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PROCEEDINGS OF CIVIL ENGINEERING PROJECT SYMPOSIUM

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DEPARTMENT OF CIVIL ENGINEERING
FACULTY OF ENGINEERING
UNIVERSITY OF PERADENIYA
SRI LANKA

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PROCEEDINGS OF THE
CIVIL ENGINEERING PROJECT SYMPOSIUM
CEPS 2012

Organized by

Department of Civil Engineering, Faculty of Engineering
University of Peradeniya, Sri Lanka

&

Civil Engineering Society, Faculty of Engineering
University of Peradeniya, Sri Lanka

Supported by

Tokyo Cement Group, Sri Lanka

31st December 2012

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HIGHWAY AND TRANSPORTATION ENGINEERING

Message from the Dean, Faculty of Engineering

Initial training in research within undergraduate academic activities is an important aspect of career development of young engineers. What we see in this document is the outcome of certain amount of embedded research into projects conducted by final year Civil Engineering undergraduates.

I am happy to write this brief message for the proceedings of the Civil Engineering Project Symposium (CEPS) 2012 as this is a display of the outcome of an exercise of exposure to research and formal technical writing. I am sure the publication serves as a good record of the work that help wider circulation and recognition, and also serves as a souvenir that reminds of activities of undergraduate life here at the Engineering Faculty, University of Peradeniya.

I take this opportunity to thank the staff of Civil Engineering department for encouraging and facilitating this outcome, and thank the Civil Engineering Student society for the hard work of the membership. Further, on behalf of the Faculty, I extend a warm gratitude to Tokyo Cement Group for the continued support.

Prof. Leelananda Rajapaksha
Dean/Faculty of Engineering
University of Peradeniya

Message from the Head, Department of Civil Engineering

It is with great pleasure that I send this message to the first Civil Engineering Research Symposium of our department. Publishing the research output of the final year undergraduate projects in the form of a bound volume is an important milestone in the progress of our department.

The Tokyo Cement Company (Lanka) Ltd. has been generously assisting the department of civil engineering for several years in carrying out the final year projects. The memorandum of understanding between our two institutions is a fine example of how Industries in Sri Lanka can assist the higher education institutions with long term commitments. The release of the proceedings of the first symposium is a direct result of this assistance. Special words of thanks are due for Tokyo Cement Company for their sponsorship of this symposium and for their continued support of our undergraduate research programmes.

I wish all the best for the day's events and hope that this will become a regular feature in our calendar and enhance the development of our department. Finally I thank the enthusiastic effort put forward by the project coordinator and instructors of the department.

Dr. A.L.M. Mauroof
Head / Department of Civil Engineering
Faculty of Engineering
University of Peradeniya

Message from Tokyo Cement Group

Tokyo Cement Group is extremely pleased to be a partner in promoting Civil Engineering Undergraduate Research Programme at the University of Peradeniya.

The training and knowledge students will gain during conducting the research projects will help on the path toward a successful carrier as a Professional Engineer.

We have strong relationship with the Department of Civil Engineering and being sponsoring various other programmes during the last few years.

We congratulate the students who take part in today's programme and completed the degree successfully and wish you the best in your professional endeavours.

Dr. M.G.M.U. Ismail
Technical Consultant
Tokyo Cement Group

Editor's Note

As Editors of the Civil Engineering Research Symposium (CEPS) 2012, we are proud to present the Inaugural issue of this series. Our aim is to create an intellectual forum to demonstrate the achievements of Civil Engineering students of the University of Peradeniya carrying out research in Faculty. We believe this volume would serve in advancing and disseminating knowledge in a multitude of research themes ranging from engineering fundamentals to applications. The CEPS 2012 sessions highlighted an array of such significant contributions in several research areas. We would like to thank the authors for their contributions and all panel members for their invaluable guidance and assistance in making this Research Proceedings volume of CEPS 2012 a reality.

We greatly appreciate Prof. Leelananda Rajapaksha, the Dean, Faculty of Engineering, University of Peradeniya and Dr. M.G.M.U. Ismail, Technical Consultant, Tokyo cement group for their illuminating plenary speeches at CEPS 2012 and for sharing their years of wisdom and experience with the participants. All the presenters and session chairs contributed with enthusiasm and shared a significant portion of responsibility, making CEPS 2012 a success. We thank them all for their dedication and fruitful effort.

Dr. A.L.M .Mauroof, Head, Department of Civil Engineering, Faculty of Engineering, University of Peradeniya is gratefully acknowledged for this guidance and encouragement during the preparation of CEPS 2012. We also extend our gratitude to Prof. K.D.W. Nandalal and Dr. A.P.N. Somaratna, for their support throughout this process. Further, we wish to convey our sincere appreciation to Tokyo Cement Group and who extended their support to make CEPS 2012 a successful event

Prof. K.P.P Pathirana and Dr. K.B.S.N. Jinadasa
Editors of CEPS 2012
Department of Civil Engineering
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ENVIRONMENTAL ENGINEERING

Characterization of Landfill Dump Sites around Sri Lanka

R. Jayathilake, K.H.P. Madusanka, D.D.S.S. Weerasooriya, G.B.B. Herath
and N.K. Wijewardena

Key Words: Calorific value, Moisture content, Municipal solid waste, Open dumps, Waste characteristics.

1. Introduction

Municipal Solid waste disposal is a major issue throughout the developing world including in Sri Lanka. Commonly used method for solid waste disposal in developing countries is open dumping. However open dumping is not a solution for solid waste disposal as it causes many environmental and health problems. Therefore finding alternative option for open dumping is urgent and has to be two folded; affordable cost effective solutions for solid waste disposal and finding methods for minimizing the risks at already existing dumps.

With the above intension, in this study five landfill dump sites around Sri Lanka located at different geo-climatic setting and different waste management approaches are investigated to ascertain the possibility of mining waste for energy generation.

2. Literature Review

Incineration, Sanitary Landfilling, Open Dumping, Composting, Recycling, land mining are the popularly used methods for solid waste disposal including Sri Lanka. Of these, the common practise for waste disposal among most local authorities in Sri Lanka is open dumping. Since open dumping has many adverse issues, among the other options available, given the high percentage of plastics and organic composition in local solid waste, waste to energy can be an attractive alternative for solid waste management in Sri Lanka. In this regard, incineration is a good option for energy recovery. Solid waste can be incinerated at 800-1050 ^{0}C. To establish a viable incineration plant average lower Calorific Value of the waste must at least be 6 MJ/kg throughout all seasons. Also the annual average lower Calorific Value must not be less than 7 MJ/kg and the annual amount for incineration should not be less than 50,000 metric tons (Municipal Solid Waste Incineration, 1999).

3. Methodology

Based on geographical setting and solid waste management systems in place, the following five dump sites were selected for this study.

Table 1: Selected solid waste dump sites

Location	Climatic Zone	Management system
UdaPalatha	Wet Zone	Abandoned
Nuwara-Eliya	Wet Zone	Engineered Landfill
Matale	Intermediate Zone	Residue dumping
Gohagoda	Wet Zone	Mixed waste
Hambantota	Dry Zone	Residue dumping

Samples for analysis were collected from top and at depth of around 0.6 to 1.0 m at each site. Waste on top was considered recently dumped while at a 0.6 to 1.0m depth was old waste.

Moisture Content (MC), Composition percentage (Japanese method) and Calorific Value (CV) of each sample were investigated. Depending on the outcome, calorific value change was investigated considering the geo-climatic differences and the solid waste management.

4. Results and Discussion

Calorific Values and Moisture Content

Table 2: Calorific Values and Moisture Content

Location		Calorific Value / (MJ/kg) Moisture content / (%)			
		Old Waste		Recently Dumped Waste	
		0.0 m	1.0 m	0.0 m	1.0 m
UPS	Top	- 26.10	- 24.30	- -	2.80 40.20
	Middle	1.46 28.80	- 17.50	14.02 55.70	15.11 58.25
	Bottom	- -	1.15 23.40	4.53 47.10	- 44.40
Matale		4.07 6.43	3.44 35.30	6.23 31.80	3.99 39.30
NuwaraEliya (0.2-0.4 m depth)		1.01 25.90		5.14 44.40	
Gohagoda		-	-	2.74 34.10	-
Hambantota		-	-	3.93 35.70	2.05 22.28

Obtained results show that the Calorific Values of recently dumped waste collected vary between 2.05 to 6.23 with the highest at Matale.

But there are two unusually high values (15.11-14.02 MJ/kg) recorded at middle locations of UPS Recently dumped waste. Those two values are much higher than the values typically expected from countries like Sri Lanka. Reason for this high value is clearly explained by its composition data. In these two locations, the soil content is much less than other locations in UPS. This is due to instability of soil cover retaining in the embankment.

Other than above these two values, the highest CV (6.23 MJ/kg) is recorded at Matale where waste segregation is done. Lowest value (1.01 MJ/kg) was recorded at NuwaraEliya Sanitary landfill. The highest value recorded from UPS, Gohagoda, Hambantota and NuwaraEliya are 4.53 MJ/kg, 2.74 MJ/kg, 3.93 MJ/kg and 5.14 MJ/kg respectively. Wastes were taken from top surface of recently and were dumped in each site show the highest calorific results. Old waste samples show lower calorific values (1.01 MJ/kg- 4.07 MJ/kg).

Comparatively the moisture content was high in fresh waste to that of the mined samples. Highest moisture content of 58.25% was obtained with the recently dumped waste at UPS and the lowest of 6.43% with mined waste at Matale. The moisture content of mined recently dumped waste is higher than the moisture content of the mined old waste.

5. Conclusion

These results show that the lower calorific value of NuwaraEliya old waste proves better biodegradation at this engineered landfill even at lower temperature. At Matale; the high calorific value is attributed to the partial separation especially the biodegradable compounds before waste are being dumped at the fill. Further the difference in geographical, climatic condition and composition of the collected waste also showed influence on the CV.

As a whole, the since the CV, for better waste to energy conversion, it is required less in many site are less than 6 KJ/kg, it is not suitable to use the waste in selected sites for energy generations.

Reference

The World Bank Washinton DC, 1999, *Municipal Solid Waste Incineration.*

Current Practices of Hazardous Waste Management in Central Province and Development of Proposal for Effective Management

I.L.M. Nifraas, A.H. Abthullah, M.M.M. Ajmal Faaique and K.B.S.N. Jinadasa

Keywords: Household hazardous wastes, Healthcare wastes, Hazard symbols, Questionnaire survey, Stratified random sampling, Analysis

1. Introduction

This document is purposed to provide the necessary information on hazardous wastes specially generated in households and healthcare institutions in the central province of Sri Lanka, and the way how these hazardous wastes are managed. Questionnaire survey was carried out in randomly selected households in several local municipalities, three major government hospitals and some private clinic centres, to study the current practices of hazardous waste management in the central province. Stratified random sampling method was used to analyse the data to obtain estimated results in both quality wise and quantity wise.

2. Literature Review

2.1 Definition of Hazardous Waste

There are three ways that a solid waste can be considered "hazardous" under Environmental Protection Agency (EPA) regulations.

i. The waste is specially listed in EPA regulations.
ii. The waste is declared hazardous by the generator based on its knowledge of the waste.
iii. The waste is tested and meets one of four characteristics established by the EPA:
Ignitable, Corrosive, Reactive, Toxic

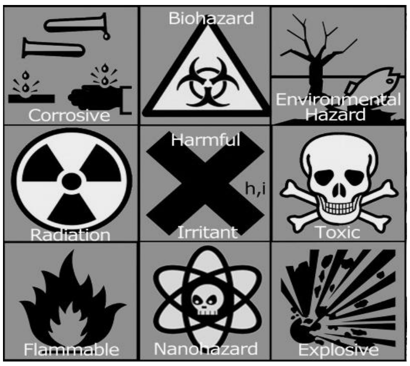

Figure 1: Hazard Symbols

2.2 Classification of Hazardous Wastes

According to the type of wastes produced from various places, the classifications are:

i. Household hazardous wastes
e.g.: CFL bulbs, Toilet bowl cleaners, Body spray, Nail polish, Pesticides, etc.
ii. Healthcare wastes
e.g.: Pharmaceutical wastes, Infectious wastes, Sharp wastes, Pathological and Anatomy wastes, Liquid and Chemical wastes, etc.
iii. Industrial and commercial wastes
e.g.: Paint, Thinner, Oil, Acidic and base wastes, Agricultural wastes, etc.
iv. Electronic wastes
e.g.: Batteries and all other electronic components

3. Methodology

Questionnaire survey was carried out in the households and hospitals to assess the hazardous wastes from generation to disposal. A well formatted questionnaire was created and issued for each respondent whoever has the educational background and oral questions were asked and filled in the questionnaire for others. Also, wastes were measured in sample wards in the hospitals using a measuring balance in order to find out the waste amount generated in a day.
All these data were fed in to computer excel sheet and Stratified random sampling equations were applied to analyze the data to estimate the results with a confident interval for the whole central province.

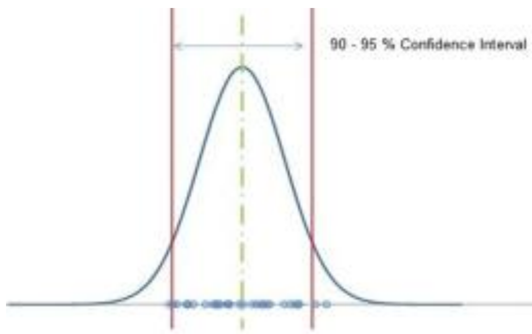

Figure 2: Distribution Curve

4. Results and Discussion

4.1 Household hazardous wastes

Table 2: Household hazardous waste amounts in central province

Hazardous waste type	Amount(Tons/Year)
E-wastes	4202
Chemical, corrosive & Toxic	653
Ignitable	646
Total	**5500**

These hazardous wastes were disposed and handled in the following methods:

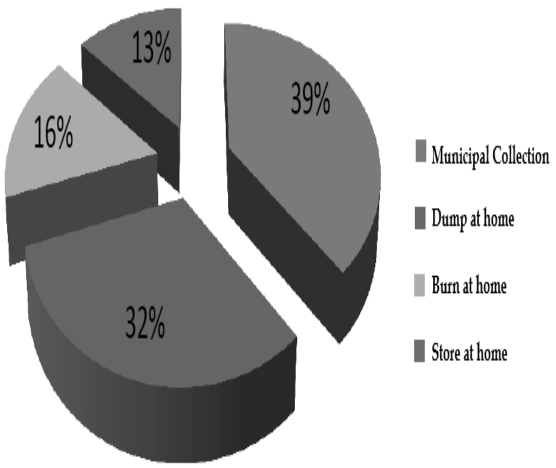

Figure 3: Current Disposal Practices

Also, assessment was carried out to find the following further aspects of the current practices and issues related with hazardous wastes among the inhabitants in the central province:

- Problems due to hazardous wastes
- Knowledge on hazardous waste
- Source of knowledge

4.2 Healthcare wastes

Assessment was carried out in the following places in the central province:

i. Matale District General Hospital
ii. Kandy General (Teaching) Hospital
iii. Nawalapitiya General Hospital
iv. Private clinic centres in Kandy

Following details were obtained from the assessment in the above hospitals and clinic centres:

i. Amount of healthcare wastes produced in each hospital per day.
ii. Current waste management systems for separate waste types.
iii. Knowledge level on hazardous wastes among labourers or waste handlers.

5. Conclusions

Human are the common producers of hazardous substances, who disposed them to the nature as wastes. These hazardous wastes will harm the human and the environment if they handled in an improper manner. According to the analysis, most of the hazardous wastes (especially household wastes) were handled in a haphazard way without a proper management system. But, most of the healthcare wastes were managed in an effective way in government hospitals where, self-management systems with treatment facilities were available and some handed over to the waste management agencies outside the hospital with a cost to treat infectious and pathological wastes. In case of private clinic centres, the waste management system needs to be improved as most of the healthcare wastes were sent with municipal solid wastes.

Also, it is important to make awareness among the public about the hazardous wastes, severe problems due to hazardous wastes and proper management system.

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Constructed Wetlands for Wastewater Treatment

S. Arani, N. Ponniah, M. Thurairajah, G.M.P.R. Weerakoon and K.B.S.N. Jinadasa

Keywords: Constructed wetlands, Nitrogen

1. Introduction

In Sri Lanka, natural streams are playing an important role in providing water for public usage including drinking, cooking, bathing and washing. But most of these natural streams are highly vulnerable for potential health hazardous. When people are densely populated around these streams, by improper excreta disposal mechanisms and bad personal habits, related with low income and less education levels of people, these streams get polluted.

This study was carried out to investigate the applicability of two types of constructed wetland systems namely horizontal subsurface flow (HSSF) and vertical subsurface flow (VSSF) wetland systems in purifying polluted stream water through a densely populated line of houses at Melfort estate, Pussellawa. The wetland units were 8m x 1m x 0.6m (Length x Width x Depth) in size and planted with emergent macrophyte *Typha Angustifolia* (cattail) as the wetland vegetation.

2. Methodology

The water of selected stream consists less amount of nitrogen compounds. To increase a significant amount of nitrogen concentrations, in order to check removal efficiency, cow dung was mixed with the stream water.

VSSF and HSSF systems are used for analysis where one day HRT and two day HRT were considered in different flow rates.

Wastewater was fed continuously to the models in a constant hydraulic loading rate for a period, and then changed into a new hydraulic loading rate. For HRT2 it was fed up to 8 weeks and for HRT1 it was fed from 9th week to 13th week. Samples were collected at the start of the hydraulic loading rate from influent and end of the hydraulic loading rate from effluent for each model periodically. Shoot height was measured every two weeks. According to the recorded data, results were analysed.

3. Results and discussions

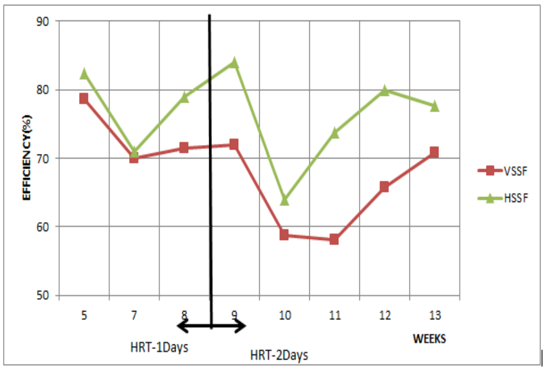

Figure 1: Removal efficiency of BOD_5

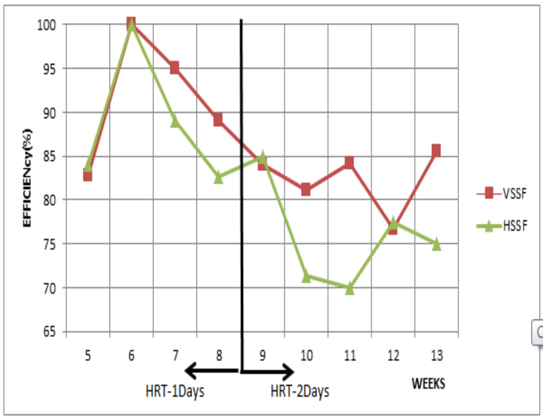

Figure 2: Removal efficiency of fecal coliform

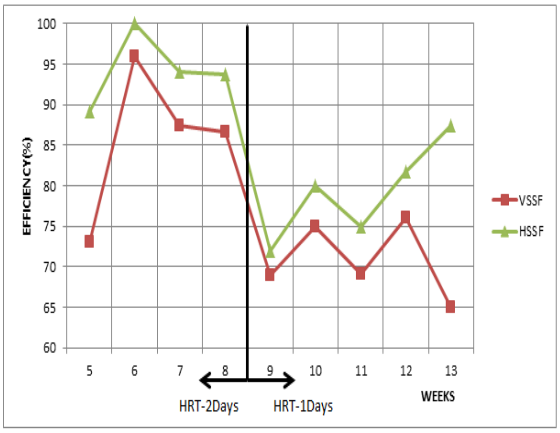

Figure 3: Removal efficiency of total coliform

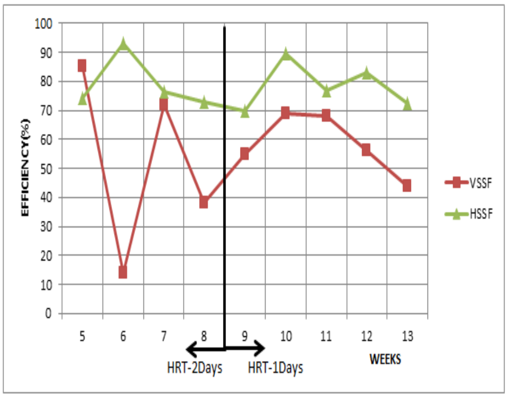

Figure 4: Removal efficiency of nitrate

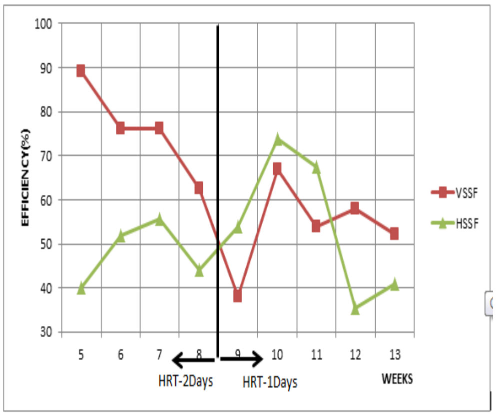

Figure 5: Removal efficiency of ammonium ion

In the above figures, the graphs illustrate the variations between the VSSF and HSSF on HRT1 and HRT2.

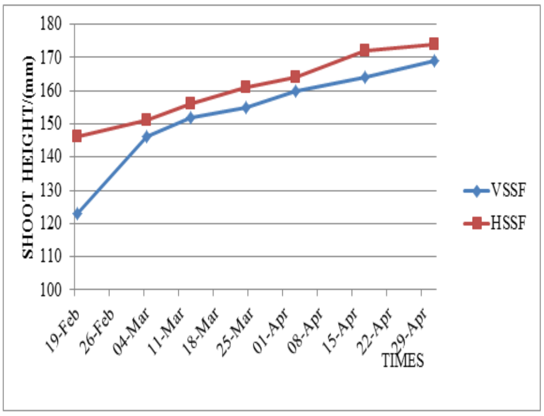

Figure 6: Variation of shoot height with time

From above figure, Horizontal flow wetland indicates higher growth than the vertical system.

5. Conclusions

From the results found, average efficiency of the treatment of water quality parameters in each HRT values were calculated. Using this it can be concluded that for all the analysis, removal efficiency of HRT 2 day is better than the removal efficiency of HRT 1 day. When comparing the VSSF and HSSF in HRT 2 day, higher removal efficiency of fecal coliform and ammonium ion was observed in VSSF. But higher removal efficiency of BOD5, total coliform, and nitrate was observed in HSSF.

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Leachate Treatment Using a Reactive Barrier

V. Kaandeepan, K. Nitharsan, P. Ravikumar, G.B.B. Herath and C.S. Kalpage

Keywords: Leachate, Reactive barrier, Clay-cement mixture, Slag

1. Introduction

Leachate, generated from landfills as a result of rainwater percolation and biodegradation of waste, causes various negative environmental and social impacts when it is released to groundwater and surface water as untreated. In order to reduce the risks, which associated, in both new and old landfills, Permeable reactive barrier, which is an in-situ treatment method is used to treat landfill leachate with many advantages compared to other treatment methods. This study is carried out in order to find out effective reactive material to treat landfill leachate on site using a reactive barrier. The material selection is based on availability, cost, stability and reactivity.

2. Literature review

2.1 Reactive media

Selected reactive materials for this study include, slag, clay cement mixture and clay.

Slag; slag is a glass/steel manufacture by-product and is a low cost material compared to other iron based materials such as zero-valent iron. The principal constituents of iron and steel slags are silica (SiO_2), alumina (Al_2O_3), calcia (CaO), and magnesia (MgO), which make up 95% of the composition. Minor elements include manganese, iron, and sulphur compounds, as well as trace amounts of several others. The physical characteristics, such as density, porosity, and particle size are affected by the cooling rates of the slag and its chemical composition. Depending on the cooling method, three types of BF slag are produced: air cooled, expanded, and granulated.

Batch tests from previous literature indicates that the slags exhibit varying degrees of reactivity ranging from nonreactive to reactivity comparable to that obtained with commercially available granular zero valent iron on a surface-area-normalized basis. (Cope DB, Benson CH, 2009).Greater reactivity was obtained with the slags having the highest iron content and the lowest reactivity was obtained with the slags having the lowest iron content suggesting that iron content is a primary factor for reactivity of slags.

Cement and clay mixture; in general, organic compounds have a strong effect on the microstructure of the cement paste. The structure and nature of the organic molecules are responsible for the microstructure characteristics. Almost all organic compounds are retarders in cement setting, and many organic acids that strongly chelate calcium also have strong retarding effects. Organic compounds retard the cement setting process by forming a protective layer around the cement grain, thus hindering the formation of calcium hydroxide, although a variety of fixation technologies have been developed. Many different additives and proprietary blends are being used to improve performance and reduce cost with specific waste streams.

It is stated that the reduction of soluble ammonia was normally enhanced by ionic attraction between ammonia ion and negative charge of clay. This represented in an decrease of ammonia as clay content increased. Cement as such and clay amended cement up to equal proportion performed no significant differences in removing organic matter. Cement to clay ratio of 1: 3 offered significantly the highest removal of ammonia and organic matter at the level of 70 %. The latter provided which is contained leachate of 20 mg NH4-N L-1, 340 mg BOD L-1 and 530 mg COD L-1 that could be treated sufficiently using the existing treatment system (Didik Sarudji ,2007).

3. Methodology

A physical model, which consists of four cylindrical columns with the dimensions of 5cm in diameter and 30 cm in height, was run using three reactive materials; Slag, cement and clay mixture and laterite. Leachate collected from Gohagoda landfill site was used to feed the above model. Sand was used as control material in the fourth column. The flow rate used is 1 ml/min to obtain feeding rate of 2.45 m^3/m^3.d and a 5 cm thick sand layer was used in each column on top and bottom of reactive material layers to achieve an evenly distributed flow. It was planned to treat organic matter, Nitrogen, Ammonia and heavy metals.

Leachate was allowed through the model and contaminant concentrations of treated leachate were measured for BOD, COD, heavy metals, phosphorous, nitrate and ammonia at inlet and outlet. Vertical flow was allowed initially at the rate of 1 ml/min.by changing the flow rate at a constant concentration removal efficiency of reactive material and the targeted contaminants was measured.

4. Results and discussions

The following graphs show (figures 1 and 2) the BOD_5 and COD removal efficiencies of the three materials used.

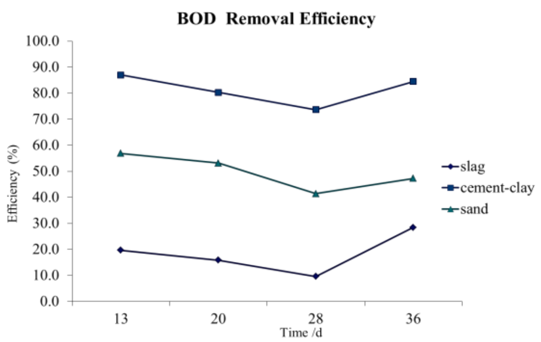

Figure 1: Comparison of BOD_5 removal efficiency

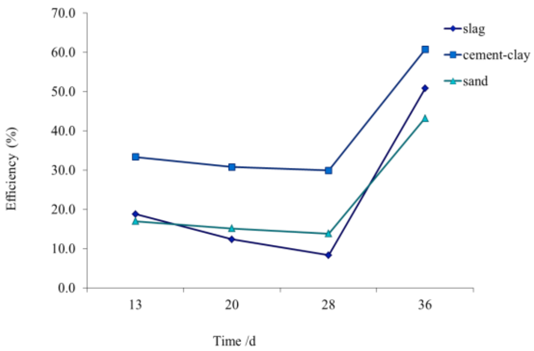

Figure 2: Comparison of COD removal efficiency

The clay-cement mixture shows the highest removal efficiency than other reactive materials in removal of BOD_5 and COD. It is believed to be due to the physic-chemical binding of the cement particles and the surface adsorption.

The concentrations obtained from different reactors for NO_3^- removal indicate that, reactor With the cement-clay mixture, is the most efficient with a maximum removal rate of 92%. This is followed by reactor containing slag with a maximum removal rate of 38 %. The removal rate in reactor containing sand is 20 %. Cement-clay mixture acts as a natural scavenger of pollutants by taking up cations and anions either through ion exchange, adsorption or both.

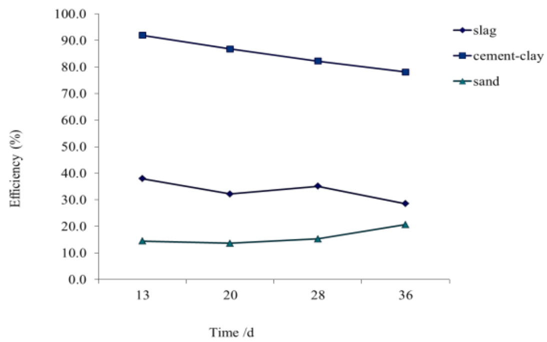

Figure 3: Comparison of NO_3^- removal efficiency

Pore space in clay soils is often filled with water. Water-filled pores of clay soils lack oxygen. Lacking oxygen, a group of soil facultative anaerobic bacteria, substitute nitrates for oxygen for respiration. When bacteria use nitrates as a substitute for oxygen, they convert nitrates to nitrogen gas through de-nitrification .Thus nitrate removal rate might be reduced due to bio film formation on the surface of cement-clay particles.

5. Conclusions

This study shows that cement-clay mixture is effective in removal of organic matter (BOD, COD) and Nitrates compared to other two materials such as slag and sand. Although Slag has higher removal efficiency in Phosphates and Phosphorous than other two materials such as cement-clay mixture and sand, it is less effective in removing phosphates and phosphorous.

The physiochemical binding process might be responsible for removing organic matter from the leachate in cement-clay mixture. It was suggested to apply more clay than cement to achieve high removal performance of nitrates and organic matter containing leachate.

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Solid Waste Management in Kandy Municipal Council

W.S.S. Weerasingha, M.N.N.M. Wijayarathna, D.S. Sampath, K.B.S.N. Jinadasa, C.S. Kalpage and A. Karunarathna

Keywords: Leachate, Bio brush, Dumping, Wetlands, Kandy municipal council

1. Introduction

Solid waste is a great problem not only in Kandy Municipal Council (KMC) but also in many countries in the world. It causes to variers problem such as social, environmental, health, economical and many more problems. So it is essential to think about solid waste management in Sri Lanka.

1.1 Objective of the Study

The objective of the study is to study the Solid Waste Management in KMC.

1.2 Scope of the study

The scope of the study is to investigate the current practices in Solid Waste Management in KMC, investigate the issues in current solid waste management system in KMC and to make recommendations to overcome these issues.

2. Literature Review

Kudaligama et.al. (2005) maintenance of an adequate amount of active biomass has been described as a key factor to a safe and stable operation. For retention of high concentration of active biomass, immobilization on inert support media has been reported to be very effective. For that, coir is the suitable raw material for producing such a media with excellent surface properties. A BOD removal efficiency of 89% percent can be achieved by using bio brush media with 200 m^2/m^3 SSA under 1.0 BOD $kg/m^3/d$ of Organic Loading Rate.

3. Methodology

Mainly solid waste management in KMC was studied under three main categories.

i. Collection
ii. Transportation
iii. Disposal

According to preliminary data collection, it was proved that transportation and collection were in a satisfactory level. But disposal section has considerable issues. The disposal was divided into four categories for easy inspection.

i. Generation
ii. Centralized processing for segregation
iii. Alternative disposal facility
iv. Environmental problems

Then the generation was divided into following four major categories and a questionnaire survey was conducted to assess the current practices, to get the people's opinion on waste segregation and to get the people's opinion on on-site composting.

i. Reduce
ii. Waste segregation
iii. Onsite-composting
iv. Polluter pay / Garbage tax

Then according to the results of the questionnaire survey, some recommendations were made to overcome the identified issues of solid waste management in KMC.

One of main environmental issues of Gohagoda open dumping site is the mixing of leachate with ground water and Mahaweli river water. As a solution for leachate issue, bio brush medium leachate treatment plant was designed by studying Nuwaraeliya leachate treatment plant.

4. Results and Discussions

4.1 Questionnaire Survey

- Current practices

According to questionnaire survey, 18% percent of people throw their waste into collection centres. Another 22% of them dispose their waste on-site (burying and burning) while the rest 60% percent of people hand over their waste to municipal council collectors.

- Waste Segregation

70% percent of people like to waste segregation while the rest of them do not like that. If the polluters pay method was introduced for those who did not like to segregate, 11% of them agreed to waste segregation.

- On-site composting

58% of people do not like to on-site composting because of lack of space and technical knowledge, sanitary problems and busyness while the rest of them like it. 56% of people from not like people like on-site composting if a composting bin is provided free of charge.

4.2 Design of leach ate treatment plant

Table 1: Results of the test samples of NuwaraEliya site

	Influent 1	Influent 2	Effluent 1
BOD_5/(mg/l)	49.9	50.8	18.5
PH	6.6	6.8	6.6
COD/(mg/l)	85	123	22
Nitrate Nitrogen/(mg/l)	290	140	65
Phosphorous /(mg/l)	20	50	10

According to above results, nitrogen and phosphorous are over range. This shows 63.6% efficiency of BOD removal via coir brush medium.

Table 2: Results of the test samples from Gohagoda site

	1	2	3	4	5	6
BOD_5 /(mg/l)	260	255	128	129	40	173
PH	7.53	7.97	7.28	8.02	7.65	7.52
COD/(mg /l)	650	399	233	352	105	240
Nitrate Nitrogen /(mg/l)	330	20	130	110	3	40
Phosphor ous /(mg/l)	70	50	60	120	0.6	10

Assuming 70% of BOD removal efficiency, a coir brush leachate treatment plant coupled with a constructed wetland was designed as a best low cost solution using existing tank system at Gohagoda dumpsite.

5. Conclusions

Using this proposed leachate treatment system, the acceptable levels can be achieved with respect to ambient water quality standards. But to achieve the proper performances, proper testing and maintenance procedures should be followed.

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GEOTECHNICAL ENGINEERING

Development of a Clay Liner Using Expansive Soil of Sri Lanka III

A.R.S.P.B. Athauda, D.A.H.J. Bandara, R.M.S.P. Rathnayaka and L.C. Kurukulasuriya

Keywords: Expansive soil, Rowe cell, Hydraulic conductivity

1. Introduction

In today's world, the solid waste material (refuse, cloths, garbage …etc.) is becoming the major problem on the environment and human health.

Landfills are used to overcome this huge problem throughout the world. Landfills are the containment system which has been constructed to minimize and control the impact of solid waste on the environment. But the main problem in the landfill is the Leachate. This leachate may affect directly or in-directly to human beings and to the environment.

To minimize the leachate coming out into contact with the soil, the Liners are used in as a barrier. The Geo membranes are very expensive. So those are not suitable for developing countries like Sri Lanka.

Thus, the suitability of an Expansive soil which is found from arid and semi-arid areas in Sri Lanka is going to be checked for Clay liner in this project.

2. Literature Review

By using Rowe Cell apparatus "Development of clay liner by using expansive soil II" Final year project, Faculty of Engineering, University of Peradeniya, 2011, had done a research about clay liners.

They had followed the same method that we will follow, but they have used waste engine oil to improve the properties of expansive soil.
Finally they found that,

- Hydraulic conductivity of soil with water was less than 10^{-7} cm/s in all the load cases and soil with engine oil was less than 10^{-8} cm/s in all the load cases.

Another research was done under expansive soil "Foundation in Expansive Soil", Headquarters, Department of Army, USA, 1983 September.

They found following results for Expansive soil,

Table 1: Tests results of research

Classification of potential swell	Potential swell (%)	Liquid limit (%)	Plasticity index (%)
Low	< 0.5	< 50	< 25
Marginal	0.5 – 1.5	50 – 60	25 – 35
High	> 1.5	> 60	> 35

Jayasekera (2006) has submitted a paper about a research "Long term effects of landfill leachate on volume change and hydraulic conductivity properties of expansive clays "
The research was done by using

- Soil
- Soil + 10% of Bentonite
- Soil + 20% of Bentonite

Finally it was found that, the original swell behaviour and hydraulic conductivity properties of the experimental clay soils are greatly influenced by the expansive clay content of the soil. The higher the expansive clay fraction, the higher the swelling potential and as a result, lower the hydraulic conductivity

3. Materials and Methodology

Firstly some Laboratory tests were done to characterize the expansive soil found from Moragahakanda area.

Determination of Expansive potential (Swelling pressure test), Determination of particle size distribution (Sieve analysis test), Determination of Atterberg Limit (Liquid Limit and Plastic Limit test).

Then the preparation of the clay liner in the Rowe cell apparatus was done using the collected expansive soil and the liner by mixing expansive soils with 5% of Bentonite (by weight).

Determination of hydraulic conductivity on liner consolidated under

- 50 kPa, 100 kPa and 200 kPa.

4. Results and Discussion

4.1 Results for Soil classification tests

Table 2: Test results for classification of expansive soil

Parameter	Limitation	Sample
Plastic Limit / (%)	< 25	19
Liquid Limit / (%)	40 - 60	44
Particle size distribution / (%)	> 35	60
Potential swell/(%)	> 1.5	4.4

The Plasticity Index = LL- PL
= 25.8 %

So the classification of this soil is Inorganic Clays of intermediate Plasticity

All requirements to be an expansive soil were satisfied for this Moragahakanda sample. So that soil was selected for our future work.

4.2 Results for Rowe cell apparatus

Table 3: Expansive soil mixed with water

Consolidation pressure /(kPa)	Thickness of sample /(mm)	Coefficient of Permeability - K / (10^{-10} × m/s)
50	35.56	6.5
100	32.77	1.5
200	30.22	0.88

Table 4: Expansive soil mixed with 5% of Bentonite and water

Consolidation pressure /(kPa)	Thickness of sample /(mm)	Coefficient of Permeability- K / (10^{-10} × m/s)
50	31.95	1.9
100	28.06	0.7
200	26.33	0.5

5. Conclusions

According to the 'Environmental Guidance Document of Nebraska Department of Environmental Quality' says that Permeability should be less than 1 x 10^{-7} cm/sec for the better clay liners. All hydraulic conductivity results which were obtained were less than 10^{-7} cm/s. It is satisfied with the liner requirement.

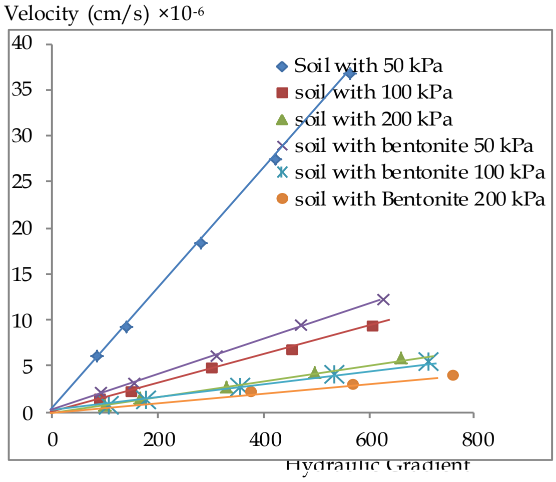

Figure 1: Results for Rowe cell apparatus

From this we can identify that the soil mixed with 5% of Bentonite has less Hydraulic conductivity than in-situ soil. Bentonite has very fine particles. So the void ratio of mixed sample is less than in-situ soil sample. Then the permeability is low. Also its Liquid Limit is very high. Therefore water absorption is high. As the result of these reasons Hydraulic conductivity may vary.

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Improvement of Durability of Coir for Its Application as Geo-Reinforcement-III

Ganeswaran U, Jeyaruban V, Yogarajah K and L.C. Kurukulasuriya

Keywords: Polymer, Polythene, Coir fibre, Tensile test

1. Introduction

Coconut fibre is extracted from the outer shell of a coconut. The common name, scientific name and plant family of coconut fibre is Coir, Cocos nucifera and Arecaceae (Palm), respectively. There are two types of coconut fibres, brown fibre extracted from matured coconuts and white fibres extracted from immature coconuts. Brown fibres are thick, strong and have high abrasion resistance. White fibres are smoother and finer, but also weaker. In engineering, brown fibres are mostly used. The use of coir fibres as a reinforcing material in mechanically stabilized wall requires the coir fibre to be non-degradable. Therefore, in this project the improvement of durability of coir fibre technique is investigated. A waste polymer (Low Density Poly Ethylene) material is used as the coating material and two different diameters uncoated and coated fibre composite specimens are prepared and its performance is investigated in the laboratory through tensile testing with varying period of exposure in acidic environment.

2. Literature review

2.1 Coir

Coir fiber made up of small threads, each about 1mm long and 10 to 20 micrometer in diameter, the individual fiber cells are narrow and hollow, with the wall made of cellulose. Coir consists of cellulosic fibers with hemicelluloses and lignin as the bonding materials for the fibers. Coir fiber has low cellulose and hemicelluloses, high lignin content and high micro fibrillar angle compared with other natural fiber. As a result, tensile strength and young's modulus of coir fiber are lower than the other plant fiber and high lignin content in coir fiber is responsible for the useful properties such as weather, fungal and bacterial resistance.

2.2 Mechanical Properties of coir

The structure, chemical composition, microfibrillar angle and cell dimension are the most important variable that affect the overall properties of fibre. According to previous research, density, tensile strength and Young's modulus are 870 Kg/m^3, 158 MPa and 3.7 ± 0.6 GPa respectively.

2.3 Chemical properties of coir

Coconut fibres contain cellulose, hemi-cellulose and lignin as major composition. These compositions affect the different properties of coconut fibres. The pre-treatment of fibres changes the composition and ultimately changes not only its properties but also the properties of composites. Sometimes it improves the behaviour of fibres but sometimes its effect is not favourable.

2.4 Thermal Properties of coir

The thermal properties of fibre were evaluated by using untreated and treated coir. Thermo gravimetric analysis (TGA) was performed under nitrogen atmosphere. Thermal degradation of hemicelluloses, cellulose and Lignin are occurring in 190^{o} C, $290\ ^{o}$ C to 360^{o} C, and 280 to 500 ^{o} C, respectively.

3. Methodology

3.1 sample preparation

- Two coir ropes of 13 mm and 18 mm were selected.
- Waste polythene is used as a coating material.
- Loose parts of the small fibers were removed from the main Coir ropes.
- Waste polythene was wrapped 30 times in middle of coir ropes. The wrapped length was 350mm. Expected coating thickness is 1mm.
- Uncoated sample was prepared.
- Wrapped ploythene samples were heated up to 160^{0}c for 10 minutes in the oven.
- Coated and uncoated samples were weighted.
- 1 M HCl solution was prepared.
- 150 mm length of the uncoated and coated samples was immersed in to the 1M HCl solution.

3.2 Experimental procedure

After two week, Coated and uncoated samples were taken out from HCl solution and was kept one week for air drying. Then weights of samples were taken. Finally all samples were subjected to tensile test and mass reduction analysis according to BS 6906: Part 1:1987.

4. Result and discussion

4.1 Result of tensile test

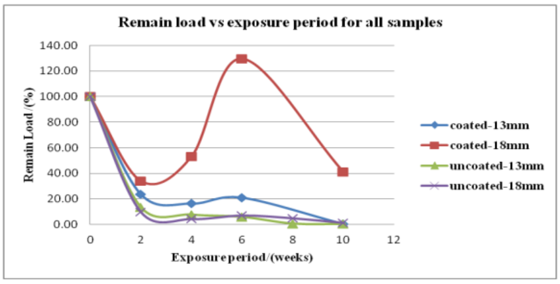

Figure 1: Variation of breaking load with Period of Exposure for all Samples with respect to initial load

4.2 Result of mass reduction

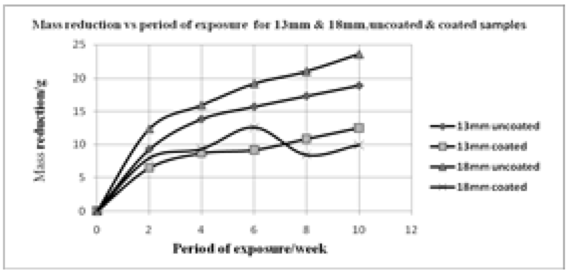

Figure 2: Variation of mass reduction with period of exposure for all samples.

Table 3: Breaking load with exposure period for full coated samples.

Exposure Period /(weeks)	Load at failure for full coated sample/(N)		
	13mm	18mm	
	60 times wrapped	30 times wrapped	60 times wrapped
2	-	3495.71	3454.57
4	1315.05	4841.02	2064.45

During the tensile test, the break was occurred at uncoated part of the coated samples and 6th & 8th week coated samples breaking load were very less than that of uncoated samples. So, 2mm thickness of full polythene coated samples prepared according to methodology and subjected to tensile test. The table shows the test results.

4.3 Discussion

- In analysis, 0th week sample failure load value is taken as 100%.then other load value are convert into percentage.
- Properties of the coconut husk and Yarning procedure is not same for particular diameter of the coir ropes. So there is a different in breaking load.
- 1 M HCl solution caused to develop holes in the coated samples. The penetration rate was varying with amount and sizes of the hole. These may be the reason to varying coated samples breaking load.
- Uncoated part of the rope became weaker compared to the coated part in the coated samples, because evaporation of HCl along the uncoated part. These may be the reason for uncoated part breaking, doing the tensile test.
- Full length coated coir rope and 2 mm polythene coating thickness samples are giving greater strength compared to 350 mm coated samples testing strength.

5. Conclusions

- The breaking load of the uncoated samples decrease with increase of exposure period and breaking load of the uncoated samples is less than that of coated samples.
- An increase in coating thickness gives more durability.
- Mass reduction of uncoated samples is high compared with the coated samples.
- Mass reduction is increasing with increase of exposure period for both 13 mm and 18 mm uncoated samples.
- Coated coir composite has the potential to be used as geo-reinforcement for mechanically stabilized wall.

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Durability Improved Geomat Reinforced Vertical Embankment Behaviour

B. Jayaprakas, K. Mahotharan, V. Manoj and L.C. Kurukulasuriya

Keywords: MSE wall, Geo-reinforcement, Coated geomats, Vertical embankment, Lateral deformation, Model test

1. Introduction

Stability of vertical embankment is very important in Civil Engineering applications. Nowadays, instead of using concrete or random rubble masonry walls, using reinforced earth embankments are very popular. But in such concept, it is essential that the lateral displacements of reinforced earth embankments are kept within a reasonable limit to satisfy the serviceability limit state. The suitability of the durability improved coir geomat made by coating natural coir with wasted plastic material as a reinforcement material, satisfying lateral displacements will be determined by laboratory model tests.

This investigation is to check the suitability of coated coir geomat as a reinforcement considering lateral displacements.

2. Literature review

When reinforcements are placed at a designed vertical spacing, each reinforcement will carry the force transferred by the soil within its tributary area which can be calculated using Rankine's active earth pressure theory. It is also possible to use coir geomats as the reinforcing material as a cost effective solution to stabilise the vertical face of an embankment making use of its engineering properties (Kurukulasuriya et. al., 2011). This is in addition to carrying out thorough subsurface investigation to determine strength properties required for settlement evaluation and stability analyses. While taking effective measures to ensure that the loss of tensile strength of reinforcement during the design life of the reinforced wall is kept within acceptable limits, it would be required to assess the performance of the wall with regard to its aesthetic appearance that could be affected due to excessive lateral deformation. Therefore, it is imperative that the designer limits the lateral deformation of reinforced earth walls under service loads. Chai et al, (2002) had conducted numerical analyses using finite element models and concluded that only when the embankment approaches to failure, the reinforcement has noticeable effect on lateral displacement of the soil.

From previous project results, coir geomat can be used as soil reinforcement material. But, its durability should be improved as nearly 80% of the strength of coir geomat is reduced after one year period (Er. Sheela Mary Cherian). Therefore it cannot be used in long run projects.

3. Methods and materials

For this experimental study the same soil type used by Kurukulasuriya et.al. (2011) was used which makes it possible to carryout comparative studies. The basic soil characteristics are given in Table 1.

Table1: Wall fill soil data

Percentage coarse particles	70
Maximum dry density	17.0 kN/m^3
Optimum moisture content	16.0 %
Effective cohesion	5 kPa
Effective angle of internal friction	31°

In order to improve the durability of geomats a coating of waste plastic was carried out. For this purpose, the ropes in the same type of geomat that was used by Kurukulasuriya. et.al.. 2011, were separated and 1 m of the rope was wrapped using fifteen pieces of 50 mm X 600 mm waste plastic bag material. Then, geomats were re-woven using the polythene wrapped coir ropes, and kept in the oven, at a temperature of 130^{0}C for 5 minutes. The geomats that were used had a rib spacing of 25 mm and 17 mm along the machine and transverse directions respectively and a density of 650 g/m^2.

In order to design the geomat reinforced vertical wall, wide-width tensile tests were carried out on polythene coated geomats to evaluate the ultimate wide-width tensile strength in accordance with BS 6906: Part 1:1987. A comparison of the wide-width tensile test results on coated coir geomat and uncoated coir geomat reinforcement is given in Table 2. The test vertical wall was constructed to have dimensions of 700 mm length, 605 mm width and 500 mm height. Coated coir geomat reinforcement was placed well stretched at a vertical spacing of 100 mm to ensure that the force developed under the surcharge pressure of 250 kPa is below the maximum load obtained along the machine direction.

Table 2: Properties of coated and uncoated geomats

	Coated Coir Geomat (kN/m)	UnCoirGeomat (kN/m)
Average Maximum load	9.70	8.25
Average Breaking load	9.70	7.40
Average Elastic limit	6.70	5.05

The soil was compacted to achieve a density of 95% of standard Proctor density at the corresponding water content of the dry side of the compaction curve. A geotextile was used to cover the front end and anchored into the upper soil layer.

A rigid steel plate was fabricated to apply the uniform surcharge pressure to the vertical wall. Dial gauges were setup along the central axis of the wall at mid-height of each of the 5 soil layers to observe the lateral deformation of each soil layer.

Then the plate was gradually loaded up to 5 tons which produced a contact surcharge pressure of 123 kPa. The pressure applied was unloaded and the lateral deformation of the wall as well as the vertical movement of the plate was monitored throughout the loading and unloading sequences. In the next stage of the experiment, a wet condition was simulated by allowing the wall fill to soak for more than 48 hours while ensuring that no surface soil eroded away. The loading was again carried out up to 10 tons and then it was slowly unloaded and the lateral deformations of the frontal central axis were monitored.

4. Results

Dimensionless analysis was carried out to compare the behaviour of the coated geomat with results that had been obtained using uncoated geomats. Since the maximum displacement was found in the top layer, the analysis was done for the top layer only.

Figure 1 gives the direct comparison of the lateral deflection ratio and the surcharge pressure ratio for the two identical walls reinforced with either coir geomats or coated coir geomats. It is evident from figure 1 that, under natural moisture conditions and soaked conditions, coated geomat reinforced wall underwent nearly the same lateral deflection corresponding to a particular applied load ratio.

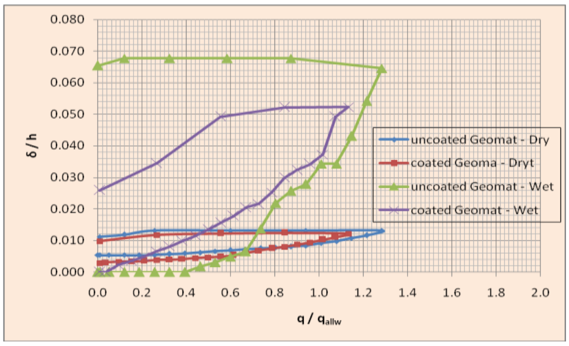

Figure 1: Dimensionless analysis for coated and uncoated geomats for the top layer

Therefore, under natural moisture conditions and soaked conditions, the coated coir geomat reinforced wall had performed in a similar manner to that of uncoated geomats.

5. Conclusions

Based on the laboratory experimental model study on the lateral deformation of coated coir geomat reinforced vertical embankment under natural moisture content and soaked conditions, the following conclusion can be made.

Irrespective of whether the loading is applied under natural moisture content of the fill or soaked conditions, the durability improved coated coir geomat reinforced vertical wall showed similar lateral deformation to that of uncoated coir geomat reinforced wall, at the same fraction of the allowable surcharge pressure corresponding to each material. Therefore, it can be concluded that the improvement of the durability by coating a waste polymer material will not affect the lateral deformation characteristics of the coir mat reinforced wall.

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Variation of Infiltration Rate with Land Use

K. Sulakxan, S. Lavaniyan, C.A. Samsen and H.K. Nandalal

Keywords: Infiltration rate, Land use

1. Introduction

Infiltration is a process of water penetrating from ground into soil. Generally infiltration is measured by infiltration rate which water enters the soil at the surface. Infiltration rate measured in/hr or cm/hr units. This infiltration rate reduces with time and reaches constant steady value.

Infiltration capacity is the maximum rate at which water can enter the specific soil. Different soil type has different infiltration capacity. High infiltration capacity will reduce the surface runoff.

Infiltration is related to initial moisture content, degree of compaction, hydraulic conductivity of the soil profile, texture, porosity, organic matter and vegetation cover. However, main factor affecting this infiltration rate is soil texture, porosity, degree of compaction and initial moisture content.

There are several ways of measuring infiltration which vary in their accuracy. Double ring infiltrometer is better than single ring to reduce the lateral movement of water.

Water inside the infiltration ring often flows horizontally through the soil as well as vertically. Double ring infiltrometer overcome this up to greater extent as shown in figure 1.

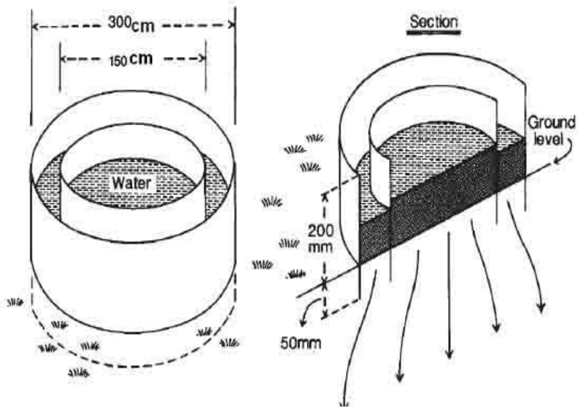

Figure 1: Set-up for the double-ring infiltrometer (adopted from Diamond and Shanley, 2003)

2. Literature review

2.1 Makungo and Odiyo (2011)

a).They selected twelve different type landsides and they classified according to the soil type.

b).They used inner diameter 32cm and outer diameter 52 cm double ring infiltrometer to measure the infiltration rate.

c).Their sites identified sandy loam, sandy clay and loamy clay from soil type maps.

d).They plot the graph between infiltration and time for each soil type and finalize the result.

2.2 Osuji et al (2010)

After studying the infiltration rates at different sites they arrived at following conclusion

a).Sandy loam have high infiltration rate.
b).Bush fallow encourages the high infiltration rates.
c).Crop land encourages the high runoff.

3. Methodology

Three different land use types were selected. Namely forest type, shrub type and turf type. Soil sample were taken to measure the soil texture, moisture content and level of compaction. The sand cone test was carried-out at selected sites and find out the dry density of soil at the site.

The infiltration rate was measured using double ring infilrometer at the selected site. The initial moisture content was measured according to BS 1377- Part 4. The sieve analysis test was done according to BS 1377 from that soil type was found. The Proctor compaction test was carried-out according to BS 1377 and the degree of compaction was found from sand cone test and Proctor compaction test. In each land use type, all those tests were done.

4. Results and discussion

The current study determined the steady infiltration rate in different land use type in selected areas in university of Peradeniya, Sri Lanka. Turf area, forest area, shrub area were identified and steady infiltration rates were completed. Totally nine experiments were conducted. Variations happened because of various factors. In this study three factors were measured and compared with the steady infiltration rate. Following tables show the comparison for each site.

Table 1: Turf area final results

Site	Steady infiltration rate (cm/hr)	moisture content (%)	Soil type	Level of compaction (%)
1	26.4	33.5	Sandy clay loam	68.5
2	12.0	19.8	sandy clay loam	90
3	14.4	22.5	Sandy clay loam	80

Table 2: Forest area final result

Site	Steady infiltration rate(cm/hr)	moisture content (%)	Soil type	Level of compaction (%)
1	48	16.23	Sandy clay loam	63.3
2	27.6	25.5	Sandy clay loam	96.4
3	31.2	20	Sandy clay loam	79.89

Table 3: Shrub area final results

Site	Steady infiltration rate(cm/hr)	Initial moisture content (%)	Soil type	Level of compaction (%)
1	30	11.4	Sandy clay loam	79.5
2	32.4	8.78	Sandy clay loam	77.8
3	34.8	8.23	Sandy clay loam	75.0

5. Conclusion

The current study determined the steady infiltration rate range in selected areas in University of Peradeniya, Sri Lanka. Each land use type infiltration rate ranges are given in table 4.

Table 4: Steady infiltration rate range

Land use pattern	Steady infiltration rate (cm/hr)
Turf area	14-27
Forest area	31-48
Shrub area	30-35

Soil type of all these places are sandy clay loam. Level of compaction has very much reduced the infiltration rate. Initial moisture content had less influence in the Steady infiltration rate. Initial Moisture content only change the time which take to get steady infiltration rate.

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Experimental Investigation of Bearing Capacity of Sand Reinforced with Randomly Distributed Plastic Strips

K.M.K.G. Karunarathna, M.I.D. Muthukumarana., P.K.M. Perera and D. de S. Udakara

Keywords: Aspect ratio, Bearing capacity, Plastic strips, Sand, Strip content

1. Introduction

This project is an investigation of an improvement of the bearing capacity of sand reinforced with randomly distributed plastic strips. Waste plastic bottles are used as the reinforced material by preparing them into small strips. Series of laboratory model tests are carried out by varying aspect ratios and strip contents of plastic strips. A loading machine and a test chamber are used for this study. From results of the experiments, the effects of strip content and strip aspect ratio on the strength of reinforced sand are determined.

2. Literature Review

Since waste plastic is a huge problem all over the world, they had been used as a ground improvement material mixing with sand to improve the bearing capacity of sand in some researches. Waste plastic exists in different aspects. Dutta and Venkatappa (2007) have used HDPE and LDPE strips as a reinforced material with sand. Shivakumar Babu and Chousksey (2011) have used plastic strips which were prepared by using plastic bottles as a reinforced material with sand.

The variables that were used for most researches are aspect ratio and strip content. Dutta and Venkatappa (2007) has used 12x12mm (AR=1) and 12x24mm (AR=2) strips with 0.05, 0.1, 0.15% of strip contents for LDPE and 0.25, 0.5, 1, 2% of strip contents for HDPE. Chaudhary (2010) has used 12x12mm (AR=1), 12x24mm (AR=2) and 12x36 mm (AR=3) strips with 0.25, 0.5,1 and 2% of strip contents. Sivakumar Babu and Chousksey (2011) have used 12x4mm (AR=3) with 0.5, 0.75 and 1% of strip contents.

3. Material and Methods

3.1 Sand

Since river sand is the most common type of sand used in construction industry, river sand was used for the study. Series of laboratory tests were carried out on chosen sand in order to identify its properties.

3.2 Plastic strips

Plastic strips were prepared by using waste plastic bottles. Laboratory model tests were done for two aspect ratios with two strip contents.

Table 1: Details of plastic strips

Aspect ratio	Size of strips	Strip content (With respect to weight of sand)
1	12 mm x 12 mm	0.25% & 1%
2	12 mm x 24 mm	0.25% & 1%

3.3 Model preparation

A test chamber was used for testing and loading was done through a foundation model.. In order to create plane strain conditions within the test arrangement, a strip footing was used. The length of the footing (90 mm) is made almost equal to the width of the chamber (95 mm) and 2.5 mm wide gap was kept at both ends to prevent contact between the footing and the side walls. Width of the footing was made as 35mm. Compression testing machine was used to load the foundation model.

First, sand was mixed with plastic strips and filled into the chamber while maintaining 75% of relative density of sand. Then, foundation model was fixed at the centre of chamber. Two dial gauges were connected to measure displacements of foundation model and sand bed.

3.3 Testing procedure

Load was applied with 5 lb (22.25 N) increments and each increment was kept until the settlement was stabilized. Then the displacements were measured. The procedure was repeated until the failure occurred.

4. Results and Discussion

4.1 Properties of sand

- Specific gravity = 2.66
- Effective size of sand = 0.33 mm
- Coefficient of uniformity= 3.4
- Coefficient of curvature = 1.3

According to the BSCS, the soil type is well graded sand.

4.2 Results of model tests

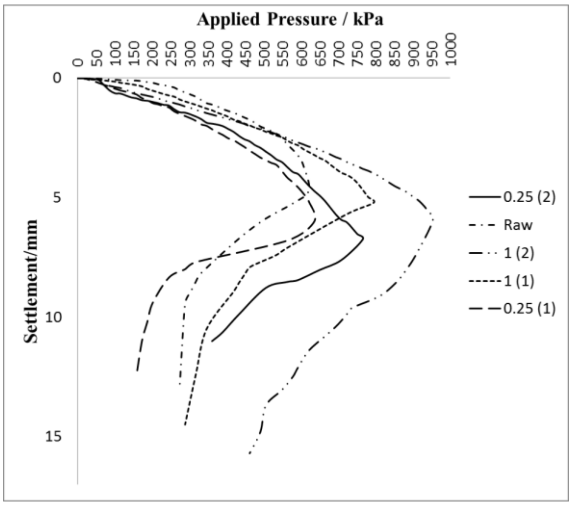

Figure 1: Variation of settlement with applied pressure

Shape of the all above graphs are almost same. First,settlement is increased with increasing of pressure. Pressure is increased upto a certain point and then begun to decreased. But, settlement is further increased. Increasing of settlement after the maximum pressure is higher than before the maximum pressure.

According to this variation of settlement with pressure, the failure mode can be identified as a general shear failure.

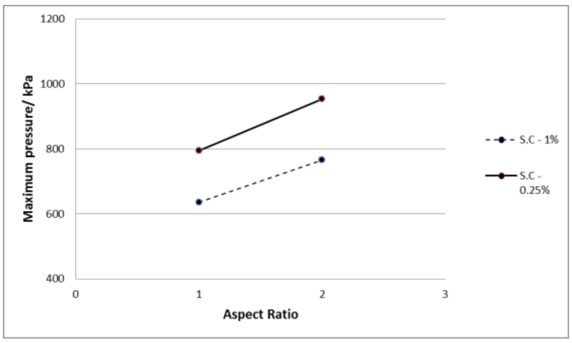

Figure 2: Variation of maximum pressure with aspect ratio for each strip content

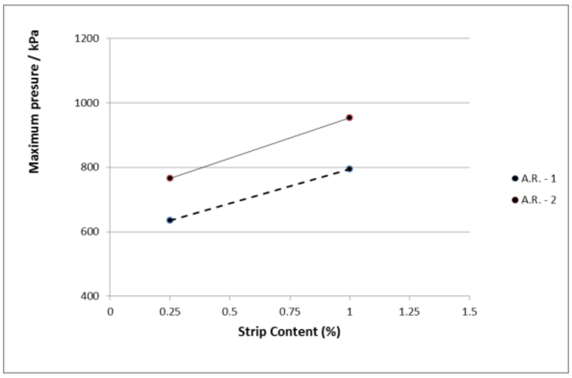

Figure 3: Variation of maximum pressure with strip content for each aspect ratio

5. Conclusions

- Bearing capacity of sand is increased by reinforcing randomly distributed plastic strips.
- Bearing capacity is increased with aspect ratio of plastic strips.
- Bearing capacity is increased with increasing the strip content.

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Experimental Investigation of the Effect of Lime and Sand on the Swelling Potential and Plasticity of Expansive Soil

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Keywords: Swelling pressure, Swelling index, Expansive soil

1. Introduction

Expansive soils are those which show volumetric changes in response to changes in their moisture content. Such soils swell when the moisture content is increased and shrink when the moisture content is decreased. Most of the problems occur in arid, semi-arid and monsoonal areas causing wide fluctuation in the soil moisture content. Lime and sand stabilization is used in this study to treat expansive soil. Cost of lime and sand stabilization is low in compared with other treatment methods. The addition of lime to soils to improve their use for construction purposes has a very long history.

2. Literature review

2.1 Expansive soil

Expansive soils contain one or more clay minerals. They are montmorillonite, kaolinite and illite. Montmorillonite clay exhibits high swelling than kaolinite. Potentially expansive soils can typically be recognized in the laboratory by their plasticity properties. Expansive soils are found in the arid and semi-arid regions in Sri Lanka. Herath (1993) reported Sri Lanka has been divided into three clay mineral zones based on composition of clay minerals namely; wet zone clay mineral province, dry zone clay mineral province and intermediate zone clay mineral province. The dry zone clay minerals mainly consist of Kaolinite – Montmorillonite with calcareous material while intermediate zone clay minerals consists of Kaolinite clays with a low proportion of gibbsite and montmorillonite. No montmorillonite was found in the wet zone clay minerals.

2.2 Treatment of expansive soil with Additives

Lime is widely used in civil engineering applications such as road construction, embankments, foundation slabs and piles. When lime is added to clay soils in the presence of water, a number of reactions occur leading to the improvement of soil properties. These reactions include cation exchange, flocculation, carbonation and pozzolanic reaction. Mhaidib (1997) found that the addition of lime to the tested expansive soil proved to be effective up to 2% after which it gave a negative effect on the swell characteristics. Amer Ali (2005) indicated thatlime treated samples were observed that the maximum reduction in swell pressure and swell percent was achieved using 6%. Hoda (2007) found the swelling potential for sand-lime cushion is minimum at 5% lime content for oedometer tests, but for laboratory model tests is at 20% lime content.Bel (1996) found the cation exchange capacity of montmorillonite was reduced when lime was added but at 6% addition, by weight, it began to increase. On the other hand, the cation exchange capacity of kaolinite rose initially with 2% addition of lime, then declined and remained more or less steady with more additions of lime.

3. Methodology

The soil used in this study was obtained from Moragahakanda (near Dambulla, coordinates 07^0 35′31.8″N, 80^0 50′ 0.5″E).The percentage additives are 1.0, 2.0, 4.0 and 8.0 lime alone, 5, 10, 15 and 20% sand alone and their combinations. The Atterberg limit and Swelling test has been carried out According to BS 1377 -2-1990 and BS 1377-5-1990, respectively. Thereafter, the variation of plasticity index and swelling index with the addition of different percentages of lime, sand and their combination were analysed. Finally the optimum percentage of lime and sand to be added to the soil was found out.

4. Results and Discussions

Table 4.1 shows the result of liquid limit (LL), plastic limit (PL) and plasticity index (PI).The Atterberg limits indicate the LL, PL and PI increase with the added lime percentage up to lime content of 2%.

Table 4.1: Atterberg Limit Test results

Type of Soil	Treatment Materials Lime %	Treatment Materials Sand %	Liquid limit/ (%)	Plastic Limit/ (%)	Plastic Index/ (%)
Untreated	0	0	42	16	26
Lime Treated	1	0	45	17	28
	2	0	46	18	28
	4	0	40	18	22
	8	0	42	18	24
Sand Treated	0	5	42	19	23
	0	10	39	18	21
	0	15	34	15	19
	0	20	37	16	21
Both Lime and Sand Treated	1	5	42	18	24
	2	10	40	16	24
	4	15	38	17	21
	8	20	39	17	22

Thereafter, Both LL and PI decrease with increasing lime percentage up to lime content of 4%, after that increase slightly with increasing lime content. On the other hand, the PL decreases slightly with increasing lime content. The LL and PI decrease with the added sand and mixture of sand and lime percentage up to sand content of 15% and combination 3 (4%lime + 15%sand). Thereafter, Both LL and PI increase with increasing sand and mixture of sand and lime percentage up to sand content of 20% and combination 4 (8%lime + 20%sand). But PL is slightly fluctuated with sand and mixture of sand and lime percentage as shown in table 4.1.

Table 4.2 shows the results of swelling pressure and swelling index.

Table 4.2: Swelling pressure test results

Type of Soil	Treatment Materials Lime %	Treatment Materials Sand %	Swelling Pressure/ (KPa)	Swelling Index
Untreated	0	0	132.2	5
Lime Treated	1	0	105.8	3.8
	2	0	98.1	4.5
	4	0	125.2	5.3
	8	0	132.2	5.8
Sand Treated	0	5	101.3	4.4
	0	10	94.8	4.1
	0	15	67.7	3.2
	0	20	76.1	3.6
Both Lime and Sand Treated	1	5	81.3	4.2
	2	10	90.3	4.7
	4	15	66.4	3.9
	8	20	67.7	4

The Swelling pressure is reduced to 98 kPa, 68 kPa and 66 kPa from 132 kPa (Raw sample) at added lime 1.6%, sand 15% and combination 3 (4%lime + 15%sand) respectively. Swelling index is reduced to 3.8, 3.2 and 3.8 from 5 (Raw sample) at added lime 1%, sand 16% and combination 3 (4%lime +15%sand) respectively, as shown in table 4.2.

5. Conclusions

This study evaluated the effect of lime, sand and combinations of lime and sand, on the swelling potential of expansive soil.

- .Swelling pressure is reduced by 26%, 48% and 50% at added lime 1.6%, sand 15% and combination 3 (4%lime +15%sand) respectively.
- Swelling index is reduced by 24%, 36%and 24% at added lime 1%, sand 16% and combination 3 (4%lime +15%sand) respectively.

It can be concluded that the sand treatment is better than lime and mixture of sand and lime treatment for Moragahakanda expansive soil from our research. The addition of sand to the tested expansive soil proved to be effective up to 15% after which it gave a negative effect on the swell characteristics.

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Study the Effect of Waste Rubber Materials on Shear Strength of Residual Soils

M.G.B.T. Balasooriya, M.A.S.N. Mallawarachchi, W.G.S.D. Kumari and D. de.S. Udakara

Keywords: Shear strength, Rubber content, Direct shear

1. Introduction

This study focuses on the utilization of the waste rubber material as reinforcement to increase the shear strength of residual soils. A series of laboratory direct shear tests are carried out on soil reinforced with randomly distributed rubber strips to determine its effect on the shear strength of residual soils. From results of the experiments, the optimum rubber content for the maximum shear strength is determined.

2. Literature Review

One of the wastes generated in engineering and transportation sector is scrap tire and it causes serious environmental problem. Some researchers showed that strength and compressibility of shredded tire can be used for the engineering requirements.

Martin Christ et al. evaluated the strength properties of rubber sand mixtures at various rubber contents as 0%, 10%, 15%, 20% & 30%. They found the Shear strength decreased at optimum rubber content is 15%.

Suat Akbulut et al. evaluated the effect of the waste rubber on strength parameters of clayey soils. The contents of scrap fiber rubber has been used is 1%, 2%, 3%, 4%, 5% by total weight of reinforced samples. They determined the shear strength increased up to 2% rubber and decreased after 2% rubber content.

Mozhi et al. evaluated the performance of soil and rubber waste mix as sub grade material. Natural soil and 30 mm to 45 mm rubber strips with 7 mm thickness were used as materials. They observed shear Strength increased with rubber content.

3. Material and Methods

3.1 Soil

The soil was collected from the university premises and Series of laboratory tests were carried out to identify its properties.

3.2 Rubber Strips

Rubber strips were obtained from scraped tire tubes and strip sizes were 20 mm × 5 mm.

3.3 Testing procedure

The entire laboratory tests are carried out according to BS 1377. The soil sample was prepared in a CBR mould using 6 kg of soil and rubber strips which were in size of 20 mm × 5mm randomly distributed with soil. The compaction was done using the Standard Proctor Compaction method by adding the amount of water which requires for 95% $\rho_{dmax.}$ Increase the rubber content with the various percentages (0%, 0.5%, 1%, 1.5%, 2%, 2.5%....) of rubber strips for each samples and three specimens were trimmed from each samples. Then, Direct Shear Test was done for each sample by applying 30 lb, 60 lb and 90 lb normal loads.

After that variation, shear stress with the shear displacement was plotted for each rubber content.

Finally the variation of friction angle (φ) and cohesion (C) with the rubber content was analysed to find out the optimum rubber content.

4. Results and Discussion

4.1 Soil classification

Sieve analysis test and attereberge limit test were carried out to identify the type of soil.

Particle percentage finer than 0.06 mm = 60%
Plastic Limit - 37%
Liquid Limit - 73%
Plasticity Index - 36%

According to BS 1377 the soil was classified as Very High Plasticity Gravelly Clay (CVG).

4.2 Specific Gravity

According to Pycnometer method, Specific Gravity of soil sample was obtained as 2.90.

4. 3 Compaction

Standard Proctor compaction test was done to obtain following results

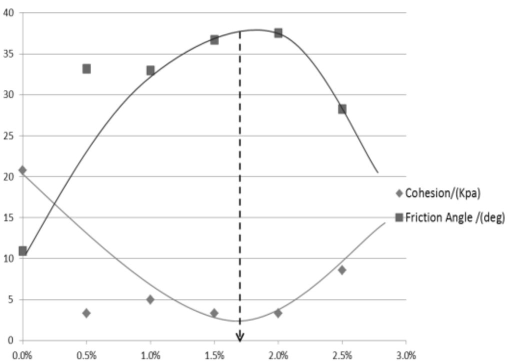
Optimum Moisture Content = 27 %
Maximum dry density (ρ_{dmax}) = 1.6 Mg/m^3
Moisture content at 95% ρ_{dmax} = 24%

4. 4 Direct shear

Following results were obtained from the series of direct shear tests, which were carried for various rubber percentages.

Table 2: Cohesion and Friction angle for each soil sample

Rubber Content %	Cohesion / (Kpa)	Friction Angle/ (deg)
0.0	21	11
0.5	3	33
1.0	5	33
1.5	3	37
2.0	3	38
2.5	9	28



Rubber Percentage / (%)

Figure 1: Variation of Cohesion and Friction angle with rubber content

According to above graph friction angle has increased up to 1.7% rubber content and it has decreased when increasing the rubber content. Cohesion of each sample gradually decreased up to 1.7% rubber content and increased when further adding of rubber. So the optimum rubber content of our study is 1.7%, is recommended to increase the strength of a residual soil which can be used as a backfill material.

5. Conclusions

Direct shear tests were carried out to determine the maximum shear strength of each sample.

20 mm × 5 mm tire tube strip sizes were used as the reinforcement of the residual soil and 0%, 0.5%, 1.0%, 1.5%, 2.0%, 2.5% increments were used until was reached to the optimum rubber percentage.

Maximum friction angle and the minimum cohesion were obtained at the sample of 1.7 % randomly distributed rubber. So according to the Mohr-Coulomb failure criteria, maximum shear strength can be obtained at 1.7% rubber mixed soil.

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HYDRAULIC ENGINEERING

Characteristics of Hydraulic Jumps over Rough Horizontal Beds

N.G.P.B. Neluwala, K.T.S. Karunanayake, K.B.G.M. Sandaruwan and K.P.P. Pathirana

Keywords: Hydraulic jump, Rough bed, Sequent depth ratio, Froude number, Roughness density

1. Introduction

The existing knowledge on the behaviour of hydraulic jump is only for smooth, horizontal channel beds and a very limited number of studies have been reported in literature on the performance of hydraulic jumps on rough beds. This research attempts to investigate the characteristics of hydraulic jumps formed on rough, horizontal channel beds under different flow conditions. A series of laboratory experiments were carried out in a rectangular flume placing artificial roughness elements at different intervals. The characteristics of hydraulic jumps formed on such channel beds were investigated to develop theoretical formulations relating the hydraulic jump parameters and bed roughness.

2 Literature Review

$$\frac{y_2}{y_1} = \frac{1}{2}\left[-1 + \sqrt{1 + 8 \times Fr_1^2}\right] \qquad \text{.......... Eq. 1}$$

where, y_1 is the initial water depth and y_2 is the sequent depth and Fr_1 is the upstream Froude number.

Equation 1 has been widely used to study the hydraulic jumps formed on smooth, horizontal channel beds and not much theories have been established for rough channel beds.

For rough beds Rao et.al, (1966) proposed a modification to above equation by incorporating a coefficient to Froude number to represent the effect of roughness. Subsequently Carollo and Ferro, (2007) estimated the value of this coefficient as 7.42 based on detailed analysis carried out. Carollo and Ferro (2007) further improved the equation by introducing a term (t/y_1) where t height of roughness elements next to the Froude number which was obtained from results of dimensional analysis and empirically.

However, these modifications to the equation 2 were not very successful as roughness density (spacing) was not incorporated in the equations. Hence, a detailed study of the hydraulic jumps characteristics on rough beds is necessary to formulate proper relationship to describe the jump characteristics on rough beds.

3. Methodology

3.1 Experimental Set-up

Laboratory flume of 12 m long, 0.3 m wide and 0.3 m high was used for the study (see figure 1). The flow was controlled by a valve at the flume. A V-notch was attached to the flume to measure the flow rate.

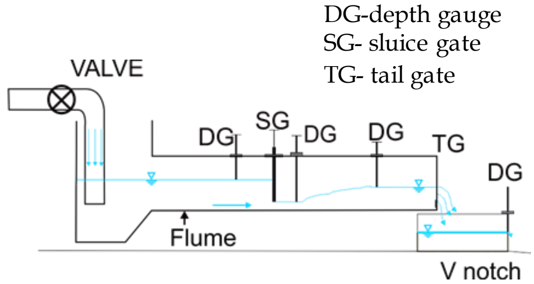

Figure 1: Schematic diagram of the experimental set-up

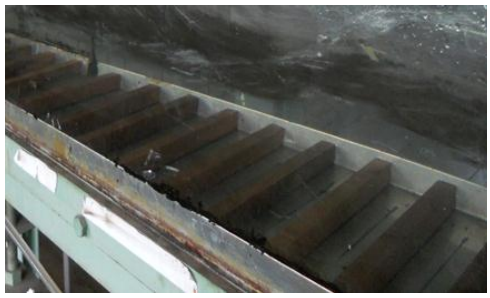

Figure 2: Roughened bed

The channel bed was roughened by fixing rectangular strips as shown in figure 2. Three types of artificial wooden strips having sizes (t) of 0.8, 1.2 and 1.5 cm were used. The spacing (s) between the roughness elements were also changed as 4, 6, 8 and 10 cm.

3.2 Procedure

For each roughness element and its spacing, the dischargers ranging from 5 l/s to 25 l/s were passed in the channel to form hydraulic jumps. The parameters y_1, y_2 and flow rate were measured in each experimental run.

4. Results and Discussion

4.1 Results

Figure 3 shows the comparison of y_2/y_1 obtained from methods suggested by previous researchers with the experimental results.

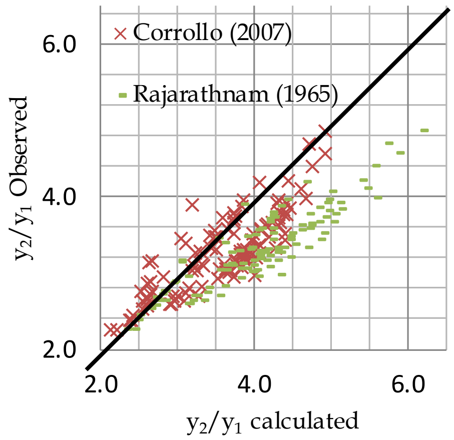

Figure 3: y_2/y_1 observed verses y_2/y_1 calculated

The above figure clearly shows that the previous equations do not predict accurate results for rough beds. This could be partly due to the fact that the roughness spacing is not considered in those formulations.

4.2 Data analysis

To identify the behavior of the hydraulic jumps, various graphs were plotted in non-dimensional form obtained from the dimensional analysis.

$$\frac{y_2}{y_1} = f_2\left(Fr, \frac{t}{y_1}, \frac{s}{y_1}\right) \quad \text{...............Eq. 2}$$

A new parameter called d (roughness density = width of roughness element / Spacing) was introduced to identify better correlations among the parameters of the hydraulic jumps on rough beds. Figure 4 illustrates the relationship developed among the selected parameters.

$$\frac{y_2}{y_1} = \frac{(0.8568\,Fr + 0.3378)}{(1-|0.23-d|)} \quad \text{.........Eq. 3}$$

To include the effect of element size into the above equation, the parameter t/y_1 was used. A new equation (See equation 4) was developed by incorporating t/y_1 with coefficients found using experimental results.

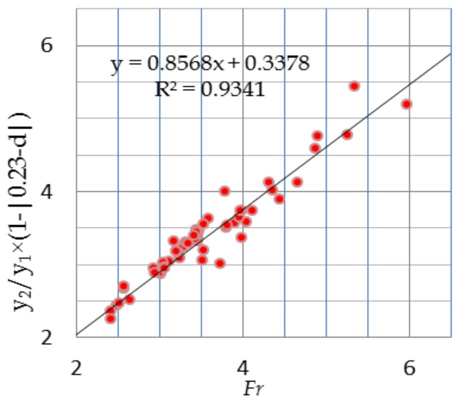

Figure 4: $y_2/y_1 \times (1-|0.23-d|)$ observed verses y_2/y_1 calculated for 1.5 cm roughness element for all spacing.

$$\frac{y_2}{y_1} = \frac{\left(0.8568\left(1-0.05\times\frac{t}{y_1}\right)Fr + 0.3378\right)}{(1-|0.23-d|)} \quad \text{....Eq. 4}$$

This equation provides a better relationship between the hydraulic jump characteristics and properties of roughened bed for all the experimental data collected.

5. Conclusions

The objective of this study was to identify the characteristics of hydraulic jumps on rough channel beds. For that several experiments were carried out and various relationships were developed between the hydraulic jump characteristics. A new equation was formulated to relate the sequent depth ratio with the initial depth and Froude Number in a more accurate manner than in previous studies by considering both effects of roughness height and roughness density.

The applicability range of Froude number for proposed equation covers 2-5 and the shape of roughness elements were rectangular. Further studies are recommended to study the applicability of proposed equation for other Froude numbers and different shape of roughness elements.

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Hydraulic Performance of Labyrinth Spillways

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Keywords: Labyrinth spillways, Hydraulic performance

1. Introduction

Labyrinth spillway is an overflow structure which is capable of releasing floods from reservoirs very effectively and is generally used for small operating head applications. Until today only preliminary design procedures are available for the design of Labyrinth spillways, which cannot yet be generalized due to so many restrictions on applicable range. The primary objective of this study is to compare the hydraulic performance of Labyrinth spillways having different geometric configurations with conventional linear spillways and to develop a relationship among different parameters of Labyrinth spillways. This paper presents the results and conclusions for an experimental study carried out to achieve the above objectives. In addition to that Piano Key weir will also be compared with Labyrinth spillway geometries.

Figure 1: Labyrinth spillway at Loggal Oya

2. Literature Review

Though there are no standard specifications established for the design of Labyrinth spillways there are two most commonly and widely used methods available (Tullis et.al, 1995). The first one was developed by Lux (1984) and the later was developed by Tullis (1995). Both these methods were empirically developed formulas for the discharge. Those formulas are given below with the notations.

a) Lux (1984)

$$Q = C_W \left[\frac{w_c/p}{w_c/p + k} \right] w_c H \sqrt{gH} \quad \text{.........Eq. 1}$$

where, Q is the discharge of one cycle, C_w: discharge coefficient, w_c/p: vertical aspect ratio, H: total head, K: constant on geometry, W_c: cycle width and p is the wall height.

b) Tullis (1995)

$$Q = C_L \frac{2}{3} \sqrt{2g}.LH_t^{3/2} \quad \text{.........Eq. 2}$$

where, Q is the discharge over weir, C_L: discharge coefficient, L: Effective length of Labyrinth and H_t is the Total upstream head.

Spill capacity of Labyrinth spillways depend on various parameters. Especially total upstream head crest length and crest coefficient have a great influence on the spill capacity. Among these factors, the crest coefficient is governed by some other geometric parameters. The crest shape may be either flat, sharp, half round or quarter round and has a direct influence on the value of crest coefficient hence the spill capacity. As it was discovered the best crest shape for Labyrinth is the quarter round shape (Greg et.al, 2006). Other than that crest height, wall thickness and the apex configuration also have effects on the value of crest coefficient.

3. Methodology

The experiments were carried out in 20 m long, 0.40 m wide and 0.48 m deep rectangular, recirculating flume in the Hydraulic Laboratory of the Faculty of Engineering. Figure 2 shows a schematic diagram of the experimental set-up. A broad crested weir was fixed and calibrated in the flume for discharge measurements.

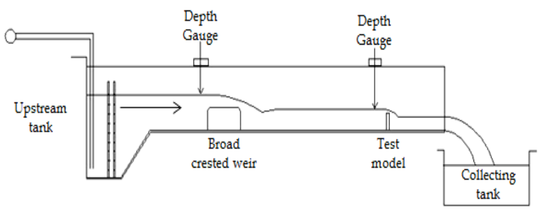

Figure 2: Experimental set-up

The model to be tested was fixed in the flume ensuring no leakages at the edges, before commencing the experiment. Then the water supply was started at a very low rate for just overtopping on the model. After a steady flow occurs, water depths on the spillway and water depth broad crested weir were measured using already installed depth gauges as illustrated in Figure 2. Water depth on the calibrated broad crested weir was measured to calculate the discharge and water depth on spillway was measured to represent the upstream depth. Ten set of measurements were taken for each model varying discharge between 1 l/s and 20 l/s. Seven spillway models having geometrical parameters as given in table 1 were tested during this study.

Table 1: Details of experimental models

Weir Type	Side wall length / (mm)	Side wall angle / (deg.)	Cycle width / (mm)	Weir height / (mm)	Apex width / (mm)
I	456	8	156	64	44
II	456	8	156	64	44
III	300	8	156	100	44
IV	300	8	112	80	88
V	300	12	156	70	44
VI	300	-	63	120	-
VII	-	-	-	63	-

4. Results and Discussion

For this study, a comprehensive dimensional analysis was carried out by considering Froude similarity. Non dimensional parameters obtained were B/p, gp^5/Q^2, H_t/p and $\tan a$ where, B is the side wall length of Labyrinth spillway. Relationship among these parameters is shown in Figure 3. The equation derived using these relationships is,

$Q = 3.77(Bgp^{1.31})^{0.5} * \tan^{3/4}\alpha * H_t^{1.345}$……….Eq. 3

This equation is valid for H_t/p ratios less than 1.00 and w/p ratios between 3 and 7, whereas the previously proposed methods are limited to H_t/p ratios between 0.7 to 0.9 and w/p ratios less than 4.

For the comparison of conventional linear spillway performance with hydraulic performance of Labyrinth spillways, variation of *Ql/Qn with* H_t/p was considered. It was observed that the *Ql/Qn* value is above 1.00 where the Labyrinth effect is prominent, at a range of H_t/p values between 0.1 and 0.4 for most of the Labyrinth geometries.

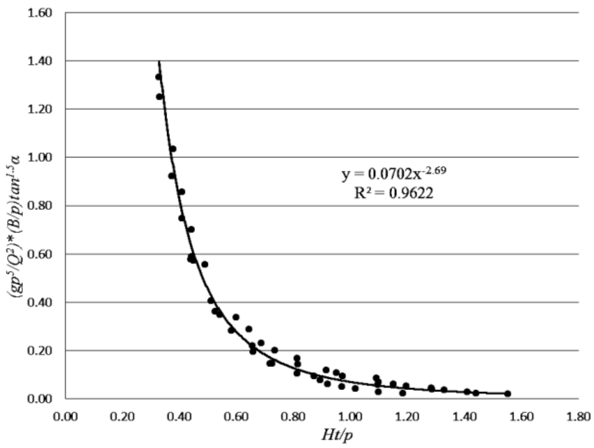

Figure 3: Variation of $\{(gp^5/Q^{2}*B/P)\tan^{1.5}\alpha\}$ vs. H_t/p

Piano key spillway shows a better performance than normal Labyrinth geometries, for very low upstream head applications usually for H_t/p ratios less than 0.4. When comparing individual hydraulic performance of prepared models, Piano Key weir model is second to the Type III model only.

5. Conclusions and Recommendations

Factors that dominate the hydraulic performance of Labyrinth spillways found in this study are side wall angle (α), side wall length (B), weir height (p) and the total upstream head (H_t). The relationship formulated to compare the hydraulic performance is valid for H_t/p ratios less than 1.0 where the currently used equations are having a limited H_t/p ratios less than 0.7. The w/p ratio for the derived equation is in a range between 3 to 7 where the available methods are limited. Piano key spillways have better hydraulic performance for very low Ht/p values compared to the normal Labyrinth spillways. Further experimentation by varying side wall angle and weir height can be used to improve the outcomes of this study.

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Simulation of Flow Patterns in a Reservoir

D.P.G.S.N. Fernando, K.S.A.U. Pushpakumara, J.A.D.S. Ranasinghe, U.R Ratnayaka and D.D Dias

Keywords: Computational Fluid Dynamic , Simulation, Retention time

1. Introduction

Computational Fluid Dynamics (CFD) is widely used in hydraulic Engineering in recent years. The physical principals used in CFD are Conservation of mass, Conservation of momentum along with Conservation of energy. But in actual situations these equations are not adequate enough to solve or get an analytical solution for most engineering problems. But we can get an approximate solution to the problem using finite element techniques.

It spreads a chronic Kidney disease in the Northern Province of Sri Lanka in recent past in the command of Padaviya reservoir, which bears water volume of 37 million cubic meters throughout the year. This project was formulated with the intention of obtaining the retention time of water at selected sections of the reservoir, using CFD technique, thus to facilitate the ongoing researches, targeted at finding whether there is a correlation between kidney disease and retention time of water.

2. Literature Review

Glynn and Shilton (1998) described how Computational fluid dynamic has been used to analyse the hydraulics of two potable water storage reservoirs. The study highlights how stagnant zones and short-circuiting pathways, which are undesirable features of reservoir flow, can be minimized using modelling techniques.

CFD simulations were carried out by Hoi Yeung for a range of common reservoirs and the effect of different inlet arrangements was studied by using the software CFX version 4.1. The water age at different parts of the reservoir were studied in their research.

3. Methodology

Reservoir data were acquired and coordinates were calculated for the construction of a computer model by using software. For the geometry definition, software called SOLIDWORK was used. Then the defined geometry was imported to the ANSYS workbench and FEM meshing was done. Thereafter conditions for the simulation were applied to the model.The material water at measured temperature (25°C) and properties of the reservoir water were given for the simulation. Then results were acquired.

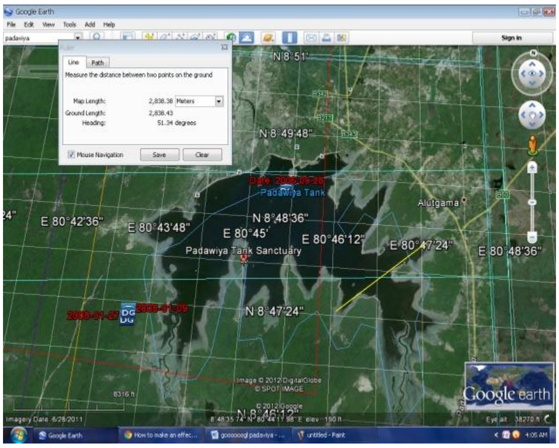


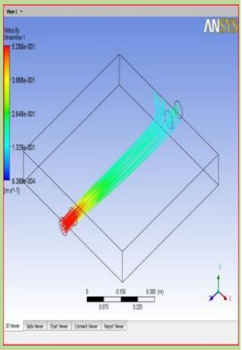
Figure 1: Padaviya Reservoir (www.Google earth.com)

For validating the software, a laboratory model was developed and conducted a series of experiments to observe the stream line patterns. Flow visualization reservoir was modified for the purpose and using the tracer method flow patterns were observed at the state.

Figure 2: Laboratory model

4. Results and Discussion

Table 1: Comparison of flow patterns

CFD simulation	Laboratory Experiment
	 

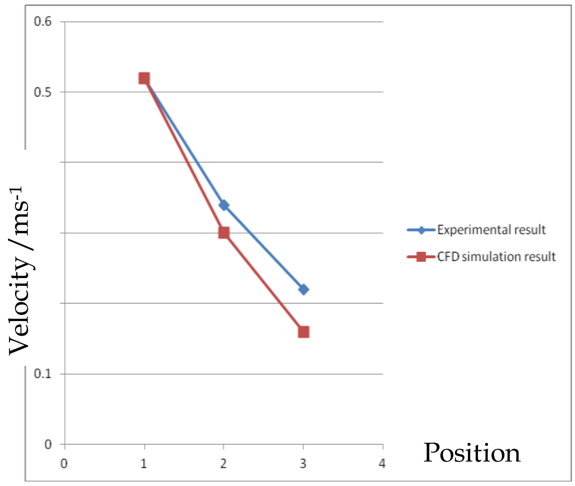


Figure 3: Comparison of velocity

The experimentally observed and the model generated flow patterns and velocities were in a good agreement and utilization of ANSYS to simulate Padaviya reservoir could be justified.

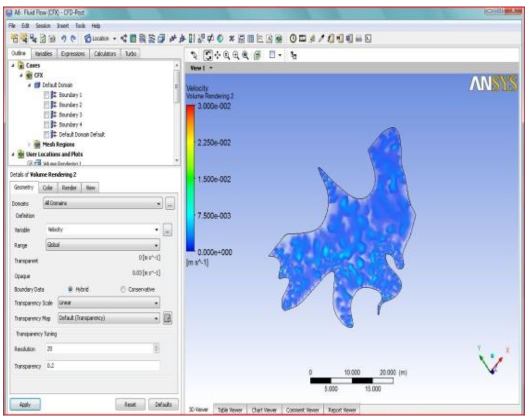


Figure 4: Horizontal velocity distribution of the Dead flood level with one inlet

Table 2: Maximum retention times of the Padaviya reservoir

Maximum retention time was occurred at the dead water level with one inlet and one outlet arrangements.

Arrangement	Location	Retention time(hours)		
		High flood level	Full supply level	Dead water level
Two inlets and one outlet arrangement	Near the inlet	11.0	6.5	12.0
	Middle of the reservoir	13.0	13.0	14.0
	Near the outlet	12.0	11.0	4.5
One inlet and one outlet arrangement	Near the inlet	13.0	9.0	14.0
	Middle of the reservoir	18.0	23.0	35.0
	Near the outlet	15.0	16.5	16.0

5. Conclusions

Resulted stream line patterns gradually diminished within several meters from the reservoir inlet. So the model generated streamline patterns did not highlight a continuity at selected levels. But the velocity field indicated the variances in water retention of reservoir body for all three levels. Retention time confirmed that Padaviya reservoir with two inlets and one outlet arrangement provides the better mixing than the one inlet and one outlet arrangement.

References

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MATERIALS AND CONSTRUCTION ENGINEERING

Improved Retrospective Analysis of Victoria Dam

N. Puvyrajinee, S. Sobija and A.P.N. Somaratna

Keywords: Finite element analysis, Victoria dam

1. Introduction

Victoria dam is an important structure that has a long intended life. Monitoring the behaviour of that structure is useful in developing predictive capabilities to assist in future operation and maintenance activities and where necessary in deciding on remedial work.

Victoria dam is a doubly curved arch dam. It was constructed during 1980 to 1984. This dam is well instrumented and from observed data it is seen that there have been significant changes in its deformation pattern. Developing a reliable model of the structure assumes great importance in such cases. Future behaviour of the structure can be predicted by analysing the model with a finite element programme package. The dam model can be verified by comparison of its estimation with observed data.

2. Literature Review

2.1 Analysis of Victoria dam

a) A static analysis of Victoria dam was carried out by the Department of civil engineering, University of Bristol (Alexander Gibb and Partners, 1980).

b) A finite element model of Victoria dam has been studied by Chandrasiri et al. (2011), who reported that the calculated displacement pattern is compatible with observed data. But, displacements do not exactly match with observed data. Therefore it is important to explore possible improvements to the model. There are certain factors that may be responsible for the mismatch of the results of the previous study of Chandrasiri et al. (2011). They are: the assumed symmetry in the geometry, fixed supports at the foundation level, exclusion of galleries and spillway openings in the model, and assumed values of linear elastic material properties.

The present study aims to examine the possible effects of some of these factors on the performance of the finite element model.

3. Methodology

The present study examined the effects of the following factors with the aim of improving the performance of the finite element model developed by Chandrasiri et al. (2011).

i. The symmetry assumption used to limit the model to half of the dam.
ii. Non-inclusion of galleries and spillway openings in the finite element model.
iii. Possible deviation of material. properties from the values used in the analysis.

The finite element displacement results were compared with the observed values to determine the effectiveness of the model.

4. Results and Discussion

In order to test the effect of symmetry assumption, full dam was modelled and compared with half dam results. (figure.1). It was seen that the half dam model produced quite satisfactory results.

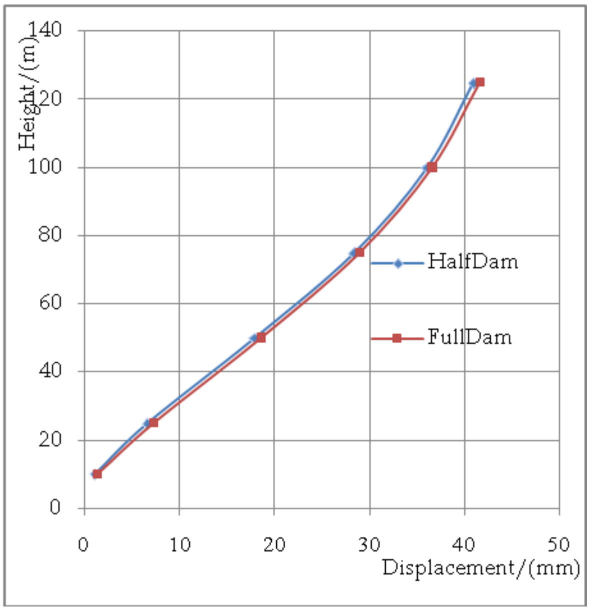

Figure 1: Comparison of results from half dam mesh and full dam mesh

Based on this conclusion, the half dam model was used in further studies.

As the next step, galleries and spillway openings were included in the half dam model. The finite element results were compared with the observed data (figures 2 and 3). Inclusion of galleries and spillway openings improved the performance of the finite element model.

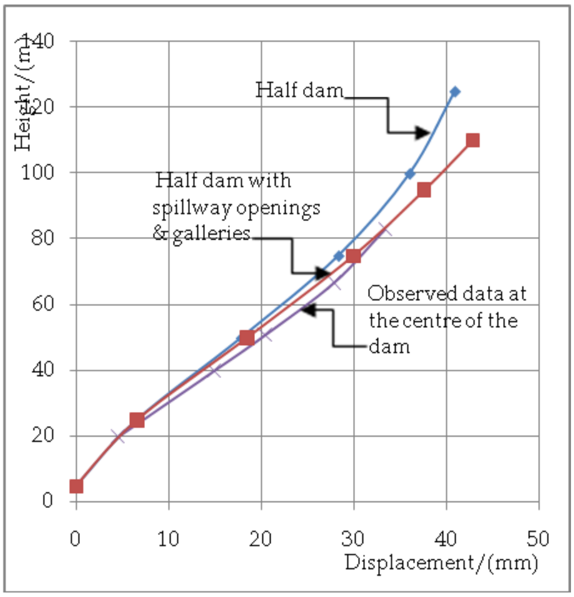

Figure 2: Effect of including spill way openings and galleries on displacements at centre

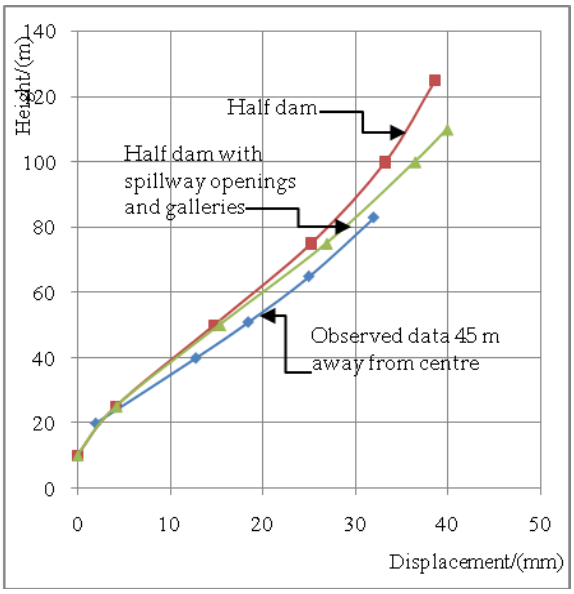

Figure 3: Effect of including spill way opening and galleries on displacements at 45 m from centre

The effect of varying the elastic modulus also was studied. The results are presented on Figure.4. Concrete test records available for Victoria dam indicate that the strength of the concrete used in the dam varies from 28 MPa to 48 MPa. Corresponding values of elastic modulus would vary between 23 GPa to 32 GPa. Figure 4 shows that the observed displacement results would fall within the envelope of finite element results for these extreme values of elastic modulus.

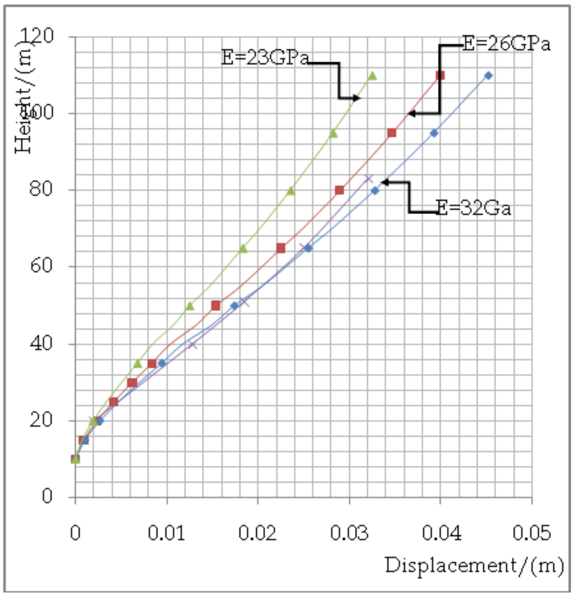

Figure 4: Effect of elastic modulus on computed displacements

5. Conclusions

i. Modeling half of the dam, assuming symmetry, is acceptable.

ii. Inclusion of galleries and spillway openings, and using appropriate and realistic values for elastic modulus has the potential to improve the performance of the finite element model of the Victoria dam.

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Compatibility Issues of OPC & Fly Ash Based Cements with Retarding & Water Reducing Admixtures

N.P.P.G.C.P. Dharmasena. , H.T.D. Hitige. , E.V.G.U.S. Kumarasinghe and H.D. Yapa

Keywords: OPC, Fly ash, Retarder, HRWR, Workability, Setting, Strength

1. Introduction

In concrete related works, admixtures are used to improve certain important concrete properties for instance workability, setting time and initial strength. Prior to the use of admixtures, it is very important to have an understanding about the compatibility of those admixtures with the used cement type. Because sometimes there are some side effects which are not desirable with some cement-admixture combinations such as rapid slump loss, strength reduction, etc.

In the current project the influence of contents of tricalcium aluminate (C_3A) and alkalis in cement on possible incompatibilities such as loss of workability, strength reduction and shortening of setting times has been evaluated. In addition, the potential to use fly ash to overcome these incompatibilities has been evaluated.

2. Literature Review

- Nkinamubanzi et al. (2003) have carried out an investigation on the compatibility issues between naphthalene - based superplasticizers and ordinary Portland cements. Information about the time interval that the slump test has to be carried out was found. In that project slump has been measured around 20 to 25 minute intervals. Slump test in the current project has been carried out at 30 minutes intervals.

- Rangaraju and Desai [2009] have carried out extensive work on effectiveness of fly ash and slag in mitigating alkali – silica reaction induced by deicing chemicals. The research reveals the followings: Fly Ashes are differing by their Lime content.
  The most effective replacement levels of those fly ashes with OPC are;
  Low lime (LL) – 15% } Class "F" fly ash
  Intermediate lime (IL) – 25% } Class "F" fly ash
  High lime – 35% - Class "C" fly ash

Since class "F" fly ash is used in this project, it was decided to replace 20% of OPC with fly ash. This is because class "F" fly ash contains LL to IL content.

3. Methodology

The major compatibility issues of using OPC with retarding and high range water reducing (HRWR) admixtures were identified as follows,

- Rapid workability loss
- Compressive strength loss
- Flash set

Three tests were carried out to check whether those compatibility issues exist in the three OPC samples with retarding and HRWR admixtures which were used.

3.1 Concrete testing

After testing materials, the three OPC samples which have been mentioned above, were tested for the compatibility issues as follows.

3.1.1 Rapid workability loss

This compatibility issue is related to loss of slump very rapidly when an admixture is added. To check whether this incompatibility exists in the cement samples, the slump test was carried out. This test was done at regular intervals after mixing concrete.

3.1.2 Compressive strength loss

This is the reduction of compressive strength of concrete when an admixture is added. This compatibility issue was checked by carrying out cylindrical compressive strength tests at different ages of concrete. Those test ages were 3day, 7 day and 28 day.

3.1.3 Flash set

Flash set can be explained as immediate stiffening of cement paste. This is due to the reaction of pure tricalcium aluminate with water. This was tackled by carrying out initial and final setting time tests.

4. Results for OPC

- When slump test results are concerned, all three OPC types have similar slump variations.
- When setting results are evaluated, all retarding and HRWR mixes of three OPC types show large delays in both initial and final setting times than standard values.
- In compressive strength test, cement II has the highest number of strength results are less than the standard strength values in ASTM.

Therefore, it can be concluded that cement II is the most incompatible cement type with retarding and HRWR admixtures used in this project.

So 20% fly ash is blended with cement II and further investigation is carried out to check whether the fly ash addition causes to overcome those incompatibilities.

5. Results for Fly ash blended cement

5.1 Slump test
There is no rapid workability loss in fly ash blended cement, workability further increased than cement II.

5.2 Setting time test
No flash set in fly ash blended cement also. Both initial and final setting times are further increased in all the mixes than the values of cement II.

5.3 Compressive strength test
Following figures shows the variation of 3 day, 7 day and 28 day compresive strength results of fly ash blended cement with Cement II.

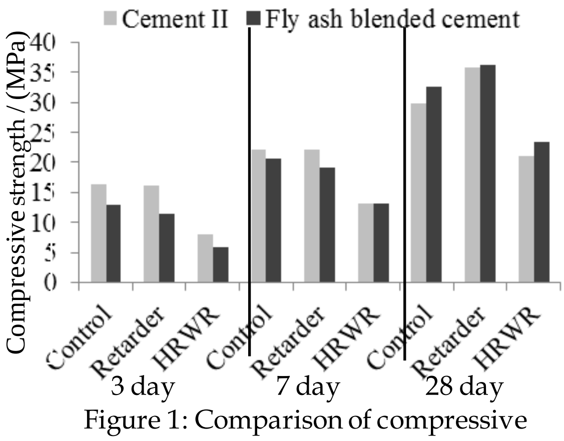

Figure 1: Comparison of compressive strength results

6. Conclusions

When compressive strength results are considered early strength (3 day & 7 day) of fly ash blended cement is less than early strength of OPC for both retarding and HRWR mixes. When standard strengths are evaluated, strength of fly ash blended cement is greater than strength of OPC in both of above mixes. The gap between strength values of OPC and fly ash blended cement is reduced when the age goes from 3 day to 7 day in both retarding and HRWR mixes. There is a trend to improve the compressive strength of concrete mixes with both retarding and water reducing admixtures when fly ash is blended with OPC.

Workability is greatly improved when fly ash is blended with OPC. Therefore, if there is any rapid workability loss in an OPC type, fly ash can be used as a solution to overcome that.

Time of setting of concrete is delayed in fly ash blended cement than OPC with retarding and HRWR admixtures. So fly ash can be blended with OPC to overcome flash set in retarding and HRWR concrete mixes.

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Construction Project Delays in Sri Lanka

M.U.M. Ijas, T.M.M. Imran, M.F.A. Najath and P.B.G. Dissanayake

Keywords: Construction delay, Contractor, Consultant, Relative Important Index

1. Introduction

The Project delays are very common in the Sri Lankan construction industry. Project delays occurasaresultofanactorfailuretoactbycontractin gpartiesaswellasby outside forces. In general, delays occur in almost all construction projects in Sri Lanka. The objective of this study was to identify the most significant causes of delays in the Sri Lankan construction industry.

Delays in the completion of construction projects are often unavoidable. The project schedule which is planned at the beginning of the project is prone to being changed many times and unfortunately leads to delays. As a result, scheduled delays may be a major problem for contractors as well as the owners, resulting in costly disputes, controversial issues and adverse relationships between all the project participants. Therefore, the identification, quantification and analysis of delays become essential. Contractors are prone to see most of the delays caused by owner, while owns usually want to put the blame on the contractor or third parties.

2. Literature Review

The literature review was carried out through internet resources, books, articles, and leading construction management and engineering journals. From the literature review all the possible causes of delay that affect construction projects were identified.

The research papers that have been gone through are "Delays in construction projects: The case of Jordan", "Construction delay: a quantitative analysis", "The effects of construction delays on project delivery in Nigerian construction industry", and "Causes and effects of delays in Malaysian construction industry" and "An evaluation of construction time performance in the building industry".

3. Aims and Objective

The main objective of this research is to investigate the major causes of construction project delays in Sri Lanka and to propose suitable measures to minimize delays.

4. Methodology

Research methodology was based on the aims and objectives of the study. Methodology focused on the literature review and the questionnaire survey. Data collection was based on questionnaire survey and Statistical methods were used to analyse the data. Preparation of the questionnaire was based on the following.

4.1 Types of delay

There are four basic ways to categorize type of delays:

i. Critical or non-critical
ii. Excusable or non-excusable
iii. Compensable or noncompensable
iv. Concurrent or non-concurrent

.

4.2 Causes of Delay

There are many factors that contributed to causes of delays in construction projects. These are owner related, contractor related, consultant related, material related, equipment related, labour related, and external factors.

4.3 Effects of construction delay

- Time overrun
- Cost overrun
- Disputes
- Arbitration
- total abandonment
- Litigation.

4.4 Questionnaire Design

A questionnaire survey was designed based on the objectives of the study, which are causes of construction delays, effects of construction delays and the method rectification of construction delays. A questionnaire survey was developed to get the opinion and understanding from industry experts regarding construction delays.

4.5 Data Analysis

Data analysis was carried to establishing he relative importance of the various factors. It included the calculation of the Relative Importance Index RII) and ranking of factors in each category based on the RII.

4.6 Data Analysis

To determine the ranking of different factors from the viewpoint of contractors and consultants, the RII was computed as:

$$RII = \frac{\sum WiXi}{\sum Xi} \qquad \text{.........Eq. 1}$$

5. Results and Discussion

RII has separately calculated for the contractors and consultants for every factor that had been identified as delay causes in the questionnaire form

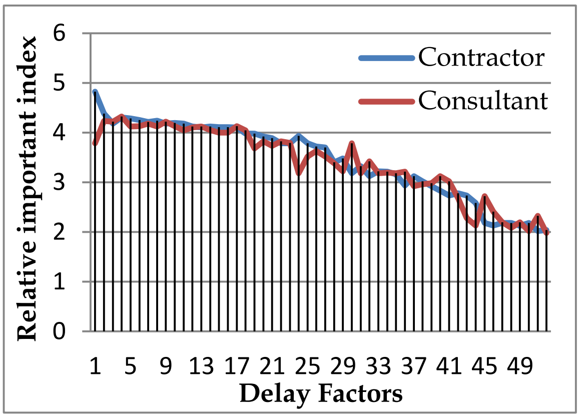

Figure 1: Correlation between contractor and consultant Vs. Causes of delays

6. Conclusion

A construction delay occurs mostly during the construction phase. This is mostly caused by the poor labour's skill, supervisor not able to coordinate the project very well and also low quality of material used in the construction projects.

Low technical and managerial skills of contractors are the problems that are faced by contractors which might cause construction delays. Therefore, contractors should organize some training programs for their workers in order to update their knowledge and improve their management skills.

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Durability Studies of Concrete Produced from Fly ash Based Cement

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Keywords: Durability, Fly ash

1. Introduction

Deterioration of concrete and corrosion of steel reinforcement are major problems for the durability of concrete. To improve the durability of concrete, partially replacing the OPC with fly ash has been identified as a good solution. Sri Lanka has its first coal power plant in operation at Norochcholai, and large amount of fly ash is produced as a by-product. This blended cement is used in several countries.

2. Literature Review

Previous studies show that after 28 days of immersion of concrete cubes in 5% concentrated H_2SO_4 , the concrete with 40% fly ash has shown a better performance in Ordinary and Standard grades. Also In higher-grade concretes, there is an improvement in compressive strength up to 10% of replacement of cement by fly ash over Conventional concrete (0% fly ash).

Previous studies show that supplementary cementing materials such as silica fume, fly ash may have a significant effect on the chemistry or electrical conductivity of pore solution, depending on the alkali content of the supplementary cementing material, replacement level and age, which has little to do with the chloride permeability.

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3. Methodology

In the current project, three experiments were done by replacing cement with various percentages of fly ash material to get the optimum percentage for good durability.

3.1 Carbonation test

This carbonation test was used to measure the carbonated depth of concrete. In this test (phenolphthalein test), carbonation depth was measured according to the CPC-18 RILEM procedure 15. And compressive strength was determined according to ASTM C-39. For that, 30 cubes were casted for the test by replacing several fly ash amounts like 0%, 10%...40% etc. and three specimens were used for each content and specimens were checked after 14 days and 21 days.

3.2 Rapid chloride permeability test

Rapid chloride permeability test was used to check the chloride ion penetrability of concrete. For this test the samples were prepared according to the ASTM C1202-07. Prepared samples were kept in a suction chamber for 3 hours to remove air voids inside specimens. Then samples were submerged in water to fill voids with water for 18 hours. After that samples were taken out from suction chamber and fixed and sealed with two cells. 3% NaCl solution was filled to one cell and other cell was 0.3M NaOH solution. The stainless steel electrodes were connected to a 60 V DC power supply fitted with a data logger. After 6 hours of the test, samples were then split open and sprayed with a silver nitrate solution. Silver chloride will precipitate and cause that portion of the sample to turn white, while the silver nitrate in the non-chloride penetrated zone turns brown. From the data logger it can be taken coulomb charged pass across the specimens.

3.3 The Sulfate Attack test (Bureau of Indian Standards)

The sulfate attack test was carried out to study the weight loss of concrete produced from cement and variation of compressive strength after sulfate attack. 15 cubes were casted and cured in separate buckets containing 5% H_2SO_4 and potable water for 28 days completely. The weights were taken initially at time of immersing in acid and at 7 days, and 28 days of curing. The percentage weight loss during curing period was observed and compared between losses of weight of cubes immersed in acidic water with different percentages of fly ash contents. The 28th day compressive strength of cubes under acid water curing was also evaluated.

4. Results

4.1 Rapid chloride permeability test

The graphical representations of results are given in the following chart.

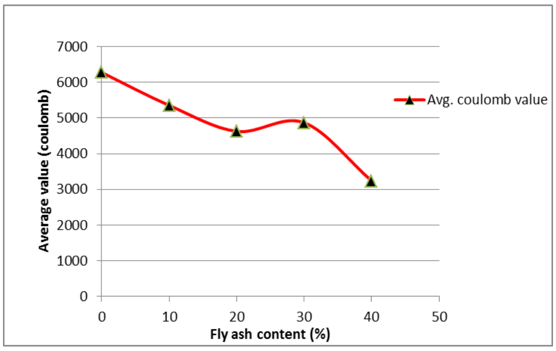

Figure 1: Variation of average coulomb value with fly ash content.

Figure 1 shows the variation of coulombs with different fly ash contents. From 0% to 20% coulomb value is continuously decreased. Then in 30 % fly ash content the coulomb value is suddenly increased and then in 40 % fly ash content there is a coulomb value drop.

4.2 Sulphate attack test

In the sulphate attack test, results were observed as in the following figure. In the figure it is included, the initial weight of concrete cubes and weight loss percentages after 7 days and 28 days with different fly ash contents.

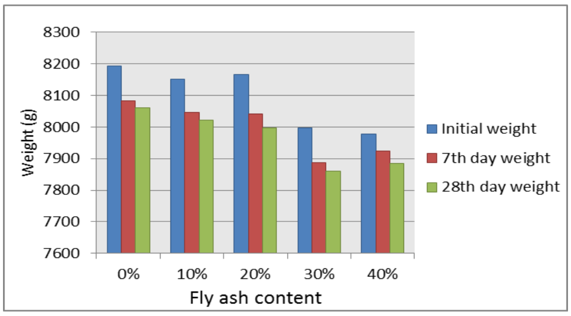

Figure 2: weight variation of cub with fly ash contents cured in H_2SO_4.

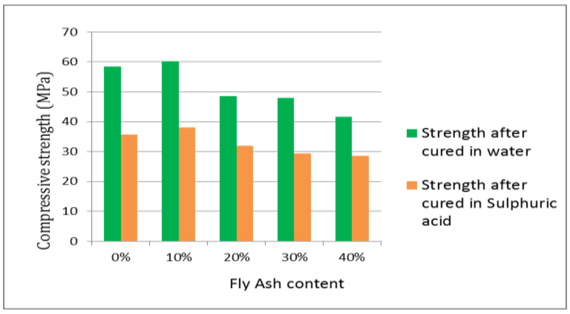

Figure 3: Variation of weight loss comparison in submerged in H_2SO_4 acid.

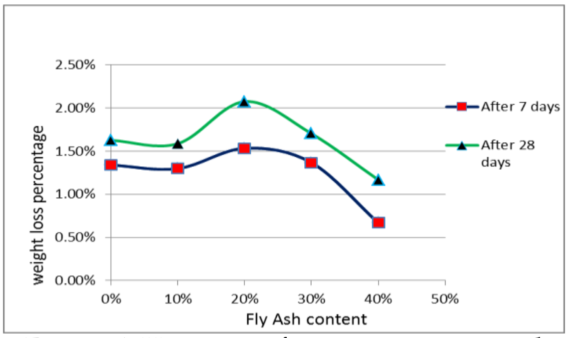

Figure .4: Variation of compressive strength with fly ash content.

5. Conclusions

In the chloride ion penetration test, results show that with the increment of fly ash content the chloride ion penetrability is decreased. So, it is better to have high fly ash contents for concrete where the chloride ion concentration in water or ambient is high, like coastal areas. After 28 days of immersion of concrete cubes in 5% concentrated H_2SO_4, the concrete with 40% fly ash has shown a better performance in used concrete mix. In the initial stages like 7 day the weight loss in concretes observed very less as fly ash replacement is increasing. 28 day weight loss variation also same as the 7 day weight loss variation. In used concrete mix, there is an improvement in compressive strength up to 10% of replacement of cement by fly ash over conventional concrete (0% fly ash).
The carbonation test results show no visual carbonation depth. There can be many reasons for that result. The time taken for the carbonation may not be enough. And there may be leakage of CO_2 gas in the carbonation chamber.

Considering durability parameters which were tested in this project and strength of concrete, the optimum fly ash content is between 10%-20%. So, under different conditions cement can be replaced by fly ash for economical usage.

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Effects of Earthquakes on Victoria Dam

S.M.A.K. Bandara, D.M.S. Dissanayake, A.P.N. Somaratna and S.R. Herath

Keywords : Victoria dam, Earthquake loading, Peak ground acceleration

1. Introduction

Victoria dam is an important structure in Sri Lanka, because of its substantial contribution to the economy of the country by producing 780 GWh of hydro electric energy. An extreme event such as an earthquake can bring about severe damages to this structure. It is therefore important to study the effects of earthquakes on Victoria dam. Such studies will be beneficial in preparing for structural appraisals after unexpected loading conditions and for disaster management as well.

1.1 Objective of the study

Evaluate the effects of earthquake loading conditions on the Victoria dam.

1.2 Scope of the study

The scope of the study is to analyse the dam numerically under selected earthquake loading conditions, ignoring hydrodynamic effects and assuming the dam to be on a rigid foundation.

2. Literature Review

Abayakoon (1997) has suggested that the maximum ground acceleration that could be expected in Sri Lanka would be of the order of 0.35g, with a return period of 300 years. Peiris (2007) has carried out probabilistic seismic hazard assessment and derived a Peak Ground Acceleration (PGA) of 0.026 g for a 475 year return period. Chopra (2012) has pointed out that in 3-d analysis of arch dams under earthquake loading (1) stresses may be overestimated or underestimated by neglecting water com-pressibility; (2) stresses may be overestimated by a factor of 2-3 by ignoring foundation-rock mass damping (3) spatial variance in ground motion can have much influence on the earthquake induced stresses in the dam. Therefore in the present study, emphasis is placed on the trends related to stresses rather than their precise values.

3. Methodology

Earthquake induced stresses in the dam were computed for different values of some of the earthquake parameters, viz. Peak ground acceleration, frequency content and Time Duration. Largest stresses on the upstream or downstream face of the dam were estimated. Finite element model of half of the Victoria dam developed by Chandrasiri *et al.* (2011) was used in the analysis.

3.1 Method of Analysis

Linear time history analysis procedure was used to determine the earthquake response of Victoria dam, using computer program SAP 2000.

3.2 Selected earthquake

Longitudinal, Tangential and Vertical components of the Koyna earthquake have been used as the basic input motion for the earthquake analysis of the Victoria dam. The Richter Magnitude of this earthquake was 6.3 and it occurred on 10th of December 1967. Its epicentre was about 10 km away from Koyna dam, India.

3.3 Design Earthquakes

Peak ground acceleration is a factor that influences stresses in a dam. For this analysis accelerograms have been modified to 3 different peak ground acceleration values, 0.02 g, 0.1 g, 0.15 g.

To identify the effects of frequency, analysis was done for the following three cases.

- Actual frequency
- Increasing frequency by 10%
- Decreasing frequency by 10%

The Duration of ground motion can have an influence on the level of induced stresses. To examine this, above analyses repeated for 10, 20 and 30 second durations.

4. Results and Discussion

4.1 Influence of Peak Acceleration on the earthquake induced stresses.

As demonstrated in figures 1, 2, 3, when the PGA is increased the Peak tensile stress also increases linearly for the different frequency contents and for all three time durations.

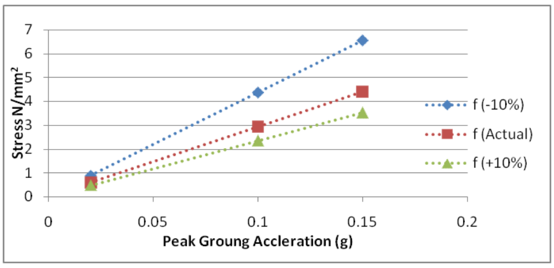

Figure 1: Variation of peak tensile stress with the PGA at 10 sec

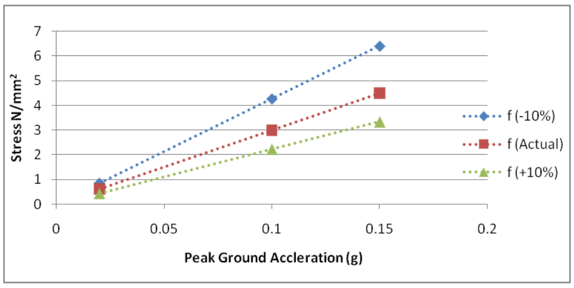

Figure 2: Variation of peak tensile stress with the PGA at 20 sec

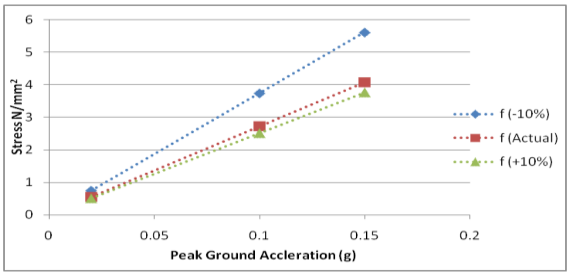

Figure 3: Variation of peak tensile stress with the PGA at 30 sec

4.2 Influence of Frequency Content on the earthquake induced stresses

As shown in figures 4 – 6, when the frequency increases Peak tensile stress decreases for all three time durations and peak ground accelerations. Natural frequencies of first, second and third modes of Victoria dam is 7.4 Hz, 11.2 Hz and 23 Hz, respectively.

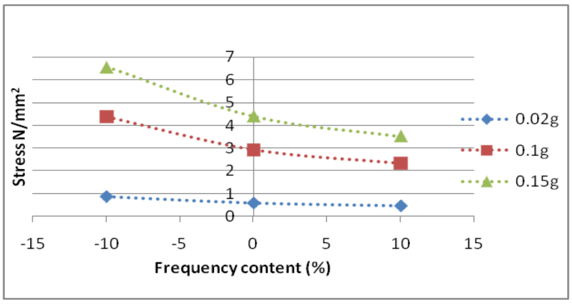

Figure 4: Variation of peak tensile stress with the frequency content for 10 sec duration

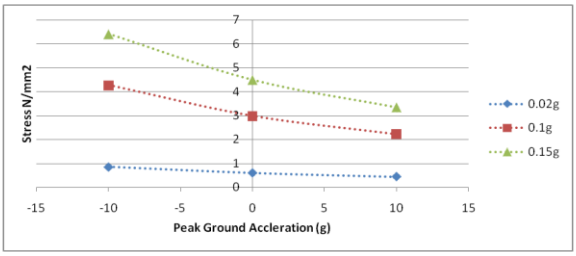

Figure 5: Variation of peak tensile stress with the frequency content for 20 sec duration

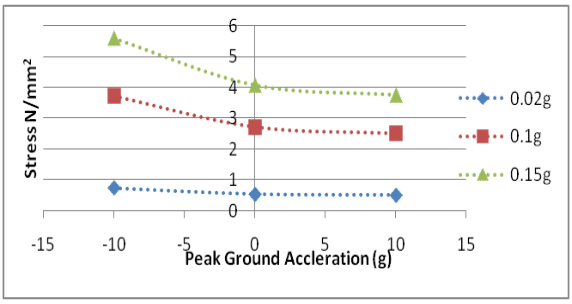

Figure 6: of peak tensile stress with frequency content for 30 sec duration

5. Conclusions

Analysis of the dam under different earthquake parameters has demonstrated that for all the earthquake loading conditions considered, the peak tensile stress occurred in the same region in the dam. Based on these results it is concluded that (1) When the Peak Ground Acceleration increases the peak tensile stress also increases linearly and the percentage variation of the peak tensile stress does not depend on the duration and the frequency; (2) When frequencies in the earthquake loading increase the peak tensile stress decreases and percentage variation of the Peak tensile stress depends on the duration and does not depend on the PGA.

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Effect of Foundation Stiffness on the Performance of a Finite Element Model of Victoria Dam

S. Robina, S. Sriranganathan and A.P.N. Somaratna

Keywords: Victoria dam, Foundation, Finite Element Model

1. Introduction

When a structure starts to show some deviation from its expected behavior it is important to investigate the probable causes for such behavior at an early stage. Victoria Dam in Sri Lanka is a case that falls within this category.

Victoria is a doubly curved concrete arch dam, constructed across the Mahaweli River in 1985 mainly for hydropower generation. It is well instrumented. The observations made using these instruments have shown certain changes in the behaviour of the dam over the last 10 to 15 years. Probable changes in structural configurations or expansion due to Alkali Aggregate Reaction (AAR) have been suspected to be the cause for this anomalous behaviour.

If there is such a problem, it is necessary to detect it early. To study this issue, an analysis has been performed earlier by Chandrasiri et al, (2011). To model the dam in the finite element analysis it was assumed that the dam is symmetric and only half of the dam was modelled. Foundation was assumed to be fixed. The results indicated that the finite element calculations showed the same trends as the observed displacements. However, the displacements did not exactly match. It was suggested that the cause for this deviation might be modelling the foundation as fixed. The objective of the present study is to investigate the effect of modelling the foundation rock also as deformable.

2. Methodology

Federal Energy Regulatory Commission Division of Dam Safety and Inspections (1999) has recommended that the extent of the foundation rock that needs to be modelled can be equal to the dam height. After some preliminary testing this recommendation was adopted.

From the borehole data of Victoria site investigation the foundation rock type was determined to be biotite gneisses. Material properties corresponding to this rock type (Jayawardena, 2009) were identified. It was assumed that foundation rock is linear elastic and isotropic. The body of the dam was considered to be perfectly bonded to the foundation rock over its area of contact.

The finite element model of half of the dam used by Chandrasiri et al, (2011) was modified by the addition of these 3D solid elements to model the rock. Analysis was done for different values of stiffness ratio E_f/E_c and the displacement results were compared with observed displacements.

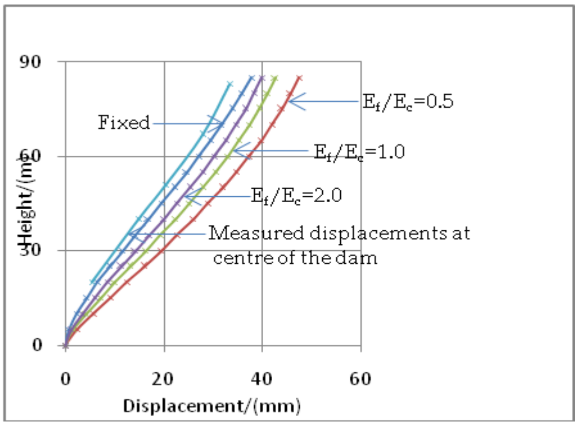

Figure 1: Effect of varying foundation stiffness on displacements at the centre of the dam under hydrostatic loading

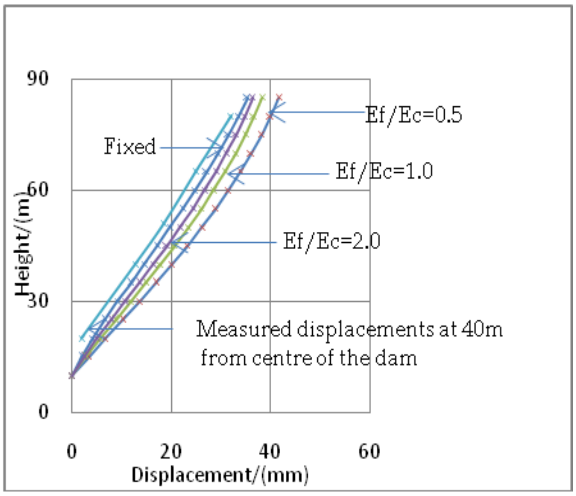

Figure 2: Effect of varying foundation stiffness on displacements under hydrostatic loading at a section 40 m from the centre of dam.

Two loading cases have been used in this study: hydrostatic load only and self-weight + hydrostatic loading. Both cases were considered because it might be more appropriate to compare the observed displacements with finite element estimates for the dam under hydrostatic loading only as these observed displacements have been measured using pendula installed in the dam

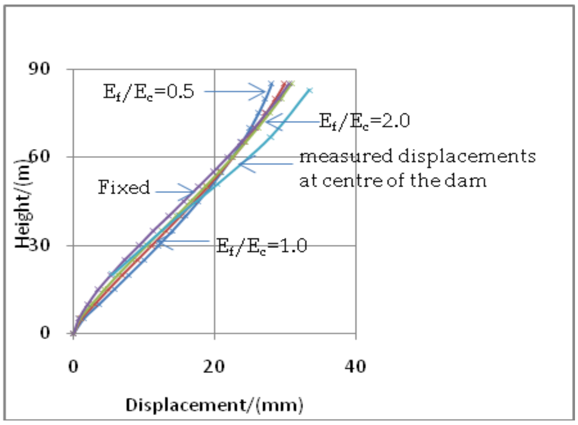

Figure 3: Effect of varying foundation stiffness on displ. at centre of dam under self weight + hydrostatic loading

after completing its construction. Therefore changes in the pendulum positions which are used to estimate observed displacements would be the result of loads other than self weight.

3. Results

In figures 1 to 4, the finite element results are compared with observed displacements. The results indicate that the stiffness of the foundation substantially influences the finite element results. Stiffer foundation seems to achieve better agreement with measured data.

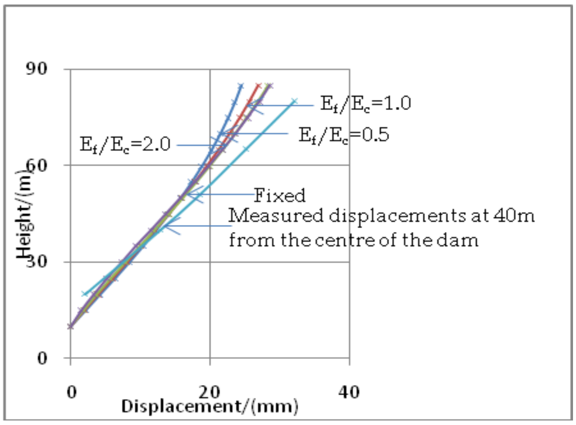

Figure 4: Effect of varying found. stiffness on displ. at section 40 m from centre under self wt. + hydrostatic loading

4. Conclusions

It is concluded that modelling the foundation rock and selecting appropriate values for the foundation stiffness is important in improving the performance of the finite element model of Victoria Dam.

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Investigation of the Effects of Possible Alkali Aggregate Reaction on Victoria Dam

W.J.M.C.S. Bandara, R.M.G.M. Rathnayake and A.P.N. Somaratna

Keywords: Alkali aggregate reaction, Numerical simulation, Victoria dam

1. Introduction

Since Victoria dam was constructed in 1985, it has been a very important structure in hydro power generation and in flood control. Therefore the safety, stability and sustainability of the dam are very important. This study is motivated by the observations on the past behaviour of Victoria dam which suggest a slightly expansive behaviour. This research mainly focused on the numerical simulation of Alkali Aggregate Reaction (AAR) which might be responsible for the observed slight expansion.

2. Literature Review

AAR has been identified as a major problem that affects the strength, stiffness, appearance, stability and safety of concrete structures. AAR is a very slow process and proceeds with the age of the structure. Usually the effects of AAR may be observed within 10-15 years after construction of the concrete structure. There are a number of concrete dams all over the world which have been identified as suffering from AAR. Much experiments and research have been carried out to investigate the effects of AAR on these dams. Most of them have been rehabilitated successfully.

There are several models and methods that have been implemented for simulation of AAR in concrete structures. Charlwood's model is one of the simple and widely used models for AAR studies. There are some expensive computer software that have been developed specifically to simulate AAR in concrete structures. In the present research, the major objective is to develop a procedure for the use of widely available general purpose software for this simulation thus producing an economical and affordable predictive tool.

3. Methodology

In the procedure developed in the present study the non-linear AAR process is approximated by a large number of small linear incremental steps. In each incremental time step, potential expansion, due to AAR, along the principal stress directions are estimated using a suitable model. These potential expansions are mimicked by equivalent thermal expansions. This is similar to the approach presented by Pretheeban et al. (2009) and Pathirana et al. (2011) to simulate AAR expansion.

At every increment it is necessary to modify the material properties and orientation of material axes. A Matlab program has been developed to perform this repetitive task. 3-D solid elements were used in SAP2000 finite element model. The finite element simulations were performed on three bench mark problems. Results were compared with analytical results presented in Pretheeban et al. (2009) and Pathirana et al. (2011). After validating the procedure and the program using these benchmark problems, a preliminary simulation of possible AAR effects on the Victoria dam also was performed.

4. Bench Mark Problems

Figures below show the comparison of the finite element results with the analytical solutions for the bench mark problems.

4.1 Column with uniform section

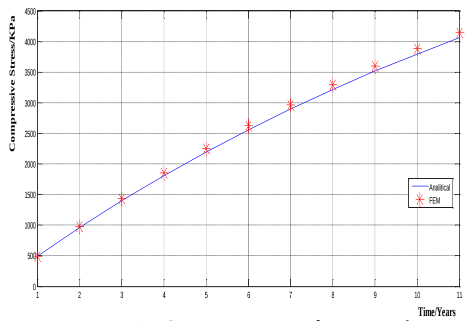

Figure 1: Stress in uniform column

4.2 Rectangular column with linearly varying width

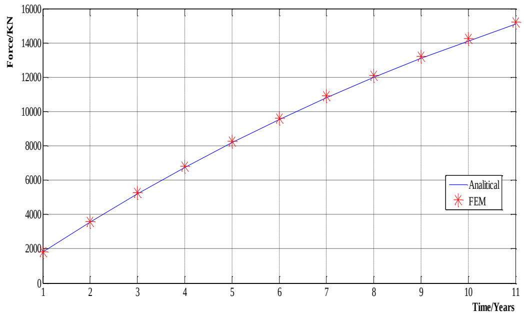

Figure 2: Compressive force in column with linearly varying width

4.3 Circular column with linearly varying radius

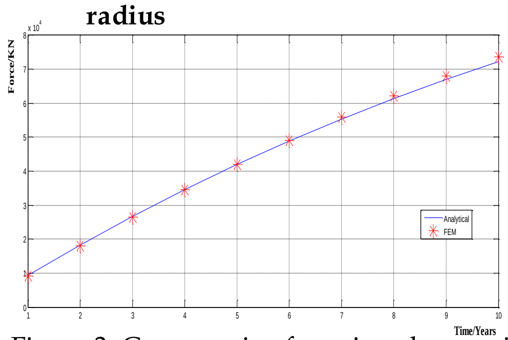

Figure 3: Compressive force in column with linearly varying radius

The results show the methodology is reliable. Therefore this methodology was used to simulate AAR on Victoria dam.

5. Possible AAR in Victoria Dam

The bench mark tests confirmed that the program performed well. It was then used to simulate the behaviour of Victoria dam if it were subjected to AAR over its entire body. The 3-D finite element model of Victoria dam developed by Chandrasiri *et al* (2011) was used for this simulation. Figure 4 shows the observed vertical displacements along the dam crest and

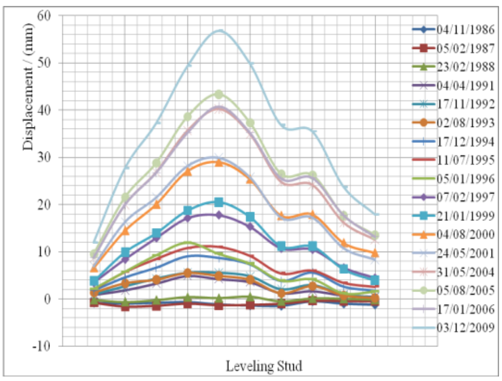

Figure 4: Variation of crest level along the dam with time

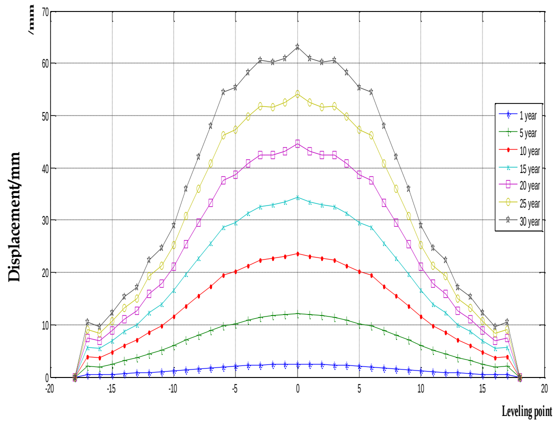

Figure 5: Finite element results for the variation of the crest level along the dam

Figure 5 shows the results of the finite element simulation.

6. Conclusions

The software developed to use Matlab to assist in the non-linear step by step SAP2000 simulation of AAR has been verified as reliable. The program can be used in further studies of AAR in any structure.

By using the program for simulation of Victoria dam we got some results similar to the observed displacement pattern. Therefore it seems that the observed behaviour of Victoria dam could be due to some form of expansion in concrete. One possibility for such an expansion could be AAR.

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Finite Element Analysis of Nanotubes

W.M.N.A.P.B. Herath, H.A.R. Subhashini, S.A.S.P. Sathurusinghe and K.R.B. Herath

Keywords: Carbon nanotubes, Single walled carbon nanotube, Isotropic, Orthotropic, Composite

1. Introduction

Carbon Nanotubes (CNT) have received significant attention due to their amazing physical, mechanical, thermal and electrical properties. Utilizing their extraordinary strength and stiffness [1], they are used in very challenging high end applications in aerospace industry, in medical and electronic devices, etc., as the reinforcing agent for ceramic, metal and polymer matrix composites. Hence, it is of vital importance that the load transfer mechanism between the CNT and the matrix material is properly investigated and potential failure modes and locations be identified in order to use in such high performance nano composite materials in high end uses.

2. Literature Review

It should be noted that most studies reported in literature has considered carbon nanotubes to have isotropic elastic properties. But, carbon nanotubes are found to have anisotropic properties, i.e. different axial, hoop and radial Elastic modulus, shear moduli and Poisson's ratios. A number of computational studies have been carried out to obtain elastic constants in the case CNTs are considered to be transversely isotropic (Li, 2008).

An end-capped single walled CNT (SWCNT), which have a hemispherical end-cap of fullerene, are found to have spatially varying material properties from the fullerene end-cap to the normal SWCNT region. Anisotropic elastic moduli of different regions near the end-capped region of one end-capped (5,5) armchair and (9,0) zigzag CNTs have been reported by Herath *et al*., 2012. They have used nano mechanics models based on the assumption that inter-atomic covalent bonds act as linear springs.

3. Methodology

Analysis is carried out using ABAQUS finite element code for two cases (9,0) zigzag and (5,5) armchair SWCNT reinforced composites, considering, i) SWCNT to be isotropic and ii) SWCNT to be orthotropic.

For each case, the following materials were considered to be the matrices.

Table 1: Material used for matrices

Matrix	Young's modulus (GPa)	Poisson's ratio
Metal-Wrought Aluminium Alloy	70	0.33
Ceramic- Alumina	250	0.25
Polymer-Epoxy	3.3	0.40

(i) Isotropic case
In the (9, 0) zigzag and (5, 5) armchair SWCNT, Poisson's ratio of 0.17 was assumed for all regions of SWCNT whereas the following elastic modulus were assumed for C_{60}, transition and CNT regions respectively, (9,0) zigzag - 0.87 TPa, 1.080 TPa, 1.411 TPa (5,5) armchair - 1.358 TPa, 1.375 TPa, 1.398 TPa

(ii) Orthotropic case
The properties in the transition region vary along the length from end capped region values to normal CNT values. Reasonable elastic properties found in literature for C_{60} fullerene, CNT and C_{60} – CNT transition region are used.

The analysis was carried out as a displacement controlled test. For each isotropic and orthotropic case, finite element models are subjected to same displacement which is gradually increased until matrices reached their ultimate strength

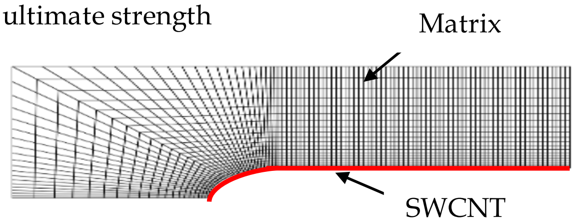

Figure 1: Finite element model for SWCNT reinforced composite materials.

4. Results and Discussion

The interface stresses and hence the high stress concentrated were locations identified in this study. For aluminium matrices, maximum value of von Mises stress and for ceramic and polymer matrices, maximum values of the principle stresses are reported in table 2 and table 3.

Table 2: Maximum stresses for zigzag SWCNT reinforced composites

Matrices r/f by zigzag SWCNT		Maximum Stress in the matrix (MPa)	Maximum Stress in the SWCNT (MPa)
Metal	Iso	395.2	241.4
	Ortho	399	361.9
Ceramic	Iso	405.2	237.6
	Ortho	398.5	344.6
Polymer	Iso	60.04	37.52
	Ortho	60.07	56.81

Table 3: Maximum Stresses for armchair SWCNT reinforced composites

Matrices r/f by armchair SWCNT		Maximum Stress in the matrix (MPa)	Maximum Stress in the SWCNT (MPa)
Metal	Iso	362.2	187.9
	Ortho	394.5	195.7
Ceramic	Iso	355.8	203.1
	Ortho	390.4	215.4
Polymer	Iso	59.51	36.74
	Ortho	60.02	35.69

The results indicate, the high stress locations are quite close in each case. But the maximum stresses developed in matrices and CNT for the same loading are higher in the orthotropic case than in the isotropic case for all types of matrices considered.

Considering about the stress distribution of both isotropic and orthotropic cases for (5,5) armchair and (9,0) zigzag SWCNT reinforced composites, always maximum stress is developed in the entrapped corner of the interface in the SWCNT and the matrix. Near the entrapped corner, gradient of the stress variation in the matrix and the SWCNT is high and at the end of the transition region, the stress variation is died down.

For (5,5) armchair CNT reinforced composites, stresses rise by 1% for metal and ceramic matrices and 0.05% for polymer matrix whereas for composites reinforced by (9,0) zigzag CNT, stresses rise by 9-10% for metal and ceramic matrices and 1% for polymer matrix. The increase in the maximum stress of CNT is 5% and 50%, observed in the orthotropic case.

5. Conclusions

From the results, the following conclusions can be reached.

i. The change in geometry at the end cap region rather than isotropy or anisotropy of the CNT, govern the general maximum stress location of the composite.
ii. Failure of composites reinforced with end capped CNTs is most likely to occur in the interface of C_{60} fullerene region of the CNT.
iii. The more realistic orthotropic description of CNT renders the composite material stiffer than the isotropic description.

Computational studies into predicting failure locations of such materials employing orthotropic CNTs have never been reported to the authors' knowledge. Moreover, an exact material property description of CNTs is yet to be pursued. It should be noted that the load transfer between the matrix and the CNT depend critically on the interfacial strength, which is governed by the bond between the matrix and the CNT. When it is week van der Waals bonds hold the CNT and matrix together, the interface is week and effective reinforcement by CNT cannot be fully reached. But formation of covalent bonds has proven to strengthen the interface. The results from this study suggest that the end cap region require relatively higher interfacial strength. Hence, the current work should be continued to analyse the effect of interfacial bonding conditions in the failure of CNT reinforced composites.

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Physical and Chemical Properties of Fly Ash based Portland Pozzolana Cement

D.L.N.B. Jayawardane, U.P.A.S. Ukwatta, W.M.N.R. Weerakoon and C.K. Pathirana

Keywords: Fly ash, Blended cement, Chemical properties, Physical properties

1. Introduction

Cement is a bonding material used with stones, sand, bricks, building blocks etc. consumed in the construction industry. It has a property of setting and hardening under by virtue of a chemical reaction.

Cement production is a significant source of global carbon dioxide (CO_2) emissions, making up approximately 5 per cent of global anthropogenic CO_2 emissions (Worrell et al., 2001). Pozzolan can be defined as a siliceous or alumino-siliceous material with very fine particles. In the presence of water, it reacts with calcium hydroxide released by the hydration of Portland cement at ordinary temperatures, to form compounds of possessing cementing properties. Supplementary cementing materials (SCM) are a class of mineral-based materials possessing pozzolanic reactivity such as coal fly ash (a residue from coal burning), blast furnace slag (a residue from iron making), or other pozzolanic materials (e.g., volcanic material). These products are blended with the ground clinker to produce a homogenous product: blended cement. Blended cement has different properties than Portland cement, e.g., setting takes longer but ultimate strength is higher. The global potential for CO_2 emission reduction through producing blended cement is estimated to be at least 5 per cent of total CO_2 emissions from cement making (Ernst Worrell, 2001).

2. Literature Review

A Number of researches have been carried out to find the properties of different types of cement. A survey of Portland cements marketed in North America was conducted in 1994 under the sponsorship of the American Society for Testing and Materials (ASTM) Committee C-1 on Cement (Gebhardt 1994). The primary purpose of the survey was to provide data on modern cement characteristics. The study reviewed 387 cements from 136 of 140 cement-producing facilities in the United States and Canada, plus several imported cements. The last survey of similar scope on cement characteristics was done on 203 cements procured in 1953 and 1954 by the National Bureau of Standards (presently called the National Institute of Standards and Technology) (Gebhardt, 1994).

In April 2005, a survey of manufacturers of Portland and blended cements in the US and Canada was conducted to collect data on cements produced in 2004. The characteristics of interest include those required to meet specifications ASTM C 150, C 595, and C 1157: physical measurements such as strength and fineness, chemical composition, and performance characteristics such as setting time. It was also comparing and contrasting results on Portland and blended cements (Tennis, 2005).

Compressive strength, fineness, soundness, setting time, and specific gravity are some physical properties of cement. Loss on ignition, chloride content, and insoluble residue are some of chemical properties of cement. Also percentage of SiO_2, Al_2O_3, Fe_2O_3, CaO, MgO, SO_3, Na_2O, and K_2O of cement can be determined. Variation of physical and chemical properties of Ordinary Portland Cement (OPC) and blended pozzolana cement has been discussed in this paper.

3. Methodology

With the blended cement, OPC sample was also tested according to the standards. ASTM and SLS test methods were used to find out above physical properties and chemical properties. The test method used to find out each property is given in Table 1.

4. Results and Discussion

Results obtained for Physical properties and Chemical properties of Cements are as follows.

4.1 Summary of Results

Obtained values for physical properties and Chemical properties are shown in table 2 and table 3, respectively

Table 1: Test Methods use to find out properties of Cement

Property	Test Method
Compressive Strength	ASTM C 109/C 109M
Setting Time	ASTM C 191
Specific Gravity	ASTM C 188
Fineness	ASTM C 430
Soundness, Al_2O_3Content, MgO Content, CaO Content and Chloride Content	SLS 107
Loss on Ignition, Insoluble Residue, Total alkalis, SiO_2 Content, Fe_2O_3 Content SO_3 Content	ASTM C 114

4.2 Chemical and Physical Property Requirements of Cement

After finding of these properties it was checked whether they satisfy the requirements given in ASTM C 150 and SLS 1247. According to the results obtained, the cement certifies to be called 'Blended'.

Table 2: Physical properties of Blended Cement and OPC

Property	Results	
	Ordinary Portland Cement	**Blended Cement**
Compressive Strength (MPa)		
3 Day	11.3	10.7
7 Day	13.2	14.3
28 Day	16.9	21.2
Setting time (min)		
Initial	120	164
Final	166	203
Specific Gravity	3.107	2.936
Fineness %	85.4	86.2
Soundness (mm)	0.5	1.0

4.3 Discussion

When consider about the results of compressive strength, Portland cement has higher early strength and Blended cement has higher later strength. The setting of Blended cement takes longer than Portland cement. Because of fly ash fineness of Blended cement becomes higher. Blended cement has low amount of CaO compare with Portland cement, resulting less CO_2 emission to the environment.

Table 3: Chemical Properties of Blended Cement and OPC

Property	Results	
	Ordinary Portland Cement (%)	**Blended Cement (%)**
Loss on Ignition	2.05	1.05
Insoluble Residue	4.1	20.0
Total alkalis	0.59	0.71
Chloride Content	0.07	0.01
SiO_2 Content	28.7	23.5
Al_2O_3 Content	13.5	12.9
CaO Content	53.6	47.0
MgO Content	2.21	1.74
Fe_2O_3 Content	2.27	2.04
SO_3 Content	2.9	2.21

5. Conclusions

According to the measured physical properties and chemical composition of fly ash based Portland pozzolana cement manufactured by Tokyo Cement PLC, it was found that the fly ash based blended cement has the properties as which required for a cement to be called 'blended cement'. As the blended Cement reduces CO_2 emission, and improves strength, workability, and durability it will be very effective in green building construction.

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Progressive Buckling of Steel Columns

A.A.H. Ahamed, A.L.M. Hakees, M.I.M. Mufras and A.L.M. Mauroof

Keywords: Progressive buckling, Energy absorption, Slenderness ratio

1. Introduction

Static load carrying capacity of a steel column is estimated using column curves in relevant standards. When the impact load is axially applied on a steel column, the load carrying capacity and the collapse mode will change due to the effect of inertial force in that member. This experimental study focused on finding the relationship between collapse mode and axial impact load of cold form square hollow steel columns. Specimens with medium slenderness ratios (30, 32, 34, 36) and width of 19.4mm are used. The energy absorption and number of folds analysed according to the change of input energy.

The past studies showed that the progressive buckling of the steel tubes depend on these parameters.

- Slenderness ratio
- Width/Thickness ratio
- Impact mass
- Dropping height

2. Experimental set-up

The following tests were carried out. First the tensile test was carried out to evaluate the mechanical properties of the square tube. Three specimens were cut according to the standard BS18: Part2: 1971. Then the static compression test was carried out to find ultimate compressive strength of the square tube. 12 samples with three thicknesses of square tube were tested. Those were 0.7mm, 1.0mm and 1.2 mm. According to the results 30% and 50% of ultimate compressive loads were calculated to find impact mass for each tube. The experimental limitation is, we can apply 600kg as the maximum impact mass but 30% and 50% of 1.2 mm and 50%of 1mm ultimate loads exceeds the limitation.

Finally the impact load test was carried out to identify the behaviour of the columns under axial impact load. 24 samples were tested with slenderness ratios of 30, 32, 34 and 36.In this test dropping height was changed, energy absorption and number of folds was calculated.

3. Calculations

Experimental energy absorption per fold was calculated using this equation

$$S = (L1+L2+L3+L4)\pi M_p + 2M_p\pi h \quad \text{.......... Eq. 1}$$

where,
$M_p = \sigma_y t^2/4$
σ_y= yield strength
b = width of the tube
t = thickness of the tube
h= fold height (shortening/No of fold)
L1, L2, L3 and L4

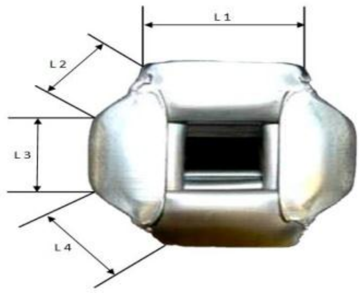

Figure 1: Parameters of the columns

4. Experimental results

Average yield strength of the square tube is 330MP. Results from the static compression test are mention below; all tested specimens undergo overall buckling.

Figure 2: Results from the static compression test

After the impact load test it was observed that all specimens have under go to progressive buckling as in figure 3.

Figure 3: Results from the impact load test

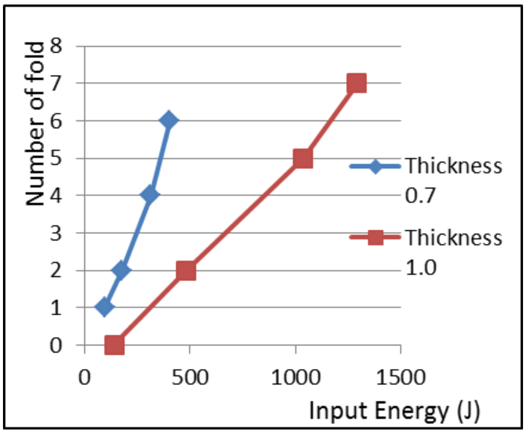

Figure 4: Variation of number of fold with input energy for slenderness ratio

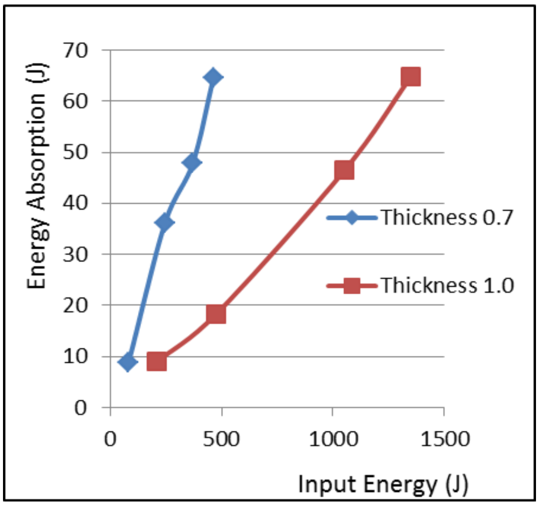

Figure 5: Variation of energy absorption with input energy for slenderness ratio 32

5. Conclusion

When the input energy increases for a particular slenderness ratio and thickness, the energy absorption and number of folds varies linearly.

When the slenderness ratio increases for particular input energy and thickness, the energy absorption and the number of folds are nearly equal, so we can say that the energy absorption of the square tube does not depends on the slenderness ratio.

When the width/thickness ratio increases for every input energy and every slenderness ratio, the energy absorption and the number of fold increase.

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Shear Strengthening of RC Beams Using External Steel Reinforcement

U.I. Samarasinghe, N.K.L. Rathnayake, K.A.D.C. Sampath and H.D. Yapa

Keywords: RC beams, Residual strain capacity, Shear retrofitting

1. Introduction

Shear failures in reinforced concrete (RC) structures are brittle, sudden and occur without giving any warnings. Application of potential remedies to overcome such risks associated with shear deficient structures in-service is important. The use of shear retrofitting methods has been identified as a better solution in this regard when compared with other possible options including re-construction and imposing load limits. Steel and fibre reinforced polymers (FRPs) are amid popular material options for shear retrofitting each of them possesses advantages and disadvantages mainly in terms of cost, durability, methods of anchoring. Thus, for local use, steel can be identified as a better material than FRP if it is carefully used for shear strengthening such that potential to corrosion is controlled. The current project investigates a possible way of using steel as an external reinforcement for efficient and effective shear retrofitting.

2. Literature Review

A literature survey is carried out explore the state of the art of shear retrofitting. An investigation conducted by Adhikary & Mutsuyoshi (2006) shows that the use of externally anchored steel stirrups running around a RC beam as an effective shear retrofitting technique. Kesse & Lees (2009) shows through an experimental investigation a possible way of using pre-stressed Carbon FRP (CFRP) straps externally to enhance the shear capacity of RC beams. They point up that strap spacing, stiffness and prestress level notably affect the ultimate shear strength. Hoult & Less (2009) has investigated on an under-slab strap installation technique for T-beams where holes drilled through the element had been used to insert the CFRP strap. This investigation exhibits that shear retrofitting can be implemented without top slab access.

3. Proposed Retrofitting Method

A method of shear retrofitting is proposed where steel is used as an external reinforcement whilst the potential to corrosion is less when compared with using them fully exposed to the surrounding. In this proposed system (see figure. 1), steel pads and bolts are used to anchor the steel bars at the top and bottom of the beam while holes are drilled through the beam to insert the external reinforcement. It is possible to apply prestress to the retrofitting steel which can be beneficially used towards efficient shear strengthening. Since the external steel is fairly confined, the retrofitting material is possibly vulnerable for less corrosion.

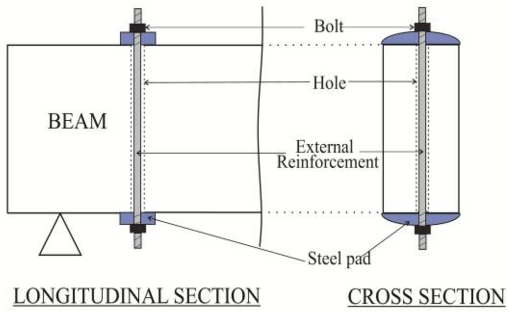

Figure 1: Proposed retrofitted method

4. Methodology

A numerical analysis is carried out for a selected case study using software called Response 2000. The investigation shows that a significant shear enhancement can be achieved for RC beams using the proposed method. Hence, in order to further test the potential of the proposed retrofitting system, an experimental investigation is conducted.

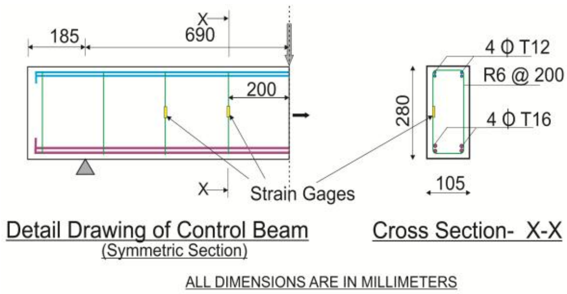

Figure 2: Detail drawing of control beam

The experimental investigation consists of two rectangular RC beams where one is a control beam and the other is a retrofitted beam. Figure 2 illustrates the details of the retrofitted beam. For the external reinforcement in the retrofitted beam, an initial pre-stress (25%) is applied.

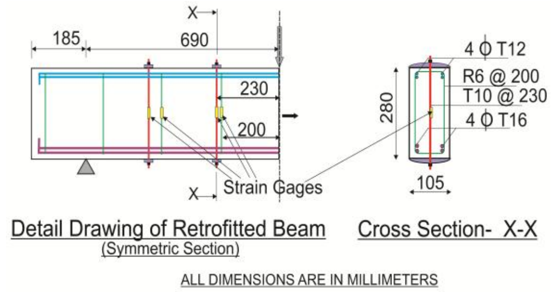

Figure 3: Detail drawing of retrofitted beam

5. Results

Table 1 tabulates the results of analytical and experimental investigation. The experimental cylinder strength for the control and retrofitted beams is 42 MPa and 40 MPa, respectively. The retrofitted beam experimentally exhibits a 29% shear enhancement. The ratio of V_{exp}/V_{theo} for the control and retrofitted beams are 1.19 and 0.97, respectively, and thus, the pattern of the results is not consistent. Hoult & Lees [2009] has shown that the Response 2000 usually slightly underestimates the shear strength for un-retrofitted beams. Similarly, the shear strength for the control beam is calculated conforming to EC 2 the prediction is 68.3 kN. It can therefore be argued that the prediction for the retrofitted beam is the inconsistent result. In addition, even though the retrofitted beam failed in shear in the experiment, flexural failure has been predicted for the beam.

Table 1: Results

Beam Type	V_{exp}/kN (failure mode)	V_{theo} / kN (failure mode)	
Control Beam	83.2(shear)	69.8(shear)	
Retrofitted Beam	107.4(shear)	f_{y1}=500 MPa	110.3 (flexure)
		f_{exp}=222 MPa	103.9 (shear)

As discussed, it seems that the Response 2000 predictions are not very much accurate, and therefore, possibilities for such discrepancies are studied. The experimental strain readings (shown in figure 4) show that the average strain in the external reinforcement is 222 MPa. Hence, this average experimental value is used as the ultimate stress instead of the actual yield stress (500 MPa) of external steel in the Response 2000 analysis. It is found the results from such refined analysis are in better consistent with the experimental results. Further investigations on this phenomenon is necessary before making a solid conclusion in regard of the potential of using Response 2000 to predict retrofitted RC beam behaviours.

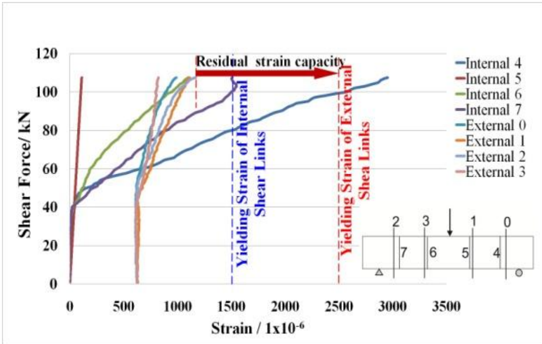

Figure 4: Strain in retrofitted beam

Figure 4 futher reveals that a notable amount of residual strain capacity is left with the external retrofiting reinforcement. This suggests that it is important to identyfy the optimum prestress level to be applied on the external reinforcement in order to obtain the best use of the proposed retrofitting system.

6. Conclusions

The experimental results show a significant shear enhancement (30%) for the retrofitted beam due to the use of external reinforcement. A somewhat discrepancy between the theoretical and experimental failure loads and modes is observed. However, by using the experimental ultimate stain in the external steel in the Response 2000 analysis, the observed discrepancy is decreased. Further investigations on this phenomenon are recommended. The experimental measurements point up that the retrofitting steel is left with considerable amount of residual strain capacity at failure. Hence, the use of an optimum prestress for the external steel is important to obtain the maximum shear enhancement and the maximum use of the retrofitting material.

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STRUCTURAL ENGINEERING

Structural Assessment of Reinforced Concrete Building

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Keywords: Ground penetration radar

1. Introduction

With the developing of new technologies lot of new concepts had been found in civil Engineering industry. As a result, the various types of complex structures have been constructed. With the aging of the structure, there has been change in resistance such as structural deterioration, structural damage due to accidentals actions, changing in loading, and extension of the design working life. Therefore, the assessment of existing structures, especially buildings is increasingly important to determine whether or not a distressed building should be demolished to build back or whether it will be cost effective method to repair or retrofitting, considering overall safety.

2. Literature Review

2.1 Assessment classification
In general, assessment procedures can be classified into three groups: measurement based assessment, model based assessment and non-formal assessment.

2.2 Assessment levels
Assessment may be carried out in different levels according to the building condition and system. Assessment procedures vary in sophistication. It is recommended to start the assessment with simple but conservative low level methods and, in case the assessment failed, move on with more refined upper levels. The grading concerns the specific methods of all above three components

2. 3 Condition assessment
Main objectives of condition assessment are to place the building into one of the following three categories.

- **A.** The building has not shown any signs of distress and it satisfies all the safety and serviceability requirements according to the codes of practice; hence no action is needed towards retrofitting.
- B. The building is seen to be deficient but it can be repaired and strengthened to satisfy the code safety requirements or performance criteria set by the user.
- C. The building is badly damaged. It is to be demolished and a new building may be built, build back better.

2.4 Ground penetration radar
In this research it is given a newest method for assessment of the same problems related with the concrete reinforced structures mentioned above, is called Ground Penetration Radar (GPR). The applications of GPR for concrete structures assessment is little new method for Sri Lanka. But it gives more accurate results related to the structural defects such as rebar placing errors, inside variation of the concrete etc.

3. Methodology

Structural assessment of reinforced concrete building is carried out in two levels.

i. Preliminary investigation
ii. Detailed investigation

3.1 Preliminary investigation
In preliminary level, it is expected to collect basic information that can be easily gathered from simple methods. That process includes three steps.

i. Preliminary inspection
ii. Collection of building data
iii. Visual inspections

3.2 Detailed investigation

3.2.1 Detailed defects investigation
The significant defects which are selected from visual inspection have to be investigated and to be analysed in detail.

3.2.2 Detailed element investigation
The properties of the defected elements which are identified from previous step are evaluated using various test methods.

4. Results and Discussion

4.1 Analysis of beams
<u>Calculation of tensile reinforcement</u>
At mid span
Calculated reinforcement area = 184.4 mm^2

Provided reinforcement area = 603.4 mm^2
With those results it was found that the provided area of reinforcement was sufficient to carry the loads.

Calculation of cracking moment
Moment at mid span = 6.77 kNm
Cracking moment =18.7 kNm

4.2 Analysis of the cracks
The analysis was done based on the code of "Guidebook on non-destructive testing of concrete structures". According to that book, all cracks we identified in Preliminary investigation need to be repaired.

4.3 Analysis of the slab
Calculated reinforcement area, = 534 mm^2
Provided reinforcement area = 1946mm^2

When comparing the areas of the reinforcement in the slab the provided reinforce area was sufficient to carry the loads.

4.4 Analysis of the column
There were not any defects identified from the column during the data collection process. The rebound was given the high strength class for the column. Therefore all the columns were in satisfied condition.

4.5 Bearing capacity check
Applied stress on the soil= 206.3kN/m^2
Ultimate bearing capacity =814 kN/m^2
With the considering of the observed results, the upper side of the foundation (at the cut) not failed. Therefore the upper side of the foundation was not in danger.

4.6 Ground Penetration Radar data analysis
According to the results that has been taken from the first survey of the building was identified there were lot of hard components in the soil, because the resulting graph was given lot of reflections at the footing zone. Also this graph was helped to identify the soil profile variation, depth of the bed rock, depth to the footings, and size of the footings etc.

According to background information, this location consists a lot of rubbles and other materials [building] in the soil due to filling. It can be seen that the point reflections shown in the above figure are potentially from those rocks and other rubbles/wastes.

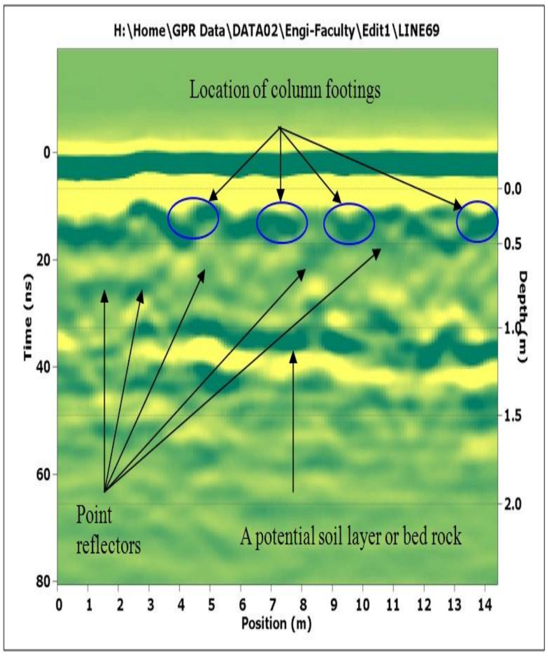

Figure 1: Observed ground profile from GPR survey

5. Conclusion

5.1 G P R test
It was given that the slope side ground of the building was in some improper condition. The graph of this survey was given more reflections. Also it was difficult to get the core sample for the direct shear test. That means the bearing capacity of the soil can be reduced. Because of that, the slope side of the building is undergoing differential settlement. Also it was given that the bed rock was very shallow. Therefore that also can be effect to the sliding of the foundation.

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Assessment of an Existing Corroded Steel Railway Bridge

S. G. C. L. Subhawickrama, S.G.P.M. Hemachandra, H.G.M.S. Edirisingha and P.B.R. Dissanayake

Keywords: Railway bridge, Corrosion, Finite Element Model, Validation

1. Introduction

Most of the Sri Lankan railway bridges have been built at the end of the 19th century, so dealing with old metal bridges can mean an age of nearly 100 years. Most of the bridges are still in operation after damages, several phases of repair or strengthening. They may have been subjected to deterioration, mechanical damages, repair or change of use. The primary cause of such deterioration is loss of steel due to corrosion. Railway lines in coastal areas are highly exposed to corrosion. This paper presents a methodology for condition assessment of a steel railway bridge situated in northern coastal belt in Sri Lanka.

The selected bridge is Holcim Bridge No.2 which is a single span (33.65 m) of semi-thorough double lattice girders. The timber abutments are temporary constructions. The cross girders are placed at the panel points of the main girders and also at the canters of each panel. There are two longitudinal girders connected between cross girders in each panel. There are bracings between main girders. Girders are supported on the bottom chords of the main trusses and they are placed at 2.33 m interval in the longitudinal direction of the bridge.

This bridge is constructed over a natural water body which is closer to the sea and the water level increases up to a greater amount affecting the structure. Hence, the structure is exposed to extreme environmental conditions.

2. Methodology

Based on the literature review, we developed a methodology to follow which will guide to assess the capacity of the bridge with the development of numerical model and at the end, to suggest suitable retrofitting mechanism.

2.1 Condition assessment

To assess the present condition of the bridge, a detailed survey was carried-out after the studying of available data. It was observed that the loading on the bridge consists of Dead and Live loads. The Dead loads are due to the self weight of the components of the bridge and the Live loads are mainly due to the weight of the train which travels on the bridge.

2.1.1 Corrosion of members

The corrosion is observed in the bridge deck, main girder and the braces. In the top chord, corrosion was relatively low, but very high at the bottom chord.

Figure 1: Corrosion on the bridge members

2.2 Field load testing

For a bridge assessment, we use a theoretical model and hence it needs to be validated .For this, it is necessary to have experimental data. For an existing bridge, either dynamic or static load tests or both together can be used previously done load testing were given. The test had been done for 5 loading cases representing dynamic and static conditions.

2.3 Laboratory testing of materials

Due to lack of data regarding the Material properties, we had to conduct a Standard Tensile Test to observe material properties. Extracted samples from site visit were used to prepare test specimens.

Table 1: Results of the tensile test

Parameters	Specimen 1	Specimen 2
Yield strength (MPa)	218	176
Ultimate tensile strength (MPa)	380	368
Elongation (%)	32.5	32.9

Based on those results, the material was identified as Wrought iron and hence Modulus of elasticity of the truss members ware select as 193 Gpa.

3. Numerical Analysis-FE modeling

For the structural analysis of the bridge using FE method, we employed the general purpose package SAP 2000. A full three dimensional FE model was used in the analysis. The bridge was analysed as a rigid-jointed Frame structure.

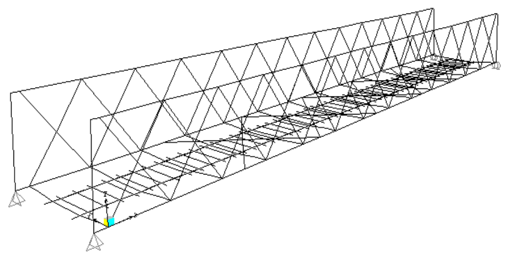

Figure 2: FE model of the Bridge

4. Validation of FE model

The results from the analytical analysis were compared with those obtained from the field measurements. The vertical and horizontal displacements and normal stresses were calculated at the same locations where the gauges were placed. Validation was based on the reduction of sectional areas of different members according to their severity

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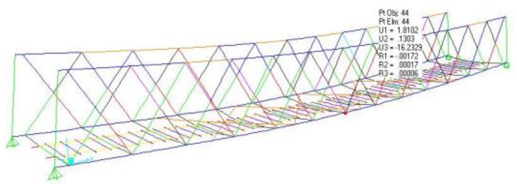

Figure 3: Validated FE model of the Bridge

Table 2: Cross sectional area reduction

Member	Reduction of percentage cross sectional area due to corrosion
Top cord and bottom cord	16%
Brace inside and brace outside	6-18%
End post	10%

5. Results

The results of displacement comparison are tabulated here. The two values are given corresponding to before and after validation process.

Table 3: Displacement comparison - Locomotive at ½ Length of the Bridge

Gauge	Experimental Value/(mm)	FEA Value/ (mm)	
		Before	After
Horizontal	2.91	1.55	1.81
Vertical	18.17	14.17	16.00

Table 4: Displacement comparison - Locomotive at ¾ Length of the Bridge

Gauge	Experimental Value/(mm)	FEA Value / (mm)	
		Before	After
Horizontal	1.46	0.99	1.15
Vertical	15.36	12.01	13.62

6. Retrofitting of the Bridge

After the validation, the analysis for the new load combinations involving higher engine weight types can be done to assess the Strength, Stability, and Functionality of the Bridge. According to the results, it was identified that the strengthening in some members are necessary. The critical members in the bridge were generally the Bottom chord, Secondary beams (cross girders) and Bracing members.

6.1 Bottom Chord

There was considerable corrosion throughout the length of Bottom chord and there is a high potential for future corrosion also. Therefore, we can introduce new additional plate to the Bottom chord. It can be installed by welding. The figure below shows the retrofitting work.

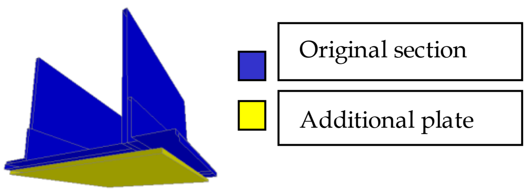

Figure 4: Retrofitting of Bottom Chord

6.2 Secondary beams and brace members

Although, these members look severe in corrosion at its first glance, the analytical results show that the bridge is not so dramatic condition. Therefore, their retrofitting work can be done by substituting the corroded elements by new additional plates

7. Conclusions

From this validation which was based on the area reduction method to consider the effect of corrosion, it is observed that the experimental and numerical results give almost equal figures.

Therefore, it can be considered that the model behavior can be considered as equal enough with the actual structure.

However, the effect of corrosion is typically a local phenomenon that does not play a big role on the global response of the structure. But because of been a local phenomenon, it has to be carefully take into account in the assessment of plates and nails (bolts) and towards retrofitting.

In the stage of designing the retrofitting and strengthening work, the engineering judgment of professional bodies should be considered rather than depending on the numerical results.

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Problems Related To Telecommunication Tower Foundations

A.M.G.S. Abeykoon, H.A.K.S. Hettiarachchi, H.C.L. Hewage, D.J. Gallage and U.I. Dissanayake

Keywords: Foundation, Uplifting, Undercut, Model, Friction angle, Unit weight

1. Introduction

Most widely used Telecommunication Tower type is four leg green field towers. These towers consist of Steel super structure & Reinforced concrete foundation. In design stage, foundation is checked for Bearing, Overturning, Uplifting, Sliding failure etc. & dimensions are provided to satisfy all the failure modes. These failure modes occur due various forces acting on to the towers. Wind is very frequently acting force & due to that tower foundation is failed by uplifting. This project describes how to economise the tower foundation by considering this failure mode. Three models were prepared on the ground and uplifting force was provided by using suitable loading arrangement. Then the actual failure load was compared with design Uplifting capacity.

2. Literature Review

For the design of foundation Friction angle (Φ) and Unit weight (γ) are required. When the foundation fails the failure plane makes an angle equal to Φ with the vertical plane. But the actual failure plane can be different. Typical Uplifting failure pattern shows in below figure.

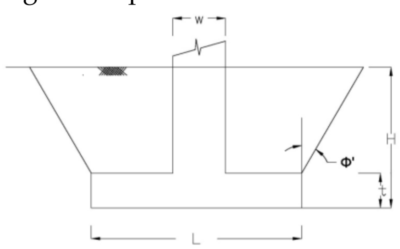

Figure 1: Dimensions of models

Generally following equation is used to calculate uplifting capacity of the foundation.
Uplifting Capacity = Weight of the soil + Weight of the foundation

It is obvious that uplifting capacity can be increased by increasing weight of the soil in the failure region. So, failure plane can be increased by providing undercut. So in this study, tried to find out how much this failure plane can be increased by providing undercut. In order to find the effect of the undercut model, testing needs to be carried out. Dimensional analysis can be used to convert the results from model to prototype. For that "Bakingham pi theorem" can be used. In this test, Area of the footing (A) and Depth of the footing (D) are selected as geometric properties, Uplifting force (P) and gravitational acceleration (g) as external effect and soil cohesion (C), unit weight of the soil (γ) and Friction angle (Φ) as surrounding effect can be considered.

So following non dimensional groups were prepared.

$$\pi_1 = \frac{P}{\gamma \times D^3} \qquad \text{..............Eq. 1}$$

$$\pi_2 = \left(\frac{A}{D^2}\right) \qquad \text{..............Eq. 2}$$

Using same scale in horizontally and vertically in prototype and model π2 can be kept constantly. So by using following equation maximum uplifting capacity of the prototype can be calculated

$$P_{prototype} = \left[\left(\frac{P}{\gamma \times D^3}\right)_{model}\right] \times \left[(\gamma \times D^3)_{prototype}\right]$$

..............Eq. 3

3. Methodology

3.1 Modal Preparation

Using design uplifting force, three different prototypes foundations were designed. Then those were scaled down by using 10:3 scale factor.

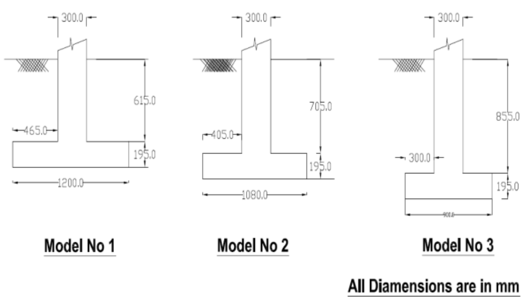

Figure 2: Dimensions of models

Then those modal were constructed on the ground.

3.2 Experimental set-up

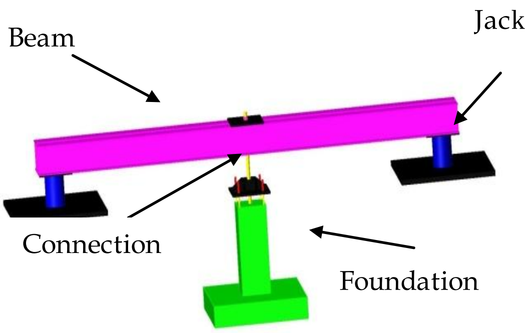

Figure 3: Loading set-up

5 m long I – beam and 25, 20 ton hydraulic jacks (least count 200 kg) were used to provide uplifting force for modal foundations.

First 1 m x 1 m x 9 mm two steel plates were laid on the ground where the jacks should be placed. It was spread for the reaction of the jacks. Then layers of concrete cubes were placed over the steel plate. After that jack is placed on the cubes & beam is placed on top of it. Then the steel plate which is inserted through the reinforcement of the column & steel beam was connected using the steel rod. Then the pumps were connected to the jacks & before applying force air voids in the jacks were removed. Then the load was increased by 400 kg intervals. This load held for 60 seconds to transfer to the surrounding soil.

So, this load increment was done until the foundation failed. Failure of the foundation can be identified in 2 ways.

- Visible Cracks appeared around the soil
- Dial gauge reading will not further increase while applying the pressuree

4. Results and Discussion

Table 1: Expected and Actual failure load of the models

Model No	Expected Failure load (kN)	Actual Failure load (kN)
1	36.31	113.47 kN
2	34.61	87.82 kN
3	35.48	104.92 kN

The above table shows the calculated failure load and actual failure load for 3 models Failure load of the prototypes are shown in below table.

Table 2: Expected and actual failure load of the prototypes

Model No	Expected failure load (kN)	Actual failure load (kN)
1	1272.8	3996.7
2	1234.9	3133.0
3	1265.9	3743.0

All the actual failure loads is greater than the expected load. But in the model No 2 there were some problems when loading. So it doesn't give actual failure load. Cracks were observed away from the foundations. So there is some effect of the undercut when foundation fails.

5. Conclusions

According to the results failure loads are greater than expected values. Major crack line between compacted soil and undisturbed soil shows that no proper bonding between compacted soil and undisturbed soil. Other diagonal cracks proved that applied uplifting load acts on the undisturbed soil.

Foundations were designed assuming that there is no considerable contribution of soil cohesion to resist uplifting force. In failure, rods of the jacks were coming out without considerable amount of decrease in the load. That is because only soil weight is acting on the foundation. If there is a considerable effect of soil cohesion, there must be reduction of dial gauge reading due to soil cohesion when separating the soil wedge.

Actual failure load is greater than the value of calculated weight of soil in cracking zone, because, Load may be applied away from the cracking zone and that load may not be enough to make the cracks, and because load may gradually reduce and at one point it would be zero. The cracks are visible only when applied load is equal to ultimate load. The point where load become zero cannot be observed.

In this loading pattern soil is not in directly in Active or Passive ranking state. So foundations were designed assuming that, angle of soil wedge which is affected on the footing equal to the friction angle of soil. If soil in active ranking state the angle of failure plane should be equal to 45°-Φ/2 (28.5°). If soil in passive ranking state the angle of failure plane should be equal to 45°+Φ/2 (61.5°). Resulting angles of failure planes of Model No1 and Model No 3 are in between 28.5° and 61.5°. So this failure pattern is in between active and passive ranking states.

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Vibration Measurement of Existing Building Due to Piling and Its Simulation

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Keywords: Ground vibration, Compression wave, Shear wave, Rayleigh wave, ABAQUS

1. Introduction

Vibration is a kind of problem not only in engineering and industry but also day to day life activities. There are cases of fail are in product, structure or disturbance to the work due to excess vibration. It is induced in several ways. In most cases it is failed to stop the vibration. But it can be controlled or reduced. In civil engineering it may cause defects in structures and delay the construction work. Vibration is a major task in high-rise buildings, because the high rise buildings reflect more than normal buildings to vibration. When a high rise building is under vibration, the top floor will get more amplitude than lower floor under single degree of freedom. So, people do not like to stay in upper floors. Not only annoyance to peoples but also can cause damages to cosmetic items and structures. Therefore the vibration in building is big task especially in high rise buildings.

2. Literature Review

When a hammer strikes the pile, energy travels through the pile. Some amount of energy is used to penetrate the pile into soil layer. While energy travels through the pile, some amount of energy is dissipated due to friction along the soil-pile interface. So the remaining energy of the blow on the pile causes wave propagation in the surrounding ground.

As the bearing capacity of piles is composed of skin friction and end bearing, the wave generation in pile is similar such that the waves are generated by two mechanisms; shear waves (S-waves) along the shaft and compressive waves or primary waves (P-waves) at the pile tip.

These are the primary type waves in ground vibration studies. These waves transmit vibrations through soil medium which they travel. Shear waves are generated along the pile skin by relative motion between the pile and surrounding soil and shear waves propagate from the pile shaft on a conical wave front for record purposes. At the toe of pile compressive waves are generated and it propagate into soil on a circular wave front. When the P-wave and S-wave reach the ground surface, part of their energy is converted to surface waves known as Rayleigh waves, and part is reflected back as P- and S-waves. Rayleigh wave has the largest potential to cause vibration on buildings those which foundations are placed near the ground surface.

3. Methodology

The steps followed to model the problem in ABAQUS are given below;

i. The geometry of the model is created
ii. The material properties are entered
iii. The finite element mesh is generated
iv. The interaction surfaces are selected
v. The boundary conditions are applied
vi. The initial conditions are entered
vii. The loading and time stepping are entered
viii. The model is solved.

3.1 Analytical Rigid Surface

In an axisymmetric finite element model, to allow the penetration of the pile into the soil, the defined nodes on the axis of symmetry should be set free of constraints. To remove those constraints and allow the soil nodes at the contact to slide on the surface of the pile elements, it is suggested to define an analytical rigid surface 1 mm away from the axis of symmetry. The rigid analytical surface that is available in ABAQUS is in frictionless contact with the pile and soil elements. This technique allows the pile to slide over the rigid surface, and the soil elements to separate from this surface during the penetration of the pile.

3.2 Artificial Non-Reflecting Boundary

In the analysis of stress wave propagation in soils due to pile driving, the reflection of waves from the far boundary causes significant problems. When these waves bounce back from the boundary, they mix with the progressing waves. Thus, the magnitudes of the waves calculated by the FE package become inaccurate. To minimize the effect of reflecting waves on the results, another alternative to damp out the excessive vibrations in a finite element model is to define an artificial boundary which damps out the entering waves. The purpose of this boundary is to minimize the reflection of shear waves from the far boundary.

3.3 Load and Boundary Conditions

The Load module allows you to specify loads, boundary conditions, and predefined fields. Loads and boundary conditions are step-dependent objects, that you must specify the analysis steps in which they are active; some predefined fields are step-dependent, while others are applied only at the beginning of the analysis.

To applying the time varying force to the pile top in ABAQUS, the forcing function is used and the maximum force to apply to the pile top is identified from the site data.

4. Results and Discussion

The amplitudes of ground vibrations during pile driving are dependent on the so many parameters. More specifically, the soil type and the released hammer energy have significant effects on the transmission of ground vibrations in the soil. The selection of these parameters is the key to the success of a numerical model to predict accurate and reliable vibration records.

Three different soil types consisting of loose sand, medium stiff clay and dense sand are defined in the finite element model to assess the effect of soil strength on the ground vibrations. 20m long and 25m depth soil layer was considered for this analysis. The dimensions and pile parameters remain same for all analysis. The analysis was done for each value of hammering energy by changing the properties of soil as loose sand, dense sand and medium stiff clay are shown in figures bellow

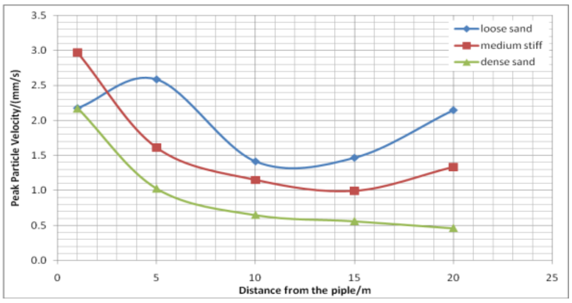

Figure 1: Variation of PPV value with distance for hammering energy of 2000kN

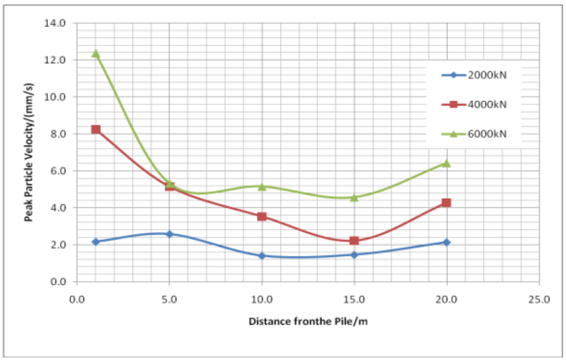

Figure 2: Variation of PPV value with distance for loose sand in different energy

5. Conclusion

When hammering energy is 6000 kN, the loose sand has the greatest PPV with a magnitude of around 12.5.0 mm/s followed by the medium stiff clay with a PPV of 9.0 mm/s. In densesand, the maximum PPV is around 6 mm/s. For all hammering energy, the loose sand has the greatest PPV except the case of hammering energy is 2000 kN where medium stiff clay has the greatest PPV with a magnitude of around 3 mm/s.

The greater amplitudes of vibrations are encountered on the ground surface. This is due to less energy dissipation in the ground during the propagation of ground vibrations in the soil layers.

The peak forces are selected as 2000 kN, 4000 kN and 10000 kN. It is observed that the increase in hammer energy causes increase in the peak particle velocities; these increments are not linearly proportional with the magnitude of the applied impact force. It is also observed that particle velocity decreases with the distance from the pile.

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HIGHWAY AND TRANSPORTATION ENGINEERING

Criteria for Setting Speed Limits for Sri Lankan Road Ways

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Keywords: Road characteristics, Speed Limit, 85th percentile speed, Speed zone

1. Introduction

Speed limits are used to regulate the speed of road vehicles for safety reasons. There is no blanket speed limit for many roads in other developed countries. The speed limits are decided by considering many road characteristics such as road class, road width, shoulder condition, road markings, etc. Although, Sri Lankan road network is prevailed only with blanket speed limits. And there are roads where users can't adopt to the current blanket speed limit, because of factors like road surface condition and topography of the road are subjected to change abruptly along the road sections. Thus, this research is expected to evaluate whether the current blanket speed limit is the best approach to be used in Sri Lanka or if not and to propose necessary guidelines for setting speed limits for different types of roadways.

2. Literature Review

Authors were unable to find any studies or researches on criteria for setting speed limits for road ways in Sri Lanka. However, other countries have carried out many studies and research in this area. Many research propose 85^{th} percentile speed to set speed limit for selected roadways. [Parker (1992), Juan and Janice] Generally a speed limit is given as a blanket speed limit, a speed zone or a specific speed limit for a particular roadway with certain features.

Some adjustment factors such as access density, road class, lane width and etc. were applied by Dissanayake and Jian (2003) to an ideal speed which limit based on actual conditions at the selected site. It was found that the model developed in the study predicted speed limits more realistically than using 85^{th} percentile speed solely.

Dissanayake and Liu (2009) stated that speed zones can be suggested for potentially hazardous location on gravel roads and the 85^{th} percentile model can be applied establishing speed zones on gravel roads.

3. Materials and Methods

According to scope of this research several steps were identified which had to be performed in order to complete the project. Those steps were; (a) select at least two roads with similar characteristics (b) collect road geometric data, speed data and traffic data (c) analyze data to find 85^{th} percentile speed and find current speed of vehicles. Finally based on analysis results to decide whether the current blanket speed limit is suitable or otherwise propose guidelines for setting the speed limits.

3.1 Site Selection

Three road sections were selected to check whether they are with similar characteristics. Those are; (1) Kandy – Jaffna A9 Road (from 67 to 70 km) at Dambulla (2) Katugastota – Kurunagala A10 Road (from 17 to 20 km) at Weuda area (3) Kurunagala – Puttlam A10 Road (from 59 to 62 km) between Wariyapola and Padeniya. Site information was collected.

Table 1: Road geometric data and AADT data

Factors	Road name		
	A9 Road (From 67 to 70 km)	A10 Road (From 59 to 62 km)	A10 Road (From 17 to 20 km)
Single lane width / (m)	3.10	3.55	3.65
Number of Pedestrian crossing(s)	1	1	5
Number of defected area(s)	3	no	no
AADT data (RDA)	5926 (2007)	9527 (2008)	8528 (2007)
Shoulder condition	Poor	Well Paved	Well Paved

3.2 Collection of Data

Field speed data were collected using radar guns at several road sections on weekdays. In obtaining the speeds of vehicles, it was ensured to collect only the speeds of free flow vehicles with the standard headway.

Besides a questionnaire survey was conducted in order to find out attitude of drivers who use those road sections frequently.

4. Data Processing and Analysing

The collected speed data was divided into speed ranges and frequency was listed which suits to the particular speed range. These frequency values for each speed ranges were plotted against the mean of the speed range in order to check the normality of collected speed data. Then cumulative percentage values were calculated. Then, the cumulative percentage values were plotted against the mean of the speed range in order to find the 85th percentile speed value for the relevant vehicle type.

5. Results

The obtained results of 85th percentile speed value of each vehicle category for both road sections and also the statutory speed limit values for relevant vehicle category are shown in the following table.

Table 2: 85th Percentile speeds

	85th Percentile Speed / (kmph)		
Vehicle Type	**Weuda - Mawathagama section**	**Padeniya - Wariyapola section**	**Statutory Speed Limit / (kmph)**
Motor cycle	65	63	70
Three wheeler	52	49	40
Car / Van / Jeep	72	67	70
Bus	69	67	60
Heavy Vehicles	62	58	60

The results obtained from the questionnaire survey were tabled as following for both the road sections considering the vehicle types. The responses of the interviewed drivers of all the vehicle types are shown in table 3.

Table 3: Questionnaire Survey Results

Vehicle Type	Weuda - Mawathagama section (From 17 to 20 km)			Padeniya – Wariyapola section (From 59 to 62 km)		
Response about current speed limit	Suitable	Should be increased	Not aware	Suitable	Should be increased	Not aware
Motorcycle	27%	67%	6%	26%	70%	4%
Three wheeler	4%	96%	0%	3%	97%	0%
Car / Van / Jeep	14%	86%	0%	26%	74%	0%
Bus	13%	87%	0%	27%	73%	0%
Light goods Vehicles	16%	84%	0%	22%	78%	0%
Heavy Vehicles	83%	17%	0%	86%	14%	0%

6. Conclusions and Suggestions

- According to the obtained results only three wheeler and buses are exceed the speed limit although majority of drivers (except heavy vehicle) convey that the speed limit should be increased. It seems that speed limit law enforcement at these road sections influence drivers to slow down their speed.
- Speed limit for three wheelers and buses can be increased by 10 kmph.
- Speed limit for Motor cycles, Car/van/jeep, Light goods vehicles can be increased.
- Speed limit for heavy vehicles is suitable for these road sections.

6.1 Continuation of the project

- Our scope of this research was limited to straight road sections with flat terrain.
- But it is required to carry out this research at road sections which have different road characteristics such as horizontal curves, vertical curves, different road conditions and geographical terrains.
- Various road sections should be considered in order to develop a generalized criterion for setting speed limits for Sri Lankan road ways.

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Development of Traffic Flow Expansion Factors for Sri Lankan Roads

K.S. Jayawrdane, R.J.K.S. Ranathunga, R.M.P.A. Thilakarathne and I.M.S. Sathyaprasad

Keywords: Average daily traffic, Traffic flow variation patterns, Stable period, Traffic flow expansion factors

1. Introduction

Estimates of average daily traffic (ADT) volumes are important in the planning and operation of highways. These estimates are used in the planning of new constructions and improvements to existing facilities, and in some cases, in the allocation of maintenance funds. Normally full-day (24 hr) traffic counts are very costly and time taking. In this research an attempt is made to estimate ADT from short duration counts that are relatively cheap and less time consuming through the development of traffic flow expansion factors that relate short counts with ADT.

Counting locations have distinct patterns of the traffic flow variation depending on the degree of urbanization, roadside developments, proximity to cities etc. Traffic flow expansion factors and the optimum times for counting (stable time) are developed for each location type statistically

2. Literature Review

2.1 Data collection:

Garber (1987) in his research used 228 road tube counters in collecting data. Aldrin (1997) has collected data from 32 existing permanent count stations.

2.2 Identification and analyzing of data:

Aldrin (1997) identified and analyzed data according to their flow variation pattern. Traffic flow variation patterns described for yearly, weekly and daily variations were analyzed for each location.

2.3 Grouping data and deriving traffic flow expansion Factors:

Aldrin (1997) divided data at the permanent count sites with (almost) complete data into a small number of groups with similar traffic patterns. After that by considering factor curve characteristics expansion factors were developed for each groups.

3. Methodology

3.1 Collecting available measured 24h data:

In this research 24-hour traffic volume data at 15 minutes totalling intervals, collected by Central Engineering Consultancy Bureau and University of Peradeniya, that cover all urbanization classes in Sri Lanka for class A and B roads were used.

3.2 Measurement stations are grouped according to their locational characteristics:

Characteristics of each of the survey stations were identified by using maps provided by survey department, CECB and Google Earth. After identifying these station characteristics at each location, traffic flow variation curves that represent the traffic flow variations with respect to time were plotted for each location. For better comparison horizontal normalizing was done for these curves. By analysing characteristic of locations and traffic flow patterns it was realized that traffic flow patterns vary from location to location due to road class, proximity class, urbanization class and connectivity characteristics of the roads. Stations were grouped considering these aspects.

3.3 Statistical analysis to find out the stable period:

Suitable time period is needed to make short counts for using expansion factors. For that time period which has the least traffic flow fluctuation in traffic flow variation curves within a group was taken. According to traffic flow variation patterns most suitable time slot for short counts (stable time) can be identified.

3.4 Statistical analysis to estimate expansion factors:

For group i,

Stable duration = t hours

No of locations inside the group = p

Ratio of t hours at station n ; $R_{t,n} = \frac{\text{t hr volume}_n}{\text{24h volume}_n}$

Average ratio inside the group

$$Rt = 1/p \sum_{n=1}^{p} Rt,n$$

Traffic flow expansion factor for group i = 1/Rt

4. Results and discussion

Table 1: Summarized Traffic flow expansion factors for "A" class roads

Traffic Flow Expansion Factors [Duration/Period]		Urbanization Class			
Proximity Class		Municipal Council	Urban Council	Pradeshiya Saba	Rural Area
Inside a town	Major Trunk Road	9.0 [2h/1000h-1200h]			-
	Minor Trunk Road	8.0 [2h/1000h-1200h]			-
In between two towns	Major Trunk Road	-	-	8.5 [2h/ 1000h-1200h]	7 [2h/ 1000h-1200h]
	Minor Trunk Road	-	-		

Table 2: Summarized Traffic flow expansion factors for "B" class roads

Traffic Flow Expansion Factors [Duration/Period]		Urbanization Class			
Proximity Class		Municipal Council	Urban Council	Pradeshiya Saba	Rural Area
Inside a town	Major Trunk Road	9.0 [2h/1000h-1200h]			-
	Minor Trunk Road	8.0 [2h/1000h-1200h]			-
In between two towns	Major Trunk Road	-	-	6.5 [3h/ 1200h-1500h]	5.5 [3h/ 1200h-1500h]
	Minor Trunk Road	-	-	5 [3h/1200h-1500h]	

For relevant groups few stations were taken for determining traffic flow expansion factors and other stations were used to examine the validation of expansion factors. For validation ADT calculated using the factors developed were checked for other stations and the percentage of error was less than 10%.

5. Conclusion

The expansion factors can be used to calculate reliable values for ADT from 2-3 hours of traffic count on any weekday. The results obtained in this study would be of immense benefit to any relevant party for forecasting trends of traffic demand, road designs, intersection / interchange designs, evaluations on road safety and traffic impact assessments (TIA)

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Effect of Road Rehabilitation on Reduction of Road Traffic Accidents

A.C.P.R. Anthony, A.A.C. Asanka, J.M.D.N. Fernando and W.M.V.S.K. Wickramasinghe

Keywords: Road rehabilitation, Geometric factors, Severity, Blackspot, Statistical analysis

1. Introduction

Safe and good road network is a most important feature of any developing country. But road accidents contribute a great loss in economic and social development in the country. It is very important to carry out a study on effect of road rehabilitation on reduction of road traffic accidents and also study the black spots along the road. Such a study will be beneficial in future road construction and improvements.

1.1 Objective of the study

Understand whether the rehabilitation of the road has been effected on reduction of road traffic accidents and study the hazardous location on the rehabilitated roads.

1.2 Scope of the study

The scope of the study is analysing the accident data considering geometric factors, using three years before and after the accident with respect to the rehabilitation. A stretch of 13 km from Peradeniya to Gampola (AA005) and a stretch of 17 km from Gampola to Nawalapitiya (AB013) are the selected roads.

2. Literature Review

Astrid H Amundsen (2002) has been studied the evaluation of the effect on road safety of new urban arterial roads in Oslo. Their scope based on before and after study of four arterial roads.

Noland (2003) The main variables included of the road geometric factors were total lane mileage, Average number of lanes, Average lane width, average median width, Mean inside & outside shoulder width and Horizontal & vertical curvature. Amundsen (2002) developed a regression model based on empirical bays methods to analysis. Noland (2003) estimated the negative Binomial model which is a generalization of the Poisson regression model. They found that increase in numbers of lanes associated with increase in both traffic related accidents and fatalities.

3. Methodology

Road segments were divided into sections by considering its road geometric changes widening, horizontal and vertical alignment. Accidents were categorized according to their severity as fatal, Injury, Property damages. Then weighted accident rates were calculated.

3.1 Data collection

Roadway accidents are recorded in police stations. They record details of accidents such as date, location, severity, technical details etc as well in Accidents Record (AR) book. The geometric data of a road can get from Road Development Authority (RDA).

3.2 Method of Analysis

Mainly this is done in three stages.

3.2.1 Pre calculation

Accident rates were calculated for each section using AADT value in order to avoid the effect of time series road traffic increments.

3.2.2 Primary analysis

In this stage number of accidents and accident rates were considered in order to compare the after and before data sets in each road stretches. Statistical indicators were calculated using these accident rates.

3.2.3 Detailed analysis

In this stage instead of number of accidents, Equivalent Property Damage only (EPDO) rates were used considering severity levels.

3.3 Hypothesis Check

Statistical part was done using computer program Minitab 16 Software. There are two data sets as before and after. Following Statistical tests are used based on some assumptions.

- Anderson-Darling test
- Two sample t-test and Paired t-test
- Mann-Whitney test
- Wilcoxon Signed Rank test

4. Results and Discussion

4.1 Check for Normality

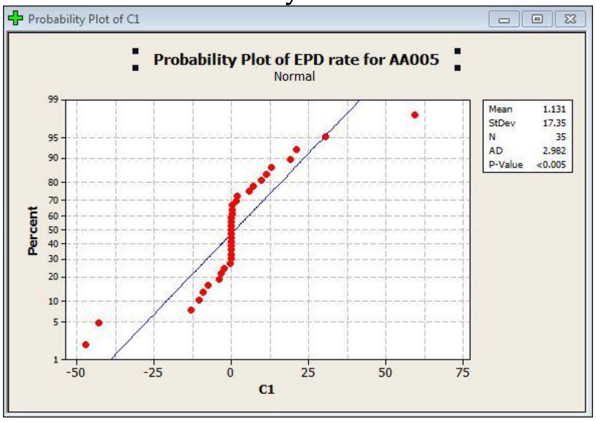

Figure 1: Normality check result for AA005 road

Using Anderson Darling test, it is checked whether the data follows a normal distribution. Distribution of equivalent property damage rates of both road segments were not a normal distribution.

So two-sample t-test or paired t-test cannot be used to get accurate results. Wilcoxon signed Rank test is used assuming after and before EPDO rates depending on same factors such as road characteristics and road user characteristics. So that Mann-Whitney test might not be given accurate results since it is used when data are independent from other factors.

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Session

Wilcoxon Signed Rank Test: Difference of EPD rates

Test of median = 0.000000 versus median not = 0.000000

         N for   Wilcoxon            Estimated
     N    Test  Statistic      P        Median
C4  35      24      169.0  0.597        0.1400
```

Figure 2: Wilcoxon Signed Rank test result for AA005 road

```
Session

Wilcoxon Signed Rank Test: Difference of EPD rates

Test of median = 0.000000 versus median not = 0.000000

          N for   Wilcoxon            Estimated
      N    Test  Statistic      P        Median
C10  40      34      329.0  0.596        0.5950
```

Figure 3: Wilcoxon Signed Rank test result for AB013 road

Probability (p value) of deviating statistics from giving a reliable and very accurate result (to say whether an effect is there or not) should be less than 0.05. But in both roads p value is high as shown in figure 2 and figure 3. So it does not give a comment on the effect of rehabilitation.

5. Studies for hazardous locations

While checking for the hypothesis for the whole road, it is important to study hazardous locations of the roads for immediate improvements to avoid danger. For that compacted behavior of accidents in the specified locations after and before the rehabilitation were observed from collected accident data and identified as removed of newly created black spots due to the rehabilitation.

6. Conclusions

Number of accidents has been increased in both roads after the rehabilitation. But the results have been given with a series of calculated EPDO rate values of two roads in both before and after rehabilitation, they did not vary significantly from each other. It indicates that there is no significant difference between before and after rates of EPDO when considering the whole road segment as a one. That means the rehabilitation has not affected significantly on reduction of road traffic accidents. But there are newly created black spots and locations with old black spots have been removed.

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Optimum Halt Locations for an Urban Rail Used as a School Transport Mode – a Case Study in Kandy

T.M.D.D.B. Pothuhera, M.M.T.D.B. Wilgamuwa, S.P. Kuruppuarachchi and I.M.S. Sathyaprasad

Keywords: Optimum halt locations, Urban rail, School transport, Mathematical model

1. Introduction

The aim of this research is to find optimum halt locations for an urban rail which is to be used as a school transport mode. The proposed cost based mathematical model and methodology is to be followed in optimizing the halt locations. The model minimizes the total cost of the students (Walking Cost and Ride Cost) and the rail operator's cost (Operating Cost). The model assumes that the trip origins are located outside the urban area and destinations (Schools) are located inside. Locations of schools with respect to the rail track and the total demand of the schools are taken as input variables.

2. Literature Review

Mainly, two approaches were thoroughly considered which have been used in the literature for modelling, analysis, and optimization of stop spacing to gain proper stop locations in a transportation system. But these approaches were derived to optimize bus stop spacing.

- Continuum models (calculus based)
- Discrete Enumeration models

Since, Wirasinghe and Ghoneim (1981) have discussed one of the continuum model which has been presented on stop spacing optimization, the paper develops an objective function using the total cost composed of access/egress time, ride time and operating time along a uniformly distributed transit route.

Stop Location Discrete Analysis Approach had been applied by Furth and Rahbee (2000). Here, approach is discrete to model the impacts of changing bus-stop spacing on the total cost which is composed of walking time parallel to the transit line, 'riding time' with discrete cumulative ons and offs throughout the route and 'operating time' that computes stopping delay with a stochastic component. Demands from the Main Street and Parallel Street are treated as distributed demands, and those from the cross streets and transfers have been treated as concentrated loads.

3. Methodology

Although there is no existing rail line for this case, it is assumed that there is a rail line and there are halts near each and every school. Considering those halts and the costs, the optimization procedure can be illustrated through the flow chart given in figure 1.

3.1 Walking Cost

$$C_{wj} = c_w P_i L_{ij} \qquad \text{………….Eq. 1}$$

where, Ct_j is Total walking cost for halt j *(Rupees)*, p is Factored demand of school *i(passengers)*, L_{ij} is walking distance between *i-j(metres)* and Cw is unit walking cost *(Rupees/metres.passengers)*

3.2 Ride Cost

$$C_r = c_t \left\{ t_t + \sum_{i=1}^{m} t_{d,i} \right\} \qquad \text{………….Eq. 2}$$

where, C_t is total ride cost *(rupees)*, t_t is Travel time to halt *(m+1)(min/passenger)*, $t_{d,i}$ is Dwell time *(min/passenger)*, m is No of halts passed and c_t is cost per unit time per person *(rupees/min.passenger)*.

3.3 Stopping Cost

$$C_s = f_s C_a t_d + C_h \qquad \text{………….Eq. 3}$$

where, C_s is Stopping Cost *(rupees)*, f_s is Idling factor, C_a is Average Hourly Operating Cost of Train (rupees/min), td is Dwell time (min) and Ch is Halt Maintenance Cost (rupees).

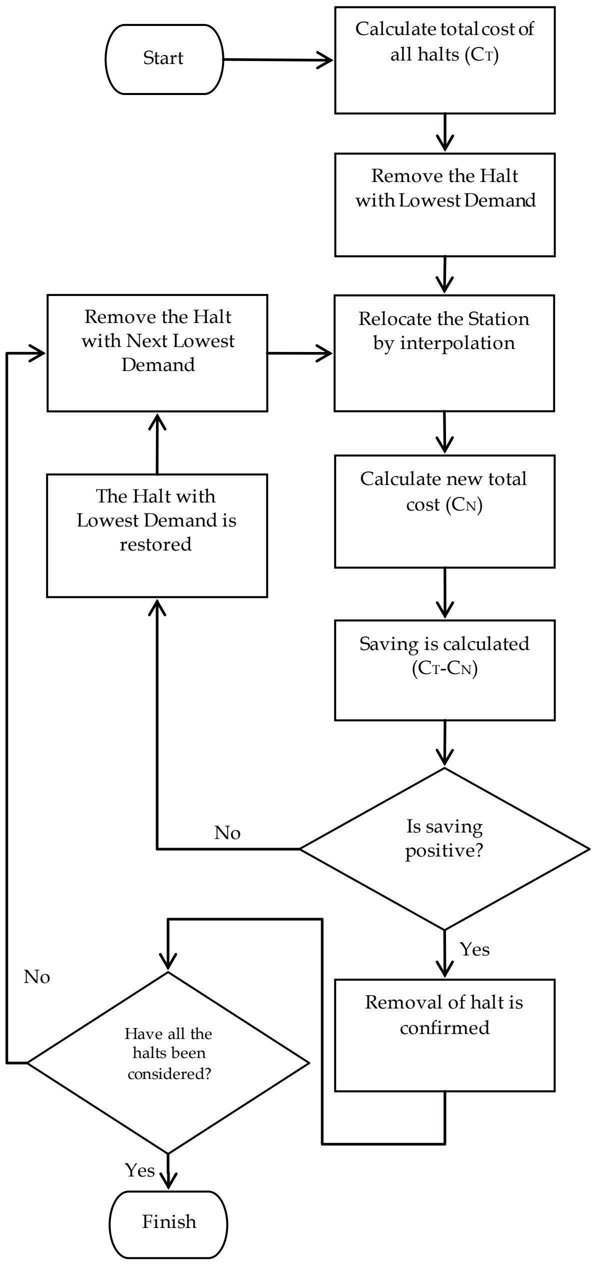

Figure 1: Optimization flow chart

The equations given in sections 3.1 to 3.3 were used for estimating each of costs.

4. Results and discussion

The developed mathematical model is used for Peradeniya – Katugastota rail line as a case study.

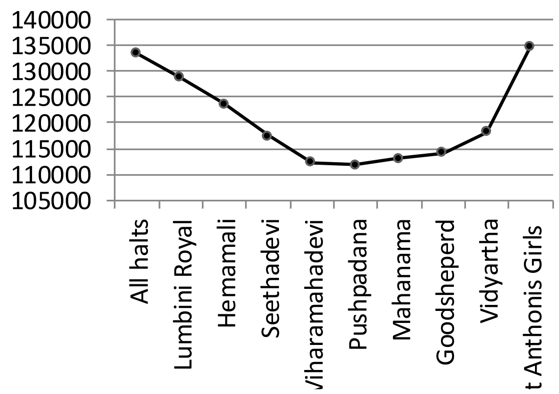

Figure 2: Variation of total cost vs removal of halts

5. Conclusion

The mathematical model developed, based on the principle of minimizing the total cost to the user and operator, was used in a case study in Kandy. The rail road between Peradeniya and Katugastota via Kandy city satisfies the assumptions on which the model is based and hence is ideally suited as a case study. Stations are first introduced at every school and then are removed one by one until the total cost is minimized.

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Development of Passenger Car Equivalency (PCE) Factors for Vehicles in Four-Lane Dual-Carriageway Roads in Sri Lanka

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Keywords: Passenger Car Equivalency factors, Four-lane dual-carriageway

1. Introduction

The term Passenger Car Equivalency (PCE) factor is defined as "The number of passenger cars that are displaced by a single heavy vehicle of a particular type under prevailing roadway, traffic, and control conditions" according to Highway Capacity Manual (HCM 2000). Though there are PCE factors for the two-lane single-carriageway roads in Sri Lanka, PCE values for four-lane dual-carriageway roads have not yet been developed.

This research is aimed at developing PCE factors for four-lane dual-carriageway roads based on field data, collected using a double pneumatic tube detector, at a location representing uncongested flow conditions in a level terrain.

2. Literature Review

Aggarwal (2008) developed his study aiming a fuzzy based model for the estimation of PCU values for buses. Fuzzy based model is of importance because of a number of independent affecting factors. In this study Effect of Pavement Width, Effect of Shoulder Condition, Effect of Surface Characteristics, Effect of Directional Split, Effect of Slow Moving Traffic was considered.

Al-Kaisy et. al. (2002) used field observations and linear programming to determine the PCE. For the case studies in their analysis, Al-Kaisy et al did not find a relationship between PCE and the proportion of trucks. However, they theorized that the PCE should decrease with increasing proportion of trucks because the interactive effect of trucks on trucks may be less than the effect of trucks on passenger cars.

However, Al-Kaisy et al have used only two types of vehicles, viz. cars and trucks, thus they have used linear regression.

3. Methodology

Since RDA identifies thirteen different types of vehicles in Sri Lankan roads multivariate linear regression is used in this study.

As the research will be conducted based on the field data and as a pneumatic tube detector will be used to collect the data, the relevant data that would be obtained are as follows,

- Individual headways of vehicles
- Individual speeds (time mean speed)
- Time of the axle hit
- Flow (in vehicles per hour)

First the data have to be classified into a time interval of 15 minutes or 10 minutes. The classification interval is to be finalized after collecting the data as the data have to be classified in order to have sufficient data in one interval.

Then the Fundamental Traffic flow equation would be used as the main equation,

$$q = k \times v_s \qquad \text{..............Eq. 1}$$

where
q = flow in passenger car units (pcuph)
k = Density in vehicles per kilometer
v_s = Space mean speed in kilometers per hour

$$k = 1/\text{ Headway} \qquad \text{...........Eq. 2}$$

Since the data which will be received from the pneumatic tube detector is in vehicles per hour,

$$PCE_{eq} \times q_{(vph)} = k \times v_s \quad \text{........Eq. 3}$$

From equation 3, the value of the aggregate Passenger Car Equivalency factor for the total flow could be calculated. But as the PCE factors for each vehicle in the flow is needed,

$$\mathbf{PCE}_{eq} = \frac{n_1 * \mathbf{PCE}_1 + n_2 * \mathbf{PCE}_2 + n_3 * \mathbf{PCE}_3 + \ldots}{(n1 + n2 + n3 + \cdots)} \qquad \text{..............Eq. 4}$$

$n_1, n_2, n_3, \ldots$ represent the number of vehicles of each category.

By using the equation 4 in several data sets the most suitable Passenger Car Equivalency factor for each vehicle type would able to be calculated.

Since the aggregate PCE and the number of vehicles of each vehicle type are known according to the equation 4, a matrix was formulated for 1200 data sets and solved by the MatLab. The function used in MatLab is a Multivariate linear regression ("mvregress").

A data logger was set up with the double pneumatic tube detector on the site. The data logger was setup on the site 24 hours per day continuously for a period of 8 days. The data file was processed using a software and the following data were taken,

- Individual Headways of vehicles (Seconds)
- Individual Speeds (km/h)
- Time of the Axle Hit
- Flow (in vehicles in hour)

By using the above mentioned equations out of 1800 data sets, 1200 were subjected to multivariate linear regression and the rest was used for validation.

4. Results and Discussion

Figure 1 shows the relation of the actual flow from the measured data and the estimated flow using the calculated PCE values. The error percentage of these two is around 10%.

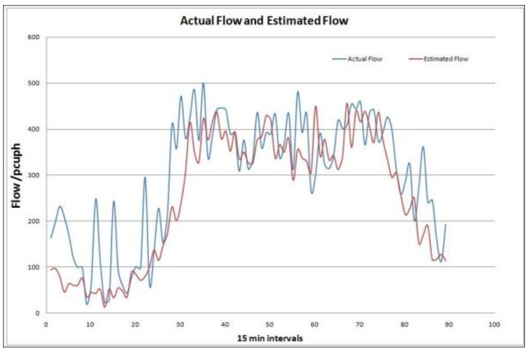

Figure 1: Actual flow and estimated flow

5. Conclusions

In this research the Passenger Car Equivalency factors were developed using the field data obtained. So the results could be considered with a higher accuracy level to the local roads.

The final PCE factors are as follows.

Table 1: Calibrated PCE factors

Vehicle class	PCE factors
Motor Bike	0.6
Car / Jeep / Pickups	1.0
Route Bus	2.8
School Van/ Passenger Van	2.0
Light Good Vehicles	2.1
Multi Axle	4.3

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