading direction for URMWNO, URMWOC, RMWOHS and RMWOVS and for push loading direction for

RMWHVS. In such way, ductility factors of 4.56, 5.9, 8.95, 13.44 and 14.46 were calculated for the specimens URMWNO, URMWOC, RMWOHS, RMWOVS and RMWHVS, respectively. The specimens RMWOHS, RMWOVS and RMWHVS were able to improve ductility by 1.52, 2.28 and 2.45 times the ductility of URMWOC. URMWNO showed slightly lower ductility with compared to URMWOC.

Adding strengthening techniques to the URM walls changes the structural system from plain URM wall to an in-fill box or frame in every direction of the wall relevant to retrofitted area. Therefore, all the three retrofitted walls showed positive behaviour compared to URMWOC. To ensure that anchorage failure did not occur, steel plates can be used to anchor the SWM to the wall using steel bolts or steel biding wires can be passed across the SWM and the wall through holes drilled in the wall. URM shear wall has continously distributed bond between bricks and mortar whole over the wall to pass the external load while widowed URM wall has disterbances from the opening for loading path. Hence, URMWNO specimen shows better behaviour in bearing load, stiffness, energy dissipation, rocking behaviour compared to URMWOC.

4. CONCLUSIONS

This paper presents an experimental investigation on the dynamic behaviour of lateral reversed cyclic in-plane loaded non-retrofitted URM wall panels with and without opening and the retrofitted wall panels with opening. Seismic belts technique was selected as retrofitting method. URM shear wall shows flexural mode of failure dominated by rocking and heavy toe crushing under the in-plane seismic shear loads. The test confirms that wall rocking is a stable non-linear behaviour in URM shear walls when no out-of-plane response occurs. URM wall with opening shows flexural mode of failure dominated by inclined shear cracking and sliding shear. Seismic belts can effectively improve the dynamic in-plane behaviour of masonry walls where they increase the lateral resistance, initial stiffness, deformation capacity and energy dissipation of the masonry wall significantly. Combination of horizontal and vertical seismic belts around the opening and running those applied bandages over the whole wall length and height continuously is more suitable where it helps the URM wall to improve those parameters greater than those improved by applying horizontal layout or vertical layout alone. Further, the vertical layout of seismic belts is more appropriate for retrofitting of URM wall with opening than the horizontal layout against the lateral dynamic in-plane loading with no out-of-plane action.

5. ACKNOWLEDGEMENT

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Simulation of Low Cycle Fatigue with Abaqus/FEA

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Abstract: *Low cycle fatigue is an important design consideration for large steel structures and metallic machine components. Accurate prediction of fatigue endurance is essential to design the elements subjected to fatigue. The design guidelines given in codes of practices are applicable only to simple shapes and laboratory experimental verification is costly. Therefore simulation using finite element software is becoming popular. This paper demonstrate successful coupling of Abaqus/FEA and fe-safe software in predicting the uni-axial fatigue behaviour of a stainless steel specimen. The simulated results are verified against experimental results available in literature. Sensitivity to surface roughness and material model were examined.*

Keywords: *Low cycle fatigue, fatigue simulation, elastic plastic fatigue analysis*

1. INTRODUCTION

A material can fail well below its monotonic strengths when it is subjected to repeated loading. This phenomenon is known as fatigue. Fatigue can happen progressively, even when the applied loads are individually too small to cause failure. Most of the structural elements are subjected to various types of vibrations over their lifespan. A structure may fail with lower number of cycles when it is subjected to higher amplitudes of vibration (low-cycle fatigue) or the same structure may fail with higher number of cycles but under lesser amplitude of vibration (high-cycle fatigue). While the mechanical engineers are always concerned about fatigue when designing machine components, the civil engineers concern fatigue damage mainly in steel bridges and tall steel structures that are subjected to wind and earthquake induced vibrations.

Fatigue damage estimation methods given in most codes of practices are for simple structural shapes. On the other hand testing large structural components for vibrations is costly. Australian code AS4100 defines a concept called the detailed category (f_{rn}) for different components. Detail category takes in to account many fatigue inducing properties to estimate the appropriate endurance curve. However there are still a number of unidentified potential problems in fatigue design specifications given in codes of practices (Dean B. & MendisP.A., 2000). Therefore fatigue modelling through computer software is becoming popular. These finite element software uses damage estimating algorithms to estimate the damage initiation and propagation. All fatigue inducing properties such as surface finish, temperature, stress concentrations can be included for accurate estimations.

Low cycle fatigue simulation has performed in the literature mostly for a selected strain amplitude level. (Glodež & Knez, 2007) (G.Staudinger & T.Reiter, 1997) This paper relates the endurance to several strain levels by performing multiple simulations.

2. FATIGUE ANALYSIS

Fatigue endurance under uniaxial fatigue for different constant amplitude strain levels are presented in the form of S-N curves. S-N curves are widely used to estimate fatigue damage by generalising for different shapes, different loading histories and different materials.

Generalisation of constant strain amplitude test data for complex loading histories is done using Miners rule together with Rain-flow cycle counting method. Generalisation for shape, mean stress and other factors are done through the correction factors. Figure 1 presents stress-strain relationship under fatigue loading approximated using uniaxial fatigue formula.

2.1. Uniaxial Fatigue

Local strain life fatigue analysis presumes that the stain life behaviour of the local stress concentration is similar to a larger uniform section tested with equalling stresses and strains. Local strain life methods are suitable for finite element models because the stress strain relationship at all locations are known. Cyclic stress (σ) strain (ε) relationship which represents the tips of the stabilized hysteresis loop coordinates is represented by,

$$\varepsilon = \frac{\sigma}{E} + \left(\frac{\sigma}{K'}\right)^{\frac{1}{n'}} \quad (1)$$

where
ε - cyclic elastic modulus
K' - strain hardening coefficient
n' - strain hardening exponent

The total (elastic & plastic) strain-life relationship for uniaxial fatigue (Coffin-Mansion formula) is defined as,

$$\frac{\Delta\varepsilon}{2} = \frac{\sigma'_f}{E}(2N_f)^b + \varepsilon'_f(2N_f)^c \quad (2)$$

where
Δε - applied strain range
$2N_f$ - number of reversals to failure
b - fatigue strength exponent
σ'_f - fatigue strength coefficient
c - fatigue ductility exponent
ε'_f - fatigue ductility coefficient

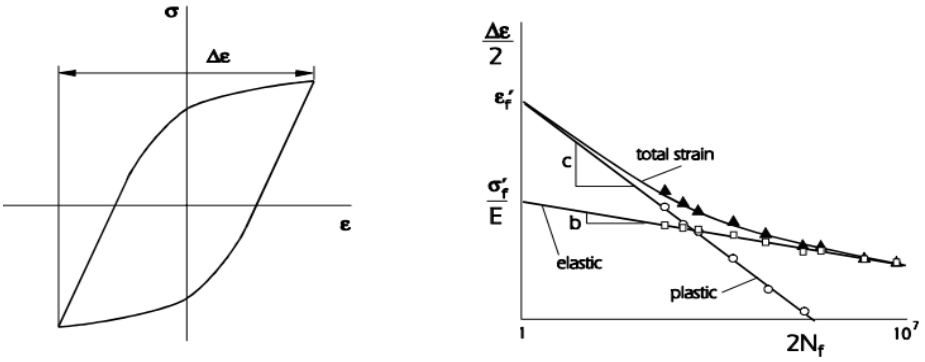

Figure 1. Strain-life Relationship (Coffin-Mansion)

Local strain-life analysis is a fatigue crack initiation criterion. There are several correlations such as Morrow's mean stress correlation to take account of the mean stress effect. (Fe-safe, 2002)

2.2. Multi-Axial Fatigue

In practice many structures experience multi-axial fatigue. Use of uniaxial methods for multi-axial fatigue may give overestimates of the life. The fatigue cracks are usually initiated from the surface. Combination of in-plane stresses and out-of-plane stresses on the surface creates tri-axial stress distribution. Multi-axial fatigue theories concentrate on this condition. Different criterions have been proposed by researchers.

Brown miller equation is a widely used multi-axial fatigue criterion. Brown miller equation with morrow's mean stress correlation can be expressed as,

$$\frac{\Delta\gamma_{max}}{2} + \frac{\Delta\varepsilon_n}{2} = 1.65\,\frac{(\sigma'_f - \sigma_{n,m})}{E}(2N_f)^b + 1.75\,\varepsilon'_f(2N_f)^c \qquad (3)$$

where,

$\Delta\gamma_{max}$ Maximum shear strain
$\Delta_{\varepsilon n}$ Normal strain range
$\sigma_{n,m}$ Mean normal stress on the plane

3. SIMULATING FATIGUE

Zhou, et al., 2008 have conducted an experimental study on the low cycle fatigue of stainless steel reinforcement bar specimens. An attempt is made to simulate the standard uniaxial fatigue test done by Zhou, et al using Abaqus/FEA and fe-safe commercial finite element software. Experiment is modelled in Abaqus to obtain the maximum stresses and strains under a specified loading condition first. Then the stress-strain data is imported to fe-safe to predict the fatigue behaviour under cyclic loading.

3.1. Material Properties

The simulation was done for stainless steel 316LN test specimens. The cyclic properties and fatigue properties extracted from the literature are given in Table 1

Table 1 Material properties of stainless steel 316LN test specimens

Mechanical Properties	**Cyclic Elastic properties** (Zhou, et al., 2008)	
	Young's modulus (MPa)	Poisson's ratio
	199817	0.33
	Cyclic Plastic properties (J.Shita, et al., 2013)	
	Yield stress(MPa)	Plastic strain
	270	0
	300	0.0025
	330	0.0075
	350	0.0125
	370	0.0175
	400	0.04
Fatigue Properties (Fe-safe, 2014)	b	-0.0835
	c	-0.5142
	ε_f'	0.476
	σf'	703.4

3.2. Material model

The hardening model is very important in simulating the fatigue behaviour. A linear kinematic hardening model or a nonlinear isotropic/kinematic hardening model can be used in Abaqus to simulate the behaviour of materials that are subjected to cyclic loading. The evolution law in these models consists of a kinematic hardening component (which describes the translation of the yield surface in stress space) and, for the nonlinear isotropic/kinematic hardening model, of an isotropic component (which describes the change of the elastic range).
(Abaqus, 2013)
The intial transient behaviour is not important since the material comes to stabilzed cyclic response easily. However the stabilized cyclic response is presnt in majority of the fatiguelife. Therefore non-linear kinematic hardening model is used which is similar to that's described in (G.Staudinger & T.Reiter, 1997)

3.3. Modelling with ABAQUS

Abaqus standard is used for this analysis.3D20R solid elements with varying mesh size are used in order to model the stainless steel specimen. A finer mesh was used to model the central region to obtain a detail stress distribution.

General static analysis is performed neglecting the geometric nonlinearity. Bottom grip was restrained to move in all degrees of freedom and top grip was allowed to translate along the longitudinal axis of the specimen as shown in figure below.

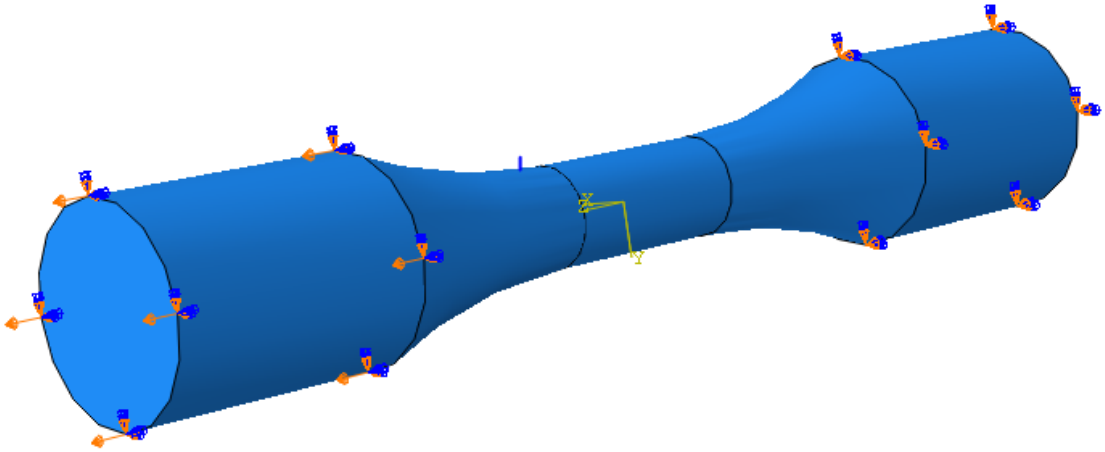

Figure 2. Idealized boundary condition in Abaqus

The model was run for different strain levels to construct the stress strain behaviour. Figure shows strain distribution for a simulation subjected to 1.77% overall strain. Figure illustrates the cyclic stress strain behaviour of a node at the centre of the specimen for the same 1.77% strain amplitude. The strain gauge in the experimental investigation performed by Zhou, et al., 2008 is located between the end points of the middle uniform cylindrical area. Therefore the strain values presented here refers to the same region.

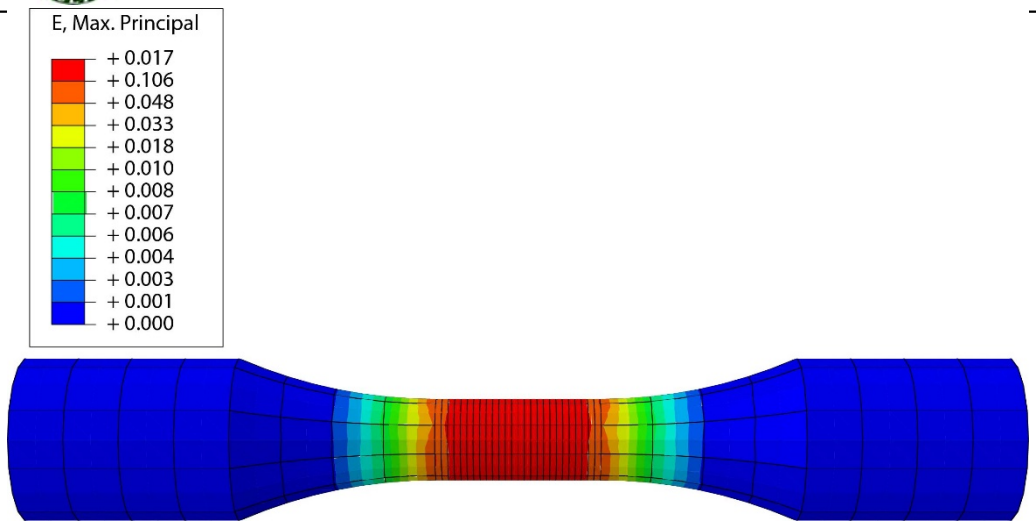

Figure 3. ABAQUS simulation of 1.77% strain amplitude

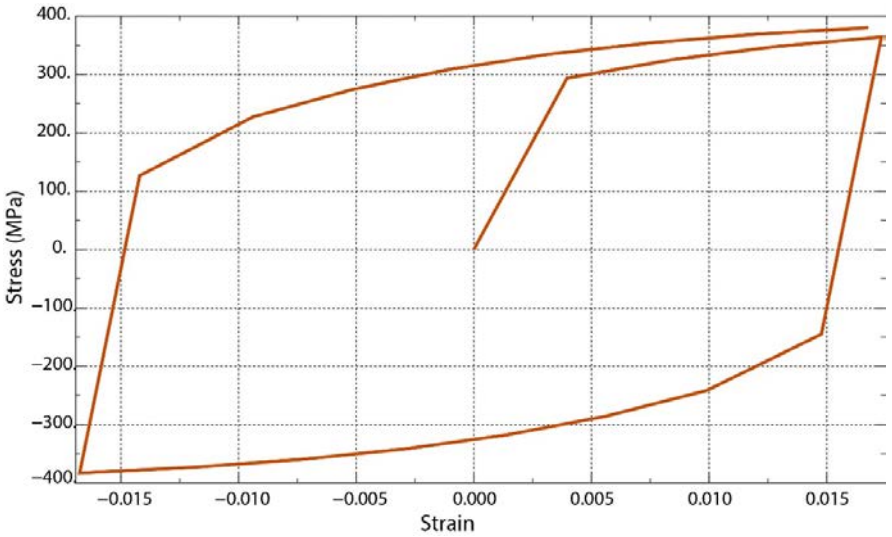

Figure 4. Principle stress strain response of an element in the central region

3.4. FE-SAFE fatigue analysis

Both the stress and strain datasets for each strain level are imported from Abaqus to the fe-safe software. By default Fe-safe assumes the stress datasets as elastic blocks. There are two methods that can be used for elastic-plastic fatigue estimation.

1. Using an elastic block with Neuber's rule
2. Using an elastic plastic block (elasto-plastic FEA is required)

The calculation of plastic strain where necessary is performed using elastic to elastic-plastic correlation (biaxial Neuber's rule) in the first method. But these correlations are applied to nodes individually. Therefore no stress redistribution is allowed. Hence this method is not suitable especially when large amount of material are subjected to plastic stresses. (Fe-safe, 2014)

Elastic plastic analysis feature in fe-safe requires elastic plastic stress strain datasets for finite elements which is more accurate. Therefore an elastic plastic finite element analysis must be carried out. By combining stress dataset and strain dataset for each increment elastic plastic block was generated. The inbuilt fe-safe material database for S316 stainless steel is used for fatigue simulation and the surface finish was assumed as mirror polished (k_t=1).

Brown-miller equation with morrow's mean stress correlation was selected as the algorithm. Although the loading is uniaxial, uniaxial strain life equations cannot be used in fe-safe for 2D or 3D objects as stress and strains are not uniaxial.

Fe-safe uses "critical plane" method to identify the most damaging plane by using brown miller equation for planes at 10º intervals between 0º and 180º in the surface of the component. (Fe-safe, 2002) The fatigue life variation in the specimen is shown below

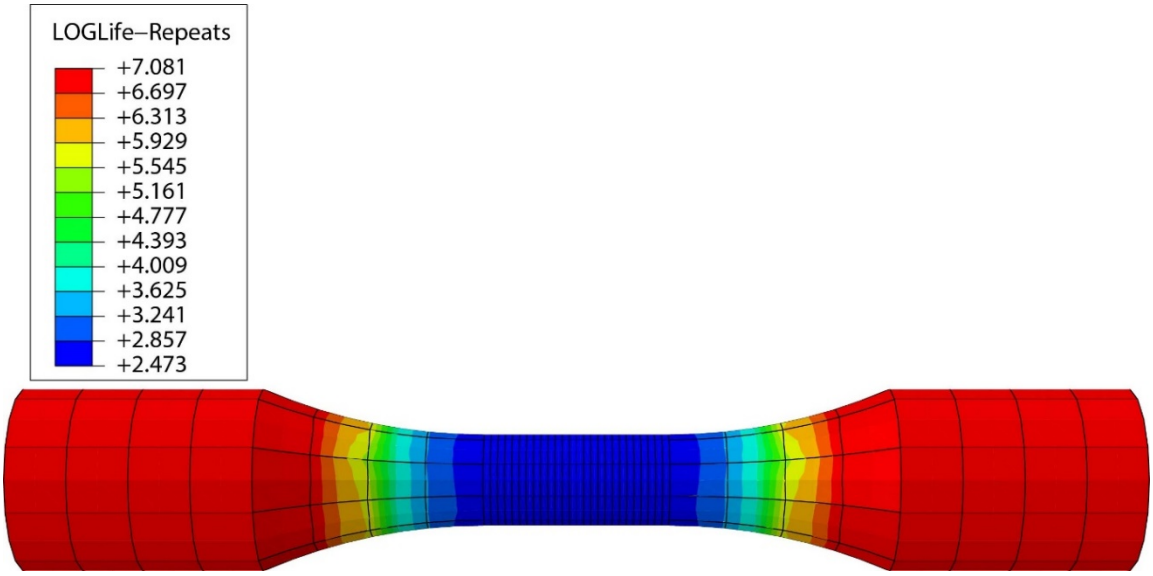

Figure 5. Fe-safe simulation of 1.77% strain Log (Life-Repeats) of the specimen

4. COMPARISON OF RESULTS

Two sets of simulations were performed using an elastic plastic block but varying the surface finish to observe the sensitivity. Another simulation was performed with an elastic block. Table 2 compares the results obtained for the simulations against the experimental results obtained by (Zhou, et al., 2008).

Table 2 Comparison of fatigue life results

Strain amplitude (%)	Number of reversals to failure (2Nf)			
	Experimental result (Zhou, et al., 2008)	Abaqus and fe-safe simulation		
		Using Elastic Plastic block		Elastic block with Neuber's rule
		Mirror polished	Rough(Ra-2um)	Mirror polished
0.99	1322	1198	552	54487
1.28	791	616	336	42369
1.5	576	411	225	35483
1.77	413	292	162	31438
2.1	293	216	120	28763
2.4	225	162	90	27064

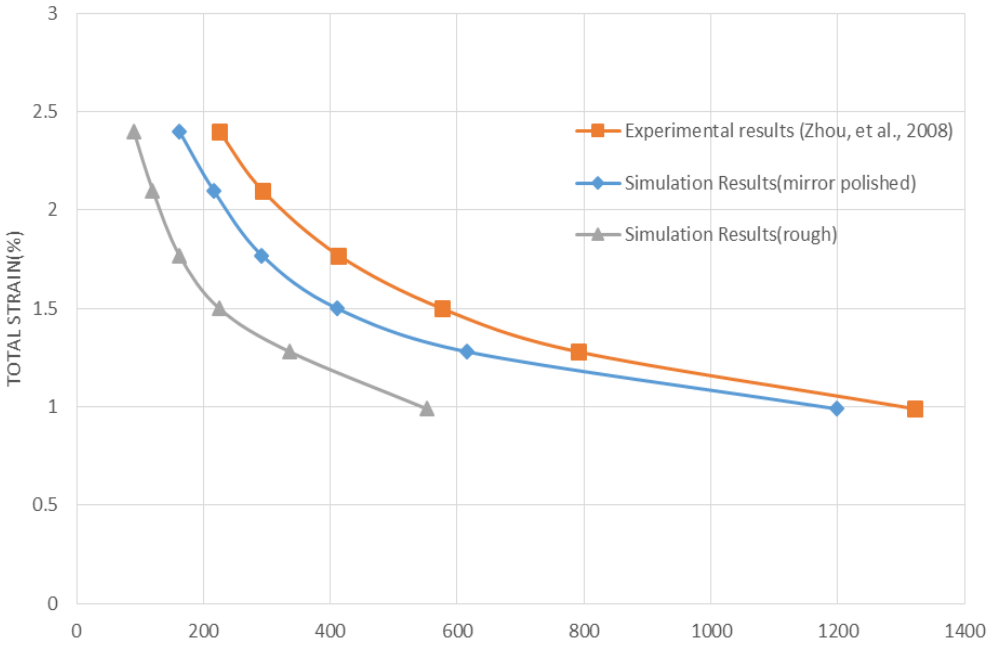

Figure 6. Comparison of S-N curves of simulation and experimental results

As shown in the Figure the mirror polished surface finish which is the actual condition in the original test gave the closest behaviour to tested results. The simulation is within 25% range of the actual values. The original experimental results indidate that there is a 10%-20% variation of the experimental curve presented here. Therefore the simulation results is in an accpetable range.

The rough surface finish that is present in type 316 rebars (Kalpakjian, 6th edition) will give fatigue limits in the range of half that was for mirror polished. This indicates the surface finish is a primary factor for fatigue life of a material. As shown in Table 2 the simulation with an elastic block with Neuber's rule gaves large fatigue lives which are not acceptable. The reason as explained in 3.4 is neuber's rule being valid only when stress redistribution is insiginifcant. In this experiment whole central region of the speciemn is subjected to plastic stresses. Therefore elastic-plastic stress block method is preferred.

5. CONCLUSIONS

- Abaqus and fe-safe were successfully coupled to predict the uni-axial fatigue behavior of a stainless steel specimen.
- Prediction of cyclic loading behavior depends on the chosen hardening model. Nonlinear kinematic hardening model was chosen to generate the stabilized cyclic response.
- Surface finish is a sensitive factor for the fatigue life. Exact surface roughness should be estimated in order for better predictions.
- Neuber's rule with elastic block is suitable for low cycle fatigue only when stress redistribution is not significant. Elastic-plastic analysis should be performed in fe-safe when the stress redistribution is significant.

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Folding Patterns for Ultra-thin Deployable Membranes

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Abstract: *A deployable structure should mainly be adequately compact and should fit into any remaining space of the launch vehicle. The main factors which will determine these are the folding pattern, ease of deployment and stresses in the fold lines. Two folding patterns are selected based on extensive literature review to investigate the possibility of using those techniques for a large solar sail mission. It is expensive as well as extremely time consuming to perform experimental investigation under reduced gravity environment for this type of large membranes. Thus developing simulating techniques are quite important. Two models are simulated using Abaqus/Explicit commercial finite element software. Quasi-static conditions and numerical accuracy are verified by comparing strain energy together with kinetic energy and artificial strain energy. It is shown that spiral folding pattern requires less energy for deployment and hence that is preferred.*

Keywords: *deployable structures, folding patterns, origami*

1. INTRODUCTION

Solar sails, solar reflectors, and sunshields are essential features for space exploration missions. These structures require a very larger surface area in the operational stage. The large size of these structures have given rise to great difficulties in designing, packaging, ground testing, and transportation.

“Deployable structures” is one of the main and widely used concepts as a solution to this problem. Deployable structures are arrangements which change its shape and size to achieve convenience in packaging and transportation. This is mainly achieved through folding patterns derived from the Japanese art of folding paper, “Origami”.

Due to the large size, ground testing of these structures in a zero gravity environment can be extremely difficult, expensive and time consuming. As a solution to overcome these issues, testing of the behaviour in a virtual environment using finite element models is adopted.

The objective of this research was to find alternative patterns which are suitable to fold a simple compact deployable structure, compare these alternatives, and select the most optimum alternative. In selecting the optimum folding pattern, a finite element analysis was carried out to simulate the behaviour these pattern under above mentioned conditions.

2. FOLDING PATTERNS

After an extensive literature survey, two simple folding patterns which wraps a membrane around a hub with a polygonal shaped cross section was selected for the research: spiral and circumferential folding patterns. Some aspects of these two patterns have been already considered by several authors.

Origami Flasher (Lang, 1997) is a famous Origami folding pattern, which was developed based on the technique “iso-area twist folding” (See Figure 1).

Spiral folding pattern utilizes the same folding mechanisms as Origami flasher, but it is a more orderly pattern which has well defined fold lines and wraps the membrane around a central hub. This removes unfavourable conditions of the Origami flasher.



Figure 1. Origami Flasher

IKAROS a 14.0 m × 14.0 m square shaped solar sail demonstrator, wrapped around a cylindrical body, which was successfully deployed in June 2010 (Mori, et al., 2009). This follows the circumferential pattern of folding.

As shown in Figure 2 the deployment and folding processes of the circumferential folding pattern is a two-step process. First the arms wrapped around the hub are deployed and then the membrane will spread out from these arms to reach the fully deployed state.

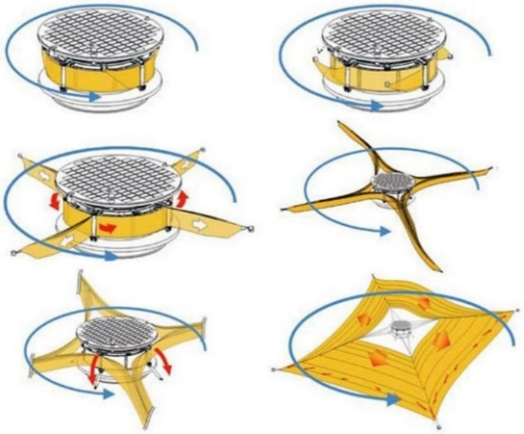

Figure 2. Deployment Process of IKAROS (Warwick, 2010)

Deployment methods for the above mentioned folding patterns are identically reversible processes. Hence simulation of either the folding or the deployment processes will be enough to identify the attributes of both the processes. Due to the convenience of simulation, the deployment behaviour of patterns is investigated.

3. SIMULATION OF THE DEPLOYMENT PROCESS

To select the optimum pattern, behavioural aspects need to be considered. Since carrying out a physical experiment under real conditions is costly, hard and time consuming, a computer based finite element simulation was carried out.

For simulation purposes, these models were taken to be square modules made from thin sheets of Mylar with a thickness of 10 µm, and an area of 50 mm × 50 mm. these modules will wrap around a square hub of 10 mm × 10 mm. Membranes will be in the most basic forms, which consist of all the folding characteristics of these patterns (See Figure 3). During simulations the effect of membrane thickness on the folding patterns is neglected. Material Properties of Mylar used for the analysis is given in Table 1.

Table 1 - Material Properties of Mylar

Material Properties	Value
Elastic Modulus (N/mm2)	5200
Poisson's Ratio	0.38
Density (kg/m3)	1390

Models were made to consist of triangular and rectangular elements, which are tied together according to the arrangement given in Figure 3. These elements are later meshed to comprise smaller quad (S4) elements. Each model consist of 3513 nodes and 2900 (S4) elements with the minimum length of an element being around 1 mm (See Figure 4).

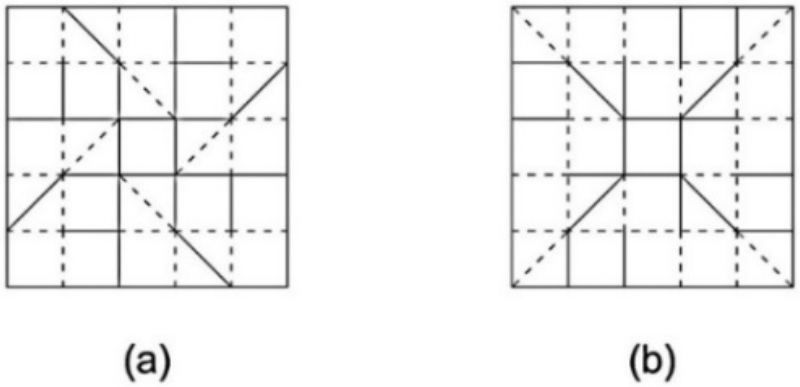

Figure 3. Models used for simulation
(a) Spiral folding pattern (b) Circumferential folding pattern

Models were pulled from all four corners in a smooth stepped function, to simulate the deployment process (See Figure 4).

Folding patterns considered here can be subjected to dynamic snapping. Therefore the Abaqus/Explicit solver is used for these simulations.

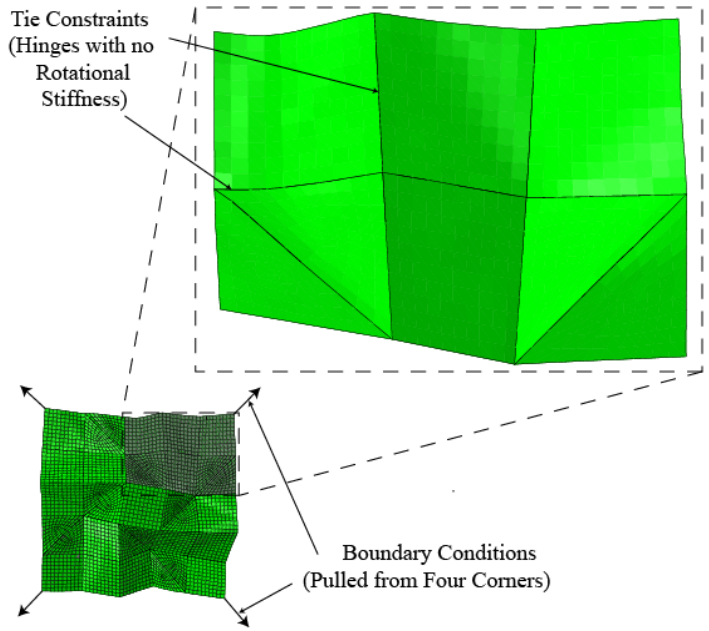

Figure 4. Abaqus model for spiral folding pattern

Figure 5 & Figure 6 will show the deployment process of spiral and circumferential models respectively.

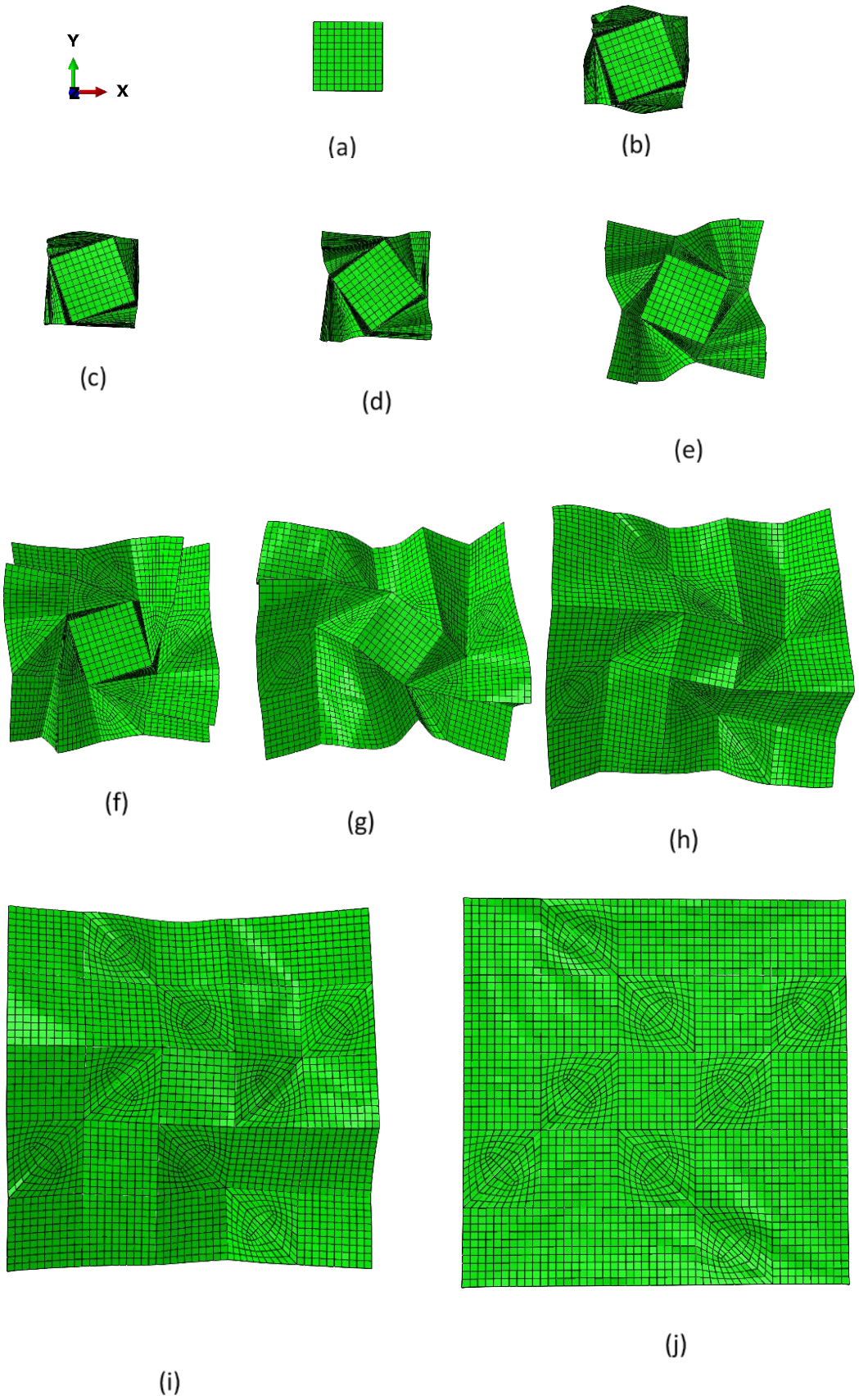

Figure 5. Deployment of the Spiral Folding Pattern
(a) Time step 0s (b) Time step 0.03s (c) Time Step 0.06s (d) Time Step 0.09s
(e) Time Step 0.12s (f) Time Step 0.15s (g) Time step 0.18s (h) Time step 0.21s
(i) Time step 0.24s (j) Time step 0.27s

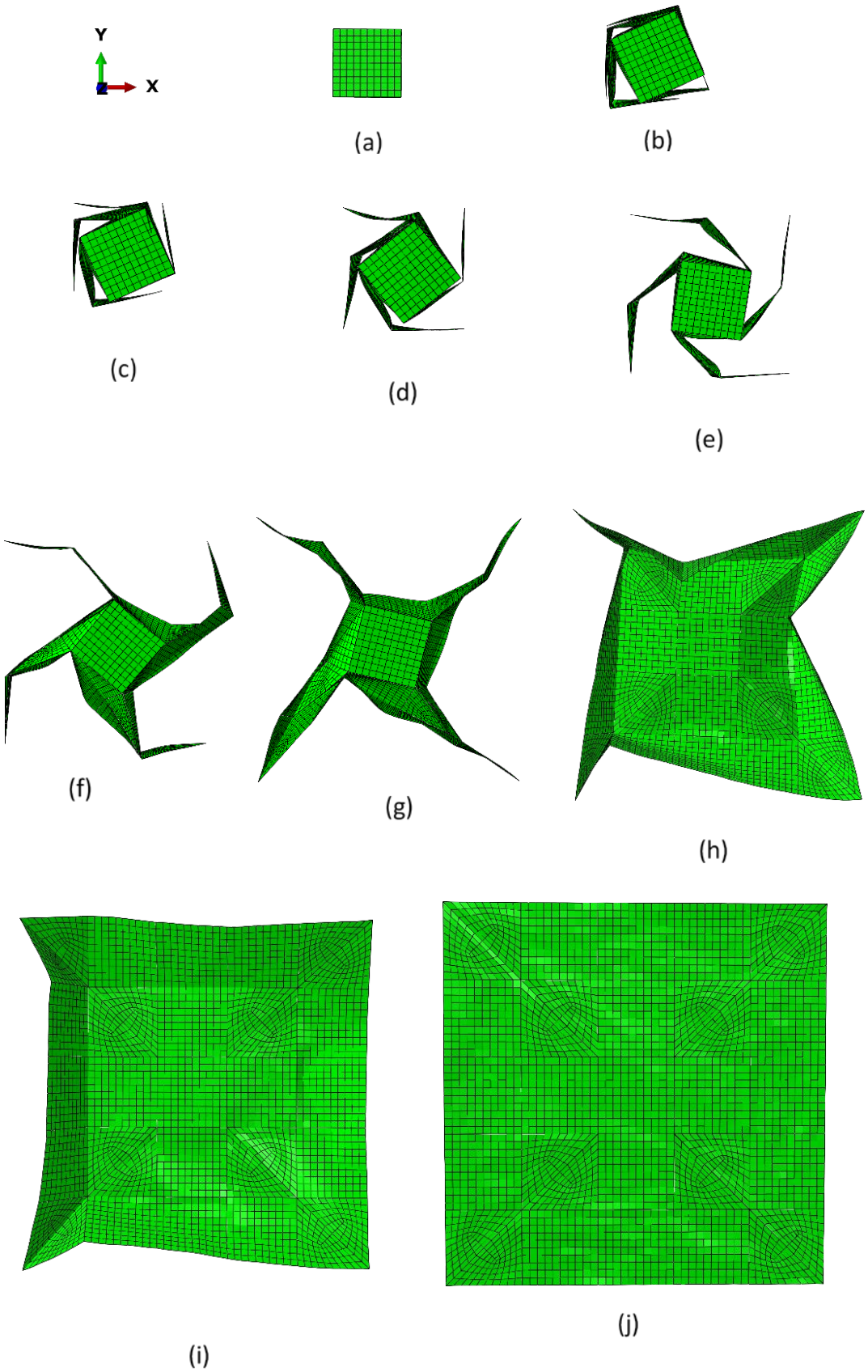

Figure 6. Deployment of the Circumferential Folding Pattern
(a) Time step 0s (b) Time step 0.03s (c) Time Step 0.06s (d) Time Step 0.09s
(e) Time Step 0.12s (f) Time Step 0.15s (g) Time step 0.18s (h) Time step 0.21s
(i) Time step 0.24s (j) Time step 0.27s

3.1. Results and Discussion

Both the models showed similar deployment behaviours. Hence both the behaviours will be explained using one energy diagram. Figure 7 below shows the variation of energy of the model (energy balance (See Equation 1)) plotted for the spiral folding pattern.

Energy Balance is based on the conservation of energy and provides a mean to check the plausibility of the Finite Element Analysis (FEA) carried out.

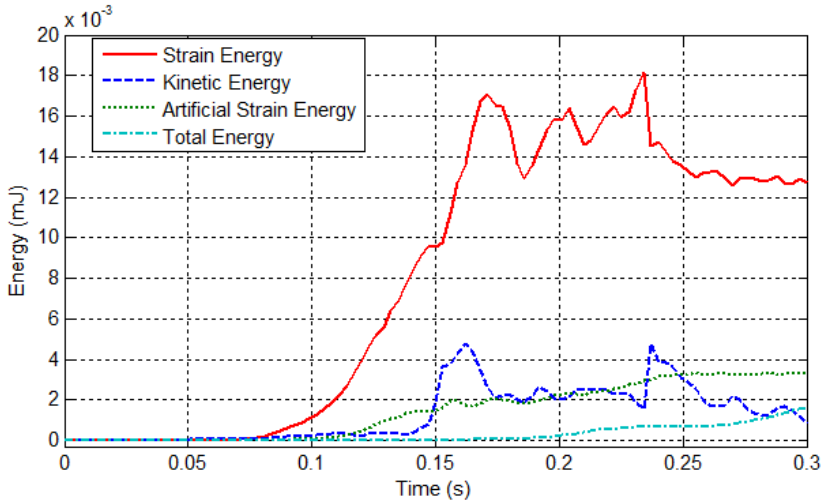

Figure 7. Energy variation of spiral folding pattern

Energy Balance Equation used in Abaqus Solver is as follows (Mallikarachchi, 2011):

$$E_b = E_i + E_v + E_k - E_w \quad (1)$$

Where:

E_i = Internal strain energy including the strain energy and artificial strain energy
E_v = Viscous dissipation
E_k = Kinetic energy
E_w = Sum of work by all external forces

In both the models, the kinetic energy is significant compared to their strain energy. This means that the quasi-static states of the models are violated. The reason for this is that, the whole deployment process occurs rapidly (In 0.3 s). If the membrane deploys slow enough, quasi-static conditions could be met.

Sudden increases can be seen in kinetic energy plots, coinciding with a sudden drop in the strain energy. This occurs as a result of dynamic snapping where the structure jumps to a more stable configuration with a release of internal strain energy. During this the kinetic energy will show significantly high values even in slow deployments.

Total energy plots provide a mean of checking the robustness of the analysis. In these simulations, total energy is negligible for majority of the time, while it shows a noticeable rise during the final 0.1 s.

Figure 8 above shows the Force – Displacement curves for deployment for the two models. These curves show very high variation in the force during the final stages of deployment. This confirms the occurrence of dynamic snapping.

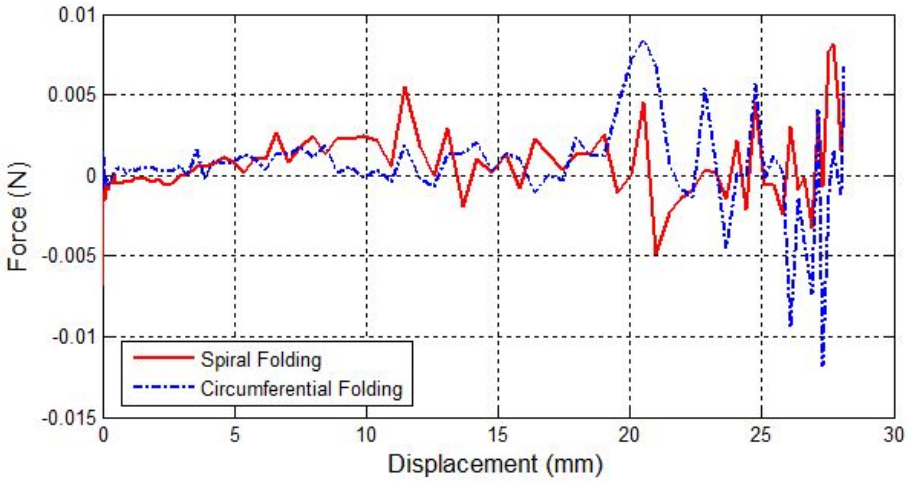

Figure 8. Force vs. displacement relationships

External energy requirement is one major deciding factor for the selection of the optimum folding pattern. The model with a low energy requirement is more attractive.

Comparing the two curves Figure 9 below, it is evident that the circumferential folding model requires a higher energy for the deployment.

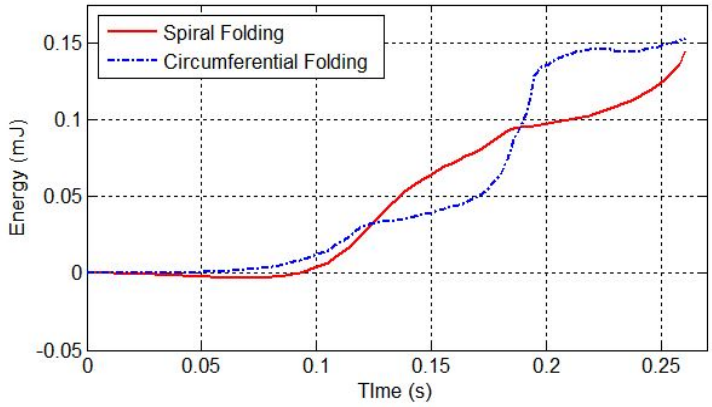

Figure 9. External work done

4. CONCLUSIONS

Deployment of the two finite element models was carried out successfully, using the Abaqus/Explicit Solver.

Spiral folding pattern shows a smooth and regular deployment sequence with very small deformations in the panels. During deployment, the stress development and reaction forces of the model too shows significantly lower values than the circumferential folding model.

From the above reasons, the spiral folding pattern proves to be the more preferred case for use in deployable structures.

All these models were checked for ideal conditions, with the assumptions of zero-thickness in membranes and no plastic deformation in the fold lines. Several authors have published ways to modify the fold line arrangement to take the thickness of the membrane into consideration. Hinge properties can be included to take into account the plastic nature of the fold lines. By including these two aspects, the results can be improved a great deal.

This research consists of computer modelling of folding patterns to check the optimum pattern. This was checked with a paper model to check whether the deployment processes approximately matches the real world scenario.

But to check the validity of the model accurately, a physical model is required to be checked while taking measurements of the force exerted by the model in the deployment process.

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Correction to Tension Stiffening Effect of Reinforced Concrete Based on Free Shrinkage and Expansion Conditions

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Abstract: *Tension stiffening effect is most trusted way to account for concrete contribution in tension at serviceability limit state calculations of deflection and crack width predictions. Shrinkage and thermal effects of concrete have major influence on the cracking and deformation of reinforced concrete elements at serviceability limit state and hence required to be account for better results. There is still lack of agreement for modelling tension stiffening effect of concrete and incorporate shrinkage and thermal effects in the calculating deflection and cracking behaviour of reinforced concrete elements. In this experimental investigation different shrinkage effects between - 460 με to + 2110 με was obtained to study the effect of shrinkage on tension stiffening behaviour using and curing conditions. Test results of reinforced concrete elements shows direct influence of shrinkage on tension stiffening effect. However it is found that adjustment for full free shrinkage is unacceptable procedure to accommodate shrinkage on tension stiffening behaviour of concrete. It is debated that extent of the shrinkage shall be limited to the extent of cracking. Eliminating shrinkage effect based on this principle show gradual agreement between the experimental and analytical situations for tension stiffening effect.*

Keywords: *Reinforced concrete, Serviceability limit state, Shrinkage, Tension element, Tension Stiffening*

1. INTRODUCTION

Unlike in the prediction of ultimate limit state, prediction at serviceability limit state performance of reinforcement concrete has been found to be inconsistent with the existing guidelines and formulation (BS 8110, BS 8007, EC2, CEB-FIP Model Code 90, and ACI). Making the matter worse, it is found that in most of the time prediction of serviceability limit state deflection and cracking has been significantly less than actual values. This has led to most of the structures designed according to existing code of practices, are deflected and cracked more during service Gilbert (2001). Major issues for improve the accuracy of serviceability limit state predictions has been to find concrete contribution in tension. As opposed to the ultimate limit state prediction, where concrete contribution in tension is neglected in the calculation for capacity of the section. Concrete contribution in tension is considered significantly important for accurate serviceability limit state prediction of cracking and deflection of concrete structures.

Thermal & shrinkage effect and creep of concrete all affects concrete contribution in tension and are important in modelling serviceability performance of concrete accurately. Given the nonlinear nature of reinforced concrete behaviour, considering tension behaviour of concrete in terms of average stress strain behaviour is considered complicated enough. But yet, there seems to be no other mean to incorporate concrete contribution in tension. It is also considered in numerical studies, that modelling the interaction between reinforcement and concrete is better achieved by average stress strain behaviour rather than modelling reinforcement and concrete separately. Finite element programme based on smeared crack approach, layered beam analysis truss modelling have all used tension stiffening effects for modelling concrete behaviour in tension. Under this background it is increasingly felt that modelling tension stiffening effect is the best mean to incorporate thermal and shrinkage effect for serviceability prediction of concrete structures under different environmental conditions. This

research is devised to come up with solutions to improve serviceability limit state prediction of reinforced concrete by means of developing a robust tension stiffening behaviour for reinforced concrete capable of handling shrinkage effect of concrete.

1.1. Tension Stiffening

Concrete is quasi brittle material with no significant post crack softening behaviour on its own. However in the presence of reinforcement, concrete continue to carry tension between crack contribute and is capable of reducing overall strain of the reinforcement bar giving additional stiffness to the overall performance of reinforced concrete. This additional stiffness in tension is called the tension stiffening behaviour of concrete. Response of reinforced concrete tension element can be broadly divided into three stages; Pre crack stage, cracking stage, Post cracking stage as displayed in Figure 1.

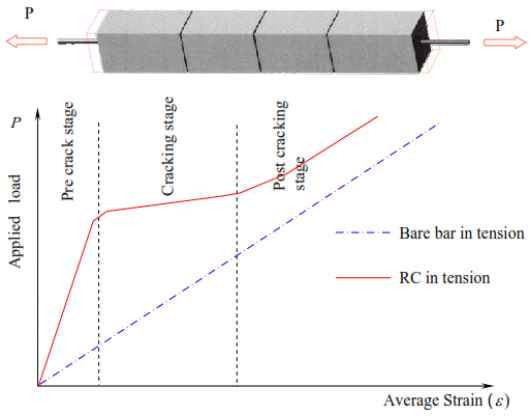

Figure 1 Typical response of a specimen subjected to direct tension tests

Figure 2 shows the actual strain gauge reading in a reinforcement bar in tension while cracking is in progress, measured using electrical resistance strain gauges attached at the middle of a specially prefabricated tension bar. Figure 3 is a close up of strain distribution and bond stress distribution along the bar between crack #1 and #2 recorded till the final failure of bar in tension Sooriyaarachchi.et al (2005).

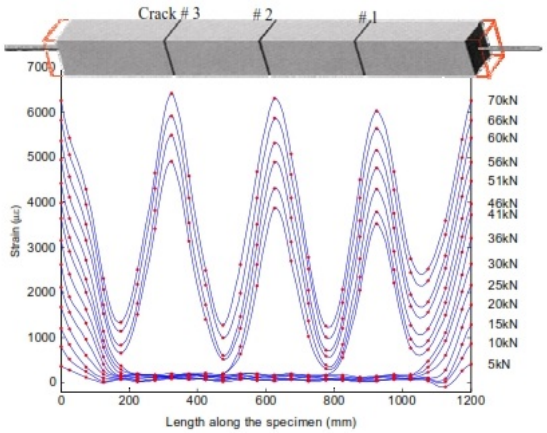

Figure 2 Strain distribution in the reinforcement during crack development and immediate before and after cracking.

From the above strain reading it is clear that, at cracks total force is taken up by the bare bar while reinforcement and concrete share the tension force between cracks. Crack stops to happen when the spacing between adjacent cracks are less than twice the length required to transfer force equal to the tensile strength of concrete. It is also vividly clear that the strain in the bar is reduced between cracks

by the concrete contribution and therefore overall additional stiffness provided by the concrete. However after yielding of steel, concrete contribution in tension is expected to be dropped extensively due to drop in stiffening of the yielding reinforcement bar and hence making tension stiffening effect little or no relevance to ultimate limit state performance of reinforced concrete. When bar bare response is removed from the overall response, concrete contribution in tension can be calculated. This when normalized by dividing it by tensile strength at first crack, is often refe bond factor (β) and is often used to method to express concrete contribution in tension

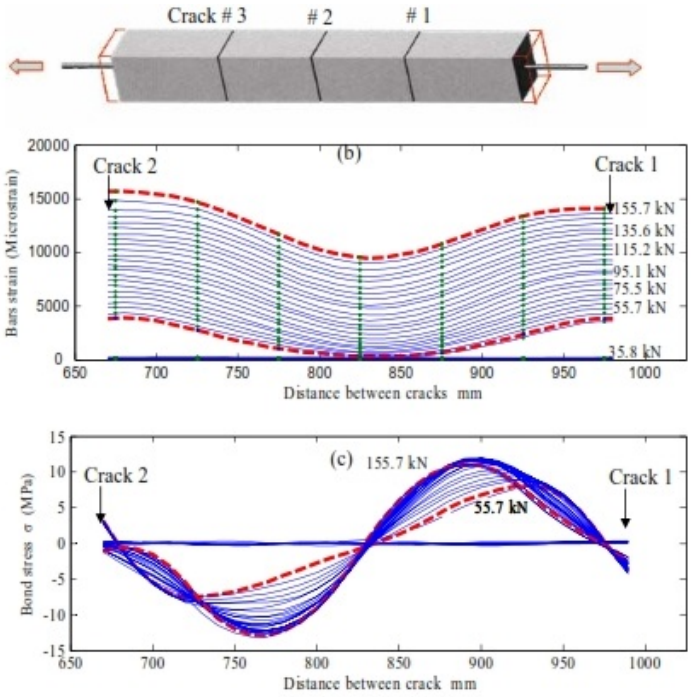

Figure 3 Strain and bond stress distribution between crack #2 and #3

Accuracy of the modelling serviceability limit state performance is primary depending on this behaviour. Figure 4 shows different model proposed Collins and Mitchell (1991), Belarbi and Hsu (1994), Johnson (1951), CEB-FIP Model Code 1990 for the bond factor. It is seen from the above models that there is no agreement between different models and that there are no provision in the models to account for thermal and shrinkage effect.

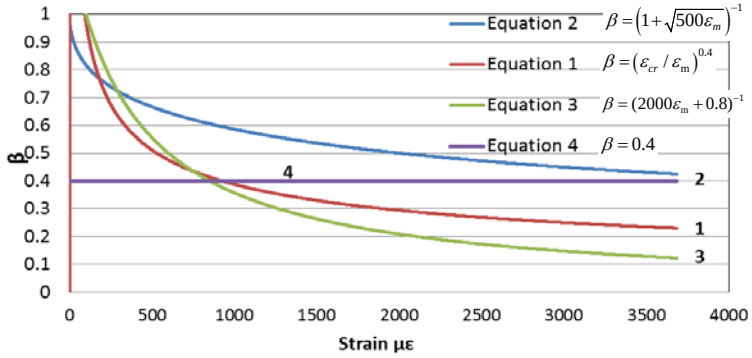

Figure 4 Softening behaviour of concrete/ bond factor predictions under different models

1.2. Method to Determine Tension Stiffening Effect for Shrinkage Conditions

Reinforcement ratio, concrete strength, bond effect, surface area to volume ratio of reinforcement (bar number), shrinkage of concrete are all considered as variables affecting tension stiffening effect of concrete. All concrete are subjected to shrinkage of one form or another. This means that, if not for

drying the more usual form of shrinkage, all concrete are subjected to some form of autogenous type (self-desiccation) of shrinkage. As shrinkage is a natural phenomena in concrete force exerted by the free shrinkage (free shrinkage strain ε_{sh}) of concrete can be quantified as $\varepsilon_{sh} A_c E_c$. However, this does not mean the reinforced concrete would shrunk to the same extend of the free shrinkage proposed above. Shrinkage force is resisted by reinforcement and concrete, together as a composite element. Thus, the shortening of the reinforced concrete element ($\varepsilon_{\mathrm{m},i}$) is limited to

$$\varepsilon_{\mathrm{m},i} = \frac{\varepsilon_{sh} A_c E_c}{A_s E_s + A_c E_c} \tag{1}$$

As a results of shrinkage, reinforcement bar is subjected to compression strain. Although concrete has shorten by $\varepsilon_{m,i}$ concrete is actually subjected to tension. This is because concrete has the desire to shrink by ε_{sh} but only allowed to be shorten by $\varepsilon_{m,i}$. The different between $\varepsilon_{sh} - \varepsilon_{m,i}$ is the magnitude of the concrete stretch in tension. The concrete tensile strain due to shrinkage ($\varepsilon_{c,i}$)can therefore be express as

$$\varepsilon_{c,i} = \frac{\eta\rho\varepsilon_{sh}}{1+\eta\rho} \tag{2}$$

Here, ρ is reinforcement ratio (A_s / A_c), η is modulus ratio E_s / E_c , A_s is steel area, A_c is concrete area, E_s and E_c are the elastic modulus of steel and concrete.

As concrete in shrinkage condition subjected to a tensile strain, concrete will crack early. Resulted reduction in cracking load in terms of free shrinkage ε_{sh} can therefore be calculated and shown to be equal to

$$N_c = \frac{-\varepsilon_{sh} E_c A_c \eta\rho}{1+\eta\rho} \tag{3}$$

or simply . $E_s A_s \varepsilon_{m,i}$

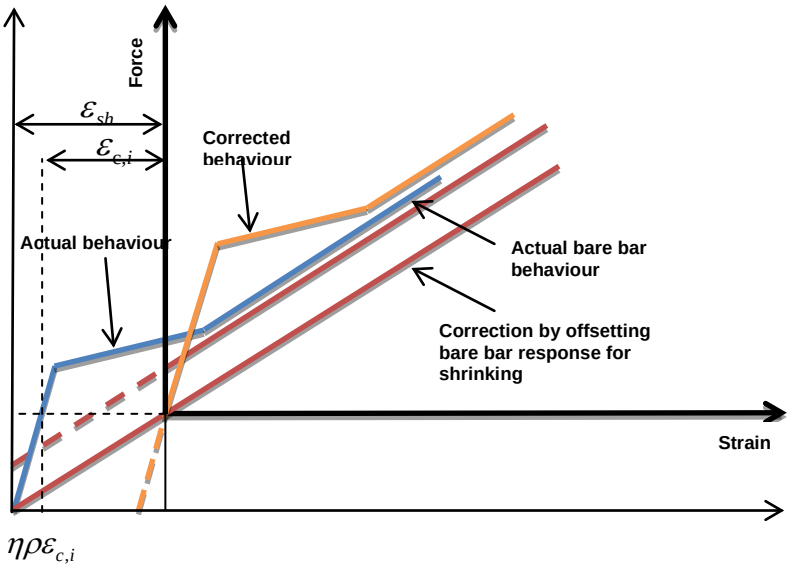

Figure 5 Transforming tension response of shrinkage concrete to non-shrinkage state

Schematic representation of tension element subjected to shrinkage is shown in the Figure 5. Similar derivation is possible for concrete subjected to expansion. The difference between recorded tension response of a specimen subjected to shrinkage and an offset drown parallel to the bare bar response to the right, by an amount equal to free shrinkage of the specimen, provides a good estimate in

transforming shrinkage concrete response to non shrinkage state and vice-a-versa.In similar argument by shifting the bare bar response to the left by and amount equal to free expansion can be used to transfer behaviour of expansive regime to zero expansive state and vise-versa (see Figure 6).

1.3. Significant of the Study

This paper is an attempt to quantify tension stiffening behaviour in shrinking, non shrinkage and expansive regimes in order to find more robust method to quantify tension stiffening effect under different conditions. In this investigation tension elements are used to quantify tension stiffening effect. From the experimental investigation on more than 41 specimen subjected to different shrinking regimes, it is found that account shrinkage and expansion effect in the same way based on free shrinkage has been problematic Based on the experimental investigation new method to make adjustment for tension stiffening effect due to shrinking based on calculation effective shrinkage value is proposed as a solution. It is shows that the proposed method provide good agreement between calculated results and actual experimental results. Theoretical explanation as to why this adjustment shall be made is also explained.

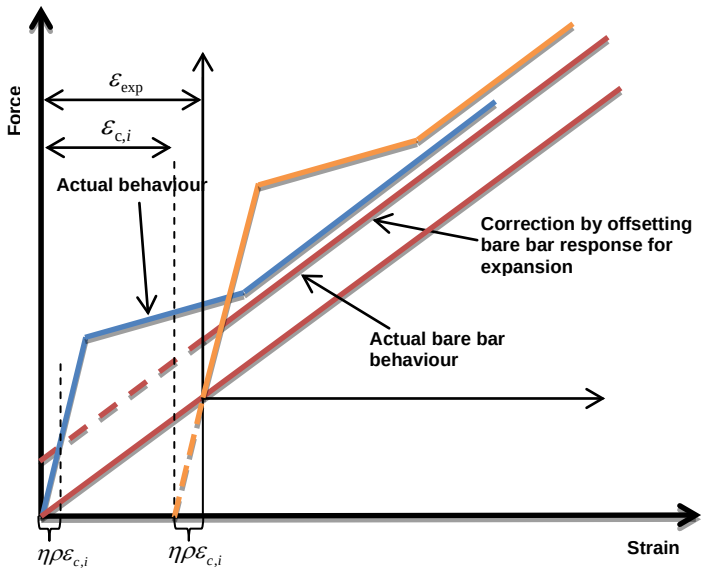

Figure 6 Transformation tension response under concrete expansion

2. METHODOLOGY

2.1. Experimental Plan

Total of 42 tension elements, belonging to four different regimes, form the discussion of this paper. Regime 1 consists of sample with normal concrete exposed to natural weathering conditions recording free shrinkage of -460 με on average. Regime 2 consists of concrete water cured to eliminate shrinkage effect and recorded neither shrinkage nor expansion. Regime 3 consists of concrete with shrinkage compensating agent adding on to concrete at 30kg per m^3 of concrete volume and water cured at room temperature recording +671 με expansion on average. Regime 4 was produced by adding shrinkage compensating agents at a rate of 60kg per m^3 of concrete recording expansion of around +2100 με. Table 1 shows all the tension element used for analysis under different regimes. Experimental results and discussion that follows uses the notation in Table 1 to identify test elements. Notations follow quite naturally from the different variable associate with the test specimen as follows; Cross section of the testing specimen/Diameter of the bar/Concrete Grade/Reinforced ratio / average shrinkage recorded at the testing regimes and to what curing regimes the specimen belong.

Table 1 Tension element used for analysis

No	Specimen Name	No	Specimen Name
1	Sqr100/T10/0.79/G30/-460 με/Reg1	15	Sqr100/T16/2.0/G30/+671 με/Reg3
2	Sqr100/T10/0.79/G30/+08 με/Reg2	16	Sqr100/T16/2.0/G30/+2110 με/Reg4
3	Sqr100/T10/0.79/G30/+671 με/Reg3	17	Sqr100/T16/2.0/G45/-460 με/Reg1
4	Sqr100/T10/0.79/G30/+2110 με/Reg4	18	Sqr100/T16/2.0/G45/+08 με/Reg2
5	Sqr100/T12/1.13/G30/-460 με/Reg1	19	Sqr100/T16/2.0/G45/+671 με/Reg3
6	Sqr100/T12/1.13/G30/+08 με/Reg2	20	Sqr100/T16/2.0/G30/+2110 με/Reg4
7	Sqr100/T12/1.13/G30/+671 με/Reg3	21	Sqr150/T16/0.8/G30/-460 με/Reg1
8	Sqr100/T12//1.13/G30/+2110 με/Reg4	22	Sqr150/T16/0.8/G30/+08 με/Reg2
9	Sqr100/T12/1.13/G45/-460 με/Reg1	23	Sqr150/T16/0.8/G30/+671 με/Reg3
10	Sqr100/T12/1.13/G45/+08 με/Reg2	24	Sqr150/T16/0.8/G30/+2110 με/Reg4
11	Sqr100/T12/1.13/G45/+671 με/Reg3	25	Sqr150/T16/0.8/G45/-460 με/Reg1
12	Sqr100/T12/1.13/G45/+2110 με/Reg4	26	Sqr150/T16/0.8/G45/+08 με/Reg2
13	Sqr100/T16/2.0/G30/-460 με/Reg1	27	Sqr150/T16/0.8/G45/+671 με/Reg3
14	Sqr100/T16/2.0/G30/+08με/Reg2	28	Sqr150/T16/0.8/G45/+2110 με/Reg4

For an example notation of specimen #1,
Sqr100/T10/0.79/G30/-460 με/Reg1, means; that the specimen; has 100 mm square concrete cross section; has reinforced with 10 mm high yield strength reinforcement bar; has compressive strength 30 N/mm^2 at the time of testing specimen in tension; has a reinforcement ratio equal to 0.79; has cast and cured according to regime 1 and records -460 με free shrinkage at the time of testing.

2.2. Experimental Set-up

Figure 7 shows the schematic representation of the test set-up for tension element. Measurement of displacement due to concrete cracking was recorded using three Linear Variable Displacement Transducers (LVDT's) connected at three of four surfaces of the concrete element, while two set of three transducers each connected at the two end of the specimen, was devised to measure the relative slip between the concrete and reinforcement bar at the two edges of the tension element. The displacement of the reinforcement concrete element at given load is equal to summation of the average of surface displacement and average slip at the two ends. All measurements were taken real time at every 0.4 second interval with all load and displacement transducers connected to the data logger. Figure 8 shows photograph of test set-up during testing.

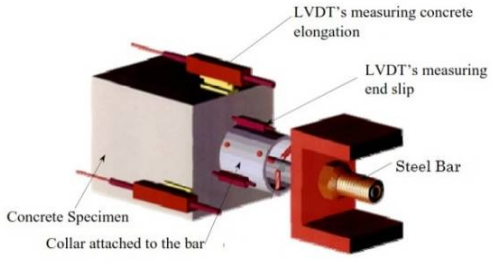

Figure 7 Arrangement of 10 mm and 100 mm Transducers to measure displacement of tension element



Figure 8 Testing Arrangement of the tension Specimens

3. RESULTS AND ANALYSIS

3.1. Concrete Contribution in Tension

Figures 9, 10 show the selected response of reinforced concrete element compared with bare bar for the four regimes. Stiffer response of the reinforced concrete is due to concrete contribution in tension between cracks. When bare bar response is deducted from the overall response of reinforced concrete element, concrete contribution in tension can be achieved. Figure 11, 12 show the concrete contribution for tension for the Figure 9, 10. It is usual practice to express concrete contribution in tension at any time as ratio to concrete strength at 1st crack. Figure 13 shows the response after the adjustment to shrinkage for the test results of Figure 9 in the way it is described at the beginning of this paper. As can be seen in the Figures 13 adjustments based on free shrinkage measurements are found grossly inappropriate to explain the results of the experimental investigation.

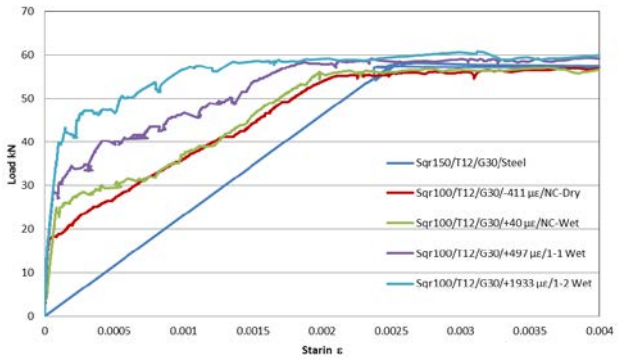

Figure 9 Axial forces vs Average Strain for 12 T Grade 30 RC element

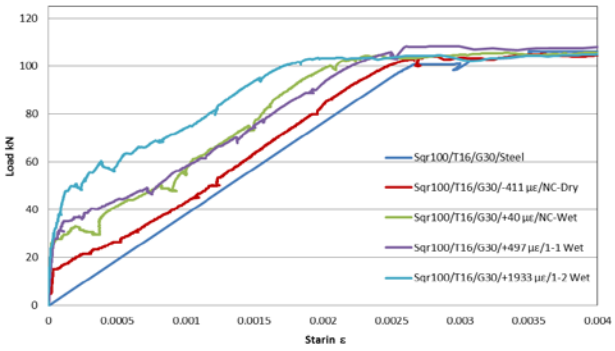

Figure 10 Axial forces vs Average Strain for 16 T Grade 30 RC element

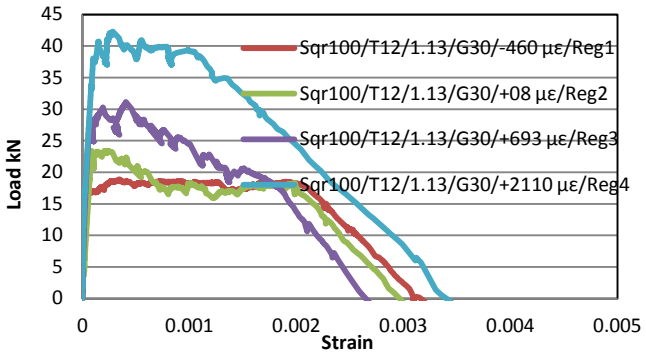

Figure 11 Concrete contribution for T12 Grade 30 RC element

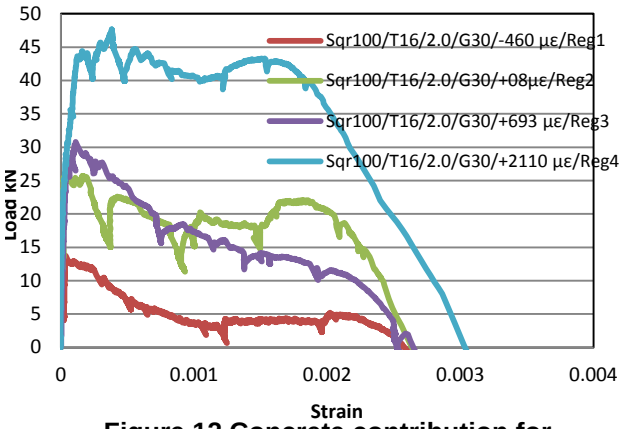

Figure 12 Concrete contribution for T16 Grade 30 RC element

Figure 14 shows the adjustment to test results using a fraction of free shrinkage values recorded. Results show good agreement between the test results for both reinforcement bars tested and describe above. Similar improvement was seen in the test results described in the Table 1. After adjustment for eliminate shrinkage effects test results also show better agreement with the existing code formulation for tension stiffening effect (CEB-FIP 78).

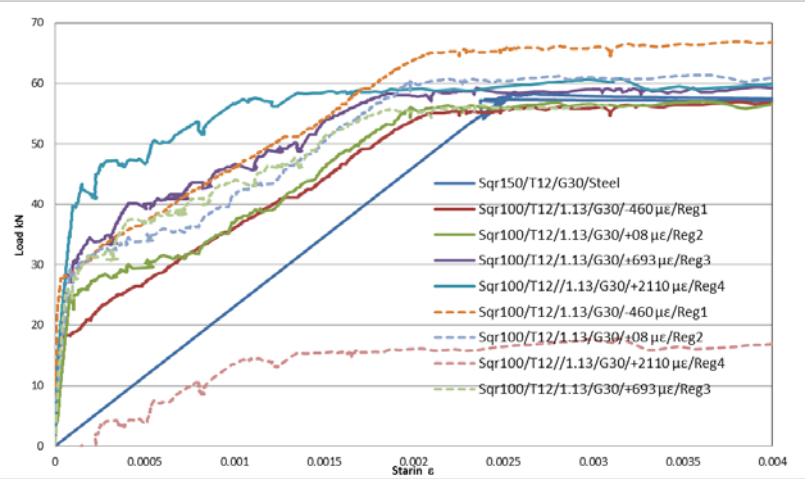

Figure 13 After adjusting free shrinkage strain for T12 G30 RC element

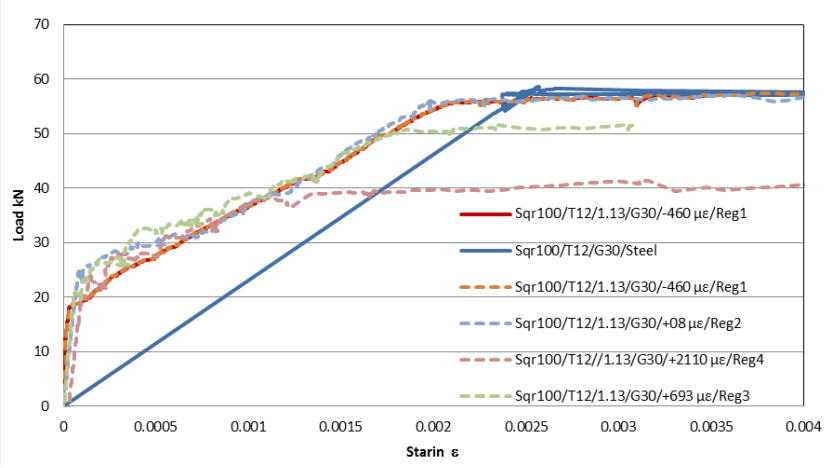

Figure 14 After adjusting free shrinkage strain for T12 G30 RC

Recommended shrinkage adjustment based on free shrinkage value measured for different specimens are given in Table 2.

Table 2 Shrinkage adjustment based on free shrinkage value

Specimen Regime	Average Shrinkage Value με	Adjusted Average Shrinkage Value με	Adjustment as fraction
Regime 1	-460	-250	54%
Regime 3	671	-322	50%
Regime 4	1500	-705	47%

Good agreement of results after adjustment seems to suggest that the proposed adjustment can account for the effect of shrinkage/expansion for tension stiffening behaviour. Above results indicate that the accounting for the tension stiffening behaviour of concrete.

4. DISCUSSION

The paper discussed the importance of accounting for shrinkage effect in tension stiffening studies. It is shown with the reasonable degree of accuracy can be made between tension stiffening effect of shrinking and non-shrinking concrete by considering the shrinkage of concrete. The paper has also shown that the adjustment made based on the free shrinkage and expansion condition according to Table 2 is the appropriate method to account shrinking and expansion condition. As the shrinkage and expansion are release only in either side of the cracks, recommendation of Table 2 values which is about half of the recorded free shrinkage is considered more logical adjustment shrinkage of concrete. Further studies are planned to investigate how the cracking and deflection of reinforced concrete vary under shrinkage and expansion conditions. With proper adjustment to shrinking and expansive conditions. It is expected that proper account of shrinkage and expansion can improve the serviceability limit state predictions of cracking and deformation of reinforced concrete structures.

5. ACKNOWLEDGMENT

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