Proceedings of the Special Session on

MSW and Landfill Management, Hydraulic Structures and Water Safety

6th International Conference on

Structural Engineering and Construction Management 2015

Kandy, Sri Lanka

11th to 13th December 2015

Abstracts of 6th International Conference on Structural Engineering and Construction Management 2015

Vision

Promoting innovative research for tomorrow's development

Mission

To meet experts, colleagues and friends in the field and to exchange findings, concepts and ideas on research for the development of a sustainable world

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Preface

It is with great pleasure that we present the Proceedings of the 6th International Conference on Structural Engineering and Construction Management (ICSECM 2015). This is the sixth conference consecutively organized following the 1st International Conference on Sustainable Built Environment in 2010, 2nd International Conference on Structural Engineering and Construction Management in 2011, 3rd International Conference on Sustainable Built Environment in 2012, 4th International Conference on Structural Engineering and Construction Management in 2013 and the 5th International Conference on Sustainable Built Environment in 2014, keeping its tradition of adhering to engineering excellence.

Taking a step forward from the last four events, the coverage of specialty areas of this conference has been diversified. This book contains the abstracts of research papers from ten different sub specialties in Construction Management, Construction Materials and Systems, Structural Health Monitoring, Structural and Solid Mechanics, Earthquake Engineering, Fatigue Damage of Materials, Water Safety, Hydraulic Structures, Tall Building and Urban Habitat and MSW and Landfill Management. We expect that all these abstracts will be presented in parallel sessions from 11th to 13th December 2015.

We would like to express our appreciation to all keynote lecturers for their invaluable contribution. We are very much grateful to the authors for contributing research papers of high quality. The research papers of these abstracts in the publication have been peer-reviewed. The enormous work carried out by the reviewers is gratefully appreciated. We are also pleased to acknowledge the advice and assistance provided by the members of the international advisory committee, members of the editorial committee along with many others who volunteered to assist to make this very significant event a success. Finally, we acknowledge the financial sponsorship provided by many organizations that has been extremely helpful in successfully organizing this international conference.

It is the earnest wish of the editors that this book of abstracts and volumes of proceedings would be used by the research community and practicing engineers who are directly or indirectly involved in studies related to Construction Management.

Editorial Committee

6th International Conference on Structural Engineering and Construction Management 2015

11th December 2015.

Message from
Conference Chairmen

It is a pleasure for us to welcome all the participants to the 6th International Conference on Structural Engineering and Construction Management 2015 in Kandy, Sri Lanka. We, the co-chairs would gratefully like to mention the previous successful conferences, the 1st International Conference on Sustainable Built Environment 2010, 2nd International Conference on Structural Engineering and Construction Management 2011, 3rd International Conference on Sustainable Built Environment 2012, 4th International Conference on Structural Engineering and Construction Management 2013 and the 5th International Conference on Sustainable Built Environment in 2014, all held in Kandy, Sri Lanka.

The theme selected for the conference - Structural Engineering and Construction Management- is extremely relevant for today's world. With the vision of promoting innovative research for tomorrow's development, we organize this conference as a meeting place of talents, knowledge and dedication. Therefore, we trust that the conference will produce great ideas from a variety of Research and exchange the knowledge of experts, colleagues and friends who are working for the world's sustainable development.

The conference focuses on different sub topics in Structural Engineering and Construction Management, Construction Materials and Systems, Structural Health Monitoring, Structural and Solid Mechanics, Earthquake Engineering, Fatigue Damage of Materials, Water Safety, Hydraulic Structures, Construction Management, Tall Buildings and Urban Habitat and MSW and Landfill Management. The proceedings of the conference are peer reviewed. The full papers are published in volumes in paper format with a book of abstracts.

The host city of the conference, Kandy, is a world heritage city famous for its unique architecture, culture, natural beauty and climate. We hope that you will enjoy your time in Kandy during the conference.

We, the conference co-chairs express our sincere thanks to our guests, keynote speakers, authors, members of the international advisory committee, members of the editorial committee financial sponsors and many others who volunteered to assist to make this very significant event a success.

Prof. Ranjith Dissanayake
Prof. S.M.A. Nanayakkara
Prof. Priyan Mendis
Prof. Janaka Ruwanpura
Dr. Y.G.S. De Silva
Eng. Shiromal Fernando

Co-chairs

6th International Conference on Structural Engineering and Construction Management 2015
11th December 2015.

Message from
Dean, Faculty of Engineering, University of Peradeniya.

I am glad to submit this message for the Sixth International Conference on Structural Engineering and Construction Management (ICSECM-2015), which is a continuation of the efforts of the organizers to share knowledge and research in the sectors. This time too, the conference is held in historic city of Kandy, in Sri Lanka.

The ICSECM - 2015 is organized as a joint effort of a number of professionals, and a number of institutions; including Engineering Faculties of Peradeniya, Moratuwa and Ruhuna Universities in Sri Lanka. The topic covered and the keynotes delivered by professionals in the field add more depth to the objectives and outcomes of the conference.

I take this opportunity to thank the organizers for their commitment and persistent effort to make the conference a success. These events facilitate a forum for many young undergraduate and postgraduate students to receive a good initial exposure to present their work, and for some few, to get a flavor of organizing events of global importance.

I believe that the organizers of ICSECM-2015 will continue their dialog of bringing concerned professionals from diverse fields, from different parts of the worlds, into the discussion forum of ICSECM.

I wish the conference a great success.

Prof. Leelananda Rajapaksha

Dean,
Faculty of Engineering,
University of Peradeniya,
Peradeniya,
Sri Lanka.

Table of Contents

SECM/15/173

PHYSICO-CHEMICAL CHARACTERISTIC OF A PETROLEUM CONTAMINATED SOIL FROM THE SPILL SITE OF JAFFNA DISTRICT

V.P.S.Richardson, G.B.B.Herath and C.S.Kalpage

SECM/15/161
INVESTIGATION OF APPLICABILITY OF DIFFERENT MODELLING METHODS AVAILABLE IN HEC-HMS FOR MODELLING KALU RIVER UPPER CATCHMENT IN SRI LANKA
D.P.H.M. Kanchanamala, H.M.H.K. Herath and K.D.W. Nandalal

SECM/15/163
CLIMATE CHANGE IMPACT PREDICTION IN UPPER MAHAWELI BASIN
H.M.V.V. Herath, R.G.A.B. Dayananda and S.B. Weerakoon

SECM/15/044

Sustainable Approaches to the Municipal Solid Waste Management in Sri Lanka

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Abstract: Municipal solid waste (MSW) is a serious environmental & socioeconomic issue in Sri Lanka and Haphazard disposal, population growth, migration and rapid urbanization will accelerates the issue further. Comprehensive and accurate measurement of waste generation and disposal continues to be an issue at national, provincial and local levels. The present composition of solid waste collection by the Municipal Councils 49.5% (1,696 Mt), Urban Councils 17.4% (594.5 Mt) and "Pradesiya Saba" Areas 33.1% (1,133 Mt). Conversely, through the several government and non- government projects were operating towards the National Solid Waste Management (NSWM). Further they were have been conducted the analyses for their internal use. However, there is no proper mechanism to coordinate this information and research, or to compile results with an intergraded approach. Appropriate estimations and evolutionary predictions will sustain new projects by minimizing difficulties. Previous data were shows approximately MSW is contain 50 - 65% readily bio-degradable waste or organic component and the balance is inorganic component. Low calorific values recoded in organic fraction of waste and it is possible to use as raw material of composting or bio-gas generation. And the receiving part of the waste should be running through the material recovery facility (MRF) and the residue has to incinerations and finally to landfilling. With the current situation there is a possibility of earning 20-22 US$ from a one metric ton of mixed MSW. There is no proper focus into the Cleaner Development Mechanism (CDM) to the current MSW management project as well as there is no focused in to Intergraded Solid Waste Management (ISWM) in the country. This paper suggested that the importance of ISWM by maintaining a sustainable composite mechanism through locally – available materials and expertise, with evidence based approach planning and strategy through eliminating the potential risks to provide a clean, healthy pleasant living environment and resource management culture for current and future generations of Sri Lanka.

Keywords: Municipal Solid Waste, Intergraded solid waste management, Material recovery facility, Cleaner Development Mechanism, locally – available materials and expertise.

1.0 Introduction

The Democratic Socialist Republic of Sri Lanka is an island in the Indian Ocean southwest of the Bay of Bengal, between latitudes 5° and 10°N, and longitudes 79° and 82°E.It is separated from the Indian subcontinent by the Gulf of Mannar and Palk Strait, It is located at the global logistic hub by intercepting with the major air and sea routes between Europe and the Far East. Sri Lanka has land area of 65,610 sq.km. and Population density of 331 per sq.km. The population growth rate was 0.9% in 2014 with respect to statistical data sheet 2015[1]. It has 25 administrative districts and 9 provinces Country economy worth $80.591 billion (2015), ($233.637 billion PPP estimate) and a per capita GDP of about $11,068.996

(PPP)[3].And Gross National Production (GNP) at current prices Rs.Mn. 9,544,608[1]. The country main industries were processing of rubber, tea, coconuts, tobacco and other agricultural commodities; telecommunications, insurance, banking; tourism, shipping; clothing, textiles; cement, petroleum refining, information technology services and construction.

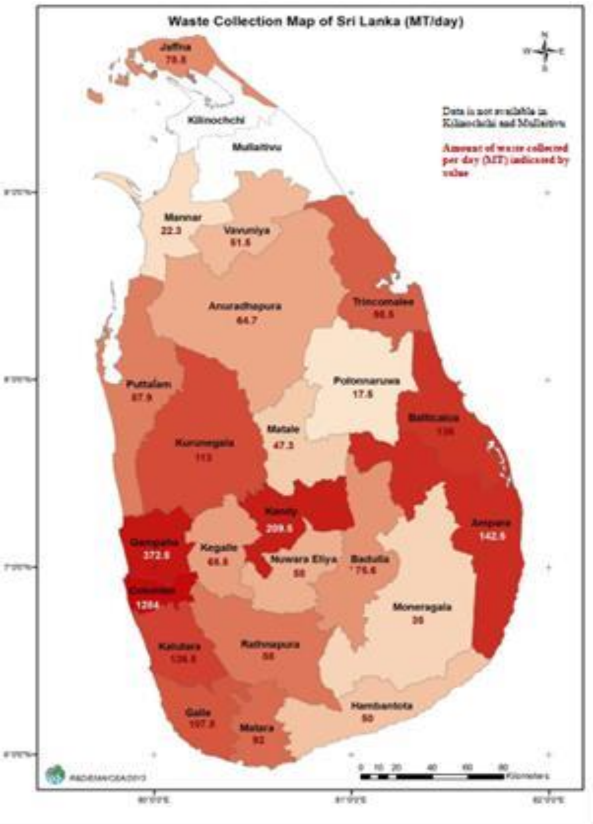


Figure **1:** Waste Collection Map of Sri Lanka
Source: Database of Solid Waste in Sri Lanka, **,**
"*Pilisaru*" *National Solid Waste Management Programme, CEA, 2012*

Municipal solid waste (MSW) is a serious issue in Sri Lanka and Haphazard disposal of Solid waste accelerates the serious environmental & socioeconomic issue further. It is a well-known fact that the waste generation has been increased due to the development, population growth, rapid urbanization, migration and accompanying changes in the consumption pattern and industrialization.

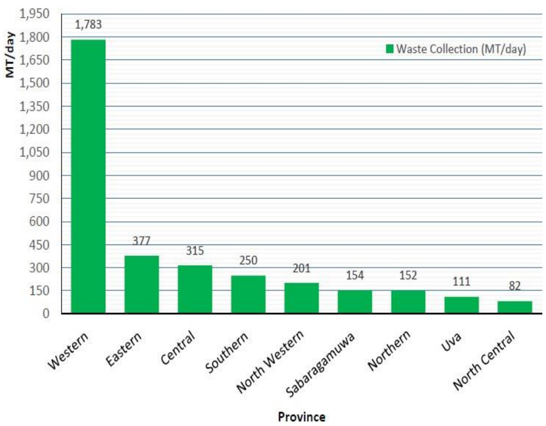
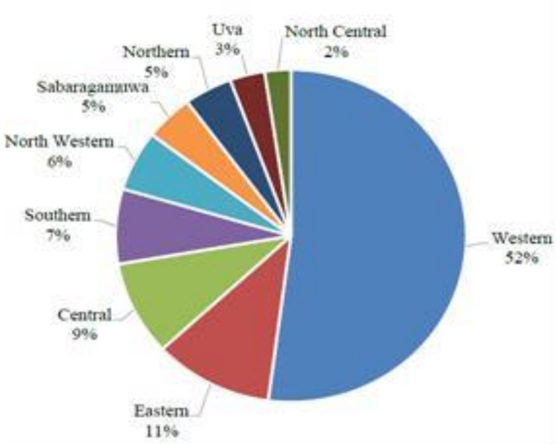


Figure 2: Waste collection provincial wise
Source: Database of Solid Waste in Sri Lanka, "Pilisaru" National Solid Waste Management Programme, CEA,

With available statistics on MSW Comprehensive and accurate measurement of waste generation and disposal continues to be an issue at national, provincial and local levels. Daily 3,242 Metric tons (Figure1, 2) were collected by the Local Authorities and the highest amount of 1,783 metric ton was collected daily by Western province (Figure2). The highset population reported from Colombo District which collects 1,284 metric tons solid waste(Figure1). The composition of solid waste collection by the Municipal Councils 49.5% (1,696 Mt), Urban Councils 17.4% (594.5 Mt) and Pradesiya Saba Areas 33.1 % (1,133 Mt). (Figure3) [4]



2012

Figure 3: Waste collection percentages
Source: Database of Solid Waste in Sri Lanka, "Pilisaru" National Solid Waste Management Programme, CEA,2012

The compositions of solid waste are mainly Polythene/plastic waste/Shopping bags, Short term bio-degradable waste (animal *& plant matter*), Long term bio-degradable waste (*e.g. Coconut shells, King Coconut shells, .etc.*), Metal waste, Wood waste (including *tree cuttings*), Glass waste, Paper wastes, Building waste, Slaughter house waste, Cloth/garment waste, Hazardous waste (*batteries, CFL bulbs, e-waste,….etc.*) and other waste (*e.g. industrial waste and other*) (Figure5) [4,7,14].54.5% is short term bio-degradable waste and 10.5% is the polythene/plastic waste/shopping bags and it was the second largest category in MSW(Figure4).[4,7,14].With the statistics of Solid Waste Management in Sri Lanka total of 11,695 employees were involved. From those 10,196(87%) were owned by Local Authority and 1,499(13%) are owned by private sector.

Expenditure on solid waste from the Local Authorities in Sri Lanka is about US$ 27 million [4].

Presently in most part of the country MSW collected and typically end up in open dumps and open burning as a primary method, and in some cases are deposited in illegal dumping sites. Meanwhile, Sri Lanka will practise the conventional waste management procedure and it focuses largely on waste collection, treatment and disposal. With respect to the characteristics of MSW, operational & maintenance cost, marketability of final product, public perception, technical & financial affordability are the factors that leads to the selection of proper solid waste management program.

Only limited attempts are made to adopt Integrated Solid Waste Management (ISWM) practices that involve waste reduction at the source, resource recovery and recycling.

There are several government programs such as "Pilisaru" National Solid Waste Management Programme 'Towards garbage free Sri Lanka by 2012'; with the meaning of the "Pilisaru" was regaining the usefulness. It is allocated approximately 5 billion rupees for the "Pilisaru" programme, through the 2008 national budget of Sri Lanka [4]. However, it is a challenge and very difficult task to provide technology, infrastructure, legal advice and political leadership. Furthermore, it is very difficult to change the long established habits & attitude of people.

2.0 Composition of Municipal Solid Waste

The Sri Lankan MSW mostly consists of Short term 54.5% bio-degradable waste (i.e. Food/Kitchen Waste, animal & plant matter…etc.), Long term bio-degradable waste (5.9%) (E.g. coconut & king coconut shells, rice husks, slaughter house waste (2.8%), leather …etc.) And Non-biodegradable as such; Polythene/soft & hard plastics waste/shopping bags (10.5%), Metal waste (1.8%) (E.g. aluminium cans, steel containers…etc.), Wooden waste (e.g. show dust, tree cuttings…etc.), Paper & cardboard waste (3.7%), cloth/garment waste (1.2%), Glass waste (3.1%), Building construction waste, hazardous waste (0.4%) (Batteries, CFL bulb, paint bottles, e-waste…etc.) and other waste (7.7%) (e.g., Industrial waste …etc.) [4] (Figure 5).

Figure 4: Waste Compositions Survey
Source: Waste characteristic survey, Kunhwa Engineering & Consulting Co.Ltd.

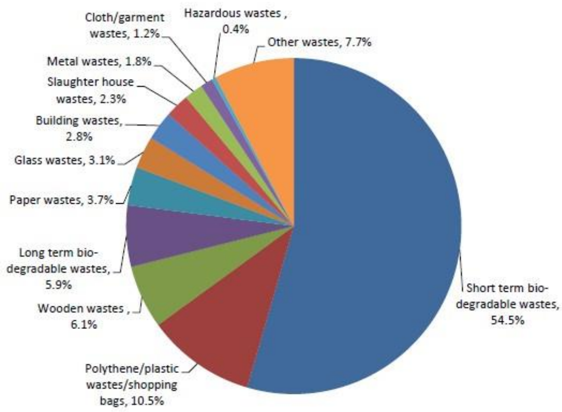


Figure 5: Compositions of MSW in Sri Lanka
Source: Database of Solid Waste in Sri Lanka, "Pilisaru" National Solid Waste Management Programme, CEA, 2012

3.0 Waste separation mechanism.

Presently most of the local authorities, there is no proper mechanism to segregate the waste at the source. The generated waste from residential, commercial…etc. are stored at polythene/polybag. Sometimes this bags will collect after 2-3 days which started the inside degradation. This will create many issues for composting/other methods as such leachate, bad odour…etc. Presently considerably good level of segregation at the source and proper waste collection mechanism was practised by several Local Authorities like, Balangoda & Kalutara Urban Councils. However most of the local authorities will collect unsegregated waste from the source.

Figure 6: Pikers in clean MRF/Waste sorting belts

3.1 Material Recovery Facility (MRF)

Through the material recovery facility it will facilitate to recover the recycles especially polythene, glass, rubber, metals…etc. (Figure6, 7). Presently in Sri Lanka MRF will basically consist of an elevated unloading area conjunction with a ramp and a conveyer belt and it will facilitate manual separation of waste (Figure7, 6).

Figure 7: Some existing MRF in Sri Lanka

There are several methods such as; jigs (device that separate less dense to more dense particles by using the difference in their abilities to penetrate a shaken bed) (Figure7), Air Classifiers (is to separate less dense, mostly organic materials from the more dense, mostly organic fraction using air as the fluid)
(Figure8,9) [6].

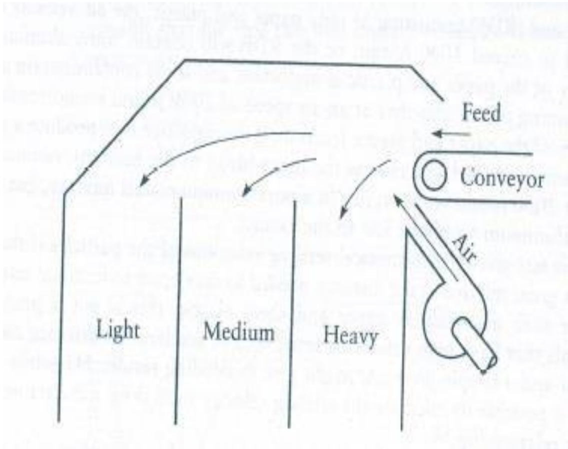


Figure 8: Horizontal Air knife:
Source: Solid Waste Engineering", Brooks/Cole,511, Forest Lodge Road, Pacific Grove, CA

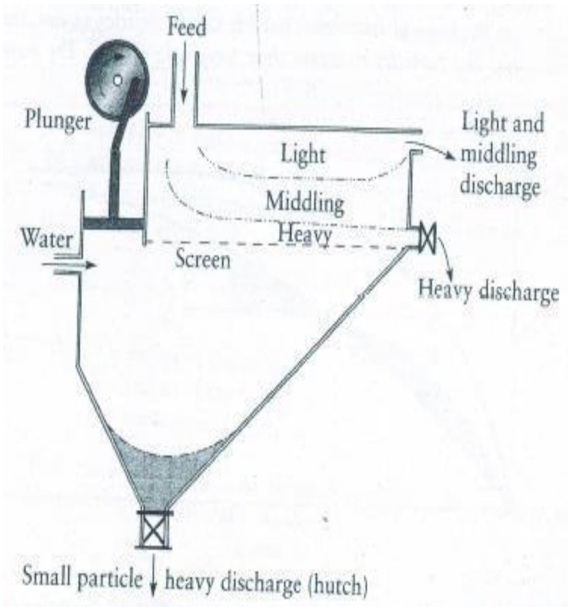


Figure 9 : Common plunger
Source: Solid Waste Engineering", Brooks/Cole,511, Forest Lodge Road, Pacific Grove, CA

Incorporate the waste separation mechanism to achieve the sustainability. Waste separation mechanism; open the waste containers/ waste bags at the starting point open the bags and segregate to bio degradable and non-biodegradable

(Polythene, plastic, glass, metals…etc.) then segregate the hazards waste(batteries, CFL bulbs…etc.) at the initial feedstock. And finally recover the plastic, metals, glass) (Figure7)

4.0 Option for Bio-Degradable Waste 4.1 Composting

Sri Lankan context approximately MSW is contain 50 - 65% readily bio-degradable waste or organic component (Figure5)[4,7,14]. Therefore converting this higher percentage portion into compost/soil conditioner will be environmental sound practice as well as economically viable option for an agricultural country like Sri Lanka [2]. It is a biological process which happens under the controlled condition of aeration, carbon nitrogen ratio, temperature and moisture. Different technological approaches to produce compost as such passive composting system, windrow composting system aerated pile system and in the vessel system[6]. From those passive aerated open windrow system (Figure10) is more feasible for the country like Sri Lanka [15]. Composting process will reduce the volume of raw waste of MSW more than 60% of its initial volume [15]. Composting of MSW will help to decrease the considerable amount of waste that must be sending to final landfilling.

Figure 10: Some existing composting plants in Sri Lanka

Quality Composting is free from methane generation, unpleasant odours, easy to handle, can be stored for a long time and it has wide variety of uses in agriculture [15]. Quality compost will improve physical properties, chemical properties, biological properties, adds organic matter, varying amounts of soil nutrients such as nitrogen, phosphorous, potassium...etc. It also improves soli structure, aeration and moisture content of soil. It also suppresses certain plant diseases and parasites and kills weed seeds thereby controlling weeds & diseases. With a comparison with chemical fertilizers compost will increase agricultural production rate. Composting has main role in organic farming which is the fairly growing industry in Sri Lanka. The demand will be increased for compost as well as organic farming products in bout local & export market.

Figure 11: Residual Composition Study of Compost

Source: Waste characteristic survey, Kunhwa Engineering & Consulting Co.Ltd.

Approximately production cost was US$ 26-30 per metric ton in Sri Lanka and wholesale price US$ 76-91 per metric ton (Table 1). Therefore it will give approximately 67% profit with composting of MSW. Area requirement for composting project is approximately 11.5 Perch/MT. Therefore composting is an attractive financial alternative and as well as value added opportunity.

4.2 Bio-Gas Generation

Readily bio-degradable component such as food waste, agricultural waste, animal dung, vegetable & fruit waste…etc. can be used as a biogas fuel generation [6]. It is produced during the anaerobic decomposition of these waste materials and contains 60% of methane composition. The generated bio gas can be used for cooking, lighting, also in conventional vehicles and also be burned in a generator to produce electricity [5,6].

Biogas Energy Systems is anaerobic digesters that create system-wide operational efficiencies in energy production and operational costs [6]. While the amount of methane that can be produced from a given feedstock is relatively fixed (Figure 12), system-wide facility design can optimize methane production and power generation [6]. With the features of redundancy, gas storage, flexibility, and disaster prevention will give more Biogas Energy Systems the ability to produce more energy from a given substrate.

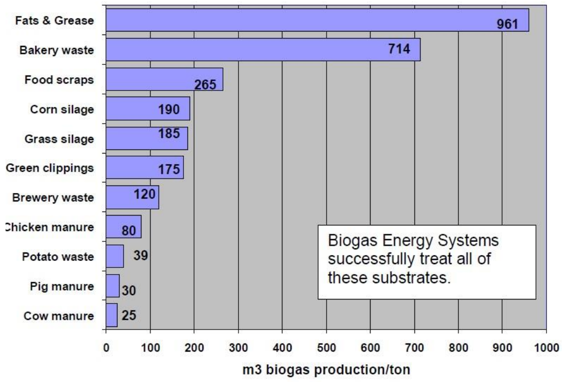


Figure 12: 1 m^3 Bio gas production /ton
Source: Basisdaten Biogas Deutchland, Marz 2005: Fachagenture Nachwachsende Rohstoffe e.v.

Countries like Germany, where more than two thousand anaerobic digesters operate, biogas production has undergone decades of continual quality improvement. They use most of them for energy production and they demand the highest efficiencies [16]. Sri Lanka is practising two type of biogas reactors (Figure 14), the Dry Batch System and the Continuous System [5]. The process of anaerobic digestion consists of three steps of Decomposition (hydrolysis or plant of animal matter. the step breaks down the organic material to usable-sized molecules like sugar), Conversion of decomposed matter to organic acids and Conversion of acids to methane gas.

Figure 13: Bio Gas Plant Dikkovita, Sri Lanka

Biogas Energy is dedicated to bringing with mature anaerobic digestion technology to easily biodegradable waste in MSW Sri Lanka and to achieving good levels of energy production with answering to the country energy requirement. As well as effective Bio-gas generation (Figure 15) will provide superior value and efficiency over the lifetime of the project.

Figure 14: Landfill Gas liquefaction Hartland unit located in Victoria, B.C. developed by CryFuels System, Inc.
Source: CryFuels Systems

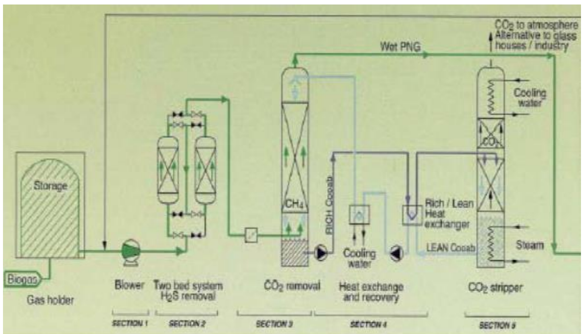


Figure 15: Effective Bio-gas generation
Source: Cirmac, undated

4.3 Animal/Poultry Food Generation

The municipal solid waste contains approximately 20 – 30 % of food waste & slaughterhouse which can be used as animal / Poultry /fish foods. This will be generated from mainly from restaurants, hotels, fish markets, supermarkets, expired foods, food & meat processing industries …etc. [17]. The waste should collect with in 12hrs/ before rotten and freeze and used for animal food processing. Countries like Korea and Japan practice in vast range and convert their waste as a value added product. The systems will be relying on more technical & financial inputs, but this will be reducing the huge amount of bio-degradable component in MSW in an economical manner.

5.0 Option for Inorganic/Non-bio- degradable Waste 5.1 Application of 3R Concept

Solid Waste Management in Local Authorities application of 3Rs will strive to reconfigure the systems to deliver better returns on natural, human and economical investments, within the same time reducing GHGs, extraction and using rare natural resources by creating less waste and reducing social disparities with promoting reusing and recycling. Recycling entered the mainstream of solid waste management from the histories by the underfunded, haulers / idealistic individuals. Nevertheless, rather multinational & national garbage companies are now involved in recycling, regardless of the price paid for recycle material, could be profitable (Table 1). While some long for the days of the idealistic recycler, no one can dispute that more material is being recycled today. Recycling business will create direct and indirect "green job" opportunities in various cross sections of the society. Reducing generation, promoting reuse and recycling of waste will reflects the balance of environmental conservation and economic growth through an effective use of virgin resources (Figure 16).

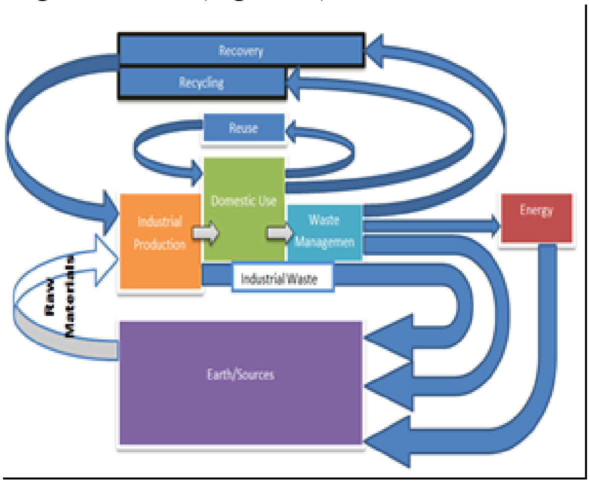


Figure 16: Resource flow through in a resource management infrastructure

5.2 Combustion/Incineration

Much of the MSW is combustible (approximately 95% of composting residue/ other residue) [7] and the destruction of this fraction can coupled with the recovery of energy as an option in solid waste management .Refuse can be burned as it is or it can be processed to improve heat value and to make it easier to handle in combustor/incinerator (Figure 17, 18). Processed refuse can also be combined with other fuels, such as coal, and cofired in a heat recovery combustor/incinerator.

MSW residues are contains 80-90% combustible materials [7]. The lower calorific value of MSW is reported 6845.77 kj/kg (1637.74 kcal/kg) [8]. Therefore from the other hand, MSW is not an insignificant source of power. However, when concerning on other effects Green House Gas emission, Water pollution, Land Pollution, eyesore and other socio economic concerns the proper combustion/incineration (Figure 17, 18) was a better solution for the residue of the Sri Lankan MSW.

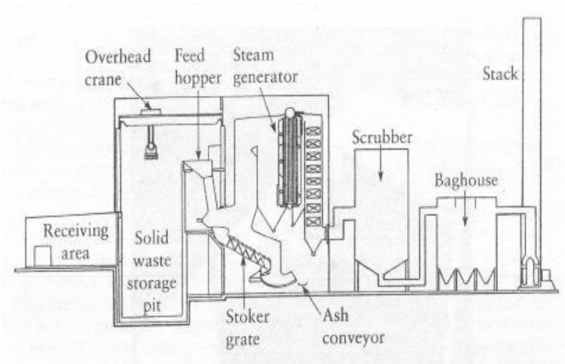


Figure 17 : A typical municipal combustor:
Source: Solid Waste Engineering", Brooks/Cole, 511, Forest Lodge Road, Pacific Grove, CA

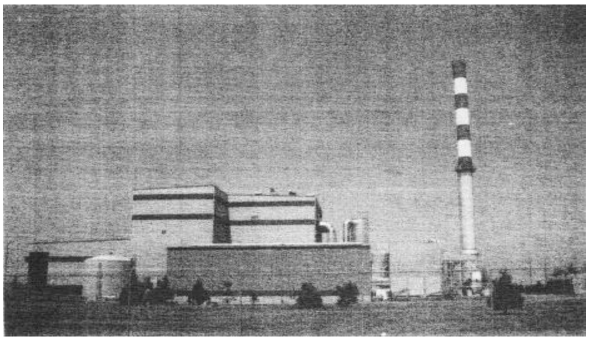


Figure 18: A typical municipal combustor
Source: Solid Waste Engineering", Brooks/Cole, 511, Forest Lodge Road, Pacific Grove, CA

From the controlled or proper combustion/ incineration ash will reduce the capacity of the sanitary landfill with a density of 3300 lb/yd^3 at this density the ash is highly impermeable, with a permeability as low as 1X 10^{-9} cm/sec[6]. Therefore land fill value will be increased compared to the other sanitary landfills. Further some of the ash include to Road base material, Structural fill, gravel drainage ditches, capping strip mines and mixing with cement to make building blocks [6].

6.0 Intergraded Solid Waste Management (ISWM)

Intergraded Solid Waste Management is a comprehensive waste prevention, recycling, composting and disposal programme (Figure 19) [9]. It will be consider how to prevent, recycling and manage solid waste in an effective way to protect human health and the environment (Figure 19). The ISWM involves evaluating local condition and requirements, through that selecting and combining the most appropriate waste management activity efficiently.

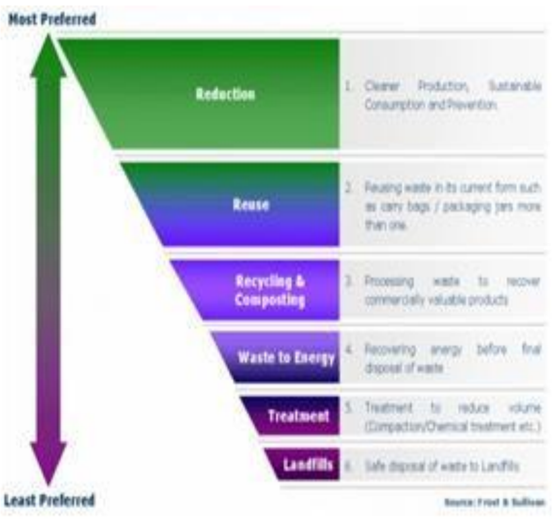


Figure 19: Integrated Waste Management Hierarchy
Source:http://www.frost.com/prod/servlet/cio/186567195

7.0 Cleaner Development Mechanism (CDM) to Solid Waste Management

The Clean Development Mechanism (CDM) is one of the Flexible Mechanisms defined in the Kyoto Protocol (IPCC, 2007) that provides for emissions reduction projects which generate Certified Emission Reduction units (Figure 20) which may be traded in emissions trading schemes [11].It is defined in Article 12 of the Kyoto Protocol and is planned to meet two objectives. First is to assist parties contributing to prevent dangerous climate change which is the ultimate objective of the United Nations Framework Convention on Climate Change (UNFCCC) and assist to parties achieving compliance with Certified Emission Reduction units of greenhouse gas (GHG) emission reduction projects under the Kyoto Protocol [12].

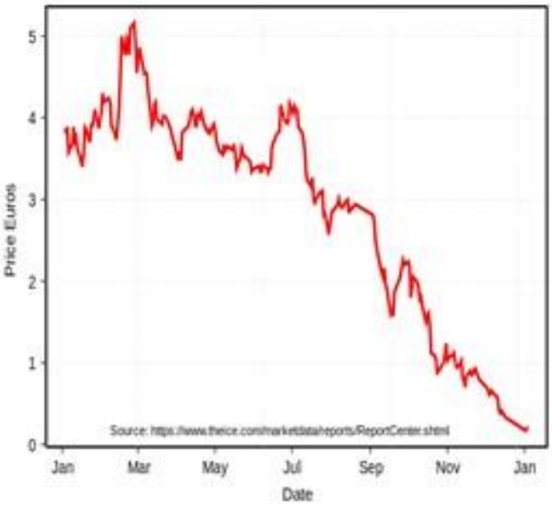


Figure 20: Certified emission reduction spot prices 2012
Source:https://en.wikipedia.org/wiki/Clean_Development_Mechanism

India generates the MSW approximately 40 million tons per Through the CDM Indian market is more cornered certified emission reduction (CERs) and dominance in carbon trading. India Inc. pocketed Indian Rupees 1,500 crores in the in the year 2005 just by selling carbon credits to developed-country clients [13].

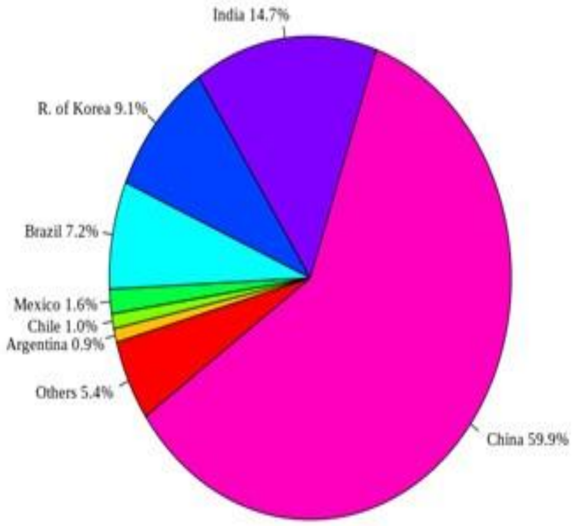


Figure 21: Certified emission reduction units by country
Source:https://cmd.unfccc.int/statistics/Issuarance/CERIssuedbyHostPartyPieChart.html

Table 1: Economical aspect of the current Solid waste Management in Sri Lanka.

Recoverable waste Type from a Mt		Consumable from recovered 90% (Kg/Day)	Total collection per month (Kg)	Unit price / Kg	expected income per month (SLR)
Composting	100% segregated Waste	180	5400	8	43,200.00
	Unsegregated waste	90	2700	10	27,000.00
Polythene	White	3.06	91.8	35	3,213.00
	Shopping	4.59	137.7	6	826.20
	Other	7.65	229.5	5	1,147.50
Plastic	PET	0.54	16.2	48	777.60
	Unusable	0.18	5.4	0	-
	Mixed Plastic	1.08	32.4	35	1,134.00
Paper & Card Board	News Paper	1.683	50.49	8	403.92
	Card Board	3.366	100.98	12.5	1,262.25
	Unusable	11.781	353.43	0	-
Coconut Shells		3.6	108	13.25	1,431.00
Glass	White	1.89	56.7	4.75	269.33
	Color	0.675	20.25	2.75	55.69
Metal	scrap iron	0.108	3.24	40	129.60
	tin & cans	0.432	12.96	18	233.28
Total Expected income from 1 Mt of MSW per month					**81,083.36**
Total Expected income from 1 Mt of MSW per Day					**2,702.78**

7.0 Discussion

Municipal Solid Waste (MSW) is a serious environmental & socioeconomic in Sri Lanka and

Haphazard disposal of Solid waste , rapidly increasing quantities and diverse characteristics of waste will accelerated, it further. Local authorities are running with a financial & technical burden for waste management as well as pressure on landfill requirements.
Hence, in most part of the country MSW collected and typically end up in open dumps and open burning as a primary method, and in some cases deposited in illegal dumping sites. As a result, massive garbage dumps such as the 'Bloemendhal" road eyesore. Furthermore urban air/water pollution, floods induced by solid waste clogging in drainage canals and land degradation, as well as the concerns of the risk to the public health and its translation into economic costs. This will reflects the overreliance on conventional waste collection, treatment and disposal is not sustainable solution and it is too costly with considering on other economical aspect.

The resource value of MSW cannot be realized unless separation of wastes is practised effectively at the source. Then, it is very difficult to change the long established habits & attitude of people. Therefore it is essential to presume on partially segregated/mixed waste from the source till the fundamental change in mindsets and attitudes toward as "waste" into "resources." And Build a "resource management infrastructure".

There is an informal infrastructure in collection of recyclables in the country, with the contribution of the huge numbers of waste pickers, who perform such a crucial service to the Waste Sector. Meanwhile, they are not only protecting and empowering the poor, but are also proceeding towards meeting relevant development goals in the country. Providing of a better occupational environment, social recognizant and insurance through formalising the informal sector workers/waste pikers to the formal sector via raising the living standards. Then it can contribute in a meaningful way for an environmentally sound and socially acceptable business.

Affordability to the "resource management infrastructure" we can predict US$ (15 to 22)/ day income from a one metric ton of MSW through the Material Recover Facility (MRF) only from the Mix-Waste/Unsegregated Waste. Further there is a tipping charge of approximately US$ 4/Mt for MSW. Therefore with this present scenario income of US$ (1926) /Mt/day income can be expected without carbon credits. Besides the income will be increases with CDM and with the total quality management.

Further to that through the global perspective introducing of policy instruments such as "volume based fees" for solid waste collection could lead to reduction of MSW generation. This tool was succussed in Korea which to a 21.5% reduction of MSW generation from 1994 to 2009. In addition to that, application of the Cleaner Development Mechanism (CDM) with the certified emission reduction (CERs) and dominance in carbon trading to the waste projects can earn economical value with selling carbon credits to developed-country clients. Hence, we can predict a better solution for the financial burden in the waste management sector and step forward to the sustainability.

Through a MRF, resource recovery has become a major factor that determines the competitiveness of firms and that excel in resource recovery could take advantages of win-win solutions to meet international obligations on climate change and Improved resource recovery could also lessen potential pressures and avoid root causes of social conflicts that could arise from resource competition.

In the planning stage there are gaps in Local, institutional and stakeholder levels that will lead to ineffective. Solid waste management in Sri Lanka is struggling to handle the MSW produced in Local jurisdictions and lack of intergraded & up to date dataset. Therefore it is the high time to strengthen institutional capacities, skill development and awarenessraising with coordinate information and research, or to compile results with an intergraded manner and formulating a Central Organization for the whole country. Further Local Authorities should not stick with everything by themselves, It is key to success is to do what they are good at, and collaborate with other sectors in the society, such as private public partnership. Such efforts by adopting ISWM and 3Rs with disposal being just the last resort or least preferred option as well as set policy directions aiming for resource efficient, recycle-based society will help Local Authorities to reduce the financial burden on Local authorities for waste management, as well as reduce the pressure on landfill requirements.

9.0 Conclusions

Municipal solid waste (MSW) is a serious environmental & socioeconomic issue in Sri Lanka. The rapid development, urbanization, migration and population growth will further pressing these issues. To way-forward to the transition to more sustainable waste management in Sri Lanka, It is better to build a "resource management infrastructure" with the fundamental change of mindsets and attitudes toward "waste" into "resources" and tap the resource value of waste. With the present condition in view of the long-term upward trend and volatility of prices material recoveries, many profitable new business opportunities are available in environmentally responsible recycling and waste disposal.

Therefore suggested that the importance of ISWM by maintaining a sustainable composite mechanism through locally – available materials and expertise, with evidence based approach planning and strategy by eliminating the potential risks to provide a clean, healthy pleasant living environment and resource management culture for current and future generations of Sri Lanka.

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Comparison on Disposal Strategies for Clinical Waste: Hospitals In Sri Lanka

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Abstract: Clinical waste is potentially dangerous because it may contain waste resulting from medical, nursing, dental, pharmaceutical, skin penetration or other related clinical activity. Therefore, it is important to exercise special caution in the management of clinical waste in order to minimize its potential danger to public health and environment. Hence, this research intends to conduct preliminary study on clinical waste management practices with special emphasis to disposal strategies and associated cost. Six case studies, both public and private hospitals were used to collect data covering nineteen semi-structured interviews. Findings revealed that the highest and the least clinical waste generated were infectious and pharmaceutical waste respectively. The cost effective disposal strategies were diesel incinerators (Rs. 28.22 per kg) and dispose in a land (Rs.12.50 per kg). In general, cost for disposal of clinical waste in public sector hospitals were Rs. 84,084.22 per day while private sector hospitals were Rs. 42,101.89 per day. Negligence of the worker's safety and issues from the outsourced companies, were the common and critical challenges for both private and public hospitals

Keywords: Clinical waste, Clinical waste management, Cost, Disposal strategies, Hospitals.

1. Introduction

Clinical waste, one special kind of hazardous wastes, which contains a mass of virus, bacteria and chemical agent, is listed as number one Hazardous Wastes at "National Hazardous Wastes List" in China [1]. The World Health Organization (WHO) recognizes that in many countries improper management and disposal of clinical waste continue a significant threat to the healthy working environment [2]. In general, clinical waste is reflecting high quantity, intensive disposal route and significantly higher costs compared to other waste categories [3]. Thus, many hospitals have faced financial difficulties in managing of clinical waste [4].

Equally in Sri Lanka, although the regulations had been gazetted by Central Environmental Authority (CEA) that improper disposal of clinical waste is an offense, still it remains as a problematic area [5]. Further, there are less special strategies have been established within the local level in order to manage clinical waste in cost effective manner [6]. According to the report identification of cost effective solutions for disposal of clinical waste is one of the main challenge face by hospitals since it require high technological and capital input. Though, few of the major hospitals operate modern treatments or outsource to a private sector, most hospitals are lacking of cost effective options to dispose clinical waste.

Thus, there is a need to develop a proper strategy for clinical waste disposal which is cost effective in long run. Within this context, this research intends to examine the clinical waste management practices in both private and public hospitals in Sri Lanka in order to identify most cost effective disposal strategies to clinical waste. This paper presents disposal strategies, waste composition, cost, challenges and remedies associated with clinical waste management in Sri Lankan hospitals, both public and private located in Colombo district.

2. Literature review

2.1 Clinical waste, Composition and Cost

There are less clear definitions for clinical waste which are generated from hospitals [7]. A study of [8] in the European Union mentioned that the definition of clinical waste can vary significantly among countries. Moreover, there are several terms used to describe clinical waste like medical waste, health care waste, hospital waste, hazardous waste and infectious waste [7]. As per the report of [9] hospital waste can be classified as clinical and nonclinical waste in broader term. Table 1 presents categories of clinical and non-clinical waste with their examples.

Table 1: Categories of clinical waste

Category	Examples
Pharmaceutical waste	Expired or unnecessary pharmaceuticals and drugs
Sharps	Needles, syringes, blades, broken glass, scalpels
Infectious waste	Lab cultures and stocks of infectious agents, wastes from isolation wards, tissues, materials or equipment that have been in contact with infected patients
Radioactive waste	Radioactive substances including used liquids from radiotherapy or lab work
Chemical waste	Solid, liquid and gaseous chemicals from diagnostic and experimental work, cleaning materials
Pathological waste	Body parts, human fetuses, blood, other body fluids
Non clinical waste	Packaging materials such as cardboard, office paper, leftover food, cans

Source: ([10]; [11])

In order to develop proper waste management strategies, it is important to characterize the composition of the waste stream with quantities [12]. It varies according to the area, scale of health care facilities, specialty and practice procedure [13].

A research concluded by [14] in UK revealed that the weight of a domestic waste bag varied between 0.6 - 5.6 kg with an average of 2.45 kg, while the range of clinical waste bag weights was 0.5 – 4.0 kg, at an average of 1.45 kg. [15] state that infectious waste is the serious waste category which is accounted for the largest amount with 1241.71 ton/year while pharmaceutical waste is accounted for the least amount with 30.64 ton/year in Croatia. [3] state clinical waste is one waste stream, reflecting high quantity, intensive disposal route and significantly higher costs in UK.

2.2 Clinical Waste Treatments and Disposal Strategies

Safe handling and disposal of clinical waste constitutes as a major challenge of the healthcare sector around the world [16]. [17] mentioned that management of clinical waste are required significant improvements in the current practices in order to ensure public health and environmental protection in Cameroon. In general there is no single disposal practice for the managing of hospital waste. In most cases, various practices including landfills, incineration, autoclaving, and recycling are used in combination [10].

The most common methods utilized in healthcare sector to dispose clinical waste in different countries are shown in Table 2.

Table 2: Disposal methods of clinical waste in different counties

Country	Disposal Methods
Algeria	Open Dumping, Incineration
Mongolia	Open Dumping or Burning, Incineration, Autoclaving
South Africa	Landfill, Open Dumping, Incineration, Autoclaving
PalestinianTerritory	Open Burning, Dumping, Incineration, Thermal Disinfection
Bangladesh	Dumping

Nigeria	Dumping, Burning, Incineration
Mauritius	Incineration, Sanitary Landfill
Libya	Dumping, Incineration
Brazil	Landfill, Incineration, Autoclave
Bahrain	Landfill, Incineration, Autoclave
El-Beheira Governorate	Dumping
Egypt	Incineration
Greece	Recycling-Reuse, Pyrolitic Combustion, Land Fill
Korea	Incineration, Autoclave, Recycling
Malaysia	Landfill, Incineration, Recycling

Source: [7]

According to the Table 2, incineration is the most common method while landfill and open dumping methods are also visible in many countries. Most of the countries used two or more than two disposal methods. However Bangladesh, ElBeheira Governorate and Egypt used only one disposal method. In South Korea, treatment on-site, such as on-site incineration and microwaving, is the cost-effective treatment of clinical waste [9].

2.3 Clinical Waste Management in Sri Lanka

Although Sri Lanka consists of impressive heath care indicators, certain shortcomings of the health care system are visible due to unequal distribution of resources, lack of funds and long term political and bureaucratic commitment towards health issues and poor macro- and micro-health planning [18]. According to the Sri Lankan [12] eight categories of clinical waste were identified, such as infectious waste, pathological waste, sharp waste, pharmaceutical waste, genotoxic waste, chemical waste, radioactive waste, pressurized containers and waste with high content of heavy metals. Table 3 has been extracted from the draft report of Situation Analysis and National Action Plan, 2001 which presented the results of an initial assessment undertaken in various medical institutes by Ministry of Health. It illustrates the production of non-risk and hazardous health care waste per district in Sri Lanka.

Table 3: Non risk and hazardous health care waste per district in Sri Lanka

District	Ton/day Non risk	Ton/day hazardous	Health Care %
Colombo	11.84	3.28	26.8%
Gampaha	4.15	1.28	10.5%
Kandy	2.98	0.91	7.5%
Kurunegala	2.28	0.76	6.2%
Galle	2.45	0.74	6.0%
Anuradhapura	2.31	0.63	5.2%
Ratnapura	1.73	0.53	4.4%
Badulla	1.89	0.53	4.3%
Kalutara	1.50	0.48	3.9%
Jaffna	1.36	0.41	3.4%
Matara	1.24	0.34	2.8%
Kegalle	0.69	0.29	2.4%
Matale	0.63	0.28	2.3%
Puttalam	0.55	0.24	2.0%
Batticaloa	0.91	0.26	2.1%
Ampara	0.48	0.21	1.7%
Polonnaruwa	0.35	0.16	1.3%
Nuwaraeliya	0.38	0.16	1.3%
Hambanthota	0.34	0.16	1.3%
Monaragala	0.37	0.16	1.3%
Trincomalee	0.34	0.15	1.2%
Vavuniya	0.25	0.11	0.9%
Mullaitivu	0.14	0.07	0.6%
Mannar	0.09	0.04	0.3%
Killinochchi	0.06	0.04	0.3%

Source: [6]

Colombo represents highest percentage of healthcare waste with 26.8% while Mannar and Killinochchi show least percentages with 0.3%.

Table 4 indicates the treatment technologies used for clinical waste management extracted from the same report [6].

Table 4: Treatment technologies of clinical waste in Sri Lanka

Treatment Technology	Description
Burial	▪ Infectious and general waste are being buried in some of the health institutions where land space is available
Open burning	▪ Mixed waste or infectious waste separated are being burnt
Barrel incinerators	▪ Infectious waste are being put to a barrel placed on bricks and then burnt
Sharp pits	▪ Deposit sharps in a pit layer wise covering with lime
Needle burners	▪ Burning of infected part of the syringes
Incineration	▪ Use low temperature (below 1000'c) single chamber incinerators ▪ Use dual chamber high temperature (above 1000'c) incinerators for incinerating infectious waste and sharps
Steam Sterilization	▪ Autoclaving- laboratory cultures and some infectious waste are autoclaved before disposal ▪ Indirect Steam Sterilizationt
Chemical disinfection	▪ Some infectious waste are chemically disinfected (Sodium Hyper Chloride)
Placenta pits	▪ Placenta are put in to a series of pits alternatively for natural digestion

Source: [6]

As per the report on Situation Analysis and National Action Plan, 2001 by the Ministry of Health Sri Lanka [6], the most popularly used technologies for the clinical waste management are autoclaving and incineration. Moreover, the choice of treatment technology is depend on the various factors such as, local conditions, impacts to public health and the environment and the overall waste management strategy of the country.

3. Research Methodology

The research was carried out under qualitative research approach. Data were collected from an extensive literature review and nineteen semi structured interviews. The interviewees were qualified professionals in clinical waste management involved in health care sector in Sri Lanka. Literature review was mainly focused to identify clinical waste types, classifications, composition, disposal strategies and cost associated with clinical waste in locally and globally.

Interviews were focused on gathering data from both private and public hospitals in Sri Lanka, mainly to identify the most cost effective disposal strategies to clinical waste. "Hospital" was considered as unit of analysis for this study. Six hospitals located in Colombo were selected from both private and public as illustrate in table 5. Colombo district is selected for data collection since it generates the highest amount of health care waste in Sri Lanka.

Table 5: Interview profile

Case (Hospital)	Sector	Designation of the interviewer
A	Private	Health and safety consultant Senior executive housekeeper Executive housekeeper Pharmacist
B	Private	Senior manager support service Senior executive facilities Chief pharmacist
C	Private	Housekeeping executive Pharmacist
D	Public	Nursing officer Public health inspector Chief pharmacist
E	Public	Infectious waste control officer

		Public health inspector Chief pharmacist
F	Public	Nursing officer Public health inspector Chief radiologist Chief pharmacist

Content analysis was used for analysis the qualitative data gathered from the cases.

4. Research Findings

The findings of the study present in four broad headings as following through the cross case analysis of the six cases covering six categories of clinical waste.

- Composition of clinical waste
- Disposal strategies and relevant challenges with prevailing remedies of clinical waste
- Cost of clinical waste disposal
- Cost effective clinical waste disposal strategies

4.1 Composition of Clinical Waste

Composition of each categories of clinical waste generate per day is presents at table 6.

Table 6: Composition of clinical waste

Case/Hospital		Pharmaceutical waste	Sharp waste	Infectious waste	Pathological waste	Radioactive waste	Chemical waste
Private kg/day	A	3.33	15	86	12	0	0
	B	3	13	245	14	0	0
	C	2.8	35	230	15	0	0
Public kg/day	D	32	25	600	50	0	0
	E	16.62	20	360	31	0	0
	F	19	216	183	25	10	0

Accordingly it is clear that infectious waste, sharp waste, pathological waste and pharmaceutical waste are the most common clinical waste types in Sri Lanka. Among them infectious waste is the critical waste category which is generated a massive quantity in almost all the cases excepting the case A. Pharmaceutical waste represents least generating quantities in all the cases excepting the case D. Further, only one hospital generates radioactive waste around 10 kg per day and none of the hospital reported on generating chemical waste. In general, hospitals generate sharp and pathological waste below 25 kg/day and more than 200 kg/day of infectious waste. [21] revealed that sharp, pathological and infectious waste reported generating less than 30 kg/day in Philippines. Accordingly, Sri Lankan hospitals generated more infectious waste than other countries.

However, study of [15] proved that infectious waste is the serious waste category which was accounted for the largest amount with 3401.94 kg/day while pharmaceutical waste was accounted for the least amount with 83.94 kg/day in Croatian counties which is more similar to Sri Lankan findings. Therefore, as mentioned by the [19] clinical waste compositions may differ from country to country.

4.2 Disposal strategies, challenges and remedies

In house Diesel incinerators, gas incinerators and outsourcing (for incineration) are the common strategies identified for the infectious waste management in the current practice of private hospitals and associated challenges as follows.

- Outsourced companies are buying limited categories of infectious waste and only more than 150 kg of clinical waste
- Having fix rate for the outsource companies
- Absence of the outsourced company to collect the clinical waste
- Breakdowns of the hospital incinerator or the outsourced companies' incinerators
- Impossibility of using invertech machine for infectious waste

The empirical findings distinguished the need of regularly conducting awareness programmes, providing Personal Protective Equipment (PPE) and signing an extra agreement with another company to face the emergency situations like machine breakdowns and absence of the outsourced companies as remedies for afore mentioned challenges.

With reference to public hospitals, hydroclavin machine and outsourcing are the common strategies identified and associated challenges as follows.

- Attitudes of the patients and Patients' behaviour
- Unawareness of the staff regarding the clinical waste management and the colour code system
- Lack of space to use incinerators
- Problems from the animal's
- Increasing outsource company charges
- Lack of safety bins to collect infectious waste

Accordingly, public and private sector hospitals are facing different challenges mostly associated with outsourcing companies.

Some clinical waste placed with the domestic waste is the critical challenge in Greece [20]. Equally, this is the common challenge faced by the Sri Lankan public sector hospitals as well. Conducting awareness programmes to each and every employee, taking action to build a closed areas, daily visiting the wards, conducting audits, appointing separate person to handle waste management of each ward are revealed as remedial actions, mostly visible in public sector hospitals in Sri Lanka.

With reference to sharp waste, gas incinerators, invertec machines and the diesel incinerators are the strategies used by the private sector while all the public sector used to outsource their waste to outsourcing companies. Both private and public sector hospitals have common challenges and similar remedies for management of sharp waste which is more similar and common to the infectious waste.

Further, findings revealed that both public and private sector hospitals has outsourced the management of pharmaceutical waste. The outsourced companies dispose these waste in separate lands as disposal strategies and associated challenges as follows.

Table 7: Pharmaceutical waste management challenges

Public sector	Private sector
▪ Changing the prescribing pattern	▪ Negligence of the workers
▪ Changing the drugs policies	▪ Issues from the outsourced companies
▪ Threats from the rats	▪ Safety issues
▪ Wrong estimations	

Exchanging the pharmaceuticals with the suppliers before expiring and providing special safety equipment for the employees were identified as remedies for the private sector hospitals and sending the pharmaceuticals to the Medical Supply Division (MSD) before expiring and transferring the unnecessary pharmaceuticals to other hospitals are distinguished as remedies for the public sector hospitals.

Referencing to the pathological waste, incinerators and burying in the cemetery are the disposal strategies used in the private hospitals while transferring to florists and outsourcing are the strategies for the public sector. Radioactive waste is identified only in one hospital and none for the chemical waste. All these categories of waste are experiencing fewer manageable challenges. Next section of the paper presents the cost of the each disposal strategies discussed.

4.3 Cost of clinical waste disposal

Table 8 demonstrates the cost in Rupees (Rs) per kg for the each categories of clinical waste.

Table 8: Cost of clinical waste in Rs. per kg

Case/hospital		Pharmaceutical waste	Sharp waste	Infectious waste	Pathological waste	Radioactive waste
Private Rs/kg	A	58.01	89.64	89.64	89.64	0
	B	62.00	66.83	92.50	10.25	0
	C	59.52	28.22	28.22	18.00	0
Public Rs/kg	D	12.50	87.00	53.52	16.33	0
	E	20.05	71.75	91.75	16.07	0
	F	21.66	29.66	29.66	39.02	29.66

Findings revealed that some private and public hospitals allocated more cost on infectious and sharp waste while some allocated less cost. According to table 8, 89.64 and 92.50 Rs/kg are the highest cost for the sharp and infectious waste respectively while 28.22 Rs/kg is the lowest cost for both sharp and infectious waste. Though, for both highest and lowest cost, disposal strategies for sharp waste are incinerators, the highest is used LP gas where the lowest is used diesel. Thus, diesel incinerator is more cost effective than the gas.

Further, findings revealed that private sector allocates high cost on pharmaceutical waste while public sector allocates less cost. According to table 8, 62.00 Rs/kg is the highest cost for the pharmaceutical waste while 12.50 Rs/kg is the lowest cost.

Most of the hospitals spend less cost on pathological waste. However, cost of case A is excessively high compared to other hospitals, since this uses incinerators. Accordingly 89.64 Rs/kg is the highest cost for the pathological waste while 10.25 Rs/kg is the lowest cost. Here, only one hospital generates radioactive waste and the cost is 29.66 Rs / kg.

In summary, figure 1 illustrates the total clinical waste of public and private hospitals per day in Rupees in Sri Lanka.

Figure 1: Total cost of clinical waste

According to the figure 1 the highest cost for clinical waste are reported Rs. 84,084.22 per day for the public sector and Rs. 42,101.89 per day for private sector. Accordingly public hospitals spend double in cost like private hospitals on clinical waste disposal.

4.4. Cost Effective Clinical Waste Disposal Strategies

According to the findings, highest cost for infectious waste represented the outsourcing strategy would be 92.50 Rs/kg while the lowest cost represented the incinerator would be 28.22 Rs/kg. Generally, the highest cost represented for gas incinerators while the lowest cost represented for diesel incinerators. Therefore cost effective strategy for infectious waste is identified as diesel incinerators.

Sharp waste cost detail both highest and least cost represented the incinerators would be 89.64 Rs/kg and 28.22 Rs/kg. However in here also highest cost represented gas incinerators while least cost represents diesel incinerators.

Pharmaceutical waste represented the outsourcing strategy would be 62.00 Rs/kg while least cost represented the dispose in a land strategy would be 12.50 Rs/kg. The reasons for this deviation is nonincrease of cost per kg align with increasing quantities of pharmaceutical waste and labourer cost. Hence dispose in a land is identified as the cost effective strategy for pharmaceutical waste disposal.

According to the pathological waste disposal cost detail, highest cost represents incinerator would be 89.64 Rs/kg while least cost represents the strategy of burying in the cemetery would be around 10.25

Rs/kg. Thus, cost effective strategy for pathological waste is identified as burying in the cemetery. There is only one strategy for radioactive waste and the cost would be 29.66 Rs/kg. Accordingly Table 9 illustrates the summary of cost effective disposal strategies and associated cost for different categories of clinical waste.

Table 9: Cost effective disposal strategies for clinical waste

Type of waste	Strategy	Cost Rs/ kg
Infectious and Sharp waste	Diesel incinerator	28.22
Pharmaceutical waste	Dispose in a land	12.50
Pathological waste	Burying in the cemetery	10.25
Radioactive waste	Outsource	29.66

5. Conclusions

Improper management and disposal of clinical waste continue a significant threat to the healthy working environment. The empirical findings recognized that, public hospital generates more clinical waste than private hospitals mainly due to high number of patients. Infectious waste reported as the serious waste category which is generated in massive quantities in all the cases. Apart from that findings revealed none of the hospitals generate chemical waste and only one hospital reported in generating radioactive waste.

Public hospitals were allocated Rs.84,084.22 per day while private hospitals were allocated Rs.42,101.89 per day for management of clinical waste. Simply public hospital cost was approximately double in amount compare to private hospitals. Gas incinerator, diesel incinerator, hydroclavin machine and outsourcing were distinguished as infectious and sharp waste disposal strategies, outsourcing and dispose in separate lands were identified as pharmaceutical waste disposal strategies, incinerators,
burying in the cemetery, transferring to florists and outsourcing were identified as pathological waste disposal strategies and outsourcing and transferred to the sea through drain line were distinguished as radioactive waste disposal strategies.

Issues from the outsourced companies, negligence of the workers, safety issues were the common and critical challenges for management of clinical waste. Finally the empirical findings recognized the cost effective disposal strategy for infectious waste and sharp waste as diesel incinerators would be 28.22 Rs/kg, dispose in a land as pharmaceutical waste would be 12.50 Rs/kg and outsource strategy for pathological waste would be 10.25 Rs/kg.

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Management of Waste Derived Due To Conflicts in the Context of Post Conflict Reconstruction

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Abstract: Increase of man-made conflicts around worldwide has created conflict waste as a major challenge. Improper management of conflict waste creates issues in public health and safety, environment, economic cost on already strained communities and post conflict reconstruction. End of three decade lasted civil war in the country has created significant amount of conflict waste affecting post conflict reconstructions. Hence, this research was mainly focused on identifying the impact of conflict waste on post conflict reconstruction with special emphasize for Construction and Demolition (C&D) waste. Five organizations involved in post conflict reconstruction in conflict areas were selected as the cases and semi structured interviews were conducted to gather data. The collected data was analyzed using content analysis. It was revealed that, lack of capacity to handle an enormous quantity, difficulty in estimating the quantity and composition, identifying dumping sites, coordination among different parties' involved, financial incapability, and lack of technical peoples as major challenges related with management of conflict C&D waste during post conflict reconstructions.

Keywords: Conflict waste, C&D Waste, Post Conflict, Reconstruction

1. Introduction

The world is confronting a growing frequency and intensity of man-made conflicts that has destructive influences to the society. Man-made conflict situations mainly rise due to political motivations or lack of foods. International humanitarian law distinguishes two types of armed conflicts, namely international, opposing two or more States, and non-international, between governmental forces and non-governmental armed groups, or between such groups only [1].

Both International and non-international (Civil war) creates direct and indirect deaths, direct injuries (both physical and psycho logical, permanent and temporary) and indirect injuries, population dislocations and environmental problems. These include direct and indirect damages and destruction to environment due to use or release of explosive, corrosive, and chemical combinations and toxic materials. Also it reduced quality of life, fulfilling basic needs, future visions of affected societies, lack of investment and exchange [2].

Among these consequences infrastructure and building damage has a huge impact on the country's economy and post conflict permanent building reconstructions. Generally, it is generated by the demolition and site clearing of the large numbers of buildings and structures which were damaged fully or partially due to conflict [3]. Many conflict and post-conflict settings, infrastructure has been destroyed and roads, power stations, water pipelines, hospitals, schools, and sewerage facilities must be completely rebuilt [4].

In this context, C&D waste management represents one of the main challenge due to lack of political interest, less awareness of respective urban managers and limited resources [5]. Specifically, the waste generated by conflict such as concrete, brick, bridges, roadways, railway structures requires heavy machinery such as excavators and bulldozers which creates a high cost [6]. Equally, in Sri Lankan due to last three decades of civil war, it has been identified that lack of awareness of the mechanisms and systems for post conflict C&D waste management is a critical challenge during post conflict reconstruction works. The common practice is managing waste by open dumping. However, lack of sites to use for waste disposal has been major impediment. Within this context, this research was focused on investigating the impacts of conflict waste on post conflict reconstruction with special

emphasize for C&D waste for future resilience. Accordingly, the paper presents conflict waste and it's impacts on post conflict reconstructions with special emphasis to C&D
waste in Sri Lanka. The aim was achieved through three objectives; identification of composition of conflict waste, investigation on the impact of construction and demolition waste for post conflict reconstruction, and by proposing remedies to overcome the above mentioned impacts. The scope of this study is limited to construction companies carrying out permanent construction buildings extensively in highly war affected areas. The paper presents brief introduction to post conflicts and impacts on C&D waste in post conflict reconstruction, methodology adopted and research findings.

2.0. Literature review

2.1 Conflict and post conflict

A conflict is an external clash with adjoining countries or internal clash with in a country together with disputes disturbing only to specific parts of a country [7]. Moreover, the conflicts are commenced in a country due to the greed and grievances against its government due to the dominant behaviour or due to the injustice against the citizens and in some cases due to discrimination of a certain part of the population [8].
A country which is having a recent end to violence can be described as a 'post-conflict' country [9]. In order to achieve a long-lasting recovery over a conflict, countries have to perform a number of improvements and changes including building lifelong peace by ensuring political stability, reconstructing or strengthening the basic functions of the country`s administration, resettling migrants and internally displaced citizens, disbanding combatants, and rebuilding basic economic and social infrastructure, reorganizing public expenses by shifting public money allocated for military expenditure into national development, reforming revenue by changing the source and method for collecting taxes and other revenues, restructuring trade and exchange by changing the method of collecting import duty and quotas as well as implementing new policies to the foreign exchange markets, restructuring financial sector by modifying the methods of managing lending and borrowing by the financial system, and reforming sector by varying policies for agriculture, industry, energy, and utilities [10].

The era of post conflict reconstruction was one of the most crucial and controversial periods which the societies that were affected by the war. Post conflict reconstruction works can be categorized into some particular key areas. Most of the time government and other donors has largely been directed into four areas as follows [11]; political reconstruction such as moving to elections, support for security such as retraining the police force humanitarian relief Reconstruction of physical infrastructure Countries making the shift from war to peace face a multi-pronged transition in the economic, legal, political, social, and security sectors. This multifaceted transition, economic reconstruction is fundamentally different from the 'development as usual' approach taken by the international community to address typical socio-economic challenges faced by peaceful developing countries [12]. It means that the economic reconstruction is not an easy task to accomplish. Economic reconstruction involves basically transforming a society's economic institutions in order to short-term growth and development as well as to resolve supposed economic and social problems. The economic reconstruction process as a whole therefore involves rebuilding infrastructure, restoring physical, social, and human capital, and restructuring fiscal, monetary, and trade policies in order to allow for a strong private sector to emerge.

2.2. Waste management in post conflict reconstruction

Post-conflict societies are burdened by various problems where the generation of waste either from demolished structures, trash from remaining weapons and other hazardous war material are most critical [13]. In general, waste streams generated by conflict situation can be categorized
as,

- Construction and demolition debris
- Industrial and toxic chemicals
- Human and animal corpses
- Landmines

During a conflict, highly considerable amount of buildings and infrastructures are damage resulting more C&D debris. In 1999, in Kosovo's more than 120,000 housing units were damaged and it was roughly estimated that the waste from damaged buildings and structures reached to a magnitude of 10 million tons. In Gaza, it was estimated as 6,300 homes were destroyed or heavily damaged [14]. Table 1 summarizes impact on building and infrastructure facilities by conflicts in different parts of the world.

Table 1: Summary of impact on buildings and infrastructure facilities

Type of Damage	Gaza North	Gaza	Middle Area	Khan Yunis	Rafha	Total
Building destroyed/ severely Damaged	585	1000	95	241	739	2660
Greenhouse severely damage / destroyed	58	74	9	25	20	186
Impact craters on road	66	82	13	16	43	220
Impact craters in field	256	172	59	83	141	711
Total	965	1328	176	365	949	3777

Sources- United Nations Environment Programme, 2009[15]

As tabulated in table 1, major percentage of the damages due to the conflict was building related damages. Therefore it is certain that the impact of C&D waste to the reconstruction works were very high. Specifically, conflict waste management become critical due to mix up materials causing various kinds of debris – hazardous to nonhazardous, biodegradable and recyclable to nonrecyclable waste. This can cause entire mounds of debris to deteriorate rapidly, making recovery and recycling more difficult [16]. This is further aggravated increasing use of land for new construction, renovation, demolition of old structures and the reconstruction or expansion of the road transportation network [17]. The collection of conflict waste happens in two stages: (1) to clear debris those obstructs emergency areas and eliminate or mitigate the exposure to hazardous waste, (2) to clear the debris to facilitate reconstruction [18].

In the current state it was identified that even though waste management strategies addressed the disaster waste there are no mush attention for the conflict waste management. It is much cleared that during the site clearing and reconstruction works, there are several opportunities for the reuse and recycling of the demolition debris, with subsequent providence of building materials to the ensuing reconstruction works and thereby reducing the quantities of waste going to the often-limited disposal sites. It should be noted that the construction and demolition waste stream does not solely include the demolition rubble, but also the construction waste generated during the ensuing rehabilitation and reconstruction works. The composition of construction and demolition waste is consisting largely of concrete, masonry, metals, plastics, glass and wood. Demolition waste may also contain hazardous materials [3].

2.3. Waste management in post conflict reconstruction in Sri Lanka

Sri Lankan Civil War was a conflict fought between the Sri Lankan government and the Liberation Tigers of Tamil Eelam (LTTE). Nearly, 30 years of civil war in Sri Lanka, was come to an end with the military defeat of the separatist LTTE in May 2009 [19]. The conflict gained greater momentum since the 1983, has killed around 70 000 of people and displaced around two million people [20]. The end of the war left past conflict zones of 2,061 Sq.km heavily contaminated with approximately 1.6 million land mines. By January 2012, demines employed had cleared 1,934 Sq.km leaving about 127 Sq.km yet to be cleared. Three years after the conflict, the requirements are evolving from humanitarian relief to early recovery and development. By the end of September 2012, 468,000 people had returned to their places of origin, while an undetermined number remain displaced in various parts of the country. In this context, preserving place of safety and ensuring protection for refugees remain as a priority [21]. The destruction of property during the war is a huge problem when reconstructing the economy of the particular society. This was further aggravated due to the deaths of young men and over hundred thousand people were injured in battle leaving them out of the labour force. Further, the loss of wealth within the country caused major problem in rebuilding the economy. The lack of industry and railroads and other facilities is decrease the manufacturing capability of the country. These are some of the common major issues in rebuilding the economy of war affected countries [22].

In respect of conflict waste management, nonavailability of disposal lands, treatment mechanism, recycling and reuse options, transportation of waste materials, accessibility to waste management facilities, environmental hazards, financial implications, labour availability, and legal and ethical responsibilities are the critical challenges [23].

As a result, there is a huge impact on reconstruction by the waste generated by the war.
Major impacts of waste on reconstructions are;

- Health and environmental issues
  The impact of debris disposed in an uncontrolled way can have a negative impact in the quality of the surface and groundwater due to contamination with a trace of hazardous chemicals.
- Institutional issues
  The municipalities have the responsibility for solid waste management and often do not have the expertise in the topic.
- Legal issues
  Lack of legal provisions and guidelines for management of conflict waste.
- Financial issues
  The uncontrolled disposal of waste can lead to huge economic losses due to loss of aesthetics especially in tourist areas and lack of possibilities for recycling or reusing of the useful materials, recuperating part of the financial losses due to the conflict.
- Socio-cultural issues

Debris management is an essential part of reconstruction. There have been several programmes that have used the cash for work approach in clearing debris. Clearance of debris may have a positive social impact, though there is scant evidence from these projects [24].

For an example, Iraq had to deal with more than 0.9 million metric tons of conflict waste incurring a total cost of approximately US$30 billion. In Sri Lanka, approximately produced 0.6 million metric tons of waste. The disposals begun with open burning which was eventually stopped due to air pollution. Then, burying of waste in existing dump-sites, coral mining areas and even playgrounds were practised to manage conflict waste [23].

Most extraordinary investments by aid agencies in waste disposal and management during post conflict activities are usually not accompanied by awareness-raising campaigns on civic responsibility, hygiene and the benefits of waste recycling thereby leading to failure of the intention.

3. Methodology

The aim of this research is to identify the impact of construction and demolition waste for post conflict reconstruction. Case study approach was used as it provide in depth investigation to collect new information that holds across many cases which can stimulate new theoretical thinking and less restrictive than other methods. Accordingly, five construction companies involved in post conflict reconstruction were selected as the cases The profile of the cases are illustrated at table 2.

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Table 2: Profile of cases

Case	*Type*	*Grade*	*No.of Participants*
A	Contractor	C1	1
B	Contractor	C1	1
C	Contractor	C1	2
D	Contractor	C1	1
E	Contractor	C1	1

Six semi structured interviews were conducted covering five case studies comprising with project managers, civil engineers and site engineers. The profile of interviewees is indicated in Table 3.

Table 3: Interviewee profile

Interviewee	*Designation*	*Experience*
I 1	Project manager	8 Years
I 2	Civil Engineer	6 Years
I 3	Civil Engineer	10 Years
I 4	Project Manager	6 Years
I 5	Project manager	9 Years
I 6	Site Engineer (Civil)	7 Years

An appropriate interview guideline was prepared by focusing the three objectives; C&D waste composition, impact of C&D waste for post conflict reconstruction and the remedies. The interviews were tape recorded with the permission of the interviewee and thereafter transcripts were developed.

For the data analysis, code-base content analysis was used, since the interviews were semistructured, which consist with open ended questions. It breaks down data into segments with the purpose of organizing data to make them easier to interpret [25].

4. Research Findings

This section presents the findings of the analysed case studies on impact of construction and demolition waste for post conflict reconstruction and the probable recommendation to overcome the identified.

4.1 Composition of conflict waste in Sri Lanka

In Sri Lanka debris from damaged buildings and infrastructure, industrial and toxic chemicals, human and animal corpses, landmines, vehicles and heavy weapons identified as the major types of conflict waste. Among them damaged buildings and partially damaged buildings, houses were the biggest issue on post conflict reconstruction. According to the respondents, waste generated by the damage buildings vary district to district. For an example in Kilinochchi District there are less permanent houses and have used clay and timber to build houses. There are only few buildings with cement and bricks in town centre. Majority of the residential buildings were done with clay bricks and mud bricks and timber frames and the kadjhan roof. However in the Mulativ District, for a long time people were establish under the LTTE control. In Pudukuduirruppu still there large houses and buildings and they are abandon and damaged. In pre tsunami period the coastal areas were fishing hunts, after the tsunami NGO's funded and people

build permanent houses and those were damaged during the war period. Therefore in these areas there are a lots of construction waste mainly damaged roof tiles, damaged bricks, foundation works. Thus district wise construction waste differ and similarly it effect to the constructions in those areas.

Findings revealed the similar impacts identified through the literature and summarised as follows with probable remedies.

4.2 Impact of construction and demolition waste for post conflict reconstruction and proposed remedies

- **Poor waste management practices**

Damaged buildings and infrastructure are often burned out, which results in the majority of the waste no longer exist. This also cause to reduced quantities of debris need to be managed. However, in managing those wastes, it is very difficult to find a properly established response plan. In general, none of the authorities promoting waste management, because it's very difficult to apportioning. In this context, responding for conflict waste should start from the community itself. Simple mechanism to separate, treat and dispose their waste need to introduce and response plans for each local authority need to be developed.

Waste response plan includes identification, and disposal of waste. In the current state without a proper plan the waste disposal is a very difficult task at the time of a disaster and people dump waste everywhere. Therefore, this issue should be addressed in the early stage of prior to the reconstructions. Due to that reason maintaining a conflict waste handling plan in municipalities in those areas is very essential. Though the plan is needed, it is very difficult and high costly to assess the waste (damage buildings) and implement properly established response plan. If construction companies can identify and separate the waste materials that they can destroy themselves and separate the materials that they can reuse, then the considerable amount of waste can be reduced. When demolishing debris, its needed heavy equipment such as excavators and bulldozers. Local authorities not having aforementioned heavy equipment. Hence the better option is that debris management left with the contractor's responsibilities and provide necessary supportive things to do that work properly. Because most of the times the contractors have such kind of a machineries. The Table 2 provides an example for a waste response plan.

Table 2: Waste response plan

Waste stream	Transportation option	Disposal options	Recycle	Reuse
Concrete/bricks	Wheel barrow or excavator/bulldozer offload into truck for haulage	Disposal at temporary site for future recycling if uncontaminated debris Otherwise disposal at dumpsite/landfill to be used as cover material	Attempt to store for future recycling. If not possible, then limited options for recycling in emergency phase.	Can extract bricks, Steel for reuse
Timber	Wheel barrow or excavator/bulldozer offload into truck for haulage	If separated, reuse Otherwise dispose at dumpsite/landfill	Possible to separate timber for heating, cooking, shelter	Can use for roof structures or any other wood requirement of house construction
Unexploded objectives	Under controlled measures by specialists	Not applicable	Not applicable	Not applicable

Environmental, risk involves with workers, and logistical consideration

Dealing with conflict waste is a high risk activity due to risks of potential building downfall, hazardous materials such as chemicals and even unexploded objectives, decomposed organic waste and sharp items such as reinforcement bars and concrete. Further, the mines always create a high risk for the construction works. Due to that workers have to be so much careful and so much thorough.

Apart from that, logistical problems are the major problems in waste demolition and clearing phase. Hence defining a proper logistical plan is very essential taking into account not only the means of transport but also the actual possibilities of getting one place to the other, alternatives for the prompt safe delivery and relief assistance.

- **Financial impact**

Misuse of funds and unethical behaviours from most of the government authorities and irresponsible people result in incomplete projects which are abandoned in long run. Mainly, due to unavailability of regulations or mechanisms to regulate the funds donated for NGOs and other organizations by the donors. Simply, there is no responsible body to govern these funds and the projects. In this context, it is needed to have a properly established management to allocate funds, and monitor the projects in long run. Specifically, government should take the responsibility and prepare effective communication chain and inspect it in a regular time intervals and see the cash inflow, allocation of funds, and progress of the projects and finally analyze the effectiveness of the usage of funds.

- **Lack of dump sites**

Suitable approach need to be proposed to manage dump sites adhering regulations and recommendations of responsible authorities for safeguarding the public health and other issues. Identification of dump sites further aggravated due to banning of paddy field for usage as dump sites by the government. Also objections by the people of use of lands come as a greater issue hence they are not aware of works going to proceed and what are the benefits that they could acquire. Making people aware by conducting awareness programs lead to mitigate this problem.

- **Lack of awareness on waste management and less expertise**

There are some expertise within the country who aware of these aspects. However it does not mean that it was enough to work with the arisen problems. Therefore, getting the international support to fill the missing areas is the best option in this regards. Specifically, by getting aware of the plans and procedures which have been used with necessary changes for the benefit of our country will be useful.

- **Lack of trainee people familiar with reconstructions**

Experienced and educated professionals' contributions for reconstruction are less in conflict areas. Due to that they have only a minimal experience with regard to the constructions. As a result, vocational training centres with the help of government and local authorities were established to enhance the technical knowledge. However, progress of training programs is very poor and participation of people related with reconstruction has to be promoted.

- **Government involvement and restrictions**

Government involvement is a huge problem in reconstruction works. Because it governs the quality of the work and control some of the basic material prices. According the opinions by various interviewees by allocating enough funds, allocating committees to investigate problems occur, government can govern the qualities and control the matters against the reconstructions in a better way.

5. Conclusion

Generally civil wars disrupt economic, political and social system in Sri Lanka as well as worldwide. War is a man-made conflict between two political parties. Conflicts generate the significant quantities of solid wastes, which are unavoidable, and it causes major impacts on built environment. Most of the common wastes from the conflicts are building related waste and chemical hazardous waste. Management of these wastes is considerable challenges for national and local capacities during the rehabilitation, and prior reconstruction works.

This study focused on the identification of impacts of construction and demolition waste for post conflict reconstruction. Lack of capacity to handle an enormous quantity of waste, difficulty in estimating the quantity and composition of waste, identifying dumping sites and coordination among different parties involved, financial incapability, and lack of technical people were identified as the main impacts. Preparation of simple waste management plan at community level, waste handling plan at municipal level, waste demolishing plan at construction site level, establish transport facilities, better access routes, and enough dump sites associate with local authorities, implement safety procedures, provide adequate funds for both community and local authorities, make people awareness importance of conflict waste management, preparation of a proper mechanism for governing funds for reconstruction works and establish proper management with welldefined goals under single authority were proposed as suggestions to mitigate the impacts identified.

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Development and Performance Evaluation of the Leachate Treatment System at Gohagoda Municipal Solid Waste Disposal Site

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Abstract: A private company with technical support of University of Peradeniya has undertaken the task of rehabilitating the Gohagoda dumpsite. Not all of the leachate collection system (*LTS*) is completed yet and runoff water too gets mixed with the leachate on one side of the dumpsite. The *LTS* consisting of leachate collection tanks, a leachate treatment bioreactor(*LTB*) followed by an algae pond(*AP*), a floating wetland(*FW*), two sub-surface constructed wetlands(*SCWs*), two charcoal filter-beds(*CFBs*). LTS_{outlet} is being discharged into a natural stream (*NS*). In this research, the existing *LTS* was improved and the performances were evaluated. To determine the surface water quality of surrounding area and performance of the *LTS*, samples were obtained from 13 pre-defined points on weekly basis for two months, analysed for 14 quality parameters.

Average pH, dissolved oxygen (DO), chemical oxygen demand (COD) and biochemical oxygen demand (BOD) of inlet leachate to the *LTS* were 7.74±0.35, 0.46±0.5mg/L, 24,552±2,612mg/L and 4,125±965mg/L respectively. LTS_{outlet} of pH (7.16±0.23) was within the Central Environmental Authority water quality discharge standards. Average salinity, EC, DO, TDS, TSS, TS, VS, VSS, BOD, COD, PO_4^{-3}, NO_3^- and $NH_4^+$of LTS_{outlet} were 0.84±0.25‰, 1.71±0.52mS, 0.63±0.6mg/L, 852±261mg/L, 1,058±199mg/L, 1,303±772mg/L, 406±220mg/L, 609±111mg/L, 217±177mg/L, 780±1,049mg/L, 2.33±3.29mg/L, 0.97±0.27mg/L, 4.38±1.59mg/L respectively. Average pH, TDS, BOD, PO_4^{-3} and NO_3^- and NH_4^+ of NS_{outlet} were 7.69±0.39, 1,457±930mg/L, 1,382±784mg/L, 5.04±6.36mg/L, 1.58±1.26mg/L, 4.3±2.02mg/L respectively. The average removal efficiency of BOD was 95%.

The lower values of the indicative parameters were when the *LTS* was stabilizing and attaining the required standards even without high growth in *SCWs*, until heavy rainfall occurred. Therefore, it is essential to install sub-surface leachate interceptor drains and those connected to the leachate treatment system. It will require a proper dumpsite cover system to reduce infiltration and thus promote runoff. It is imperative to monitor and evaluate frequently the system and improve it with an aerated biological indicator pond.

Keywords: Clay-polythene-clay biofilter liner, Dumpsite, Rehabilitation, Leachate treatment

1. Introduction

The final disposal of municipal solid waste (MSW) is yet a major problem and an immense burden on the environment [4,5,6] and the local authorities, particularly in the most urbanized areas such as in Colombo, Dehiwala-Mt Lavinia and Kandy in Sri Lanka [2]. All of the dumpsites are located in environmentally sensitive areas and near residential, commercial or institutional establishments [2]. For instance, the Gohagoda dumpsite in Kandy is bounded by the river Mahaweli to the east with very little rock exposures but only weathered soil profiles [7] which is the major fresh water supplier for down-stream communities for drinking, agricultural and sanitary requirements. The Gohagoda dumpsite which has an aerial extent of 2.5ha [7] has been used since 1960s for open dumping of MSW collected within Kandy City limits and Harispaththuwa Pradeshiya Shabha (HPA). At present, about 150tonnes of MSW is disposed daily [1]. A project was developed by the EcoTech Lanka Limited with the collaboration of University of Peradeniya and Kandy Municipal Council (KMC) to rehabilitate the Gohogoda dumpsite and establish an integrated solid waste management system [1]. A Leachate management is one of major challenges in the rehabilitation efforts of the dumpsite. The estimated leachate generation was 30,810m^3/year [1].

Therefore, during the rehabilitation, an integrated leachate treatment system (*LTS*) was designed and established by combining landfill bioreactor technology with clay polythene clay composite liner system developed at University of Peradeniya which can reduce its strength to manageable level [3]. The *LTS* consisting of leachate collection tanks, a leachate treatment bioreactor (*LTB*) followed by an algae pond (*AP*), a floating wetland (*FW*), two sub-surface constructed wetlands (*SCWs*), two charcoal filter-beds(*CFBs*). Not all of the leachate collection system is completed yet and runoff water too gets mixed with the leachate on one side of the dumpsite. LTS_{outlet} is being discharged into a natural stream *(NS*). In this research, the existing *LTS* was improved and the performances were evaluated.

2. Materials and method

Improvements of the *LTS* which were done by the company and performance evaluation leachate treatment system were undertaken from 16th September 2014 to 25th October 2014.

2.1 Improvement of the leachate treatment system

The *LTB* was re-established with two leachate pumps, starter switches and level sensors. The blocked pipes with biofilms of *LTB* were cleaned and replaced with easy maintenance system. New pipelines to pump leachate to *AP* were laid. A gravel screen and a wetland around the *LTB* were constructed in order to collect and treat permeate.

After completing the construction of the *AP*, it was filled with fresh water to test the liner integrity for a month (20/08/2014 – 20/09/2014). The inflow and outflow pipe line installations were completed. An aeration system was installed and oxygenating rate tests were carried out before sending effluents of the sewage treatment system and *LTB*. DO concentration of the *AP* in 10 different locations during day and night time (26/09/2014 - 27/09/2014) were measured. Water hyacinth (*Eichhorniacrassipes)* plants were planted in the *FW*. The fencing around the *AP* and *SCWs* was done. Re-planting of cattail (*Typhalatifolia spp.*) plants in the SCWs was done and also a charcoal filter bed was converted to a wetland with water hyacinth (*Eichhorniacrassipes)* in order to improve efficiency of the treatment system.

In the absence of a proper leachate collecting network in the North East (NE) part of the dumpsite, a leachate collecting system comprising of two de-silting wells and a leachate storage tank was established and the pumping of the collected leachate commenced on 07th November 2014 with a leachate pump and floater switch and a delivery PVC pipe line in to the *LTB*. After connecting this leachate storage well to the *LTB*, most of the leachate flowing in to the natural stream was arrested, notably not the washouts during heavy rains as runoff.

2.2 Wastewater / water sampling locations for performance evaluation of the leachate treatment system

In order to obtain wastewater/ water samples in and around the site, sampling locations were selected as shown in Figure 1. Sampling was done on weekly basis from 14.10.2014 to 25.11.2014. SP1, SP2 and SP3 were sampling locations of the dumpsite, leachate collecting tank and leachate collection system of NE side respectively. SP4 and SP5 were the outlet of the LTB and the outlet of the sewage treatment system respectively. SP6 and SP7 were positioned in the in the *AP* while SP8 was placed in the *FW*. SP9 and SP10 were outlets of the *FW* and *SCWs* (LTS_{outlet}) respectively. Then SP11 was located on the natural stream which is flowing directly to the Mahaweli river. And SP12, SP13 were located on upstream and down steam of the river, respectively as shown in Figure 1.

3.3 Analytical parameters

Samples were obtained on three occasions before activating the leachate treatment system. The collected samples were tested for physical, chemical and biochemical parameters namely pH, electrical conductivity, salinity, DO, total dissolved solids (TDS), total solids (TS), volatile solids (VS), total suspended solids (TSS), volatile suspended solids (VSS), chemical oxygen demand (COD), biochemical oxygen demand (BOD), $N(NH_4^+)$, $N(NO_3^-)$, $P(PO_4^{-3})$ by using standard methods. Laboratory analysis was done at the University of Peradeniya. Removal efficiency (RE) was calculated by using equation1 and data was analyzed descriptively.

$$RE (\%)=(I - O) / I \times 100 \quad (1)$$

Where, I - Inlet and O - Outlet concentration

3. Results and Discussion

3.1Overview of the performances of leachate treatment system

3.1.1Leachate collecting network

Leachate collecting network is consisting of sub surface drains, PVC pipe lines and collecting tanks. Leachate is treated inside the collecting system too As reported by [1],biofilms have been formed with time on the inner surface of the leachate collecting system of backfill and slotted pipes due to that leachate is treated. Also due to crust formation and pipes are being blocked, so a flushing system is needed to clear the collecting network otherwise there could be a negative impact on the system. Leachate collecting network should be further improved to collect all the leachate without contaminating soil, because leachate flow spreads on the ground, creating vast pollution effects on soil, surface water and groundwater.

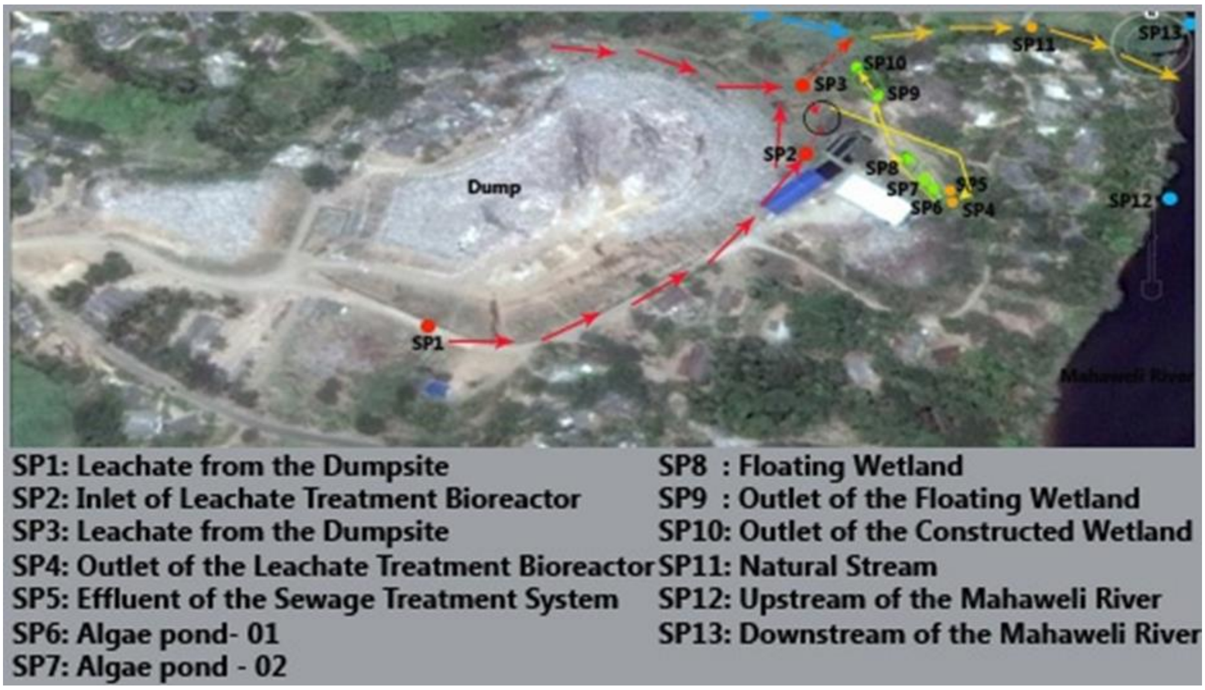


Figure 1: Sampling locations

3.1.2 Leachate treatment bioreactor

In the *LTB*, the pipes which are used for recirculation and pumping out of the system also suffer from the same fate of crust formation, thus flushing of the system is required as in the case of pipe network maintenance. Due to collection of runoff with heavy rainfall, the amount of leachate collected though the leachate collection system did increase far beyond the design capacity of the leachate treatment system. Therefore, the hydraulic retention time of the reactors drastically reduced. Consequently, the system did not function properly. This situation will continue, unless otherwise interceptor sub-surface drains are installed and connected to the leachate treatment system. After installation of interceptor subsurface leachate collection system, the dumpsite should be covered with bio-filter liner system immediately as proposed by Ecotech Lanka Ltd.

3.1.3 Algae pond and floating wetland

AP is full of water, up to 1.0 m in depth and algae lives with the uptake of nutrients of the leachate for their biological activities digesting the materials. But the nutrient content in the leachate is low for the growth of algae. Therefore, sewage treatment system effluent which is having a good carbon and prosperous sources was also directed to the *AP*.

DO is an important and critical factor for living aquatic organisms like algae. Therefore, with the supply of the required and optimum oxygen levels, the treatment effect can be enhanced in aerobic treatment systems. Figure 2 shows the DO concentration variation with time throughout on 27.09.2014 after fixing the aerator and before discharging the effluent to the *AP*. According to that DO concentration in day time was higher than night time that’s because of respiration and photosynthesis of algae. Those two biological activities are occurring at day light and not in dark time.

Consequently, in early morning DO concentration is much lower due to respiration effect and microbial uptake during night. *FW* is consisting of water hyacinth plants which have also treatment effect on wastewater. These plants do uptake organic and inorganic materials from wastewater for their biological activities. As expected, all the parameters did decline when the effluent went through the *FW*. Highest treatment can be seen before flowering stage for better performance because after flowering, the growth is lesser and precipitates nutrients like phosphate, heavy metals etc. *AP* and *FW* needs higher DO concentration and sunlight for photosynthetic organisms. Due to overload of the reactors during rainy period, algae died and plant growth reduced due to high concentration of leachate. Treatment effect also did decrease due to less retention time. So *AP* and *FW* were diluted by adding water, therefore algae and plants are growing again, thus the system was improving.

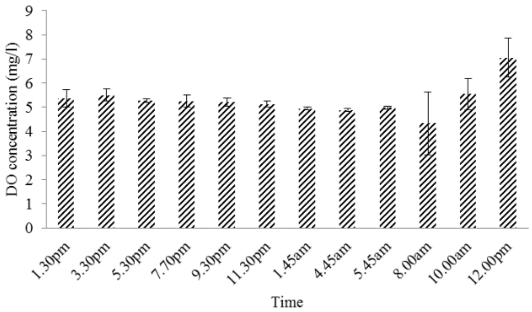


Figure 2: Do concentration variations of algae pond on 27.09.2014

3.1.4 Subsurface constructed wetlands

The effluent from the *FW* is then directed to the *SCWs*. Cattail plants are growing in the *SCWs* by up taking organic and inorganic compounds from the effluent of partially treated leachate and sewerage for their biological activities. There are also two charcoal filter beds that adsorb the organic and inorganic compounds of the effluent. Plants are not well grown yet, but there is an impact on leachate treatment. Cattail plants should be in the growth phase for better performance.

After plants grow to reproductive phase, plants should be pruned and some replaced. These plants need protection from animals too, thus fences are constructed surrounding the *SCWs*. *SCWs* should be followed by a bio-indicator pond. According to results, all the parameters have been declined after *SCWs* treatment.

3.2 Natural stream and river

At early stage of study period, the natural stream was directly contaminated with leachate and wash down to the River *Mahaweli*. After establishment of 2nd leachate collecting tank, direct contamination of leachate was arrested. With heavy rainfall interception, due to absence of dumpsite cover system, leachate was mixed with runoff water and therefore the quantity of leachate drastically increased and the treatment system got overloaded. Subsequently, during the rainy period, leachate was not directed through the leachate treatment system. Therefore, according to the study results, still the quality of natural stream water was poor and did not reduce to the required standards. Even though, the direct flow of leachate to natural stream is arrested, the quality of water may not be improved up to permissible level rapidly due to remaining deposits on the boundaries of the stream. Further, there is an abundant paddy land adjacent to the natural stream which was contaminated with landfill leachate for many years. So the runoff and washouts from this paddy land contaminate the stream water. This ecosystem will recover and rejuvenate slowly; otherwise intervention should be to expedite the recovery process. Although, effluent is treated to acceptable standards, it gets re-contaminated and it is an important issue for the stakeholders. According to the results, water quality of downstream was lower than upstream. This is certainly due to inflow of contaminated water from the natural stream.

3.3 Performance of the leachate treatment system

3.3.1. pH, salinity, conductivity, DO

The leachate quality and quantity generated from dumpsite are strongly affected by hydrological conditions and the conditions of the dumpsite. Recorded pH, salinity and EC values are higher in stabilized leachate. Leachate quality reduced with aging. pH, salinity and EC reduced with time during the study period, this may be due to interception of heavy rain during this period. With rainfall interception, dumpsite leachate gets diluted. Average pH, salinity, EC and DO concentration variation in sampling locations during the study period are shown in Figure 3. pH, salinity, EC declined when the effluent went through the system as shown in Figure 3 and with time as shown in Figure 4.

Leachate is normally in basic condition. Average pH of inlet of the LTB was 7.74±0.35 during the study period, which indicates that the dumpsite is under methanogenic conditions. Average pH value of the outlet of the LTB was 7.78± 0.04, thus indicating that the LTB was also under anaerobic conditions. pH of the leachate decreased when it did flow through the treatment system. Highest reduction of pH did occur in the algae pond and the sub-surface constructed wetland. According to stipulated wastewater discharge standards by the Central Environmental Authority (CEA), pH value of the effluent should be in the range of 6.0-8.5. The average pH value of outlet of the leachate treatment system was 7.16±0.23, so it was under permissible limit. The high values of leachate conductivity reflect the large content of soluble inorganic. Although, EC of the system declined to low levels, but at the end of the study period, it was high because the system got overloaded. Permissible EC level according to CEA standards is 2,250µS. In the system outlet, the average EC was 1,715±516µS. The DO concentration of the dumpsite leachate is very low indicating that the dumpsite is under anaerobic conditions. DO concentration of the LTB outlet did fluctuate with time but it reduced in latter part of the study period, indicating the LTB is becomes the substrate for the acedogenic and methanogenic microorganisms reaching complete anaerobic conditions with time. The DO concentration of the algae pond was very low that varied between 0.25-0.92 mg/L. Less population of algae was due to higher intensity of rainfall during the study period with overcast sky conditions. Under those conditions, aeration system was used to provide adequate oxygen for microbial activity. However, algae are growing rapidly at present, thus providing oxygen for the microbes

3.3.2 Variation of solids

At the latter part of the study period, solid concentrations increased because of system failure due to heavy rains. Average solid content variations in sampling locations during the study period are shown in Figure 3. According to this study, average TDS and TSS of the inlet of LTB were 5,592±698.2 mg/L and 7,453±2,640.7 mg/L respectively. TDS comprises mainly of inorganic salts and dissolved organics. According to stipulated wastewater discharge standards by the CEA, maximum allowable limit of TDS concentration for discharge effluent is 2,100mg/L. The average TDS value of the discharge effluent of the leachate treatment system was 852±261mg/L. So it did not exceed the allowable limit. As can be seen, TSS reduced at each treatment step of the system and with time also. According to CEA standards maximum permissible TSS level is 50mg/L but the average of the system outlet was 1,057.8 ± 199.4 mg/L during the study period which is much higher.

3.3.3Variation of chemical oxygen demand (COD)

BOD is an index of the oxygen demanding properties of biodegradable material in water. COD is a measure of the oxygen equivalent of the organic matter content of a sample that is susceptible to oxidation by a strong chemical oxidant. Recorded average BOD and COD values in sampling locations during the study period are given Figure 3. The BOD and COD values recorded for the leachate is very high. This may be due to the reason that with time the solid waste material gets degraded and the waste constituents percolate down along with rain water thus polluting groundwater nearby to MSW landfill site. BOD value varies according to age of landfills. Recorded average BOD and COD values of the leachate in this dumpsite were 3,919±576.56 mg/L and 21,535±2,224.72 mg/L respectively. BOD and COD reduced with aging of the dumpsite, but the dumpsite was still operational. BOD is an important parameter in water quality. When microbial activities are taking place, the demand of oxygen is increasing because of break-down of biodegradable materials.

According to the results, BOD of leachate reduced through the treatment system during the study period. Average BOD value of the system outlet was 216.7 ± 177.2 mg/L. The highest BOD reductions were observed from the LTB where anaerobic microorganisms are functioning without any oxygen. According to Central Environmental Authority (CEA) standards, the BOD value of the effluent of the leachate treatment should be 30 mg/L, so the BOD of the effluent should be further reduced before releasing to the natural stream. COD is also important parameter in water quality. When chemical reactions are occurring oxygen is used. As in Figure 3, COD did decrease when flowing through the treatment system. The accepted COD level is 250 mg/L according to CEA standards. Although the average is 868.86± 1,028.4 mg/L, in the last week of the study period, it did reach permissible levels in this system. According to Figure 3, highest reductions of COD were from LTB where anaerobic conditions and mineralizing taking place.

3.3.4 Variation of available phosphorous, nitrate nitrogen and ammonium nitrogen

Recorded average available phosphorous, nitrate nitrogen and ammonium nitrogen values during the study period are given in Figure 4. According to stipulated wastewater discharge standards by the CEA, the maximum allowable limit of available phosphorous and ammonium nitrogen of the discharge effluent are 5mg/L and 50mg/L respectively. Average available phosphorous and ammonium nitrogen concentration of discharge effluent of the leachate treatment system were 2.33±3.29mg/L and 4.38±1.59mg/L respectively. Phosphorus and nitrate content of leachate fluctuated with time and nitrate increased whenever there was high rainfall. Ammonium nitrogen was higher due to anaerobic digestion of organic nitrogen and generation of ammonium which was not converted into nitrite and nitrate because of lesser oxygen concentration in leachate. Nitrate and phosphate were decreasing via the system components.

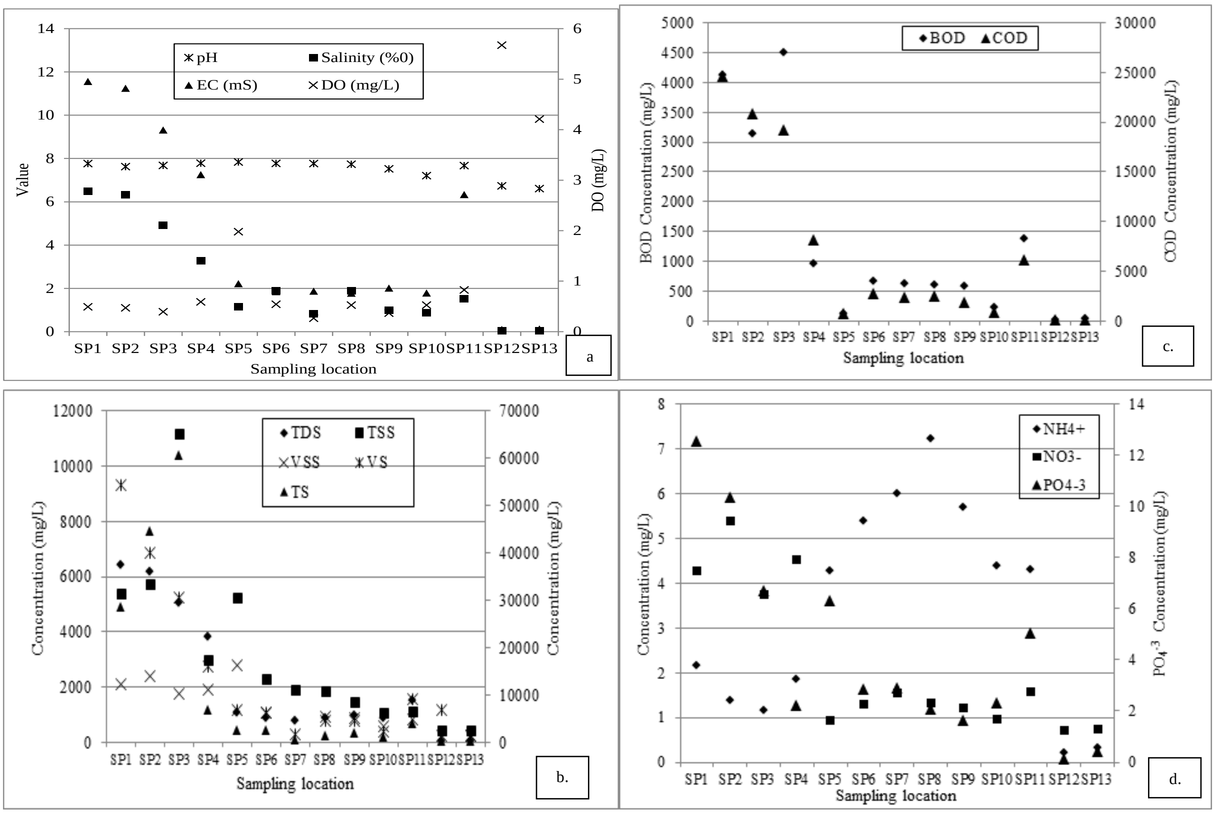
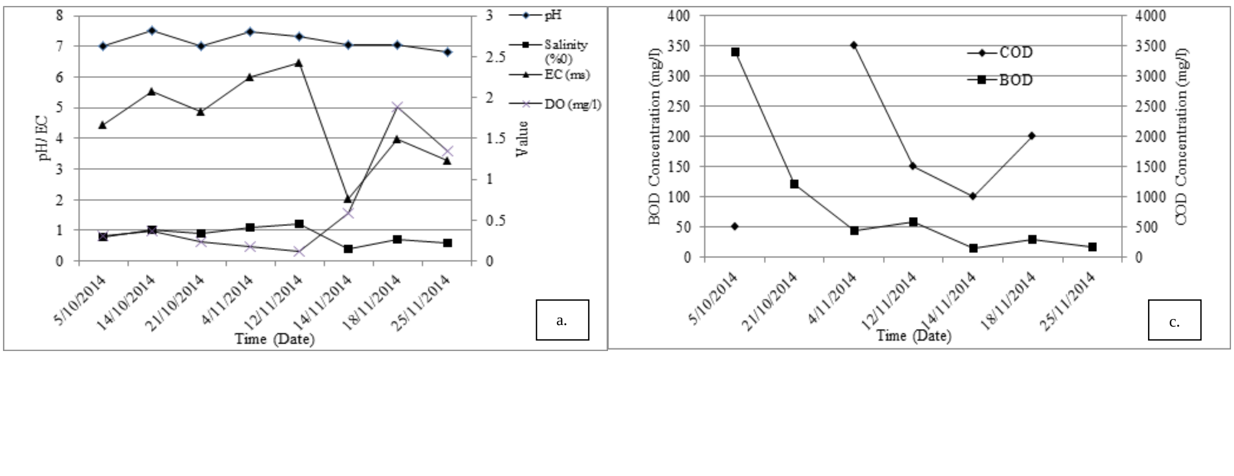


Figure 2: Recorded average values of quality parameters of the sampling points during the study period (a.) pH, salinity, EC and DO; (b.)Solid content; (c.)BOD and COD (d.) NH_4^+, NO_3^-, PO_4^{-3}



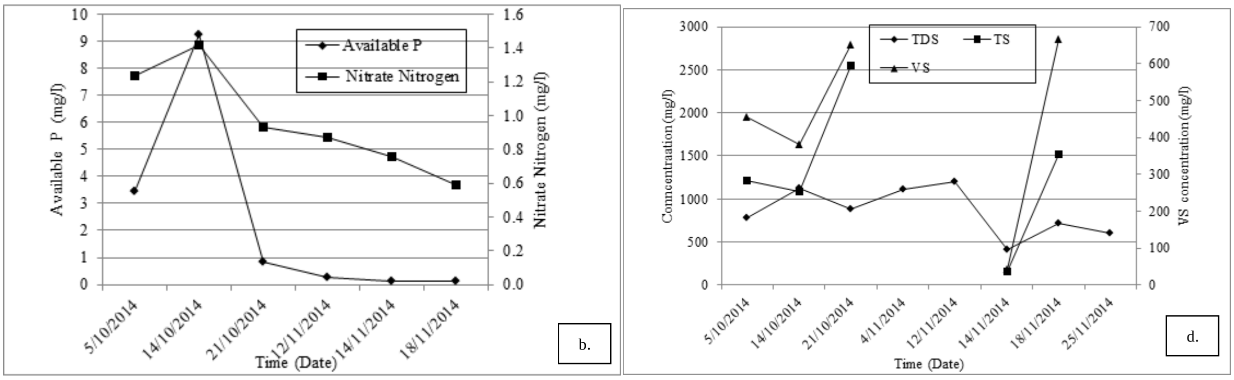


Figure 3: Quality parameters of the leachate treatment outlet variations with time (a.) pH, salinity, EC and DO; (b.) Solid content; (c.)BOD and COD (d.) NO_3^-, PO_4^{-3}

3.4 Removal efficiency (%) of the leachate treatment system

Table 1 shows components and total removal efficiency (RE) of the *LTS*. According to the Table 1, highest salinity RE is from *AP*. High solid RE is from the *LTB* and the *AP*. Highest RE of COD and BOD is from *AP* and *SCWs* respectively. Highest RE of PO_4^{3-} is from *LTB*, NO_3^- is from *AP*, NH_4^+ is from *SCWs*. During the study period, RE of salinity, EC, TS, TDS of the leachate treatment system was 72.18±17.73%, 72.03±22.63%, 93.13±7.39%, 75.82±16.65% respectively which is indicating higher performances of the system even at the initial stabilization period. The removal efficiency will be increased with time through the stabilization of the system and interventions.

Table 1: Removal efficiency of the leachate treatment system

Parameter	Average Removal Efficiency %				
	LTB	AP	FW	CW & CFB	Total
Salinity	53.16±29.61	75.29±0.78	8.33±11.78	24±32.52	72.18±17.73
EC	54.47±37.62	72.24±0.81	10.66±8.44	24.76±37.74	72.03±22.63
TDS	55.87±36.51	74.33±0.87	12.32±10.55	26.61±36.84	75.82±16.65
TS	79.31±19.7	71.19±25.73	33.57±11.28	54.82±38.28	93.13±7.39
VS	53.39±2.74	55.26±18.3	68.86±31.8	34.82±0.66	87.09±9.23
TSS	68.23±17.02	61.58±22.67	40.74±11.88	34.5±55.99	90.15±11.8
VSS	53.2±0.34	47.94±9.13	19.77	26.76±21.78	70.34±21.08
BOD	62.2±36.49	27.01±21.13	70.79±36.02	77.48±21±82	94.77±2.87
COD	58.5±5.13	66.04±2.89	34.37	42.58	86.92±7.63
PO_4^{-3}	55.79±26.96	10.81±7.03	27.43±17.59	39.53±38.96	57.23±11.45
NO_3^-	41.56±12.64	69.86±8.73	17.23±12.9	31.85±32.28	74.16±9.35
NH_4^+	-101.12±30.3	-231.24±41.44	21.26±0.98	45.76±36.15	-218.39±114.7

4. Conclusions and Recommendations

The dumpsite generates high strength leachate and the strength may be reducing with time. pH is an important indicative parameter and it is within the CEA discharge water quality standards. BOD, COD, EC, TDS, phosphate and ammonium have reached CEA discharge water quality standards. Salinity values were low. The system efficiency seems to improve with time and the combined system will be able to comply with the CEA standards, on condition that biological systems are maintained and have adequate growths of algae and plants.

However, VSS far exceeded CEA standard because of high microbial activities in the algae pond and inadequate removal in the constructed wetland. Nevertheless, the system was stabilizing even without much growth in sub-surface constructed wetland and it was attaining the required standards, until heavy rainfall occurred. The system cannot cope with total rainfall going through it, thus drastic reduction of retention time to treat the effluent to the required standards. This situation will continue, unless otherwise interceptor sub-surface drains installed and connected to the leachate treatment system. The dumpsite will require a proper cover system to reduce infiltration and thus promote runoff. It is advisable to construct a biological indicator pond with adequate aeration, so as to convert ammonia to nitrite and nitrate. It is imperative to continue the monitoring and evaluation of the process.

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General Characteristics of Hospital Wastewater from Three Different Hospitals in Sri Lanka

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Abstract: The hospital wastewater in Sri Lanka is a particular concern possibly due to the hazardous and toxic nature and its direct discharge into water bodies. Hence, this study focuses the characterization of wastewater generated from hospitals in Sri Lanka to assess the spatial and temporal variations. Wastewater samples were collected monthly from three different hospitals over a period of 3 months and they were tested for quality parameters: pH, temperature, electrical conductivity, total solids (TS), total dissolved solids (TDS), total suspended solids (TSS), volatile solids (VS), volatile suspended solids (VSS), biological oxygen demand (BOD_5), chemical oxygen demand (COD), nitrate-N, phosphates and heavy metals. The results revealed that hospital wastewater exceeds the allowable limits of Sri Lankan wastewater discharge standards for many of the parameters. The maximum recorded values for TS, TDS, TSS, VS and VSS were 2658, 560, 314, 126 and 235 mg L^{-1}, respectively. The demonstrated values for BOD_5, COD were falling into a large range, 6-1950 and 130-1183 mg L^{-1}. Nitrate-N and phosphate concentration varied and upper limit were reported as 3696 and 103.74 mg L^{-1}. Apparently, maximum concentration of Cr(VI), Mn and Pb were reported as 0.23, 0.52 and 0.90 mg L^{-1}. Further studies are undertaken to analyze volatile organic compounds (VOCs) and pharmaceuticals.

Keywords: BOD_5, COD, Heavy metals, Pollutants, Water quality

1. Introduction

Wastewater is defined as any water, whose quality has been adversely being abused by anthropogenic influence. This includes liquid waste discharged from domestic homes, industries, hospitals, agricultural and commercial sectors. Many of the pollutants detected in wastewaters are categorized as non-regulated “emerging pollutants” [1]. The contact of this kind of wastewater with the surrounding environment results in adverse effects on the biological balance of aquatic ecosystems, causing imbalance at different trophic levels possibly related to the action of toxic and genotoxic agents and indirectly by eutrophication [2].

Over the last few years, hospital effluent has been gained a significant attention in various countries in the world facing different issues. It is well established that hospitals may consume extensive amount of water in a day, ranging between 400 to 1200 L day^{-1} [3] and consequently, generate equally significant volume of wastewater load. Hospital wastewaters (HWW) are generated in different sectors of a hospital including patient wards, surgery units, laboratories, clinical wards, ICU, laundries and possess a quite variable composition depending on the activities involved [4]. In this context, HWW consist a numerous persistent chemical compounds and complex mixtures of organic matter including pharmaceuticals, radionuclides, detergents, antibiotics, antiseptics, surfactants, solvents, medical drugs, heavy metals, radioactive substances and microorganisms [4-5]. After usage, some of these compounds and non-metabolized drugs excreted by patients are detected in HWW and then, enter the municipal sewer network without preliminary treatment. For this reason, this composition leads to extensive levels of toxicity, genotoxicity and organic load and subsequently, causes an adverse impact on the natural ecosystem and inherent hazard to human health [6].

More recently, a study by Jean et al. [7] showed that 15-20% of medicines utilized in hospitals are potentially bio-accumulative. The HWW reveals the presence of potentially toxic heavy metals such as Hg and Ag as well as chlorinated molecules in high concentrations. Additionally, significant concentrations of COD and BOD_5, 1900 and 700 mg L^{-1}, have been assessed in these effluents [3]. Laundry wastewaters from hospitals were characterized by Kern et al. [8] in Brazil and COD and BOD_5 concentrations were as 477 and 305 mg L^{-1}, respectively, when washing stages were not subdivided. However, when washing steps were subdivided into different stages, the first rinsing was demonstrated higher COD and BOD_5 concentrations, 3343 and 1906 mg L^{-1}, respectively.

Pharmaceutical drugs given to people and to domestic animals including antibiotics, hormones, strong painkillers, tranquilizers, and chemotherapy chemicals given to cancer patients are being measured in surface water, groundwater and drinking water as well. Hospitals discharge plenty of undesired potentially pathogenic propagules including antibiotic resistant bacteria, viruses and may be even prions. As a result, in some developing and industrialized countries, the outbreaks of cholera are periodically reported. Moreover, sewers of hospitals where cholera patients are treated are not always connected to efficient sewage treatment plants, and sometimes municipal sewer networks may not even exist.

One of the major environmental concerns due to hospital effluent is their discharge into urban sewerage systems without adequate treatment. HWW could be negatively affected to the ecological balance and public health. If left untreated, pathological, radiaoactive, pharmaceutical, chemical and infectious components of HWW lead to outbreaks of communicable diseases, diarrhea epidemics, cholera, skin diseases, enteric illness, water contamination and radioactive pollution [3]. On the other hand, HWW sludge from on-site treatment plants are to be carefully managed with the precautions as municipal waste sludge. Such sludge must not be utilized as manure without proper pretreatment for food crops [3].

Most often, conventional treatments have been adopted for HWW, however, they are not properly managed and slightly low removal capacities are achieved even for common parameters including BOD_5, COD, TSS and total coliform [1]. On the other hand, only a simple primary treatment such as primary clarification and prechlorination is applied for hospital effluent, anyhow it is not efficient. Moreover, no treatment is adopted at all and direct discharge of HWW into water bodies is a common practice in the developing world. Sri Lanka is one particular example for direct discharging of HWW into surrounding environment. However, no studies have been carried out to examine the quality of HWW in different contexts. No baseline data available to drive the authorities to have a proper wastewater treatment or management system. Hence, the main aim of this study was to investigate conventional physico-chemical parameters including BOD_5, COD, TSS, N and P compounds, pH and selected heavy metals for hospital effluents collected from different locations in Sri Lanka with an aim of assessing their temporal and spatial variations.

2. Materials and methods

The HWW characterization was conducted from March 2015 to June 2015 with one month time intervals. The HWW samples were collected from the three hospitals, Kandy teaching hospital (latitude 7º 17′ 10″ N and longitude 80º 37′ 53″ E), Diyathalawa base hospital (latitude 6º 48′ 20″ N and longitude 80º 57′ 22″ E) and Badulla general hospital (latitude 6º 59′ 30″ N and longitude 81º 03′ 08″ E), Sri Lanka. The respective locations are designated as KTH, DBH and BGH. Samples were collected at five sampling points with three replicates from each location to ensure standard quality control procedures. The respective sampling points are denoted as KTH-01, KTH-02, KTH-03, KTH-04, KTH-05, DBH-01, DBH-02, DBH-03, DBH-04, DBH-05, BGH-01, BGH-02, BGH-03, BGH-04 and BGH-05. The characterization of the collected HWW samples was performed at field conditions and consequently, transferred into the laboratory environment under 4 ºC. Table 1 summarizes different chemical constituents investigated and the analytical methods used.

3. Results and discussion

Wastewater composition describes the actual quantity of physical, chemical and biological constituents present in the wastewater. Depend upon the concentrations of these constituents, wastewater is categorized as strong, medium or weak. According to the results, average pH values were well within the range of 6 to 8.5 [9] in despite of several sampling points. EC of HWW samples varied within the range of 110 and 1120 μS/cm and the average EC was 434.88 μS/cm.

Table 1: Tested quality parameters in HWW

Constitute	Method
pH	Ross sure-flow combination epoxy body electrode
Temperature	Temperature meter (HANNA)
EC	Conductivity meter (Orion 5 star)
TS	Membrane filter paper techniques
TDS	
TSS	
VS	
VSS	
BOD_5	Winkler method
COD	Spectrophotometer (HACH DRB 200)
Nitrate-N	Cadmium reduction method
Phosphate	Ascorbic acid method
Cr(VI)	1,5 dipenyl carbozide method
Mn	Atomic absorption spectrophotometer (GBC 933)
Pb	

Studying the variations of different solid contents is important from a wastewater management perspective because many recommended standards are focused substantially on solids. Maximum TS value recorded was 2658 mg/L at DBH-02. Average TSS of HWW samples was 31.97 mg/L which is within the maximum tolerance limit (MTL) of 50 mg/L given by central environmental authority (CEA) Sri Lanka [9]. However, the maximum TSS value was recorded as 314 mg/L at KTH-04 which is exceeds the MTL given by the CEA. Additionally, the average value recorded for VS and VSS were 125.6 and 235 mg/L, respectively. TDS values were ranging from 50 to 560 mg/L and the average TDS was reported as 144 mg/L.

Figure 1a shows BOD_5 in the three respective hospitals. According to the CEA guidelines, any water to declare as non-polluted the BOD_5 value must be less than 30 mg/L [9]. The average BOD_5 recorded as 416 mg/L in this study. In the case of COD, most of the HWW samples showed higher values than in the MTL established by CEA, which is 250 mg/L [9]. The average COD for HWW samples was 556 mg/L. The COD for

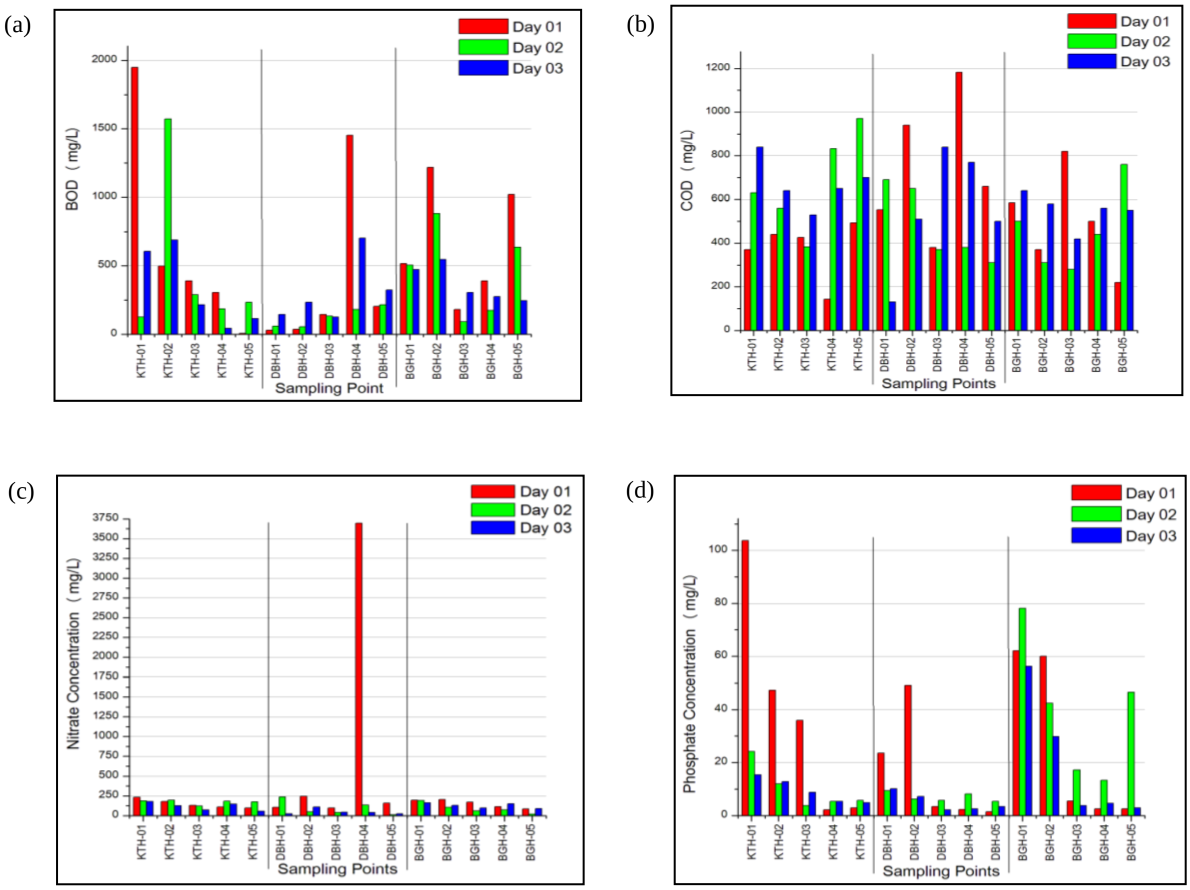

Figure 1: (a) BOD_5 (b) COD (c) nitrate and (d) phosphate concentration in three respective HWW sampling locations

HWW ranged from 130 mg/L to 1183 mg/L (Figure 1b). The higher BOD_5 and COD values can be attributed to the septic matter collected into the drainages and mare of the kitchen waste added into the system.
Among the nutrients, nitrate concentrations were observed in high concentrations ranging from 12 to 3696 mg/L (Figure 1c). Phosphate in the HWW varied in a range of 1.5 to100 mg/L. The average value for phosphate in the samples was 19 mg/L which exceeds the MTL, 5.0 mg/L in wastewater [9]. It is a 381% of increment than the MTL value for wastewater. The maximum phosphate concentration dissolved in the wastewater was 104 mg/L at KTH-01 (Figure 1d). Kitchen and septic waste as well as sodium tri-phosphate (STPP) which is a cleaning liquid used frequently may be the possible sources of phosphate in HWW.

It is known that heavy metals can accumulate via food chain and reach living organisms causing serious effects. Inspection of Fig. 2 (a) and (b) reveals the variation of some selected heavy metal species, Mn and Pb, during study period. The MTL for the Mn is 0.5 mg/L [9]. However, the maximum Mn concentration

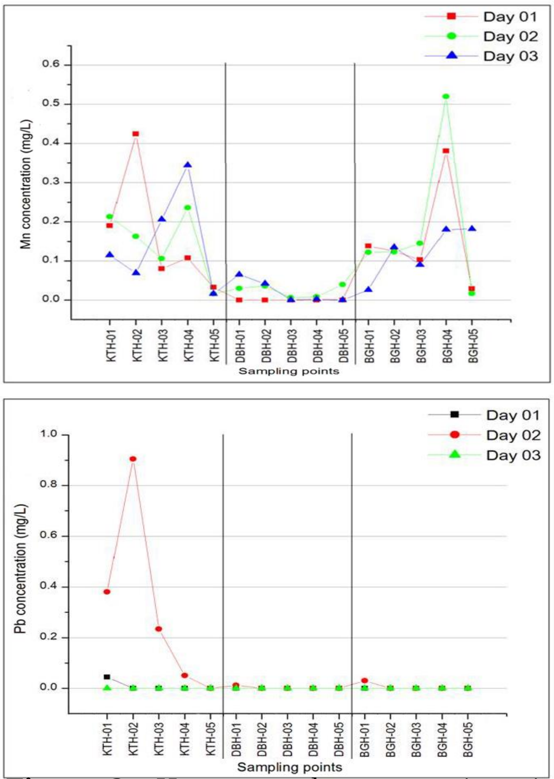

Figure 2: Heavy metal concentrations in different sampling locations (a) Mn (b) Pb

recorded in KTH, DBH and BGH were 0.424, 0.065 and 0.520 mg/L, respectively and only the location BGH exceeds the permissible level. The MTL for the Pb is 0.1 mg/L. The maximum Pb concentration recorded in KTH, DBH and BGH were 0.91, 0.01 and 0.03 mg/L, respectively and among them, KTH exceeds the MTL value nine times higher. Additionally, Cr (VI) concentration varied around 0.01 to 0.225 mg/L. The average value for Cr (VI) concentration was 0.106 mg/L, which is above the maximum tolerance limit, 0.1 mg/L [9] of the wastewater.

4. Conclusions

In summary, this preliminary study describes the physico-chemical composition of HWW discharged from KTH, DBH and BGH, respectively. In fact, this study provides a general overview of the contaminants relative to the spatial and temporal variation of HWW. Accordingly, some solids such as TSS showed a maximum of 314 mg/L was higher exceeding the country's MTL. The demonstrated values for BOD_5, COD, nitrate and phosphate were falling into a large range, exhibiting spatial and temporal complexity and exceeding MTL in most occasions. Additionally, heavy metals including Mn, Pb and Cr(VI) are present in considerable concentrations leading to deleterious effects on living organisms. Thereby, it is seems obvious that suitable management practices should be initiated for HWW treatment making it safe to discharge into the surrounding environment. On the other hand, it is timely needed to investigate VOCs, chlorinated byproducts and pharmaceuticals in HWW since they are capable of adversely affect on human health.

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SECM/15/094

Heavy Metals in Atmospheric Deposition in Kandy City; Implications for Urban Water Resources

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Abstract: Atmospheric deposition is a serious issue in the context of biogeochemical cycling of heavy metals (HMs) and the resulting impacts on urban water resources. In this study, dry deposition (DD) and bulk deposition (BD) samples were collected from four sites located in heavy traffic areas in Kandy, Sri Lanka. Wet deposition (WD) was determined from the difference between DD and BD. Samples were analyzed for Al, Cr, Mn, Fe, Ni, Cu, Zn, Cd and Pb using inductively coupled plasma-mass spectrometry. Relatively high concentrations of Fe, Al and Zn were found in both DD and WD samples collected. In BD, percentage dissolved fraction of Ni, Cr, Cu, Cd, Al and Pb was 94, 86, 81, 78, 61, 46% respectively. Presence of high dissolved fraction of highly toxic HMs such as Cr and Pb can result in significant ecosystem health impacts due to their ready bioavailability. The presence of metals such as Al, Fe, Zn, Pb, Cr and Cd in WD was consistently more significant compared to DD. The presence of high concentrations of mobile forms of metals in WD will contribute to the pollution of urban stormwater, resulting in the degradation of urban receiving water environments and associated ecosystems.

Keywords: Atmospheric deposition, heavy metals, stormwater quality, urban water pollution

1. Introduction

Atmospheric deposition is a global issue since it leads to the degradation of terrestrial ecosystems. Among different pollutants in atmospheric deposition, heavy metals play an important role due to its availability to biogeochemical cycling. Heavy metals that are emitted from different sources into the atmosphere; natural and anthropogenic sources such as soil, sea water and volcanic eruptions, emissions from traffic, fossil fuel combustion, industrial activities and solid waste incineration, tend to deposit via dry and wet deposition processes [1]. Heavy metals can be toxic to humans at even trace levels, especially Cd, Cr, Hg and Pb like metals [1, 2]. Based on the WHO air quality guidelines the threshold levels Cd, Pb and Mn are 5 ng m^{-3} $year^{-1}$, 0.5 and 0.15 μg m^{-3} $year^{-1}$ respectively. As heavy metals are non-biodegradable and transport through biogeochemical cycling, they can be accumulated in fauna and flora, water bodies, soils and even in humans bodies [2, 3].
Either via wet or dry deposition, heavy metals will be transported to surface and groundwater [2]. Since wet deposition has a higher percentage of the dissolved fraction, the urban water resources can be directly affected [2]. Furthermore, heavy metals deposited as dry deposition on soil or impermeable surfaces will be transported by stormwater runoff to receiving waters [4]. Studies have shown that atmospheric deposition consists of a diverse range of heavy metals in high concentrations [5, 6]. Also, past research studies have provided detailed insights to urban water pollution by deposited heavy metals via stormwater runoff [7]. Therefore it is important to study about atmospheric deposition of heavy metals and its effects on urban water bodies.
Kandy is the second largest city in Sri Lanka, located at a valley in the hill country which shows typical characteristics of a mountainous city in the developing world. Although there have been very few studies on Kandy's atmospheric pollution, it is proven to be highly polluted by gases and hydrocarbons [8]. However, no studies have focused attention on the atmospheric deposition of heavy metals which can directly pollute the surface water bodies such as the Mahaweli River (the potable water source for the population of Kandy), via stormwater runoff. Hence, the objective of the study was to quantify the heavy metals loads, namely, Al, Cr, Mn, Fe, Ni, Cu, Zn, Cd and Pb in atmospheric deposition samples in Kandy in bulk,

dry and wet deposition which will contribute to evaluating the effects on urban water bodies.

2. Material and methods

2.1 Sampling sites

For the preliminary study, four locations, Fire brigade site, Police station site, Railway station site and at National Institute of Fundamental Studies (NIFS), were selected within Kandy City area to collect deposition samples. Sampling sites were selected considering traffic volume and distance to major roads. The sampling station at NIFS was the control site since traffic volume is relatively low. Samples were collected during February 2015 based on rainfall events.

2.2 Sample collectors

Sample collectors were designed as a funnel bottle connected system [9]. Two polyethylene bottles connected to polyethylene funnels were used to collect deposition samples. Each bottle was placed inside a PVC column and two columns connected to a pole. The system was fixed at a height of 1.5m above ground. One bottle was used to collect dry deposition while other for bulk deposition.

2.3 Sample collection

Same collection bottles were placed just after a rainfall event. Dry sample collection was done just before the next immediate rainfall event and the bulk deposition sample was collected just after the same rainfall event. After collection, the samples were transported to the laboratory as soon as possible (within maximum ½ hour) following standard quality control procedures.

2.4 Sample preservation and digestion

After sample bottles transferred to the laboratory, meniscus was marked in bulk deposition samples to determine the total volume of the sample. Funnels were rinsed with de-ionized water. For bulk deposition samples, pH was measured just after the samples received by the laboratory. Approximately 50 ml of sample was filtered using 45 μm membrane filter for analysis of dissolved elements and 50 ml of original sample separated for analysis of total recoverable elements. Remaining sample volume was measured using a measuring cylinder and added to a pre-weighed crucible to determine the weight of total solids. Finally, the bottles were filled with water to the meniscus and volume was measured as sample volume.

In dry deposition samples, funnels were rinsed with 100 ml of de-ionized water.

Samples separated for total recoverable elements and dissolved elements, were preserved until digestion. For preservation, samples were acidified to pH<2 with 1:1 HNO_3 solution. After 16 hours, samples were verified to be pH<2. Samples were kept in refrigerator until digestion.

Preserved samples were acid digested prior to analysis. 50 ml of sample transferred into a Griffin beaker and added 2 ml (1:1) HNO_3 acid and 1 ml (1:1) HCl acid was added. The temperature was maintained at no higher than 85 ^{o}C. The sample was evaporated until it approached 15 mL using gentle heating with avoided boiling. Then the solution contained in the beaker was allowed to cool and gravity settling of any solids. Samples were stored in 15 ml centrifuge tubes for further analysis.

2.5 Analysis

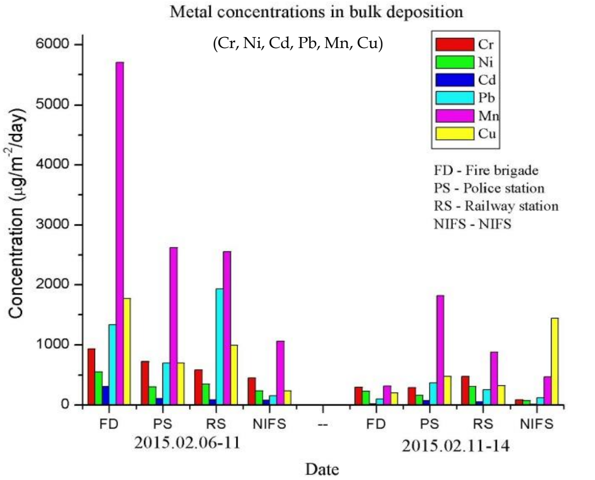
Digested samples were analysed for Al, Cr, Mn, Fe, Ni, Cu, Zn, Cd and Pb using Inductively Coupled Plasma – Mass Spectrometry (ICP-MS) at Queensland University of Technology (QUT), Australia, laboratory which is a certified laboratory. For total solids, sample contained in the crucibles were placed in an oven at 105 ^{o}C until evaporation of all liquids. Total solids was measured by the weight difference between the oven dried and empty crucible.

3. Results and discussion

3.1 Bulk deposition

Heavy metal concentrations in bulk deposition are shown in Figure 1a and 1b. It shows the concentrations of heavy metals based on their loads. Al and Fe have been reported with relatively higher concentrations than the others. Both Al and Fe demonstrated significant difference from all other tested heavy metals. However, no significant difference was observed between them. At two consecutive rainfall events, the metal concentration change pattern in bulk deposition is Fe = Al >> Zn = Cu = Mn = Cr = Ni = Pb = Cd. There is no other significant difference between the two different time periods for heavy metals deposition in Kandy City. Accordingly, the deposition pattern of heavy metals was the same in Kandy City throughout the month of February 2015. For Al, Fe, Cu, Zn and Mn, no significant difference was observed for the four sites. However, Ni, Cr, Cd and Pb exhibited a significant difference among sample collection sites. The averages of metal concentrations among the sites varies as Railway station > Police station ~ Fire brigade > NIFS.

The percentages of dissolved element fraction of heavy metals are illustrated in Figure 2a and 2b, respectively. Although, Cu and Zn have low total metal concentrations in bulk deposition compared to Al, Mn and Fe, they were high in the dissolved fraction. This is attributed to the higher solubility of Cu and Zn compared to the other elements. In the literature, the solubility change pattern is given as Zn > Cd > Cu > Ni > Pb > Cr [10]. However, in our study it was Ni > Cr = Al = Zn = Cd = Cu = Mn > Fe = Pb. This is attributed to the chemical characteristics of the heavy metal species present in the urban atmosphere in Kandy City. The presence of high soluble fraction indicates the studied metals are ready to impose eco-toxic impacts in the water-soil environment and on biota [11].



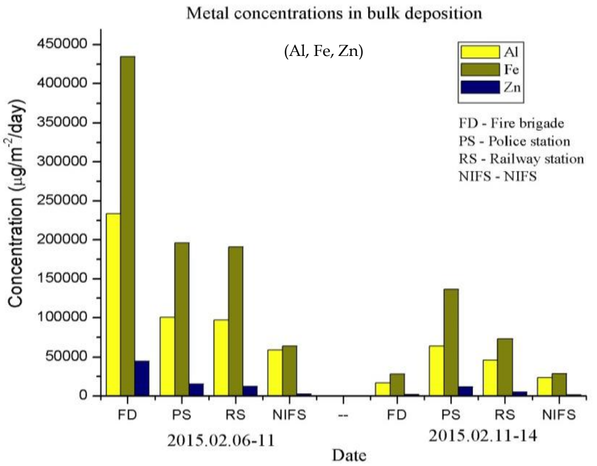
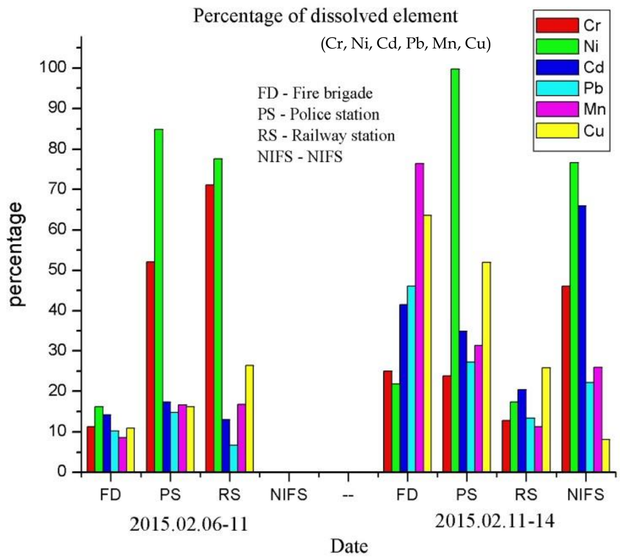


Figure 1a (top): Metal concentrations for Cr, Ni, Cd, Pb, Mn and Cu in bulk deposition in Kandy City during February 2015. 1b (bottom): Metal concentrations for Al, Fe and Zn in bulk deposition in Kandy City during February 2015

3.2 Dry deposition

Heavy metals in the dry deposition during February 2015 in Kandy City depicted in Figure 3a and 3b. Similar to the bulk deposition data, Al and Fe showed significantly high concentrations in dry deposition and there was no significant difference among rest of heavy metals.

Taking into consideration three different rainfall events, overall low heavy metal concentrations in dry deposition loads have been reported in Figure 3a and 3b for February 6th to 9th which was Friday to Monday, February 11th to 12th which was Wednesday to Thursday and February 14th to 16th which was Saturday to Monday.



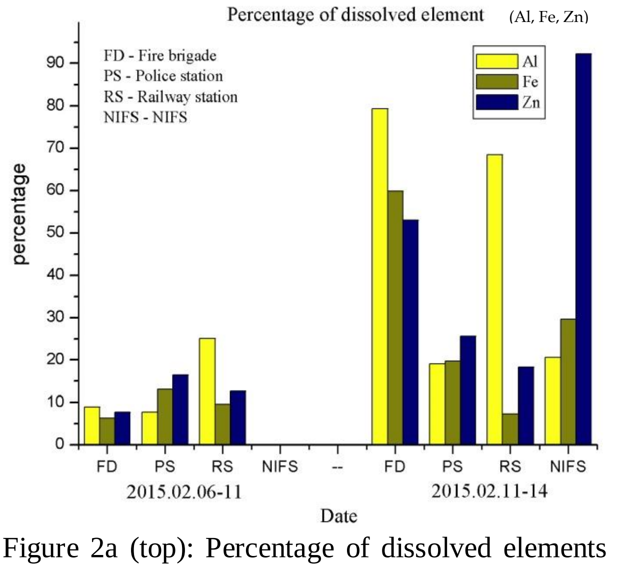


Figure 2a (top): Percentage of dissolved elements from total deposition during February, 2015 – for Cr, Ni, Cd, Pb, Mn and Cu. 2b (bottom): Percentage of dissolved elements from total deposition during February, 2015 – for Al, Fe and Zn.

Fridays and Mondays are the days where the highest traffic volumes are experienced due to the large number of vehicles entering and leaving Kandy City. The highest average metal concentrations in dry deposition have been reported for the period 14th to 16th February. As Kandy City has historical importance, even during the weekend, a high number of vehicles can be expected. The emissions from the large number of vehicles may be the reason for high heavy metal concentrations in dry deposition during February 6th to 9th and 14th to 16th. In the case of February 11th to 12th, which were Wednesday and Thursday, no significant difference was observed for Al, Fe, Zn, Mn, Cu, Cd and Ni among four sites whereas a significant difference among sites was observed for Cr and Pb. Similar data have been observed in Sha-Lu, Taiwan, during the year 2003 [12]. The average dry deposition fluxes reported for day time and night time as 54.1 and 26.2 g m^{-2} respectively. The higher day time dry deposition fluxes have been reported due to high traffic incidences. But in night time due to low traffic incidences the dry deposition fluxes are low [12].

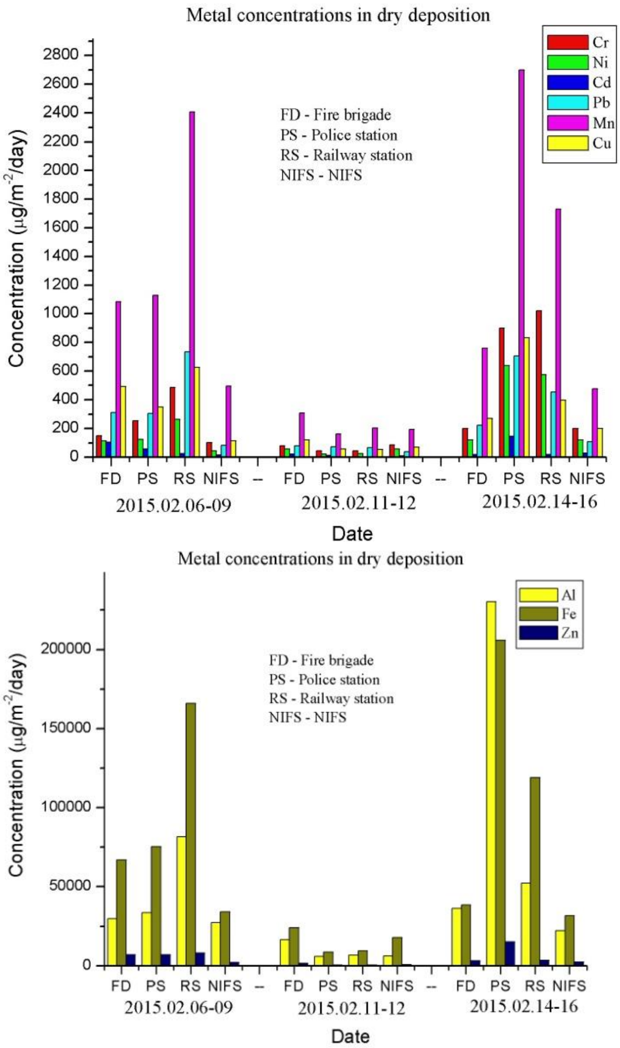


Figure 3a (top): Metal concentrations for Cr, Ni, Cd, Pb, Mn and Cu in dry deposition in Kandy City during February 2015. 3b (bottom): Metal concentrations for Al, Fe and Zn in dry deposition in Kandy City during February 2015

The average heavy metals in dry deposition was distributed as Railway station > Police station ~ Fire brigade > NIFS.

3.3 Wet deposition

Wet deposition concentrations are shown in Figure 4a and 4b. Wet deposition has been calculated on the basis of the difference between bulk and dry deposition for a particular rainfall event. Similar to the dry and bulk deposition, wet deposition showed comparatively higher concentrations for Al and Fe than the other metals. In Izmir, Turkey during year 2004 the HM changing pattern in wet deposition is same as present study [11]. For both of studies it was as Zn > Pb > Ni > Cd [11]. The highest average wet depos (Cr, Ni, Cd, Pb, Mn, Cu) ported at the Railway station and lowest at the NIFS. The wet deposition is more critical than dry deposition in terms of urban water pollution. Also the wet deposition is higher than dry deposition for studied HMs. Similar data was shown in Izmir, Turkey [11].

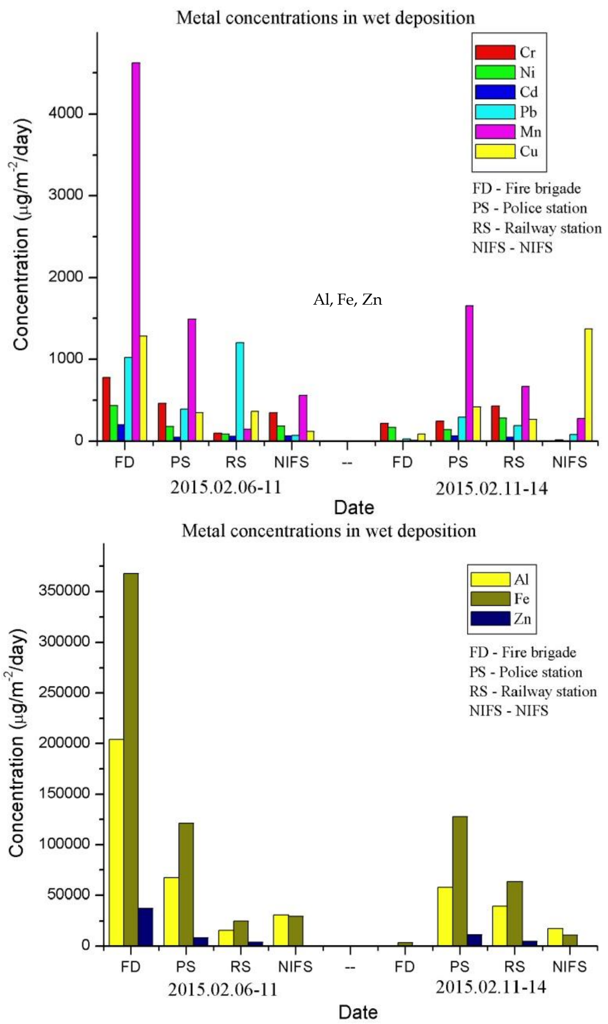


Figure 3a (top): Metal concentrations for Cr, Ni, Cd, Pb, Mn and Cu in wet deposition in Kandy City during February 2015. 4b (bottom): Metal concentrations for Al, Fe and Zn in wet deposition in Kandy City during February 2015.

4. Conclusions

The bulk and dry deposition were collected in Kandy City during February 2015 encompassing two consecutive rainfall events. Al and Fe were reported in significantly higher concentrations in deposition loads than other heavy metals that were tested. For Fe, the reported highest values are 434.86 g/m^2/day for bulk deposition during February 6th to 11th at Fire brigade and 205.53 g/m^2/day for dry deposition during February 14th to 16th at Police station. The lowest Fe values are 28.122 g/m^2/day for bulk deposition during 11th to 14th at Fire brigade whereas it was 8.845 g/m^2/day for dry deposition during 11th to 14th February at Police station. Other heavy metals, Mn, Cu, Zn, Ni, Cr, Cd and Pb did not show a significant difference among each other for bulk, dry and wet deposition. The Railway station showed a significant difference among other sampling sites exhibiting highest average heavy metal concentrations.

The dissolved fraction represents the bioavailable form that is contained in atmospheric deposition. Even though Cu and Zn showed lower total metal concentrations in bulk deposition, percentage of dissolved fractions is higher at 64% and 53%, respectively. This indicates that high percentages of deposition are already available in bioavailable form.

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Detection of benzene in landfill leachate from Gohagoda dumpsite and its removal using municipal solid waste derived biochar

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Abstract: Numerous landfill associated volatile organic compounds (VOCs) are emerging concern due to their potential risk of health. Benzene is one of the most common VOCs in landfill leachate. Biochar has bulged as a universal sorbent for the removal of contaminants from water and soil. Hence, this study examines the potential of biochar derived from municipal solid waste (MSW-BC) on adsorption of benzene so that MSW can be recycled to treat its own pollutants. Landfill leachate was collected from five sampling points at Gohagoda MSW dumpsite and analyzed for benzene. In order to evaluate the potential of MSW-BC for removal of benzene from water, the equilibrium adsorption was procured by using headspace GCMS. The effects of pH, contact time and sorbent (1-10 g/L), sorbate (30-300 µg/L) concentration were investigated using a batch sorption technique. Benzene was detected in landfill leachate, approximately 20 µg/L. The batch experiments indicated that benzene adsorption was dependent solution pH and percentage removal became higher above pH 7 and highest adsorption 42.7 µg/g was observed within 24 hours at pH 9. Maximum saturated sorption capacity of MSW-BC for benzene was 87.0 µg/g. Preliminary experiment data suggest a potential of MSW-BC to be utilized as a material for VOC remediation from MSW dumpsites.

Keywords: Adsorption, Benzene, Landfill leachate, MSW biochar

1. Introduction

Waste management is a serious problem faced by many countries around the world (Zurbrugg, 2002). Although open dumping is an environmental threat, many Asian countries still allow open dumps as final the disposal method due to financial constrains and operational easiness. However due to the uncontrolled solid waste management, numerous environmental and health problems simulate within the site and the adjacent area of dump. Prioritization of issues lay on landfill toxic gas emission associate air pollution, with global warming potential, and soil water contamination by landfill leachate.

Landfill leachate is formed from the degradation of waste mass by the infiltrated water. It produces organic and inorganic pollutants in a soluble form. There are four major groups of pollutants that can be observed within the leachate; dissolve organics matter, inorganic macro component, heavy metals and xenobiotics organic compounds (Christensen et al., 2001). The xenobiotics organic compounds mean organic compound that are form and emit due to the mixture of chemical products incorporate interaction in landfills. The origin of the xenobiotics belong to anthropogenic sources and include volatile organic compounds (Baun et al., 2004).

The most frequently detected VOCs in landfills are Benzene, Toluene, Ethyl benzene and Xylene (Först et al., 1989, Harkov et al., 1985, Sabel and Clark, 1984). Moreover, VOCs are found in leachate r at low concentrations (Dincer et al., 2006). Nevertheless, VOCs can pose health risks even at very low amounts; such asppb or even less (Leidinger et al., 2014). However, only a few studies in Asian countries have reported regarding

VOCs in landfill leachates although they are highly toxic pollutants.

Benzene is one of the most common VOCs detected in landfill leachates (Först et al., 1989) and it is the primary raw material for polymer production and use in several industries such printing & lithography, paint, rubber, dry cleaning, adhesives & coatings, detergents, extraction and rectification (USEPA, 2003). This may be the source of benzene in landfills. The generated benzene may volatilize and the remaining may leach to groundwater or surface water bodies. Both acute and chronic effects dominant in benzene (Cotruvo and Regelski, 1989). Therefore remediation of benzene from landfill leachate is often crucial for health and the environment. Although many different studies have shown different technologies and media for benzene removal, few studies have focused their attention on biochar for its remediation by using sorption technique. Sustainable MSW-BC utilization for remediation of heavy metal in landfill leachate were reported by Jin et al. (2014) and removal of the organic contaminant pesticide by using green waste biochar was successfully (Zheng et al., 2010). Apart from that, Bornemann et al. (2007) postulates the existence non-linear adsorption behaviour of benzene and toluene in to biochar and it showed the Langmuir isotherms fitting with indicative pore filling process. Biochars (BCs) can be produced by many different feedstocks and use of MSW would be a double benefit in terms of waste reduction and waste reuse. Hence, the objective of this research was to assess the benzene levels in the leachate from Gohagoda MSW dumpsite in Sri Lanka and produce MSW-BC to test its effectiveness on benzene removal.

2. Materials and methods

2.1. Chemicals

Commercially available EPA 524.2 analytes, analytical standards were used in 2000 µg/mL ampules (Sigma Aldrich) and Oxygen-free water was obtained by N_2 gas purging at 30 mins through milli-Q water (resistivity 18 MΩ.cm).

2.2. Leachate

Leachate collection was performed in Gohagoda dumpsite that is located in Kandy centralized around latitude and longitude of 7º 18′ 47.85″ N and 80º 37′ 19.02″ E, respectively (Wijesekara et al., 2014). The open dumpsite spatial distribution is extent up to 2.5 ha. Recent investigation of solid waste generation in Kandy municipal council (KMC) was 152 tons, while few of places are located for waste segregation and application of 3R within the KMC (WACS, 2014). However, the majority of KMC waste from households, fish market, slaughter house and non-infectious hospital pharmaceutical waste are daily dumped without any pre-treatment (Wijesekara et al., 2014).

The leachate samples were collected from points of GS-9, GS-8, GS-7, GS-5 and GS-1 (Figure 1). Triplicate samples were performed in each location and 1 g of NaCl electrolyte stabilization as a peak resolving agent was added into headspace vials. The leachate sample, 10 mL, was filled to reach the gauge line of headspace vial. After the leachate collection, immediately, the vials were crimped tightly to minimize loss of VOCs. Then container transferred to laboratory environment under 4 ºC and samples were analysed for benzene under the headspace GCMS.

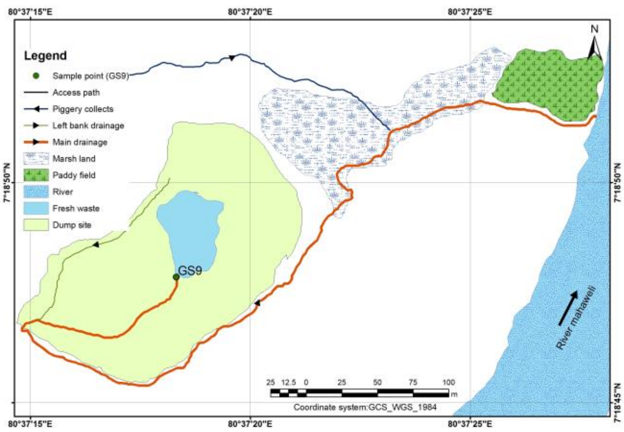

Figure 1: Sampling points in a Gohagoda dumpsite

2.3. Biochar

Segregated organic fraction of the MSW has been used for producing MSW-BC and pyrolysis was performed in batch reactor built in Gohagoda dumpsite under slow pyrolysis. Proximate analysis was conducted based on the experimental procedure described by Ahmad et al. (2013). The moisture content of MSW-BC was determined by drying samples at 105 ºC overnight. For determination of mobile matter content, BCs were heated in covered crucibles at 450 ºC for 1 hr in muffle furnace. Meanwhile ash content was measured by heating samples in open top crucibles at 750 ºC for 1 hr. Finally rest of the resident matter, analogous to fixed matter was calculated from ash, mobile matter and moisture contents. In order to measure pH and electrical conductivity (EC), MSW-BC was dilluted ··· . Elemental analysis was performed after oretreatment using microwave (···, ···) .

The elemental analyzer (Vario MAX CN, elementar, Germany) was used for determination of elemental composition (C,H,N,S and O). Molar H/C and O/C ratios were calculated from the elemental analysis for supportive indications of aromaticity and polarity. Concentrations of heavy metals on biochar were determined by inductively coupled plasma optical emission spectrometry (ICP-OES) (Perkin Elmer Optima 4300 DV ICP-OES, US).

2.4. Batch experiments

Batch experiments were performed for MSW-BC and identification of suitable pH to conduct sorption experiment. The 1 g/L of biochar with initial and 4 different pHs (3, 5, 7 and 9) were tested to determine suitable pH. Prior to the addition of benzene the biochar solution was hydrated for 4 hours at shaker in 100 rpm after initial pH adjustment. The sample of 10 mL was filled to reach the gauge line of headspace vial and consequently 25 µL of benzene primary dilution standard amount were spiked into the headspace vial. The samples were shaking overnight at 100 rpm. The kinetic experiment was performed in controlled laboratory conditions and at different time intervals, 0.5, 1, 2, 4, 12, 18 and 24 hr. The isotherm, sorbate (30-90 µg/L) concentration at 24 hrs length was investigated using a batch experiment. For all sorption tests, oxygen free water were used to preparation of samples, immediately headspace vials was crimped tightly to minimize loss of benzene, remaining concentrations data were collected by using GCMS. Non linear curve fitting were performed for identification of possible mechanisms of MSW-BC with Benzene.

2.5. Analytical method

Quantitative analyses of the Benzene standards and samples were performed using gas chromatograph (GC, Shimadzu QP 2010 plus, Japan) equipped with mass detector (QP 2010 ultra-MS), and a Shimadzu HS-20 head space auto-sampler. Chromatographic separations were accomplished with a 20 m Rtx-624 column in 0.18 mm i.d. and 0.001 mm film thickness (Restek Scientific Inc.) using injections in the split mode (1/30). The oven temperature was held at 35 ºC for 2 mins and then increased up to 230 ºC with a gradient of 20 ºC min^{-1} holding for 3 mins at final level. The temperatures of the injector and detector were 200 ºC and 200 ºC, respectively. Ultra high purity helium was used as the carrier gas at a flow of 24.7 mL/min with a constant rate.

3. Result and discussion

3.1. Characteristics of leachate

All the leachate samples of exceed MCL (0.005 mg/L) of benzene in water. The obtained values are summarized in Table 1.

Table 1: Benzene concentration in Gohagoda leachate

Sample point	Benzene concentration µg/L
GS-1	18.3 ± 1.9
GS-5	11.4 ± 5.2
GS-7	21.5 ± 1.3
GS-8	18.4 ± 2.5
GS-9	21.7 ± 3.1

3.2. Characteristics of the biochar

The analytical results for the MSW-BC are presented in Table 2. However, ultimate analyses indicated that the low molar O/C ratio is due to high pyrolytic temperature. Pyrolysis temperature on benzene adsorption can be distinguish according to the (Bornemann et al. (2007) and it shows the high temperature derived biochar favourable for benzene adsorption. Atomic ratios of the H/C and [(O+N)/C] are recognized as indicator for aromaticity and polarity of BC, respectively (Sizirici and Tansel, 2010, Florez Menendez et al., 2004). In addition, lower values of both H/C and the polarity index [(O+N)/C] ratios of the MSW-BC indicated that the high temperature-derived BC are highly carbonized, visualizing a highly aromatic structure. The reduction of O/C and H/C ratios further explain by Edil (2003) and processes of dehydratation, decarboxylation, and decarbonylation exhibit due to pyrolysis process at higher temperatures.

Table 2: Analytical data for the MSW- BC

Proximate analysis		
pH	9.7	± 0.05
EC (mS/m)	31	± 2
Moisture (%)	6.3	± 0.1
Mobile matter (%)	31.6	± 2.2
Ash (%)	15.6	± 3.3
Resident matter (%)	46.5	± 4.0
Ultimate analysis		
C (%)	60.8	± 0.12
H (%)	2.79	± 0.05
O (%)	14.6	± 0.02
N (%)	1.33	± 0.01
S (%)	0.16	± 0.03
Molar H/C	0.04	± 0.01
Molar O/C	0.24	± 0.02
Molar [(O+N)/C]	0.26	± 0.002

Table 3 shows concentration of heavy metal on the MSW-BC and municipal sewage sludge biochar (Liu et al., 2014). Compared to the sewage sludge derived biochar, the amount of heavy metals are very low in MSW-BC. Therefore potential utilization of MSW-BC as sorbent can be implemented without any constrain on environment. The Table 3 shows concentration of heavy metal on the municipal sewage sludge biochar and MSW-BC.

Table 3: Elemental analysis data for the MSW- BC

Element	**Average concentration** (mg/kg)	**(Liu et al., 2014)**	**Element**	**Average concentration** (mg/kg)	**(Liu et al., 2014)**
As	Nd	Nm	Mo	Nd	Nm
Be	Nd	Nm	Ni	1.81	Nm
Ca	5920	Nm	Pb	2.48	67.5
Cd	Nd	4.12	Sb	Nd	Nm
Co	Nd	Nm	Se	Nd	Nm
Cr	9.27	92.2	Sr	14.3	Nm
Cu	10.9	125	Ti	15.8	Nm
Fe	1810	Nm	Tl	Nd	Nm
Li	Nd	Nm	V	Nd	Nm
Mg	2320	Nm	Zn	82.8	749
Mn	305	Nm			

Nd- not detected , Nm- not measured

3.3. pH dependent sorption of benzene on MSW-BC

The effect of solution pH on benzene adsorption onto biochar is shown in Figure 2. The highest removal was at pH 9 where as maximum adsorption (39.6 µg/g) of benzene was observed for MSW-BC. High pH has shown also higher removal than the lower pH values (Wibowo et al., 2007). In addition to that, lower interaction effect of the field level biochar application into leachate can be expected from aqueous phase due to the similar pH potential in both leachate and biochar solution (Wijesekara et al., 2014).

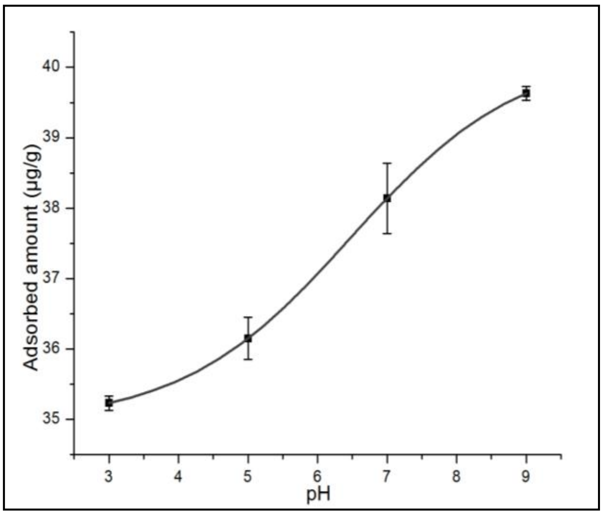

Figure 2: Effect of solution pH on benzene adsorption onto MSW-BC

3.4. Adsorption isotherm for MSW-BC

An adsorption isotherm relates the sorbent/sorbate interactions. The maximum adsorption capacity in the experiment was recorded as 87.0 µg/g of benzene. The model of best fit was Dubinin Raduskevich (DR) which is applied to isotherm data (Figure 3) and respective free energy of adsorption (E) per molecule of the adsorbate is 266.4 $KJmol^{-1}$. Physisorption process can be explained by E value and E <8 $KJmol^{-1}$ denotes the possible physisorption process. Therefore, the mechanism of benzene adsorption is not a physisorption process and it could be chemisorption or another process (Horsfall et al., 2004). The data goodness of fit were R^2 of 0.86 and 0.93 for Freundlich and Temkin models, respectively. This suggests the fit can be attributed tp heterogeneous adsorption with chemisorption process (Mohan et al., 2011).

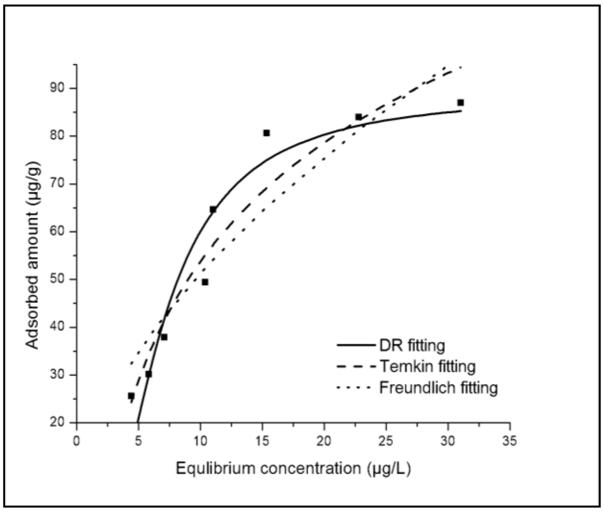

Figure 3: Freundlich, Temkin and DR adsorption isotherms of benzene by MSW-BC. The symbols represent experimental results and lines show the model predicted data fittings

3.5. Kinetics of benzene adsorption

The experimental runs were done for the effect of contact time on the batch adsorption of benzene. Result under condition at initial pH of 9 and an initial concentration of 50 µg/L is shown in Figure 4. Data modeling provided the best correlation (R^2 = 0.99) of the experimental data to pseudo-second order chemical reaction and it can be postulated that the rate-controlling step is chemisorption. The best fitting order of kinetic model is determined to be pseudo second order >> Elovich > power function.

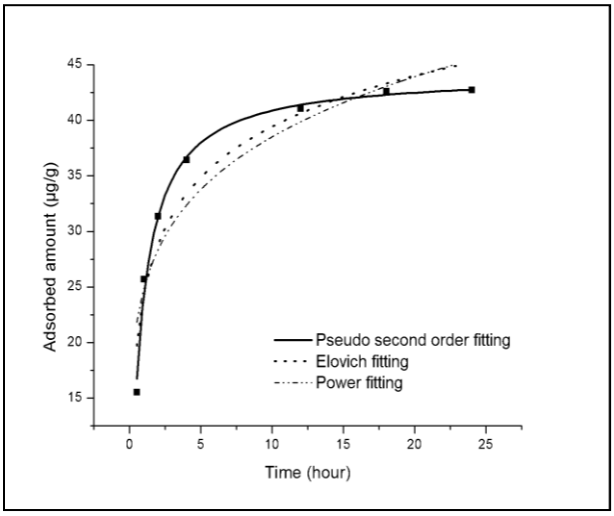

Figure 4: Pseudo-second order, Power and Elovich models on kinetic data. The symbols represent experimental results and lines show the model predicted data fittings.

4. Conclusions

In summary, all the leachate samples obtained from Gohagoda dumpsite exceeded the MCL. The adsorption isotherm, rejection of physisorption process by higher value of free energy of adsorption (E) per molecule, mechanism skewed towards chemical interaction. Therefore chemisorption can be suggested as the mechanism behind benzene removal process. The low availability of heavy metals in MSW-BC and the pH of maximum adsorption bearable with leachate environment explain the possibility of potential use of MSW-BC for Benzene removal from leachate. Hence, this study showed the possibility of VOCs removal from landfill leachate using its own biochars. More studies will be undertaken to test other VOCs.

Acknowledgements

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Landfill Leachate Treatment by Using Two Stage Anaerobic Aerobic Systems

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Abstract: In Sri Lanka, the Municipal Solid Waste is disposed in to sanitary landfills or dumping sites. The generated leachate at dumping sites causes many problems leading to various social, cultural, environmental impacts. Hence, it is decided to conduct this research to find out suitable solution for proper treatment for the leachate generated at waste disposal sites. In this research, two stage biological treatment processes were used to treat the leachate. At first stage up flow anaerobic sludge blanket (UASB) is used. The important of the UASB is it removes high amount of BOD_5 and COD in efficiently. However the UASB process is not capable for removing fair amount of nutrients. To overcome this problem an aerobic system sequence batch reactor (SBR) was used after the UASB. The specific task of this introducing this SBR system is to study the capabilities of removing nitrogen in leachate treatment. In the total system, the BOD_5 removal efficiency is around 92% and COD removal efficiency is reached to 54%. Therefore this system can be used as secondary treatment system, to treat leachate.

Keywords: Municipal Solid Waste, Leachate, Biological treatment processes, UASB, SBR, Nitrogen removal

1. INTRODUCTION

Along with the rapid growth of human population and their needs, the solid waste generation has increased dramatically all over the world. Hence, solid waste management is an inevitable term of sustainable development. “Landfill” is one of the most widely used, viable and economical method of solid waste management in the modern world.
When the degenerating waste of those solids in the landfills contact with moisture it generates a liquid called leachate, This is a serious environmental problem, which environmental mangers and engineers face today, because this leachate contains a substantial, amount of organic and inorganic substances and has the potential to pollute ground and surface water. Therefore, this leachate should be treated. Various methods have been used for leachate treatment. The biological method is a widespread method, which is adopted in many countries all over the world. The biological treatment method includes anaerobic process, anoxic process and aerobic process. At present pre-treatments such as mechanical or chemical-mechanical methods are used before these biological methods. Anyhow this kind of biological systems were not studied much for landfill leachate treatment. This system generally consists of an anaerobic system which can resist high loading capacity as the first stage of the combined system, followed by a low loaded aerobic bioreactor. In both systems activated sludge treatment process is used. In this research, this system was used to study the performance of two stages UASB (Upflow Anaerobic Sludge blanket)-SBR (Sequencing Batch Reactor) for landfill leachate treatment.

2. LITERATURE REVIEW

The municipal solid wastes is treated and disposed in different ways. Landfill is the most popular and widespread method in the modern era 1[4]. The types of landfills can be classified in different perspectives. However, in general it is not only limited to one perspective. A combination of requirements and criteria can be used in practical [4].

Landfill leachate is a liquid which is mainly produced by the physical mixing of garbage and chemical reactions with rain water which, falls on the landfill and infiltrates into the garbage [4]. The leachate usually contains larger number of pollutants and it can be divided into four groups. They are (a) Dissolved organic matter, (b) Inorganic macro components, (c) Heavy metals, (d) Xenobiotic organic compounds (XOCs) [3]. Landfill leachates can also be classified according to the landfill age. The three types are young leachate, intermediate leachate and stabilized leachate.

To treat this leachate biological system can be used due to its reliability, simplicity and high cost effectiveness. In biological treatment, there are different mechanisms such as aerobic treatment, anoxic treatment, anaerobic treatments and natural systems [1]. Microorganisms are the primary agent of this type of treatment.

Anaerobic bio treatment process is used primarily for the treatment of waste sludge and high strength organic waste. This process is advantageous because of the lower biomass yield. Furthermore energy can be recovered in the form of methane, from the biological conversion of organic substrates [2]. Up flow anaerobic sludge blanket reactor is a popular anaerobic reactor for both high and low temperature. This is a single tank process in an anaerobic bio treatment system achieving high removal of organic pollutants. Wastewater enters the reactor from the bottom, and flows upward. A suspended sludge blanket filters and treats the wastewater as the wastewater flows through it. Bacteria living in the sludge break down organic matter by anaerobic digestion, transforming it into biogas. Solids are also retained by a filtration effect of the blanket. The up flow regime and the motion of the gas bubbles allow mixing without mechanical assistance [6].

In the aerobic treatment process, the conversion of organic matter is carried out by mixed bacterial cultures in general accordance with the stoichiometry with the presence of oxygen. The SBR is one of an aerobic system which, complete mix activated sludge process is happened [2].The conventional SBR contains four steps: filling, reacting, settling and idling without a secondary clarifier.

To get the maximum performance of the biological systems, coupling can be done. In this research biological system coupling of UASB and SBR was established to treat landfill leachate in order to enhance organic and nitrogen removal. Combining anaerobic and aerobic biological treatments is effective and efficient because it can produce good quality effluent. Moreover, these systems are cost effective, simple and do not cause pollution. Therefore a biological approach that combines anaerobic and aerobic systems have been recommended as a feasible method for removing organic and nitrogen from landfill leachate [5].

3. Methodology and implementation

3.1. Experimental setup

An integrated treatment system was operated. The treatment train consists of a UASB followed by an equalization tank and then an SBR as shown in Figure 1.

The UASB has a working volume of 8.2*l* and HRT is 8hrs. The SBR had an effective volume of 10*l*. An equalization tank was designed to adjust the conflict between continuous effluent in the UASB and intermittent influent in the SBR.

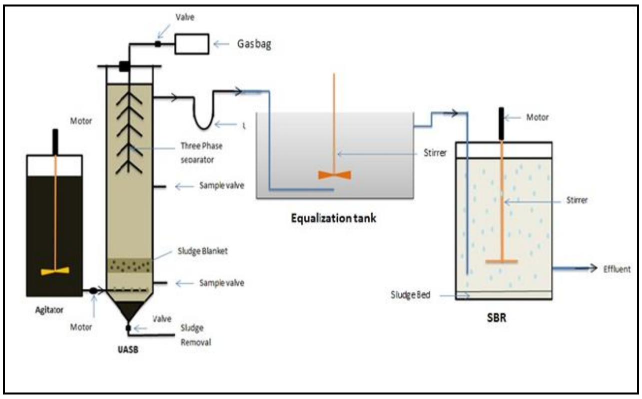

Figure1: Schematic diagram of the UASB-SBR biological system

3.2. Operating conditions

The system was run more than 4 months; this included the start-up of the system. The UASB was seeded with anaerobic sludge from UASB digester in the ice cream factory treatment plant (Ceylon Cold Stores Pvt Ltd plant at Ranala). The aerobic sludge was seeded to the SBR from the wastewater treatment plant at Temple of Tooth Relic. For the accumulation of the sludge the reactors were run with synthetic wastewater. After that, synthetic leachate was introduced to the system for 10 weeks. There onwards the UASB was operated with the avg. organic loading rate of (OLR) 84kg.COD/m^3.day. The SBR was fed with the UASB effluent, and it was operated under alternating aerobic and anoxic conditions. The cycle of the SBR consisted of 10-min filling, 90-min aerobic I, 205-min anoxic I, 60-min aerobic II, 150-min anoxic II, 120-min settling and 10-min decanting and idling period.

3.3. Sampling and analytical methods

The performance of the treatment system was evaluated by monitoring the quality of the influent and the effluent of each treatment unit. The physico-chemical analysis covered: pH, temperature. NH_4^+-N, NO_3^- -N, COD, BOD_5, TN.

For the UASB the MLVSS and gas yield was measured additionally. Inside the SBR, pH, temperature. NH_4^+-N, NO_3^- -N, COD, BOD_5, TN parameters were evaluated in each phase and the MLVSS also measured.

4. RESULTS AND ANALYSIS

4.1 BOD_5 removal of the UASB-SBR system

Figure 2 shows the system performance related to the BOD5 removal throughout the 10 weeks. In the system, the organic matter removal was around 92% which was in the expected efficiency range. The effluent BOD5 level of the system was below 30mg/l. These results have proven that, the combined system can be used to remove organic matter efficiently instead of using these two apparatus separately.

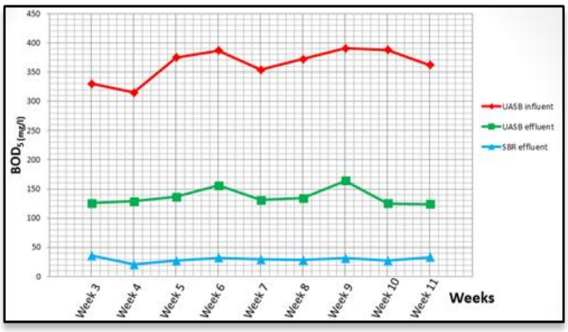

Figure 2: BOD_5 variation of the System

4.1 COD removal of the UASB-SBR system

The COD removal was measured during the experiment period. The figure 3 shows the variation of COD removal during the study period.

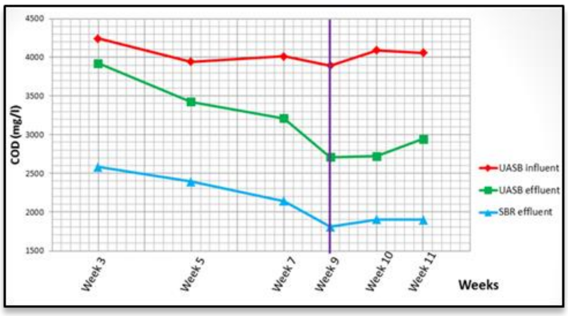

Figure 3: COD variation of the System

The COD removal efficiencies of the UASB, SBR and system were shown in Fig.4. And from the first week onwards, the COD was maintained around 4000 mg/l in the UASB influent. In the UASB, COD removal efficiency was not stabilized during the experimented period. In first eight weeks the gradual increment of removal efficiency was observed. Anyhow after fist six weeks, the increment gradient was slight. From the beginning, the SBR shows acceptable removal of the COD.

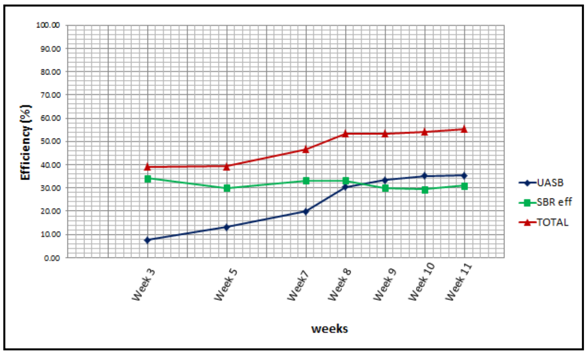

Fig.4. COD removal efficiencies of the UASB-SBR and the system

4.1 TN removal of the UASB-SBR system

The total nitrogen shows overall reduction in the effluent from the beginning in UASB and SBR effluents. The reason for the drastic drop of total nitrogen content at the influent from the seventh week in the UASB influent was the change of nitrogen source of the synthetic leachate. The total nitrogen removal was shown in figure 5. and it is clear that organic nitrogen was very high at the initial synthetic leachate composition which was not a significant feature of real landfill leachate. After the introduction of NH4Cl, the total nitrogen content was dropped but the NH3-N amount was almost equal with the previous leachate.

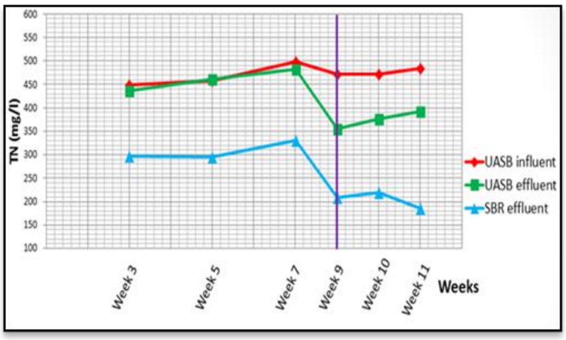

Figure 5: TN variation of the system

5. CONCLUSION

In this study, synthetic leachate was treated by using anaerobic and aerobic coupled system in laboratory scale model under tropical conditions in Sri Lanka for 10 weeks. BOD5/COD ratio of the synthetic leachate was between 0.09~0.14 throughout the study and this leachate had medium to low biodegradability. It was observed that the combined system's organic matter removal efficiency was about 92%. Effluent BOD5 was below 30mg/l, which satisfy the standard to discharge into the inland surface water bodies. The organic matter removal was significant with this combined system. Considering all of these aspects, it is concluded that, the combined system was effective for treatment of leachate under tropical conditions.

Acknowledgement

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SECM/15/173

Physico-Chemical Characteristic of a Petroleum Contaminated Soil from the Spill site of Jaffna District.

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Absrtact: Contamination of soil with petroleum products is among the most common sources of pollution in an industrialized world. This poses severe threats to the local communities and the ecosystem. Consequences of soil contaminations by petroleum products are multi-dimensional and thus their assessment has remained major problem. The presence of total organic carbon (TOC), heavy metals, electrical conductivity (EC) and pH were determined from petroleum contaminated soil samples from the spilled location of power plant premises of Chunnakam, Jaffna district. Three spilled locations have been identified and samples were collected from each location for this study. Control samples were collected from the uncontaminated location from the area same as the geology of the affected area. Results revealed that the heavy metal content of lead and nickel were higher than those of the control site and the recommended permissible limit. Evidence of severe hydrocarbon contamination was confirmed by presence of elevated level total organic carbon in the contaminated soil. Other analyzed metals including ferrous and manganese and physical parameters such as electrical conductivity and pH in the impacted zone have not shown any significant differences, while compared to the control samples.

.**Keywords:** Heavy Metals, Total organic carbon, Oil Spills, Soil Contamination.

1. Introduction

Past power plant activities in the area of power plant premises of Chunnakam, have resulted in heavily polluted soils. Plant effluents and oily wastes had been disposed to the nearby land plots over to several years without any mandatory regulations being followed. Contamination of soil by petroleum products is a severe environmental problem. This contaminated soil could poses a constant threat to local communities, due to the possibility of pollution spreading to the surrounding areas (Radu and Diamond,2009).Remediation of soil compared to water and air is difficult, due to the complexity of assessment of overall toxicity (Rank and Lawrence, 2013) and complex spatial distribution of the contaminant in the subsurface.

Refined petroleum products are complex mixtures of variety of hydrocarbons, heavy metals, antioxidants, corrosion inhibitors and other additives (Callot and Ocampo, 2009; Essien and John, 2010). Heavy metals in the refined petroleum product may have resulte

from the natural occurrences and resultants of

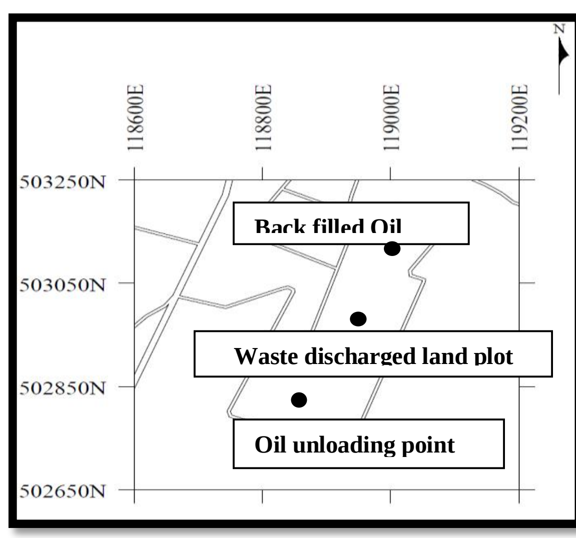

Figure 1: Soil sampling locations in side the power plant premises

the cracking process of crude oil. These pollutants may persist for a long time in the environment and act as a continuous source of hazardous compounds to the soil and as well as the groundwater (Ostendorff, 1990; Essaid et al., 1995). Marwood et al., (1998); Kelly and Tate

(1998) and Amadi et al., (1996) measured the heavy metal concentration to quantify the degree of contamination in soil matrix There are several evidences, but are not limited to, (Coyne, 1999; Kelly and Tate, 1998; Amadi et al., 1996) that the soil ecosystem could significantly loss in quality due to the impact of petroleum hydrocarbons. Testa, 1997 observed diminished microbial biomass and microbial activities in contaminated soil profile due to the toxicity of petroleum hydrocarbons. Muniz et al., 2004 discussed the seriousness of contaminant accumulation in organisms through the food chain, due to the contaminants uptake by plants.

Similar to the inorganic constituents, soil consist of natural organic matters which derived from decaying animal and plant residues and from soil biomass. Several anthropogenic activities also contribute to the soil organic matter. As petroleum hydrocarbons consist of the vast

diversity of complex organic carbon compounds, oil contamination substantially increase organic carbon level in the soil matrix. Therefore, determination of the amount of organic carbon may have helpful to delineate the contaminated soil from the uncontaminated zone.

The main objective of this study is systematic comparison of the soil matrix in the vicinity of the Power Plant Premises and at a control site. This may facilitate to delineate the contaminated soil zone from the uncontaminated zone and helpful in the estimation of the magnitude and the level of contamination. The specific task of this phase is intended to quantify the degree of contamination of soil matrix on the basis of heavy metal and total organic carbon presence.

2. Material and Methods

2. 1 Soil Sampling and Preservation

Top soil samples were collected from the three identified spilled locations within the Power Plant Premises. Individual soil samples were put in labeled polythene bags and flame sealed. Sealed samples were stored in a cooler box until brought to the laboratory. Samples were stored in the laboratory at 4^0C in a refrigerator until analyses were made.

2. 2 Preparation of soil samples

Samples were individually air dried at room temperature and sieved with the 2 mm sieves. Sieved samples were checked through the hand lens and remaining plant debris and calcium carbonate particles taken by autoclaved forceps

2.3 Analysis for heavy metal and other metals

Individual soil sample were weighed (250 mg) and transferred to digester vessels for soil digestion process. The samples were left in contact with the acid solution overnight and digested using a microwave assisted reactor system. Digested soil samples were filtered and analyze for ferrous, lead, manganese and nickel with the atomic absorption spectrophotometer.

2. 4 Analysis for total organic carbon

Analysis of the organic carbon by dry combustion method was performed by organic elemental analyzer (Flash 2000 Organic elemental analyzer). 50mg of individual soil sample were weighed in a clean ceramic blocks and allowed to react with fresh concentrated hydrochloric acid in order to remove any inorganic carbon. Reacted samples were heated in the oven for one hour at 60 ^{0}C to drive off residual acid. 15mg of treated soil samples were wrapped in tin capsules and capsules introduced to the analyzer.

2. 5 pH and Electric conductivity

Soil samples were weighed (2 g) and transferred to a Teflon (PTFE) bottle. Distilled water was added (10 ml) into each bottle with soil, and the bottles were placed on a shaker, and were mixed for 4 hours. Then, the solutions were filtered through Watchman 42 filter papers and solutions were measured for pH and electrical conductivity by pH meter and electrical conductivity meter.

3. Results and Discussion

Table 1: Heavy metal content and total organic carbon content for all soil samples collected from three contaminated sites and the reference site are presented in Table. Electrical conductivity and pH of the samples are also presented

Location	pH	EC µS/cm	TOC (w/w%)	Pb (ppm)	Ni (ppm)	Fe (ppm)	Mn (ppm)
Back filled oil lake	7.82	127	1.66	5-6	13.5-14	164	82
Oil unloading point	8.5	190	2.09	2-2.5	4-4.5	162	46
Waste discharged plot	7.42	100	16.23	8-8.5	24-25	162	61.5
Control site	7.61	111	0.54	1-2	0.5-1	157	52.1

Heavy metals lead and nickel in the soil samples from the contaminated zones were compared with the uncontaminated zone. The results clearly demonstrate that concentration levels substantially increased at the suspected contaminated sites. These observations with respect to heavy metal (Pb, Ni, Cu and Zn) were consistent with the reported results by Abdhullah et al. (1972), Scerbo et al. (2001), Figueira et al. (2002) and Fatoba et al. (2013**)**. No remarkable changes were observed in ferrous and manganese contents at the contaminated sites compared to the reference zone.

Soil samples from the location waste discharged point showed substantially higher level of organic carbon. The TOC content of the zone is 30 times higher than the uncontaminated reference zone. This elevated levels of TOC values than the uncontaminated reference zone, could be attributed to the heavy impact from organic compounds from the petroleum wastes and oily effluents (Ellis and Adams, 1961;Coker and Ekundayo, 1994). Other two locations also showed elevated level of TOC values compared to the control sample. Electric conductivity and pH associated with most of the soil samples has not been shown significant difference in comparison with the control samples.

4. Conclusion

It can be concluded that the evidences of severe contamination of the soil profile by petroleum contaminants of the impacted sites compared to the un-impacted site on basis of total organic carbon presence. Results of the heavy metal analysis obtained from the soil samples also confirm high soil pollution from heavy metals in the area. High concentration of these pollutants could be very hazardous to human health and as well as the living organisms, when too much level of pollutants are biologically accumulated through the water sources or by food chain.

5. Recommendations

Having successfully analyzed the soil samples from the plant premises, it has confirmed that the soil was polluted. Hence, it is pertinent, that adequate remediation and complete cleanup measures should be carried out on the sites to save the environment as soon as possible.

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Domino Effect of River Training in Large Sand-Bed Braiding Rivers

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Abstract: Large sand-bed braiding rivers such as the Brahmaputra River form an enormous challenge to understand and to control. For efficient and sustainable management of these rivers, it is vital that we can predict the effects of river training works on the channel pattern and dynamics. In this study we used a computer model to simulate the dynamics of bars, islands and channel branches in a large sand-bed braiding river. We applied river training works in it to evaluate the nearby effects and the far-away effects. The results showed that a single river training work like bank protection or a groin can significantly affect the locations of bars, islands and channel branches far downstream. The downstream propagation of the effect of a river training work is a domino effect by means of bifurcation instability and bar shape adjustment. It means that a training work can adjust navigation channels, bank erosion and flooding over many kilometres downstream of the training work. Thus, a training work in a large sand-bed braiding river not only has local effects on the flow and river bed, but can also have major economic and social impacts. This is both a sign to be very careful with river training in these rivers, and a great opportunity to change a long river reach by a single, relatively cheap river training work.

Keywords: Discharge, Modelling, Rivers, River pattern, Sand transport

1. Introduction

The Ganges and the Brahmaputra are two of the largest rivers on Earth located in one of the most densely populated areas in the world. The rivers are monsoon driven and every year major flooding occurs, causing thousands of death and loss of residences and property. River banks are eroded with rates of hundreds m/year, taking away valuable and scarce land (Sarker et al. [1]). This is just one example and many other inhabited river plains and deltas worldwide face similar problems. Moreover, expected climate change and sea level rise will increase these problems. Yet the populations also depend on the rivers for drinking water, irrigation and transportation. In fact, maintaining fairways to guaranty navigability of rivers is a major challenge in view of the desired economic growth.

Multiple attempts to curb the rivers by training works only had limited and temporary effects (Mosselman [2]). Or, they solved local problems, but introduced new problems at other locations in the river.

In large sand-bed braiding rivers, the interaction between bars, islands and channel branches plays a major role in the dynamics of the river, with bifurcations being an important link between them (Schuurman et al. [3]).

In this study, we evaluate the effects of river training works on the channel morphology and morpho dynamics of large braided sand-bed rivers. We used numerical modelling in which we applied different kinds of river training works.

2. Methods

We used the well-established physics-based numerical model Delft3D to construct a ‘data set’ of large braiding sand-bed rivers an example of river training works.

Delft3D computes the hydrodynamics by solving the depth-averaged flow equations for momentum and mass balance. More detail about the Delft3D model can be found in Schuurman et al. [4]. The hydrodynamics are used to compute sediment transport and bed level change. In addition, the model accounts for bank erosion, spiral flow and bed slope effect.

We applied uniform sediment, constant discharge and a straight initial channel with flat bed. The model settings are given in Table 1. The channel dimensions, channel slope and discharge were inspired by the Brahmaputra River, but we had no intention to reproduce the Brahmaputra River. In one model run without river training works, the main channel of 3200 m wide had

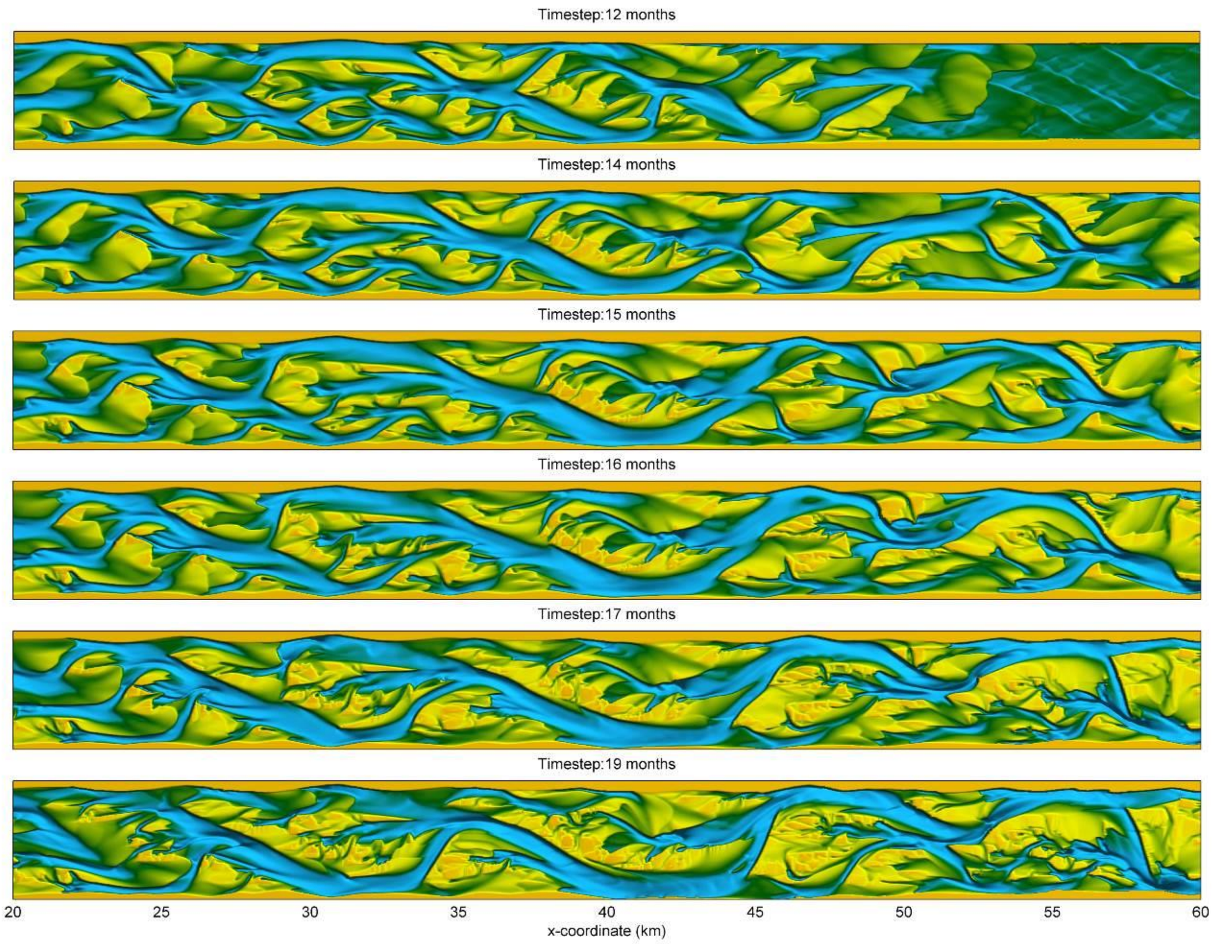

Figure 1: Bed level evolution in a model run with erodible floodplains. Flow is from left to right.

erodible floodplains along both sides that remained dry during the discharge of 40,000 m³/s. In the other model runs, we applied fixed banks without floodplains

Table 1: Model settings

Parameter	Value
Discharge	40,000 m³/s
Channel length	80,000 m
Channel width	3200 m
Channel slope	3,2 E^{-5}
Initial water depth	8.6 m
D_{50}	0.2 mm
Bed roughness	k_s = 0.15 m
Grid cell size	50 x 20 m

3. Results

3.1 Braiding channel pattern construction

Figure 1 shows the initiation and evolution of a braiding channel pattern in a straight channel, including the initiation of bars and channel branches. The bars were initiated at the upstream boundary and formed a downstream expanding front of bars. Thus, each bar triggered the initiation of a new bar further downstream.

Furthermore, the locations of the bars were linked to the locations of bank erosion. This is attributed to redirection of the flow towards the outer banks by the bars. This was a self-reinforcing process, as the bank erosion provides space for sedimentation and thus expanding of the bar. The bed evolution in the modelled channel also showed another interaction between bars and channel branches through bifurcations. New branches were formed when bars were dissected by cross-bar channels (e.g. at x = 52 km), and other branches where closed by expansion and migration of bars (e.g. at x = 28 km).

The channel reached a stable Braiding Index of about 3, which means the channel had on average about three parallel channel branches. Although this shows that the channel was braiding, some

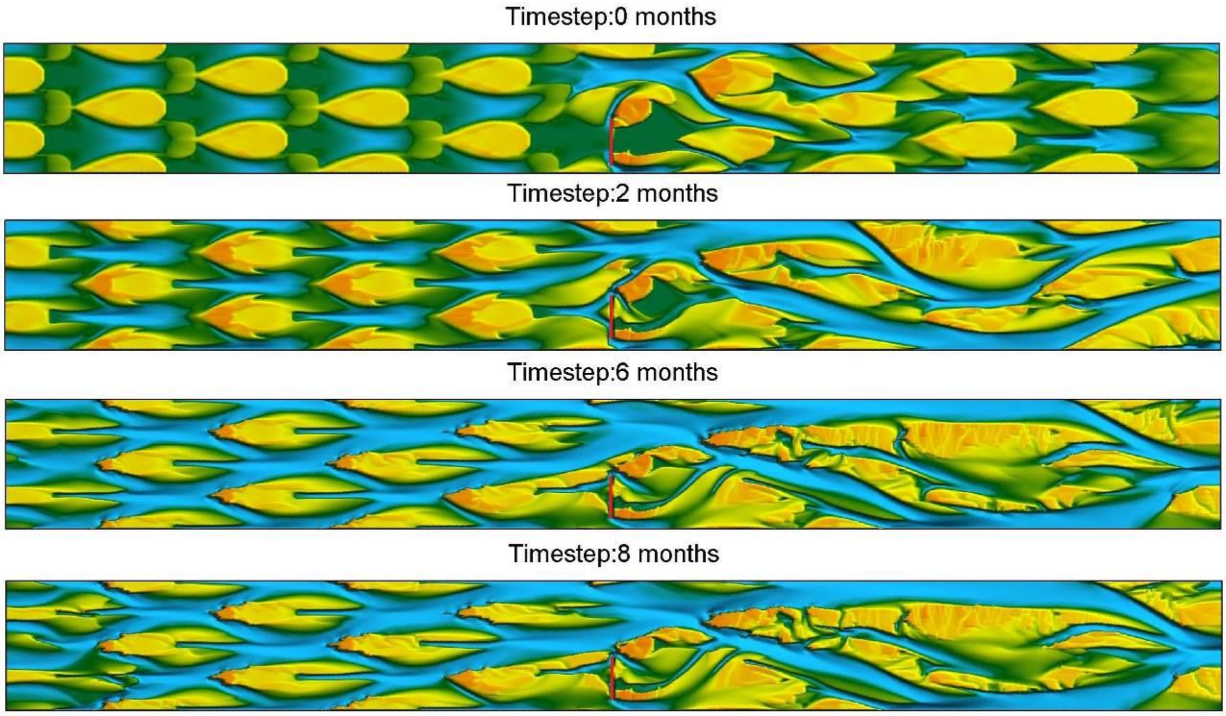

Figure 2: Bed evolution in case of a training work in an initially symmetric bar setting. The red line denotes the training work dam. Flow is from left to right. Adopted from Schuurman et al. [5].

channel branches had a meandering behaviour, with bank erosion in the outer bend and bar expansion in the inner bend (e.g. at x = 43 km in Figure 1).

3.2 Effect of river training works in initially symmetric bar settings

Now, we added a river training work in a channel with initially a symmetric pattern of mid-channel bars. In this case, the river training work was a dam to close one of the channel branches. The bed evolution as a response to this training work is shown in Figure 2.

The symmetrical bars upstream of the training were symmetrical at the start of the simulation and remained nearly symmetrical after 8 months. In contrast, shape of the bars near and downstream of the training work completely changed. This started near the dam, where the flow was steered around dam and incised the bed due to local flow acceleration. The eroded sediment was deposited further downstream, resulting in the formation of an enormous bar by merging of the initial bars.

Thus, here we could identify three zones of influence: the area in the vicinity of the training work that was directly affected by the training work; an area of adaptation to the directly affected area; and a long zone in which the bars and channel branches were indirectly affected by downstream propagation of the influence.

3.3 Effect of river training works in a complicated bar settings

Next, a training work was introduced in a complicated, more realistic setting of mid-channel bars and channel branches. Here we used a self-formed bar pattern and closed again one of the channel branches.

The result is given in Figure 3, with a closure dam built at the northern branch along bar D. Similar to the symmetrical bar setting, the dam redirected the flow, causing flow acceleration and local bed incision. In this case, the bed incision dissected bar D, forming two smaller bars D1 and D2. The training work in km 32 had enormous impact on the locations of bars and channel branches further downstream. For example, a 5 km long bartail-limb formed downstream of bar D2, connecting bar D2 with bar F. Along bar F, no clear main branch existed anymore in months 13 and 14, which would have serious economic consequences if navigation was important. It also had implications for the shape of bars G and H further downstream.

The downstream propagation of the effect of the training work in km 32 is shown in Figure 3B. Two months after construction of the dam, the influence zone expanded to near km 70. This implies a

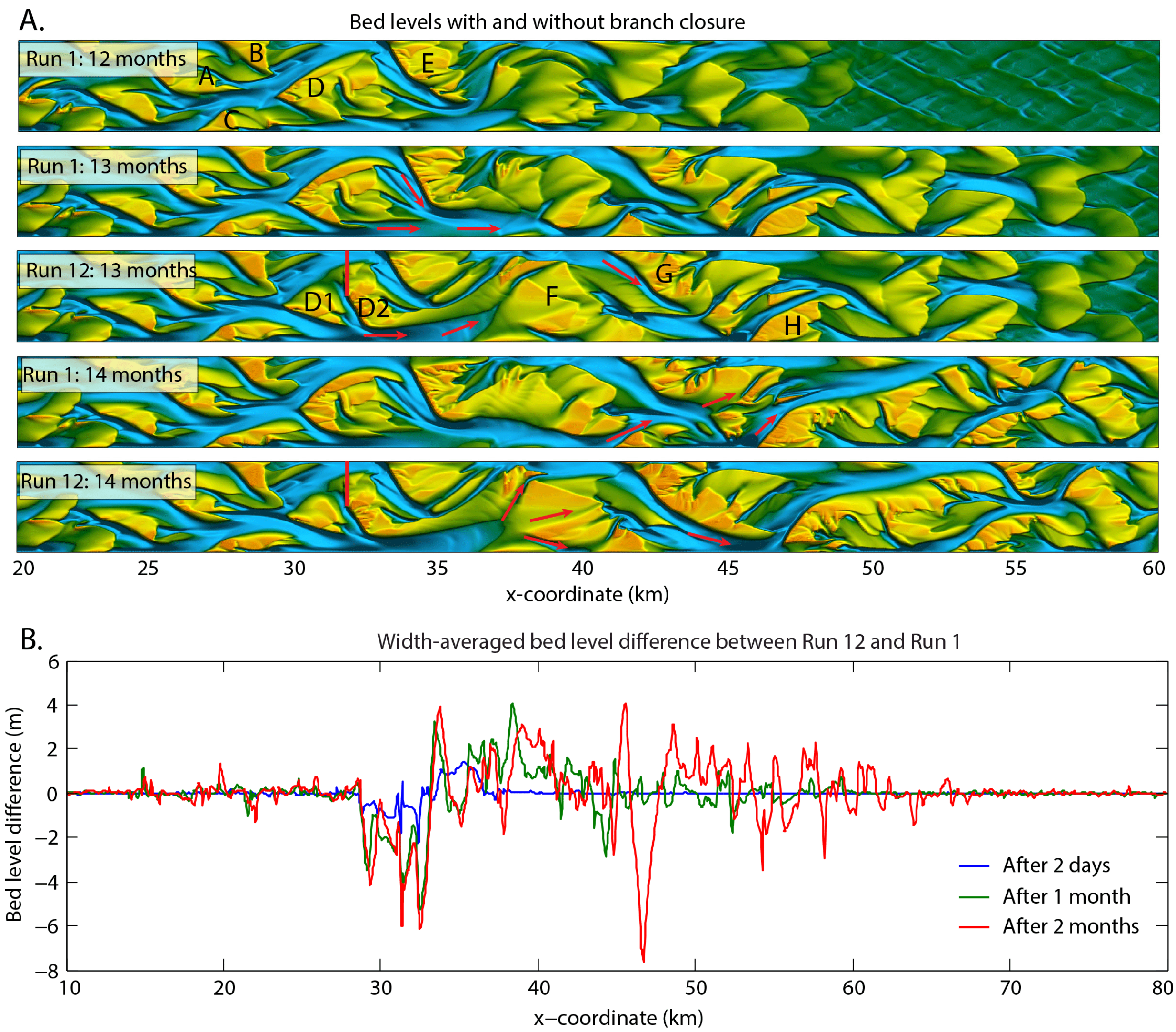

Figure 3: A) Bed evolution in a complicated bar setting with training work (Run 12) and without training work (Run 1). The red line at x = 32 km in Run 12 denotes the training work dam. Flow is from left to right. B) width-averaged bed level difference between the model run with and without training works, showing the downstream propagation of the influence of the training work. Adopted from Schuurman et al. [5].

propagation celerity of nearly 20 km/month. At the same time, the upstream effects of the training work was relatively small (Figure 3B).

4. Discussion

This study showed the effects of a training work in a large sand-bed braiding river using a state-of-the-art computer model.

The computer simulations showed different influence zones (Figure 4): a local, direct effect of the training work, which was channel incision in our case; an indirect effect by adaptation to the direct effects, which was deposition in our case; and an indirect effect due to adjusted locations of channel branches and bars.

Based on the interaction between bars, branches and bifurcations, plus the identified influence zones, we propose a conceptual model of the downstream propagation of the effect of a training work in a sand-bed braiding river. This conceptual model is illustrated in Figure 5 and further explained in Schuurman et al. [5]. The conceptual model comprises of the following steps, with numbering according to the numbering in Figure 5:

1. The training work adjusts the nearby flow and discharge division over the channel branches.
2. This both adjusts the shape of the bar by either erosion or bar expansion, and it

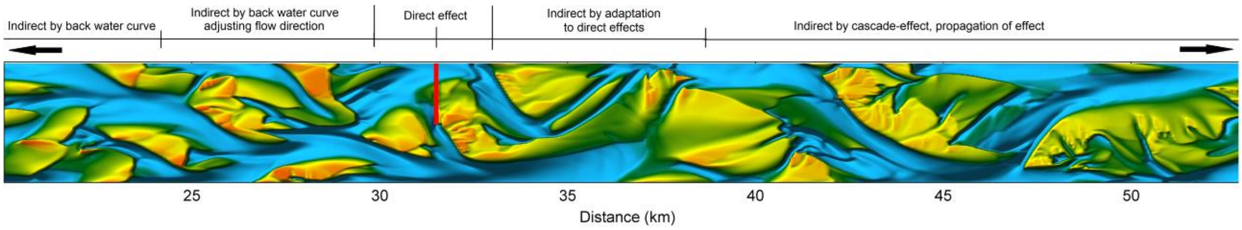

Figure 4: Zones of influence by a training work in a braiding channel

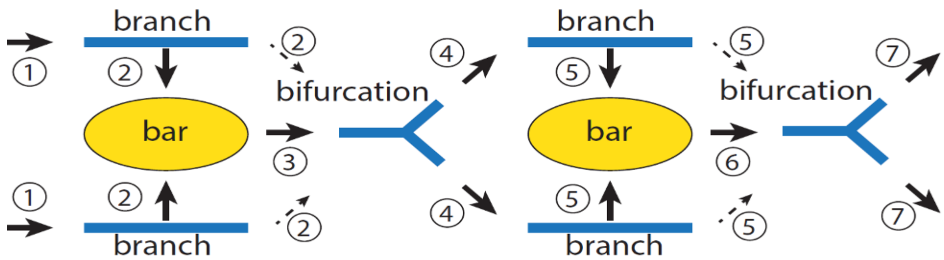

Figure 5: Conceptual model of downstream propagation of the effect of a river training work in a braiding channel, based on Schuurman et al. [4].

adjusts the flow approaching the downstream bifurcation.

3. The adjustment of the bar changes the approaching flow at the downstream bifurcation even more.
4. The discharge and sediment distribution at the bifurcation is changed.
5. The discharge and sediment in each branch is changed, which affects both the second bifurcation and the section bar.

These cyclic processes propagate in downstream direction like a domino game, and eventually affect the entire downstream river morphology.

The conceptual model is only valid for braiding rivers, as meandering rivers have no bifurcations. It was shown by, among others, Schuurman et al. [6] that a perturbation in a meandering river only propagates in downstream direction in case of meander bend, and the effects on alternate bars are limited to a few bar lengths. Also, narrow river sections might block the downstream propagation in a braiding river, for example in case of rock outcrops or engineering works. If these sections are too narrow for mid-channel bars, they may block the downstream propagation.

Nevertheless, it is important to understand that a relatively small adjustment to a braiding river, for example a training work, can have enormous effects further downstream. These effects include locations and amount of bank erosion, and the location and depth of navigation channels. Thus they may have social and economic impacts that are difficult to predict. However, it is also an opportunity to adjust a braiding river over large distance by a single, relatively cheap training work or other kind of perturbation. This requires more research to the natural response of the braiding river morphology to perturbations.

5. Conclusions

This study showed that a single river training work might have enormous consequences for the downstream morphology in large sand-bed braiding rivers. The hydrodynamic and morphological effects of a training work propagate in downstream direction by means of an interaction between mid-channel bars, channel branches and bifurcations.

Acknowledgement

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Comparison of Failure Mechanisms of Coastal Structures due to the 2004 Indian Ocean and 2011 Tohoku Tsunami Events

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Abstract: By analysing and comparing the results of post-disaster field studies and literature regarding the mechanisms by which coastal structures failed due to the 2004 Indian Ocean Tsunami and the 2011 Tohoku Tsunami events (the focus being on defence structures where applicable), trends were identified and examined. This paper highlights the most commonly occurring / major failure mechanisms identified in the various locations affected by the two tsunami events. The failure modes found in over twenty locations throughout the Fukushima, Iwate and Miyagi Prefectures of Japan were categorised into seven failure modes: a) leeward toe scour, b) crown armour failure, c) leeward armour failure, d) parapet wall failure, e) overturning, f) seaward toe scour, and g) sliding. Leeward toe scour was found to be the major failure mechanism in seawalls and dikes, and sliding was found to be the major failure mechanism in concrete breakwaters. The failure modes found throughout regions affected by the Indian Ocean Tsunami were categorised into five failure mechanisms: a) scouring of foundations, b) beam/column failure, c) joint failure, d) wall failure, and e) total disintegration. The 'total disintegration' caused by seismic forces, debris collision and hydrodynamic forces was the major failure mode throughout the studied regions. Some of the major tsunami induced forces found to have been among the causal factors of structural failure included hydrostatic and hydrodynamic forces. Flow velocities as high as 13.4m/s were found in areas of Japan, and flow velocities of up to 10.4m/s were found in regions affected by the 2004 Indian Ocean Tsunami. Potential strengthening measures were suggested for structures such as seawalls and coastal dikes, which were most vulnerable to scouring at the toe. By producing armoured components to protect the toe of the structures, they would become less susceptible to toe scour failure.

Keywords: Coastal structures, Failure mechanisms, 2004 Indian Ocean tsunami, 2011 Tohoku tsunami, Toe scour, Total disintegration.

1. Introduction

Recent extreme events such as the 2004 Indian Ocean Tsunami and the 2011 Great East Japan Earthquake and Tsunami (2011 Tohoku Tsunami) have been the cause of excessive amounts of damage not only to coastal structures, but also to the lives and economies of many. In order to be able to prevent such large-scale disasters in the future, it is imperative that research on the probable failure modes of the coastal structures is carried out. It is also very important that the relevant findings are used to design, strengthen and refine existing and future coastal structures to resist against such events. The purpose of this study is to compare the various failure mechanisms present in two tsunami events and in turn, to identify correlations between the structures and failure modes present in different locations. This paper also focuses on identifying some of the major destructive tsunami-induced forces that are responsible for the various failure mechanisms observed. By examining the identified trends, this study aims to investigate the vulnerabilities of the coastal structures and suggest viable solutions to strengthen them.

2. Identified coastal structures, failure mechanisms and tsunami-induced forces

The various coastal structures and failure mechanisms identified in this section, for both tsunami events, were obtained by examining and comparing field notes, reports, case studies, photographs and various literature documenting post-tsunami field surveys. Field studies that were analysed and compared included the works of Jayaratne et al. [1, 5], Kato et al. [2], Chock et al. [3], Saatcioglu et al. [4] and Shibayama et al. [6], which collectively covered field surveys that were carried out in over twenty locations throughout the Fukushima, Iwate and Miyagi Prefectures of Japan and also regions throughout Indonesia, Sri Lanka, Thailand and the Maldives Islands.

2.1 Summary of observed coastal structures

The various coastal structures observed in each of the surveyed locations were studied and analysed to determine by which mechanisms they failed. Only the most commonly observed coastal structures are summarised as follows:

Coastal structures observed in 2011 Tohoku Tsunami:

1) Coastal Dikes
2) Seawalls
3) Breakwaters

Coastal Structures observed in 2004 Indian Ocean Tsunami:

1) Seawalls
2) Residential buildings

2.2 Summary of identified failure mechanisms

This section briefly summarises the different failure mechanisms that were observed in the various coastal structures summarised in Section 2.1.

Failure mechanisms found in 2011 Tohoku Tsunami:

1) Leeward toe scour failure
2) Crown armour failure
3) Leeward armour failure
4) Parapet wall failure
5) Overturning failure
6) Seaward toe scour failure
7) Sliding failure

Failure mechanisms found in 2004 Indian Ocean Tsunami:

1) Foundation/Scouring failure
2) Beam and column failure
3) Joint failure
4) Wall failure
5) Total disintegration

2.3 Calculation of tsunami-induced forces

The observed failure mechanisms were often caused by a combination of different tsunami-induced forces which are shown below, along with the method by which these forces were calculated.

Flow Velocity

As tsunami waves have very long wavelengths, they act like shallow water waves. For celerity of shallow water waves Eq. (1) was used:

$$V = \sqrt{gd} \qquad (1)$$

where:
V = velocity of the tsunami flow (m/s)
g = gravitational acceleration (= 9.81 m/s^2)
d = flow depth (m)

Hydrostatic Force

According to the Federal Emergency Management Agency (FEMA),USA [7], hydrostatic load can be determined using the following equation:

$$f_{sta} = \frac{1}{2}\gamma_w\, {d_s}^2 \qquad (2)$$

where:
f_{sta}= hydrostatic force per unit width (kN/m)
γ_w = specific weight of fluid(10.1 kN/m^{3}for seawater)
d_s = design still water flood depth (m)

Hydrodynamic Force

Hydrodynamic load can be determined using Eq. (3) (FEMA [7]):

$$F_{dyn} = \frac{1}{2} C_d\, \rho\, V^2 A \qquad (3)$$

where:
F_{dyn}= horizontal drag force (N)
C_d = drag coefficient (-)
ρ = mass density of fluid (1025kg/m^3 for seawater)
V = velocity of water (m/s)
A = surface area of obstruction normal to flow (m^2)

2.4 Summary of calculated tsunami-induced forces

This section shows the various tsunami-induced forces that were calculated for both events, using the equations given in Section 2.3 above.

Table 1: Inundation heights and calculated flow velocities (2011 Tohoku tsunami)

Location	Inundation Height (m above MSL)	Velocity (m/s)
Otsuchi Town	12.2	10.9
Kamaishi Port	9.0	9.4
Minamisanriku	15.9	12.5
Onagawa	18.4	13.4
Hitachi Port	3.0	5.4

Table 2: Calculated hydrostatic and hydrodynamic forces (2011 Tohoku tsunami)

Location	Hydrostatic Force (kN/m)	Hydrodynamic Force (kN)
Otsuchi Town	750	7513
Kamaishi Port	402	5547
Minamisanriku	1269	9768
Onagawa	1710	11340
Hitachi Port	46	1849

NOTE: ‘Surface Area (A)’ = $100m^2$ was used for hydrodynamic force calculations.

Table 3: Inundation heights and calculated flow velocities (2004 Indian Ocean tsunami)

Location	Inundation Height (m above MSL)	Velocity (m/s)
Kaddhoo	1.3	3.6
Kuchchaveli	6.7	8.1
Palatupana	11.0	10.4
KhaoLak	9.6	9.7
Banda Aceh	7.9	8.8

Table 4:Calculated hydrostatic and hydrodynamic forces (2004 Indian Ocean tsunami)

Location	Hydrostatic Force (kN/m)	Hydrodynamic Force (kN)
Kaddhoo	8	789
Kuchchaveli	227	4129
Palatupana	611	6779
KhaoLak	465	5916
Banda Aceh	318	4893

NOTE: ‘Surface Area (A)’ = $100m^2$ was used for hydrodynamic forces calculations.

2.5 Analysis of failure mechanisms

2011 Great East Japan Earthquake and Tsunami

Leeward and Seaward Toe Scour Failure

Failure by leeward toe scour was found to be the major failure mode by which seawalls and coastal dikes failed (see Figure 1).But toe scour did not always lead to the failure of the leeward armour. However it was found that armour failure was commonly caused by toe scour. There were instances where no toe scour was found, but leeward armour still failed. A possible explanation for some of the leeward armour failures may be due to negative pressures caused by fast overflow that imposed suction on the armour and removed it.

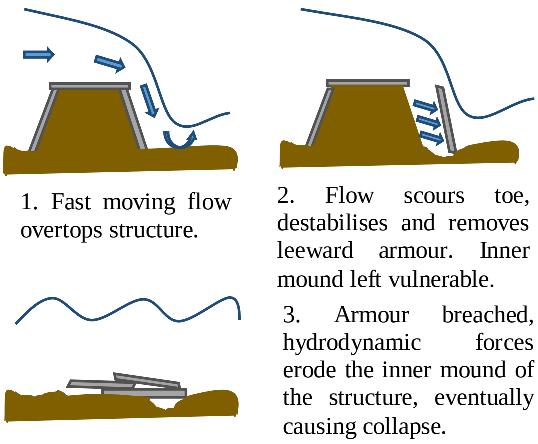

Figure 1: Leeward and seaward toe scour

It is commonly accepted that the shear forces that are induced by rapid flow on the structure toe generated by the overtopping waves are responsible for the toe scour. In order for mitigation methods to be implemented, it would probably be necessary to calculate the shear forces induced by the tsunami waves. Another tsunami induced force that could be useful to calculate in the case of further research would be overturning forces. It was found in the work of Jayaratne et al. [1], Kato et al. [2] and Chock et al. [3]that though leeward toe scour was the main causal factor of failure in seawalls, many seawalls were found to have failed ultimately by overturning. It could be argued that by making the seawalls more resistant to overturning, the scouring would likely have far less of an impact on the structure and may no longer be deemed a mode of failure but simply as some erosion being present. In order to mitigate the problem of scouring completely, the overturning moments induced by the tsunami waves could be calculated and then incorporated into the design process.

Crown Armour Failure

Crown armour failure was found to be one of the more common failure modes in coastal dikes. The major causal factor for this failure mechanism was proved to be negative suction pressure being induced by rapid flow overtopping the structure (Kato et al. [2]). When the suction force was greater than the resisting force (i.e. holding the armour in place), the crown armour was removed and left the inner mound vulnerable to scouring. As shown in Figure 2, once the armour was breached, the enormous hydrodynamic forces often eventually led to complete collapse of the structure.

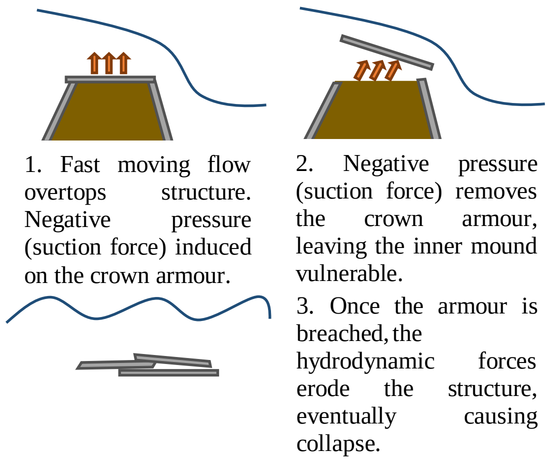

Figure 2: Crown armour failure

Overturning and Parapet Wall Failure

These two failure mechanisms have been grouped together because they work more or less in the same way. It was found that the majority of parapet walls that did fail (this failure mechanism was found to be very common), failed as a result of wave impacting forces. As it is shown in Figure 3, when the impacting forces imposed by the waves exceeded the resisting strength of the parapet wall, the structure was cracked or in some cases destroyed completely. This is very much the same for the overturning of structures such as seawalls and breakwaters. It was found that in many cases where structures failed by overturning, they were caused by the impacting force of the tsunami waves; whether it was the run-up process or the draw-down process. When the overturning moment induced by the waves exceeded the restoring moment, the structure overturned. It was also found that hydrostatic forces induced by differences in water level either side of the structure often resulted in overturning failure.

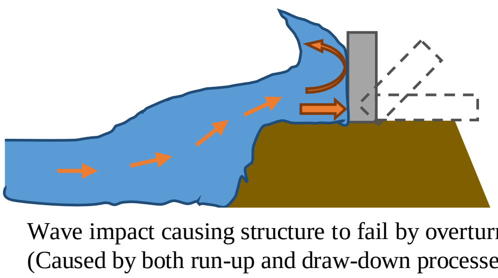

Figure 3: Overturning and parapet wall failure

Sliding Failure

Sliding failure was the major failure mechanism by which breakwaters failed. As tsunami waves overtopped the structure, a difference in water level on either side of the structure induced lateral hydrostatic forces. These forces pushed the structure, destabilised it and in many cases caused them to fail by sliding.

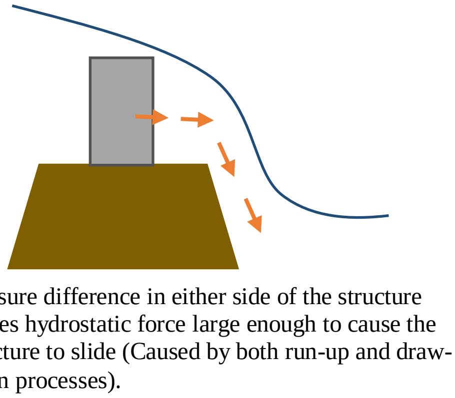

Figure 4: Sliding failure

It was noted that scouring would most probably have occurred and destabilised the mound, making it more susceptible to sliding failure. The hydrostatic forces and the presence of scouring were the possible factors in sliding failures. By computing shear forces induced by scouring, relevant measures could be taken to resist these forces. Hydrostatic forces were also calculated in this paper. Based on the calculations, it can be said that structures such as breakwaters would need to be designed to be able to resist against, on average, a maximum force of 1710kN/min regions of Japan, and 611kN/m for regions surrounding the Indian Ocean. Now it would seem that building a structure to resist such large forces would be difficult and expensive. It could also be argued that such a strong structure would not be necessary as hydrostatic forces that high would not occur regularly.
Chock et al. [3] also mentioned that caisson-type breakwaters that were founded on rubble mounds were more susceptible to sliding and overturning failure. It was assumed that this was because of the effects of scouring induced by shear forces on the rubble mound. There are however several different types of breakwaters. It could be interesting to see whether or not other types of breakwaters also failed commonly by sliding and overturning failure or if they failed by different mechanisms. This is an area that could be focused on if further research were to be carried out in future.

Foundation Failure

Saatcioglu et al. [4] found that most of the residential buildings along the shorelines of urban

Thailand had spread footings which are shallow foundations.

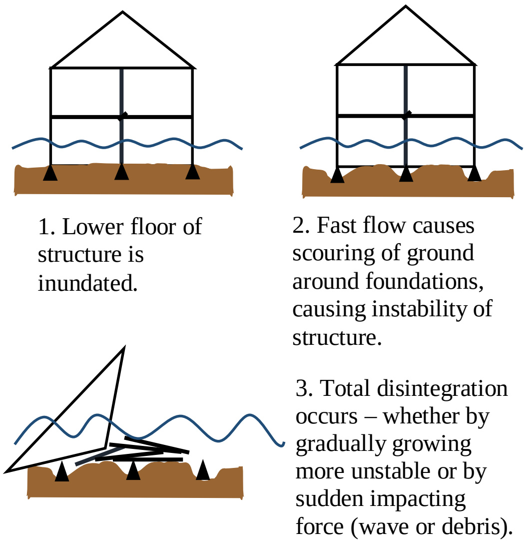

Figure 5: Foundation failure

It was found that shear forces were induced by the rapid flow, which in turn scoured the ground around and under the foundations of the structure. This resulted in unsupported foundations. Such lack of foundation support had obvious effects on the stability of the structure and in some cases where the foundation was not quickly repaired, it even led to collapse. In other cases where the structure had already been made unstable, debris impacts and hydrodynamic forces from the waves eventually led to collapse.
As there are not many other studies focussing on the failure mechanisms of coastal structures in Thailand due to the 2004 Indian Ocean tsunami, it is difficult to evaluate the views of Saatcioglu et al. [4]. It can however, in this case, be seen quite evidently from photographs that there wash eavy erosion around the footings of marine buildings. It would be logical to say that the erosion of the footings would have had a large effect on the stability of the structure, and inturn have been a governing factor in the overall collapse of the building.

Wall Failure

Wall failure occurred as a result of differences in water levels on either side of the wall, which imposed hydrostatic forces. These forces, when greater than the resisting strength of the walls, led to punching failures. As the walls serve as part of the structures' frame to support roofs, floors and ceilings, failure of these walls often led to instability of the supported components and eventually to the collapse of the structures.

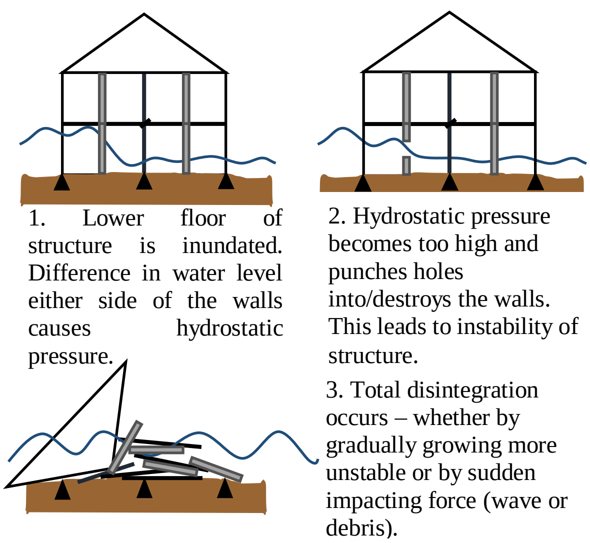

Figure 6: Wall failure

Total Disintegration

Total disintegration was found to be the major failure mechanism by which structures failed in regions of Indonesia and Thailand especially, under the 2004 Indian Ocean tsunami. It was found that these failures were often caused by systematic failure whereby foundation failure, beam and column failure, joint or wall failure eventually led to the total collapse of the structure. Unlike the wall failures shown in Figure 6, beams, columns and joints serve an essential role in the distribution of loads in a structure. In many cases, when the beams or columns or joints were damaged (due to seismic action, hydrodynamic forces, debris impact etc.) the overall stability of the structure was severely affected. Such instability, when not repaired quickly, often led to collapse. In other cases structures were made unstable and vulnerable by beam, column and/or joints being damaged, and then completely swept away (totally disintegrated) by wave or debris impacts. In other cases structures completely disintegrated under seismic excitation alone, before the waves even reached them. It was more commonly observed however where structures that had already been weakened by seismic excitation from the earthquake, were completely swept away by the tsunami waves.
One of the fundamental reasons why so many structures were devastated by this failure mechanism was the poor quality of design and construction. It was observed that even though a few structures were structurally well-engineered,

they were not built to withstand seismic forces. This may have been because the surveyed areas were previously not as prone to seismic activity as Japan.

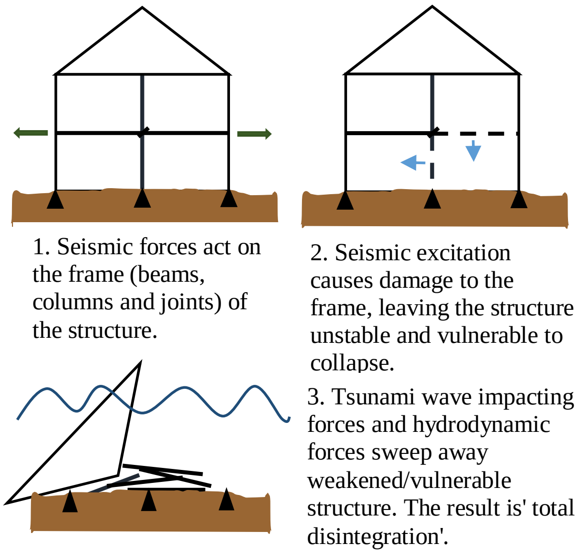

Figure 7: Total disintegration

Thus, when designing for future structures, it would be necessary to incorporate concepts of earthquake engineering into the structures to be at the least somewhat resistant to seismic action. It was discovered that many structures had strong beams and weak columns, which is seen to be one of the worst possible combinations in earthquake engineering. Ideally, a structure would need to remain stable and strong enough to withstand the hydrodynamic forces imposed by tsunami waves even after seismic excitation. According to the calculations carried out by the authors, a structure of surface area 100m^2 in the Indian Ocean regions would need to be able to withstand drag forces of up to about 6.8 MN. Now as was the case with the hydrostatic forces, it would be difficult to design a structure to withstand such a force, and even if it were possible it would likely be very expensive and inefficient. An alternative would be to develop methods of minimising the hydrodynamic forces to some extent and also to incorporate strengthening measures to structures so that the combination of the two methods would allow for structures to resist the hydrodynamic forces without failing.

3. Viable solutions and strengthening measures

Scouring

In this study, it was found that toe scour posed a significant threat not because the scouring itself rendered the structures unusable, but because they led to the destabilising of the structures and made them more susceptible to overturning and sliding failures. As this is the case, two logical approaches could be taken in order to strengthen the structures against failure:1) Strengthen the toe region of the structures to avoid/resist erosion in the first place,2) Second approach would be to make the structure more stable so that even if scouring were to occur, it would not be enough to cause the structure to fail.

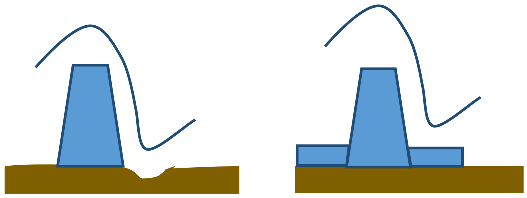

Figure 8: Toe scour (left), Protected toe (right)

If the first approach were to be taken, various materials could be used to create an elongated toe part that would come into contact with the rapidly overtopping flow. If the toe of the structure is elongated as shown in Figure 8, the structure is more stable in two ways;1)The elongated toe part will be the part of the structure the rapidly overtopping flow will impact [as opposed to ordinary soil as shown in Figure 8 (left)]. This means that worst case scenario, the elongated toe part will be damaged, but the soil underneath will not be scoured and the structure will therefore remain stable. The elongated toe effectively works as a sheet of armour protecting the soil on either side of the structure against scouring. Considering that such methods could prove to strengthen the structure against failing completely, it could be deemed a worthwhile investment despite the costs. 2) The elongated toe part of the structure is directly connected to the base of the structure, giving it more stability against overturning failure. It is commonly known that structures or objects with a wider base are more difficult to overturn. By implementing such elongated toes to these structures, they would be protected against the effects of both scouring and overturning failure. In order to make this method affordable and sustainable, cheap but durable materials could be used, perhaps materials that are recycled/recyclable or reusable such as aggregates.
These elongated toe parts could also be designed to be removable/ attachable to the main structure. This would allow for just the toe parts to be replaced in the event of one of them being damaged, rather than having to repair/replace the whole structure which would be more expensive and inefficient. This would also be beneficial as such parts could be added to already existing

structures as well as ones that are to be built in future. One thing that should be considered however is the method by which these attachable parts will be held in place. In crown armour failure, it was understood that the tsunami-induced negative pressures lifted and removed the crown armour from their existing places. It is possible that such negative pressure could remove these attachments if they are not securely fixed to the structures.

The second approach mentioned above, involved making the structure more stable so that even if scouring were to take place, the structure would remain unaffected. This could be done by embedding the foundation of the structure deeper into the ground, much like pile foundations, which would give the overall structure greater stability than if a structure has shallow foundations. This approach could be somewhat problematic however, especially for existing structures. Such methods could be implemented into proposed new construction. An alternative method would be to increase the self-weight of the structure which would make it more resistant against overturning forces.

Total Disintegration

It is fundamental when designing for structures to be resistant against seismic excitation, to have a frame with strong columns and weak beams (the opposite of that found in the structures in Indonesia). The reason for this is earthquakes usually cause a lateral movement of the earth. Such excitation is known to cause stiff columns to snap and flexible columns to sway in the direction of the seismic movement. If columns are strong, they are able to withstand the seismic action without snapping/breaking. Beams on the other hand connect column to column. When columns begin to sway due to seismic excitation, the beams if too stiff would simply snap. By adopting a strong columns and weak beams configuration, the structures would become more resistant to seismic excitation. This configuration could easily be applied to future design and construction and also to already existing structures by using various methods of reinforcement. It was found through interaction diagrams that the columns observed in regions of Thailand could not even sustain half of the moments that were imposed, before failing. But columns that had lateral bracing provided by in-plane infill walls were found to survive against the seismic action as well as the tsunami waves. Such information could be taken into consideration and additional lateral bracing could be applied to columns, whether by infill walls or by alternative methods.

Other practises included in earthquake engineering could also be incorporated into design. Factors such as stiffness and orientation of the building's shape could also be considered in designing process. For example though uniform distribution of stiffness is ideal (i.e. the top through to the bottom of structure have the same stiffness,) when this is not possible, lower floors of the structure should be made stiffer and the upper floors should be made increasingly less stiff to avoid a 'soft storey mechanism', in which case the lower floors would simply snap or collapse and bring down the rest of the structure. The changes in stiffness between each floor should also be gradual. It must also be ensured by careful design that there is a balance in stiffness of the columns and structure in both directions. Again these strengthening measures could be applied to structures being designed and built in the future, and they could also be implemented by methods of reinforcement to already existing structures.

4. Conclusions and further studies

4.1 Conclusions

The failure mechanisms by which coastal structures failed in two extreme events were identified; both patterns identified by researchers in the past and new patterns were found. Some of the major tsunami induced forces responsible for the failure patterns were identified and quantified through theoretical models and simple calculations. Trends regarding these forces were also found and linked back to the failure mechanisms. Numerous weaknesses, vulnerabilities and patterns in the coastal structures were discovered and some basic strengthening measures and concepts were produced in order to account for these vulnerabilities. The major failure mechanism found in coastal dikes and seawalls was leeward toe scour and sliding failure was most common in breakwaters for the 2011 Tohoku Tsunami. For the 2004 Indian Ocean Tsunami the major failure mechanism found in coastal and residential structures was total disintegration.

Some of the major tsunami induced forces were identified and quantified. These included:

- Flow velocities
- Hydrostatic forces (lateral)
- Hydrodynamic (drag) forces

Numerous vulnerabilities were discovered in the coastal defence structures. Seawalls and coastal dikes were found to be weakest at the toe of the structures. The toe must be strengthened to withstand overturning failure. Concrete breakwaters (caisson-type) were weakest where the toe of the structure linked onto the mound. Again the toe must be strengthened to resist scouring and sliding in breakwaters.

Residential structures found in Indonesia and Thailand were poorly designed against seismic action. A ‘strong column, weak beam’ configuration must be adopted. Reinforcing measures must be implemented to the columns and joints especially.

4.2 Further studies

Sediment characteristics, geomorphology and terrain should be analysed to see if these have an effect on the stability of structures. Perhaps this could be done by carrying out sieve analyses and by studying the samples. Also the failure mechanisms of ordinary residential coastal structures in Japan (not defence structures) should be compared with those found in residential structures that failed due to the 2004 Indian Ocean Tsunami event. It would be interesting to see how the failure mechanisms differ between well-engineered and non-engineered structures.

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Design and Construction of a Large Shiplift Facility in India

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Abstract: To meet the growing shipbuilding needs in Indian subcontinent, L&T Shipbuilding has set-up a modern shipyard facility at Kattupalli near Chennai, India. As part of shipyard infrastructure, a state-of the art Shiplift and Transfer System has been implemented. With facility to handle ships of beam up to 43m and lifting capacity of 21000 MT (expandable to 26000 MT), this is one of the largest shiplifts in the world. To optimize the project cost and develop technology capability, entire planning, design, development and construction of Shiplift system including equipment and control system were handled indigenously. This along with stringent time schedule posed numerous challenges to the project team. Innovative solutions were adopted to overcome challenges and to maintain the project cost within budget. This paper covers the design and construction aspects of Shiplift system with focus on civil works. Challenges faced and solutions adopted are provided. The conclusion of the paper is that world class solutions can be developed and implemented successfully by pooling the available expertise and integrating it with specialist support.

Keywords: Drydock, Marine Structure, Shiplift**,** Shipyard.

1. Introduction

This paper describes the shiplift and transfer system constructed as part of L&T Shipyard at Kattupalli near Chennai, India with focus on civil works. This is one of largest shiplifts in Asia with a lifting capacity of 21000 MT (expandable to 26000 MT).

2. Project background

L&T Shipyard is located on the east coast of India, at Kattupalli near Chennai. It is part of an integrated marine complex consisting of Container Port, Shipyad and Modular Fabrication Facility. The facility was set-up to cater to the growing needs of modern shipbuilding and repair facility in Indian subcontinent. At present most of the new build / repair works are carried out in East Asian or Middle East yards.

Location and Overall Layout the facility are shown in Figure 1 & 2.

3. Shiplift concept and technology

During planning phases of Shipyard, two alternate dry docking concepts were evaluated, one is conventional dry dock and other being modern shiplift technology. Considering the need to cater to new build and repair business as well as other operational considerations, it was decided to adopt shiplift system for dry docking needs of the yard.

At a basic level, shiplift is a large marine elevator which can be lowered into water for lifting the ship to the yard level or lower the ship from yard level into water.

A shiplift consists of a steel lifting platform suspended by wire ropes, raised and lowered vertically by a series of hoists. The hoists are distributed in equal numbers on either side of the platform and located on piers or foundations. By synchronising all the hoists, the platform is raised or lowered uniformly and in a horizontal plane. Fine level adjustment is also available in the system.

Shiplift platform is the largest component in the system and is built with high tensile steel. Both rigid and articulated designs have been implemented in existing shiplifts with each having its own distinct advantages. The shiplift platform's vertical lifting speed is generally in the order of 150mm - 200mm per minute, but lifting speed never being a critical design requirement [1].

Transfer system is used to move the ship from yard to shiplift platform or from platform to its intended dry berth in the yard. It consists of series of heavy duty steel beams (Cradles) supported by a pair of hydraulic trolleys. The trolley system moves on a pair of rails and has hydraulic jack to support cradle beam and electric motor drive with fine control mechanism. This is used to move and position the ship accurately in its position.

The shiplift is controlled and operated from a control system and operating console. The control system has a number of in-built safety mechanisms to prevent incidents like overloading.

4. Shiplift system

After the selection of shiplift, available technology options were reviewed. As procurement of total system from a technology provider was much costlier, it was decided to develop the system in-house. As the company had sufficient expertise in such complex heavy engineering systems, this was considered feasible. A cross functional expert team was formed covering different disciplines to plan and develop the system. Independent experts with shiplift functional and implementation background were recruited to provide guidance and carry out independent reviews.

After numerous design trials and value engineering exercises, design configuration shown in Table 1 was adopted.

Table 1: Design Parameters

Parameter	Value
Length of shiplift platform	200 m
Width of platform	46 m
Available Vertical Travel	18 m
Hoists – 590 MT capacity	68 Nos.
Spacing of Hoists	4.8 m / 7.3 m
Articulated Steel Platform (with timber decking)	8500 MT
Design life of Platform	50 years
Classification / Certification	Lloyd’s Register

5. Project structure

Due to the criticality of project, the project was managed by a task force team headed by senior management team. All the stack holders were represented in the team. Major decisions were discussed and decided by the task force. This helped in resolving issues quickly. This task force approach to managing the project was one the key success factor.

6. Design

6.1 General

The design of shiplift system was carried out by an interdisciplinary team of specialists drawn from different engineering units of the company. Civil works team included marine structures, Pavements and Utilities design specialists. Platform and Lifting / Transfer system was handled by two teams, one for mechanical / structural works and another covering electrical, Instrumentation and Control Systems. These specialist teams were guided by the independent shiplift expert.

6.2 Layout and structural systems

Different layouts options like wet basin, finger type piers and hybrid systems were evaluated. After functional and preliminary cost comparisons, it was decided to adopt a finger type layout with platform located beyond shoreline in basin area. Two outfitting jetties are integrated on either side of the shiplift structure. Transfer areas and Dry berths used for positioning of ships are located behind the jetty area within shoreline. Cross section of shiplift is shown in Figure 3. Layout of shiplift facility is shown in Figure 4.

The shiplift main piers and end transfer areas were designed as piled deck structure with appropriate combination of marine / structural loads. Reclamation area behind Shiplift was retained with a combination of Sheet pile wall and rock bund structure. Reclamation area adjacent to shiplift was provided with rock revetment protection.

The onshore structures consisting of transfer / dry berths, crane foundations and pavements were designed as ground supported structures using beam on elastic foundation approach.

6.3 Marine structures

During preliminary design, different types of pile systems viz., Steel / Precast / Hybrid were compared from both design and construction aspects. Due to local expertise and its inherent cost advantage, bored cast-in situ pile system was chosen as preferred system. All the marine structures were designed with large diameter (1300mm / 1200mm) vertical bored cast-in situ piles with permanent casing. No raker piles were used in the system, mainly to improve constructability. Initial Pile loads were specified to verify the design parameters used in geotechnical design.
Superstructure was designed with a combination of Precast and In-situ elements. This helped in

achieving the right balance between design requirements and ease of construction. Maximum precast element weight was kept below 20 MT.

Considering durability requirements, following requirements were specified for concrete works:

- Concrete Grade: M40 (Splash Zone), M30 (above Splash Zone)
- Cement content : 400 kg/cum (Min.)
- Water cement ratio of 0.40 by weight (Max.)
- Minimum cover to reinforcement: 75mm (Splash Zone), 65mm (above Splash Zone)
- Chloride penetration in 30 years < 5mm

7. Construction

Materials for the works were sourced locally except for specialist items like Jetty furniture. Plant and Machinery for the works were also used from existing pool or sourced locally.

Major materials required for the work were Concrete and Reinforcement. Concrete was produced in batching plants established at site. Aggregate for concrete was sourced from quarries located 100km away. Reinforcement was procured from primary producers. Reinforcement was processed in central rebar yard for various works.

7.1 Marine piles

Large diameter piles were installed using Hydraulic Piling rigs mounted on temporary steel platforms and Jack-up barge. Piling operation was assisted by crane, primarily to install casing and reinforcement cage. Casing was installed by vibro hammer. After casing driving, piling rig completed the boring operation to required depth. Bentonite slurry was used to stabilise the bore. Concreting was done by tremie method. Capacity of regular piles was verified with High Strain Dynamic tests conducted on randomly selected sample piles.

Pile bore collapse due to presence of very loose sand layer (N<10) below dense layer of sand posed a serious problem during piling operation. This was overcome by extending the permanent casing below this loose sand layer.

Position of piles was very critical, especially for piles at winch location, as large deviation will create eccentric loading and affect the capacity / design parameters. Stringent control procedures were adopted to ensure pile installation tolerance (+/-75mm).

7.2 Superstructure

Superstructure elements were precast in a central casting yard set-up at site. Installation was done by crane from completed deck. In-situ portion of deck was taken-up sequentially. Installation of jetty furniture (Fenders, Bollards, and Stairs, Mooring rings) was done subsequently. More than 50% of superstructure works were precast, greatly enhancing the quality and speed of construction.

7.3 Shiplift platform

Shiplift Platform fabrication was carried out at project site itself. Assembly and installation was taken-up after substantial portion of civil works were completed. Winches were erected and integrated with Platform subsequently. After completion of electro-mechanical works, testing and commissioning was taken-up and completed.

7.4 Commissioning

Entire shiplift system was commissioned and certified by Lloyd's Register in Jan 2012. Since then the shiplift system has successfully performed many dry docking operations to date meeting or exceeding the performance requirements set out during design.

Figure 6 shows the shiplift facility in operation.

8. Conclusions

Shiplift and transfer system, one of the largest in the world at present was successfully completed as per plan within the budget. Design and construction of such large capacity complex shiplift system within short schedule was not achieved elsewhere without the active involvement / support from a technology provider. This was made possible by innovative solutions using the expertise available in the company with specialist support and focussing on the functional requirements through value engineering.

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Figure 1: Location of the Project

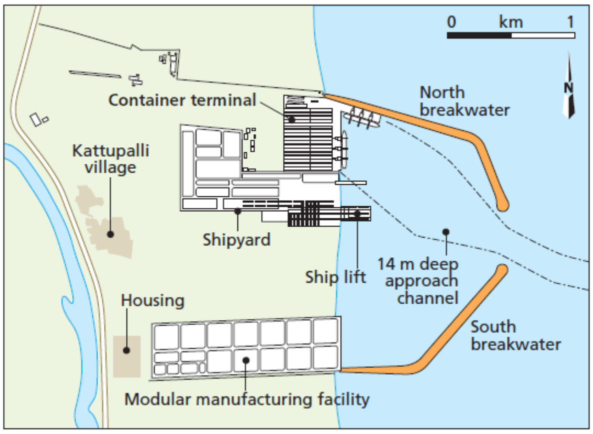

Figure 2: Overall layout of the facility

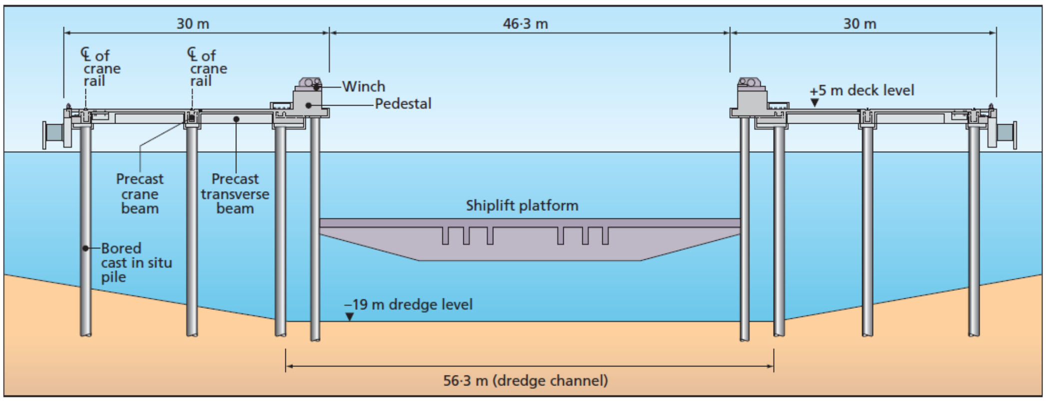

Figure 3: Cross section of Shiplift

Figure 4: Layout of Shiplift facility

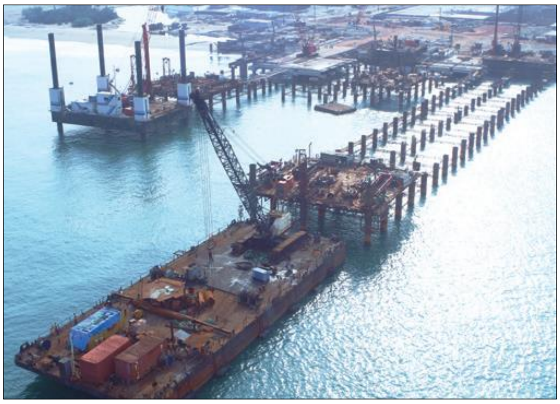

Figure 5: Shiplift under construction

Figure 6: Shiplift facility in Operation

Development of a Computer Model of a Drainage System with Uncertainties in External Inflow and Channel Cross-section

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Abstract: In urban development, stormwater drainage is an important aspect of infrastructure planning and design. Computer modeling has often been used to assist the design of the drainage system. With this type of modeling, external inflows to the system and channel configurations are important model inputs. As such, it is challenging to develop a model when there are uncertainties in external inflows and channel cross-sections.
This paper presents the development of a computer model of a drainage system in Singapore. In the development, it is necessary to resolve the uncertainties in the external inflow at various points along the drainage system, as well as the uncertainties in channel cross- sections. The software package Stormwater Management Model (SWMM) was used and the model has been developed and calibrated with on-site measured data. The results show that the external inflows have significant effects on the simulated hydrographs, while channel cross-sections do not affect the simulated hydrographs. On the other hand, the channel cross-sections have significant effects on the simulated water levels in the drainage channels.

Keywords: Modelling, Stormwater, Uncertainty

1. Introduction

In urban development, storm water drainage is an important aspect of infrastructure planning and design. Computer modeling has often been used to assist the design of the drainage systems. For example, Jang et al. (2007) used SWMM to simulate the hydrologic assessment of natural catchments and verified its applicability in both pre- and post-development considerations [3]. Kim et al. (2014) used numerical simulation to assess three alternatives of a drainage design in response to the use of an underground storage facility as an underground cistern for drainage [4]. In many of the publications, peak discharge of stream-flows, times to peak are important results to be analyzed. Models of runoff are used not only for forecasts and predictions of runoff, but also as inputs to the environmental processes. Numerical models are also widely used as a research and education tool to gain further understanding of the processes and to test hypotheses ([5]). On the other hand, model results are highly dependent on model inputs ([1], [2]). In particular, external inflows to the system and channel configurations are important model inputs. As such, it is challenging to develop a model if there are uncertainties in external inflows and channel cross-sections.

This paper presents the development of a computer model of a drainage system in Singapore. In the development, it is necessary to resolve the uncertainties in the external inflows at various points along the drainage system, as well as the uncertainties in channel cross- sections.

2. Methodology

This paper presents the case study of a catchment in Singapore with a total area of approximately 182 ha. The drainage system of the catchment was built for draining the surface runoff and external inflow resulting from industrial activities. The system was also subjected to tidal influence downstream.

In this catchment, rainfall and runoff data were monitored at nine stations over a period of one year from August 2012 to August 2013. Figure 1 shows the layout of the nine monitoring stations in the catchment. The data being monitored at the nine stations are summarized in Table 1. Rainfall was measured at six stations, discharge at three stations, and water level at all stations.

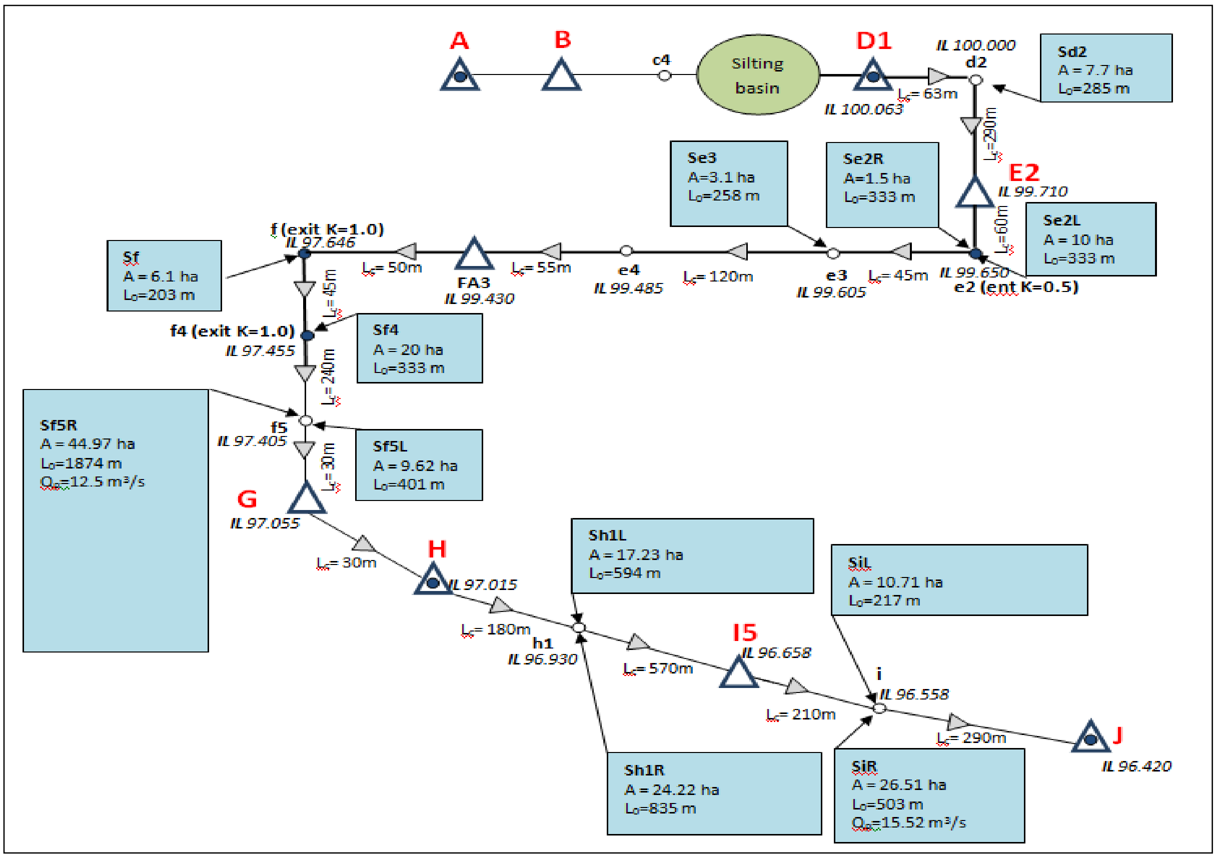


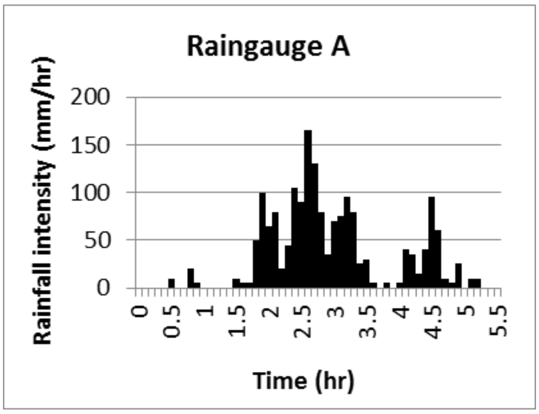
Figure 1: Layout of the monitoring stations

Table 1: Parameters being monitored at the different stations

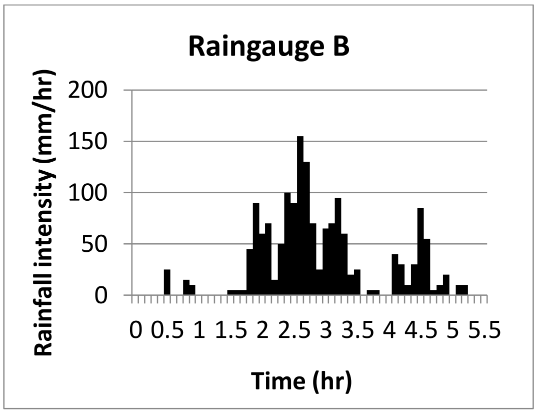
Station	Rainfall	Discharge	Water level
B	X		X
A	X		X
C (silting basin)			X
D1	X	X	X
E2	X	X	X
G	X		X
H			X
I5		X	X
J	X		X

Using the package Storm water Management Model (SWMM) developed by the US Environmental Protection Agency (USEPA) [5], a computer model of the catchment has been developed. The model consists of 12 subcatchments, 18 drain sections and a silting basin. In the model, considering the flat topography of the site, the slopes of the subcatchments were set at the value of 0.1%. Both ends of the drainage system were subjected to tidal influence. According to SMWW manual, the recommended values of Manning's n for an impervious concrete surface for overland flow (N-Imperv) is 0.013 and for a pervious surface (N-Perv) is 0.024. The recommended value of Manning's n for concrete conduits or channels is 0.015. The depths of the depression storage on both impervious and pervious areas (i.e. Dstore-imperv and Dstore-perv) were set at 0 mm ([5]).

Of all the measured data (August 2012-August 2013), the event on 15 December 2012 had the heaviest recorded rainfall. Hence, the computer model was calibrated and verified using the data on this date from 12:30 to 18:00. Figure 2 shows the rainfall data recorded at all stations, namely A, B, D1, E2, G and J.



(a)



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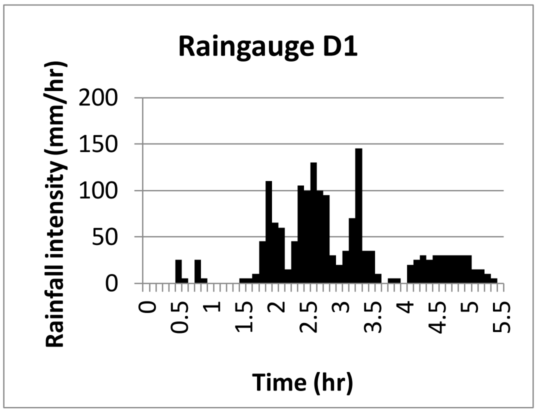

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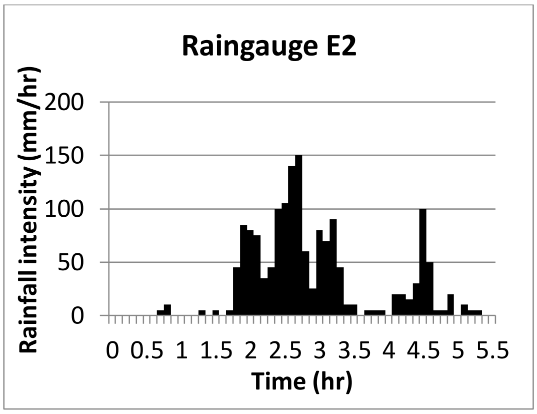
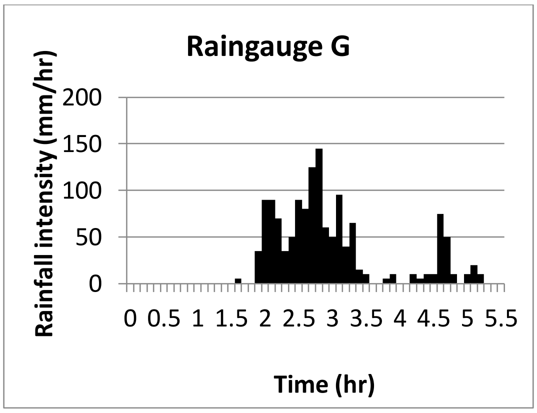
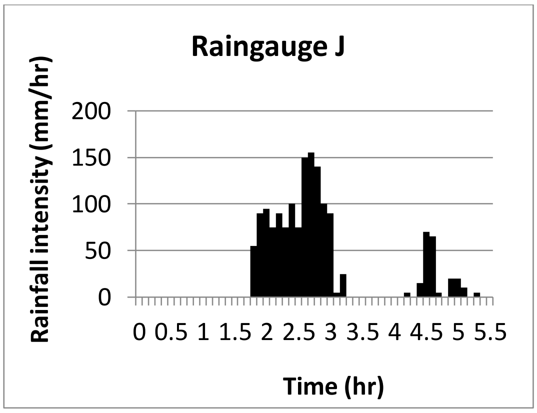


(d)



(e)



(f)

Figure 2: Rainfall recorded at six stations on 15 December 2012 from 12:30 to 18:00 hrs

To calibrate the model, the measured water levels at Stations A and J, which correlate with the tidal inputs, were applied at Nodes A and J in the model, respectively. Since there was no direct measurement of the external inflows, the first step in the model calibration was to determine the probable external inflow by assigning fixed values to all the other model parameters.

To determine the probable external inflow rates, Table 2 contains the values of process flow at nodes f4, f5 and I5 at various percentages of the design flow rates. These flow rates were then used as input for the model to determine the probable external inflows that would correspond to the discharge data collected on 15 December 2012.

The model also considered two sets of channel configurations that correspond to the design scenario and the measured channel configuration at the time of the monitoring. The design and measured channel configurations are shown in Table 3 and Table 4.

The simulated hydrographs were then compared with the measured hydrographs. The probable external inflow is taken as the flow rate with which the simulated hydrograph best fits the measured hydrograph.

Three values of runoff coefficients were considered in the simulations (C = 0.7; 0.8; and 0.9).

Table 2: External inflows (in m^3/s) at various model nodes and at various fractions of the design flow rates

% of design external inflow	Node f4	Node f5	Node I5	Total External Inflow (m^3/s)
100%	15.95	12.50	15.52	43.54
90%	13.48	11.25	13.97	39.19
80%	12.42	10.00	12.42	34.83
70%	10.86	8.75	10.86	29.08
0%	0	0	0	0.0

Table 3: Design cross sections of the channels

Channel section	Reach	Shape	Length (m)	Bottom width (m)	Height (m)	Side slope (H:V)
Section 1						
20m temporary earth drain	c4→ B → A	Trapezoid, open	190	5.5	3.75	1.93
Section 2						
4m drain	D1→d2→E2 →e2	Rectangle, closed	413	4	3	0
4m drain	e2→f	Rectangle, closed	270	4	3	0
6.5m culvert	f→ f4	Rectangle, closed	45	6.5	4	0
8m-wide canal	f4→f5→G → H	Rectangle, open	300	8	6	0
26m-wide canal	H→h1→I5→i→J	Rectangle, open	1250	26	6	0

Table 4: Measured cross sections of the channels

Channel section	Reach	Shape	Length (m)	Bottom width (m)	Height (m)	Side slope (H:V)
Section 1						
20m temporary earth drain	c4→ B→A	Trapezoid, open	190	4.7	2.8	1.30
Section 2						
4m drain	D1→d2→E2 →e2	Same as design	Same as design	Same as design		
4m drain	e2→f					
6.5m culvert	f→ f4					
8m-wide canal	f4→f5→G					
	G→H			11.8	4	0
26m-wide canal	H→h1→I5→i→J	Trapezoid, open		12	6	1.94

3. Results

3.1 Model calibration

In the calibration, the probable flow rates and runoff coefficient were determined based on the comparison of the measured and simulated hydrographs at Station I5.

a. Effects of external inflows

Figure 3 shows the simulated hydrographs at Station I5 with different values of the external inflows. The results showed that the simulated hydrograph that best fits the measured hydrograph is the one with 80% of the design flows.

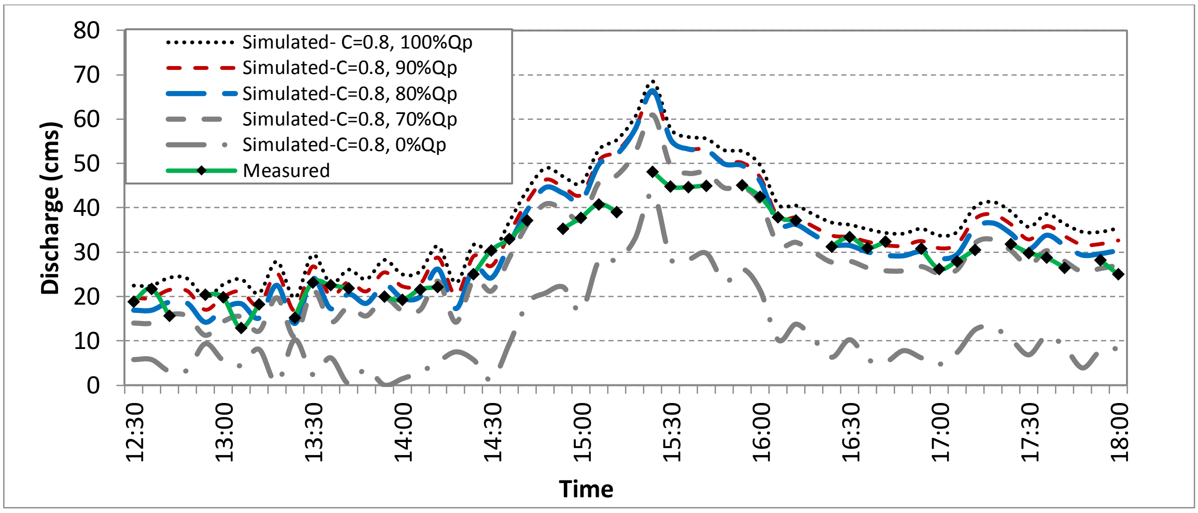


Figure 3: Comparison of measured and simulated discharges at various external inflows at Station I5 for the 15 December 2012 event

b. Effects of runoff coefficient

Figures 4 and 5 show the simulated hydrographs with C = 0.7; 0.8; and 0.9. From the figures, it is apparent that runoff coefficient has negligible effect on the simulated hydrographs and the simulated water levels.

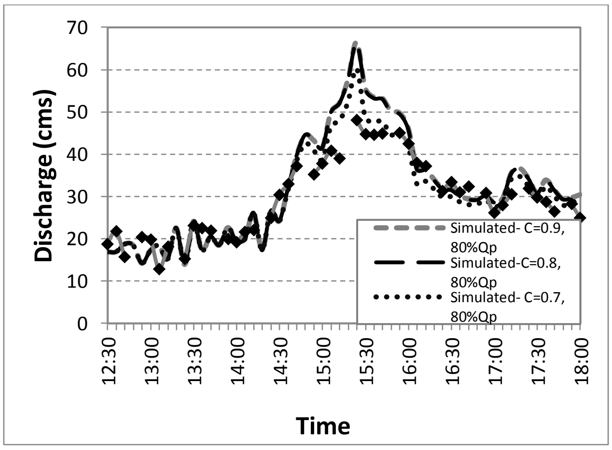


Figure 4 : Variation of the simulated hydrographs with different values of runoff coefficient.

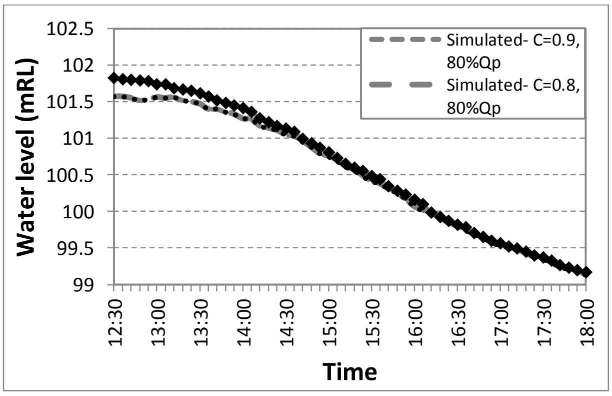


Figure 5: Variation of the simulated water levels at Station I5 with different values of runoff coefficient

c. Effects of channel cross-sections input

The simulated hydrographs during the event on 15 December 2015 using the input of cross sections as the design condition are shown in Figure 6. While the simulated hydrographs from the two simulations with the design and measured cross sections were similar, the change in cross sections has more impacts on the simulated water levels (as shown in Figure 7).

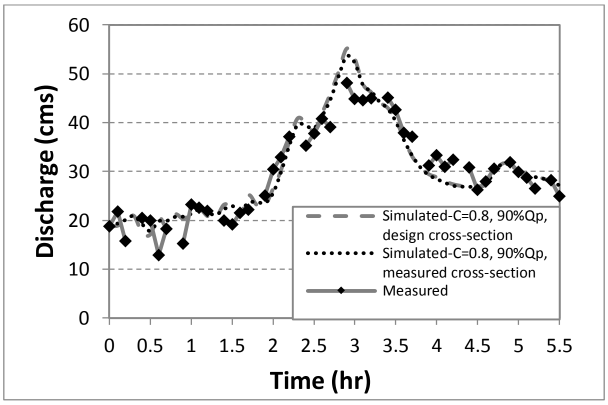


Figure 6: Variation of the simulated hydrographs at Station I5 with design and measured channel cross sections

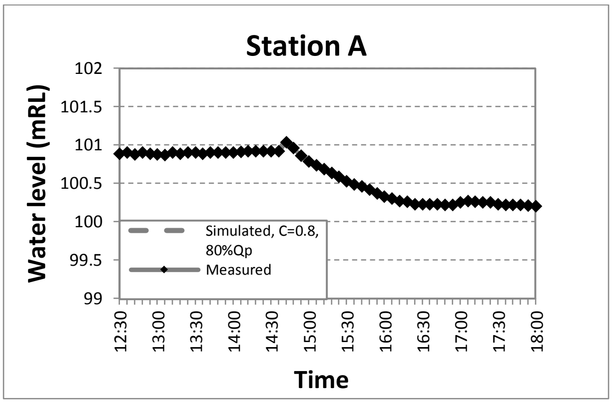


Figure 8: Comparison of the simulated and measured water levels at Station A for the 15 December 2012 event

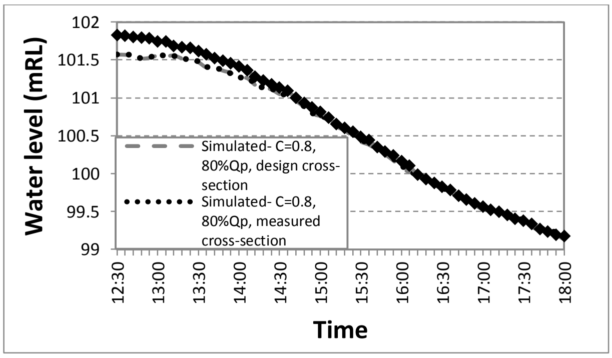


Figure 7: Variation of the simulated water levels with design and measured channel cross sections

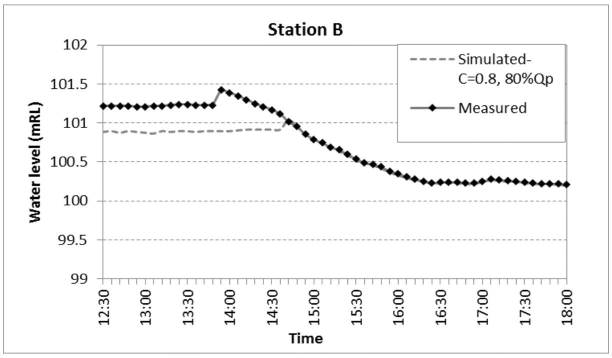


Figure 9: Comparison of the simulated and monitored water levels at Station B for the 15 December 2012 event

3.2 Model verification

Using the calibrated probable external inflow and runoff coefficient, the simulated discharge and water levels at Stations A, B, D1, E2, G, H and J are shown in Figures 8-16. The simulation was based on 80% design external inflows, runoff coefficient of 0.8 and design channel cross-sections.

Figures 10 and 12 show that the simulated discharges agree well with the measured discharges at Stations D1 and E2.

Figures 8 and 16 show that the simulated water levels match perfectly with the measured water levels at Stations A and J. This was because the measured water levels had been used as input values for the computer model. Figures 9, 11, 13, 14 and 15 show reasonable agreement between simulated and measured water levels for Stations B, D1, E2, G and H.

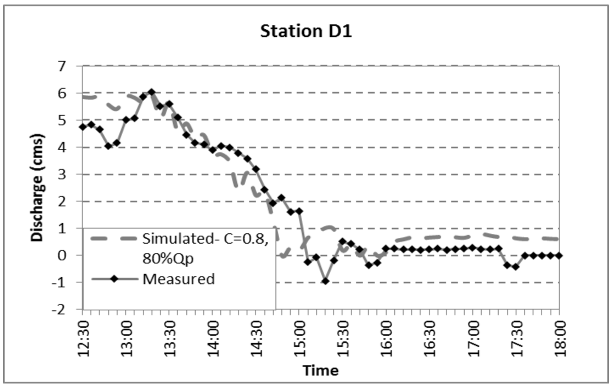


Figure 10: Comparison of simulated and measured discharges at Station D1 for the 15 December 2012 event

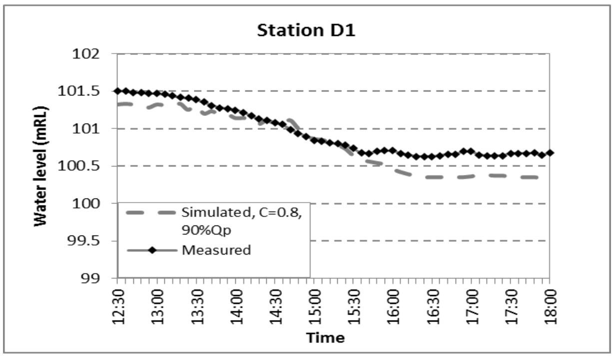


Figure 11: Comparison of simulated and measured water levels at Station D1for the 15 December 2012 event

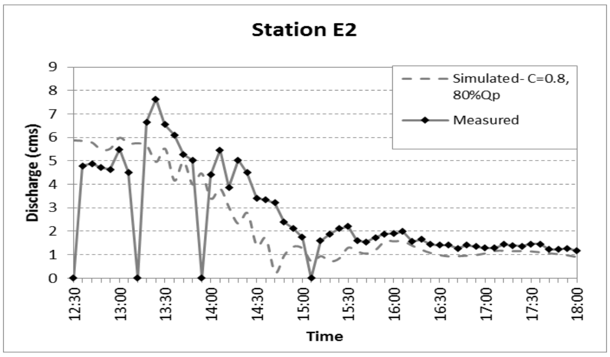


Figure 12: Comparison of simulated and measured discharges at Station E2 for the 15 December 2012 event

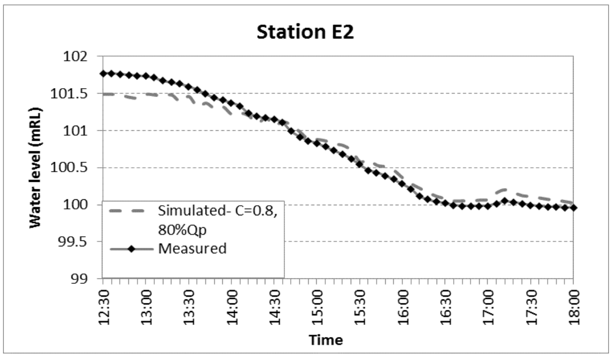


Figure 13: Comparison of simulated and measured water levels at Station E2 for the 15 December 2012 event

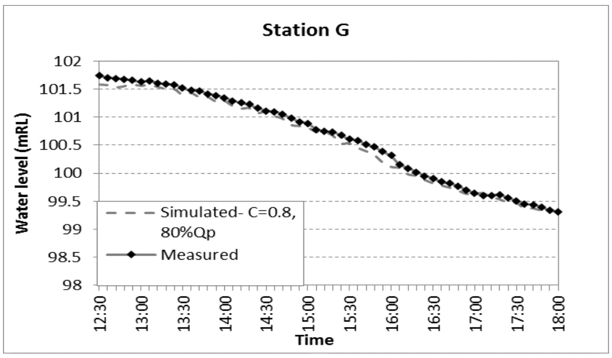


Figure 14: Comparison of simulated and measured water levels at Station G for the 15 December 2012 event

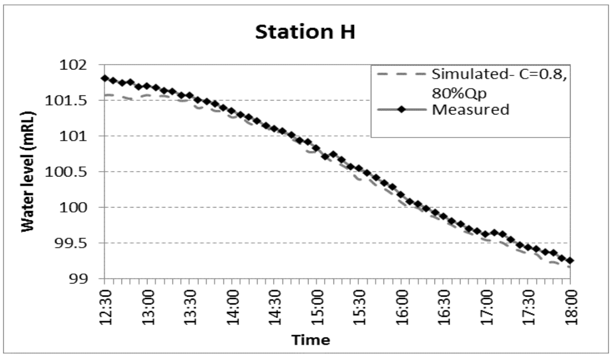


Figure 15: Comparison of simulated and measured water levels at Station H for the 15 December 2012 event

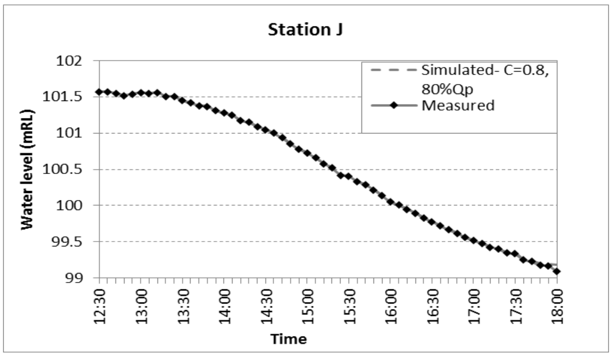


Figure 16: Comparison of simulated and measured water levels at Station J for the 15 December 2012 event

4. Conclusions

Using the Storm water Management Model (SWMM) software, a computer model has been developed for a drainage system in Singapore and subjected to tidal influence. The model was calibrated using the measured rainfall, discharge and water level data for the 15 December 2012 event. The results show that external inflows have significant effects on the simulated hydrographs, while channel cross-sections do not have any impact. On the other hand, channel cross-sections have significant effects on the simulated water levels in the drainage channels.

The model was then verified using the discharge and water level data at the other eight stations. The simulated discharges and water levels agreed reasonably well with the corresponding measured data at all stations.

Acknowledgement

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&Water Research Institute, NTU, Singapore) for his valuable suggestions on the paper.

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Design Process of a Sandy Convex Shaped Beach Layout

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Abstract: The planned Colombo Port City (CPC) development, shown in Figure 1, comprises 450 acres of reclaimed land, 3 km of offshore breakwater, two additional breakwater revetments and a central canal. In its final stage, the reclamation will be closed off by a sandy beach at the seaward side. This beach will be partly sheltered by the offshore breakwater and lagoon in front of it. The beach layout is convex, implying that all beach angles are offshore directed. This layout with respect to erodibility, poses multiple design complexities. These problems will be tackled by a converging design approach, focusing on reduction of risks and increasing knowledge (from measurements and modelling) at the one hand and a highly adaptable design at the other hand. This engineering management approach is described in the present paper.

Sediment transport along the beach is influenced by the complex hydraulic climate in the lagoon area: the combination of wave overtopping and transmission through the breakwater, waves diffracting around the breakwater heads, local waves and residual currents. The anticipated sensitivity to beach erosion should not negatively affect the development. Therefore, to quantify the beach stability, the hydraulic climate inside the breakwater has been assessed by numerical modelling to form a basis for the spatially distributed sediment transport computations. To acquire a reliable translation from the offshore wave and current climate to the climate within the lagoon area, extensive physical and numerical model studies have been performed.

The preliminary beach stability analysis indicates that mitigation measures will most probably be required. This requirement, as well as assessment of the type of mitigation measures, is key to the adaptive engineering approach that has been adopted here. The adaptive approach aims at arriving at a practical design for the beach to secure the functional (public) requirements within economical (maintenance) and practical (constructability) boundaries. A groyne scheme is a relative simple and adaptive way to stabilize an unstable beach, whilst providing flexibility as it can be implemented in a phased way and be adapted rather simply when required. Along with the design, we focused on optimization of the construction strategy and sustainable material usage.

The present paper presents the numerical analysis part of the iterative design process, which has not yet been completed. As such, this paper is the launching paper regarding the CPC beach stability, providing a baseline for the design, and will be followed up by further paper(s) at a later stage of the design and construction.

Keywords: Beach Protection, Colombo Port City, Convex layout, Erosion, Sediment transport.

1. Introduction

The Government of Sri Lanka together with the Project Company has initiated the development of a high- end urban development on newly reclaimed land near the port of Colombo: the Colombo Port City (CPC) Project. CPC Project will be constructed against the existing Colombo South Port in the North and against the Galle Face in the East. The development comprises 450 acres of reclaimed land, 3 km of offshore breakwater, two additional breakwater revetments and a central canal. The primary protection of the development is a semi-circular shaped offshore breakwater which obtains a calm wave climate along the reclaimed area to facilitate amongst others the construction of an ocean facing beach. Between the offshore breakwater and the beach a 300m wide so called “lagoon” will be created. The beach layout is convex-shaped and serves as secondary protection for the development. In principle such a beach layout is considered morphologically unfavourable. However, due to the anticipated relatively mild wave and current conditions caused by the sheltering effect of the offshore breakwater, as well as by the expected large sediment grain size on the beaches, the beach stability has been considered as potentially feasibly during the first design stage.

Figure 1: Artist impression of Colombo Port City

This paper elaborates further on the anticipated beach stability and the adopted engineering management cycle to arrive at a practical beach design. The emphasis of the design is on integration of functional and safety requirements. It is the responsibility of the developer to combine these aspects within an integrated design, which favors the development and the people of Colombo. Due to the large number of uncertainties and functional requirements of the beach, the engineering cycle is a highly iterative process.

The beach design has not yet been finalized and is still continuously subject to progressive insights from increasing data and information on prevailing hydraulic conditions. The complexity of the design of the convex-shaped beach demands the use of state-of-the-art models and latest scientific view-points and inputs. Therefore, this paper is the launching paper regarding the CPC beach stability, providing a baseline for the design, and will be followed up by further paper(s) at a later stage of the design and construction.

2. Engineering design process

2.1 Requirements

One of the most recent designs of the CPC development can be seen in Figure 2 with the beach indicated in yellow. The beach area needs to fulfil a number of functions:

- The beach is a major landscaping element in the spatial city environment that connects the water and urban environment.
- The beach is part of the land revetment, minimizing wave overtopping and forming the foundation for the retaining wall / staircase as land revetment.
- The beach will facilitate leisure activities, such as: strolling, sun-bathing, swimming, boating and wind-surfing.
- The beach enhances an attractive water front development (e.g. restaurants, beach apartments, sites with attractions), and can be equipped with an attractive boardwalk.

The convex-shaped beach layout as currently designed, however, is also a highly challenging element as regards the vulnerability for sand losses in the anticipated exposed situation with waves coming from one predominant direction. Major requirements, connected to the above functions are:

- The beach should sufficiently protect the land revetment.
- The beach should facilitate leisure activities at minimum risk.
- The beach in itself should be sufficiently maintainable to fulfil its safety and recreational functions at acceptable costs and efforts.

Protective function

The beach should maintain a minimum width and volume to fulfil its function as protection of the land revetment to prevent overtopping of the revetment crest or flooding of the reclaimed land in case of extreme events.

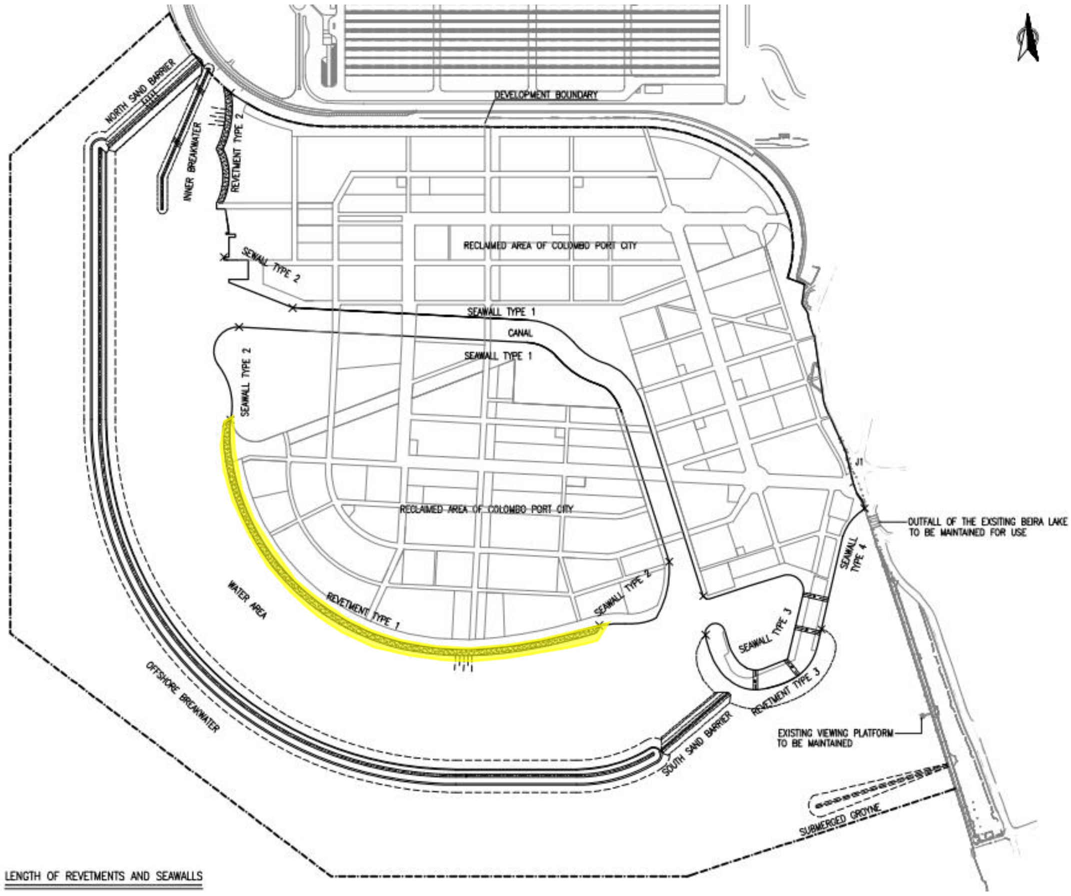

Figure 2: Layout design Colombo Port City with main beach indicated in yellow

Risk for people

People that are near or in the water at the beach zone should be safe at all times, as far as they cannot be held responsible for their own safety. A gentle and relative flat beach profile, to be maintained over a sufficient area, will reduce the danger considerably. This also increases the safety for swimmers.

Maintenance

The fully unprotected beach will experience cross-shore and long-shore adaptation of the beach profile in order to naturally adjust to the prevailing hydraulic conditions. The degree of protection by the offshore breakwater and potential additional beach stabilizing measures will determine the remaining morphological activity. Hence, the sand volume fluxes should be considered carefully in relation to the sand volume available within the beach area, to arrive at a feasible and acceptable maintenance.

2.2 Integrated design

Keeping the functional requirements as indicated above in mind, the risk can mainly be expressed as function of the anticipated morphological changes, governed by the wave and current action in the lagoon. A potentially feasible and flexible solution to improve the stability of the beach, if required, is by means of a groyne system. For an optimized design of the arrangement of the groyne system, the morpho dynamics of the beach has to be known in detail. However, accurate prediction of the morphological changes in the design stage is highly challenging because of:

- Uncertainties in the wave transmission / overtopping parameters of the offshore breakwater.
- Uncertainties in the 3D hydraulic effects within the lagoon.
- Uncertainties in the properties of the beach sand.
- Lack of information and data on the beach system, or similar systems.

For these reasons 2D and 3D physical modelling as well as numerical wave modelling studies are performed to obtain more insight in the erosive and wave propagation processes. This paper only focusses on the numerical study on the sand stability, in a later stage results of the numerical and physical modelling study will be combined to obtain a clearer and broader picture of the occurring processes.

As a consequence, a flexible and dynamic design approach is applied with ‘no regret’ measures where required and flexibility where allowed. A basic idea is to let nature shape the beach first in the different seasons during the construction period to obtain more (reliable) knowledge. Subsequently, in order to meet the functional requirements best, an optimum design can be obtained with the least amount of stabilizing measures. This approach may be maintained until progressive insight proves otherwise.

2.2 Uncertainties

Although a groyne field will significantly improve the stability of the beach, uncertainties in spatial distribution of the erosion and sedimentation patterns require intensified monitoring, even when such a groyne field has been designed properly. The maintenance requirements of the beach are dependent on the spatially and temporal distributed sand loss volumes in time. Accurate assessment of these losses will allow for an optimum beach maintenance program. Improving insight on the behaviour of the beach system during early stages of construction will accelerate the iterative design process. The iterative design process is characterized by the continuous decrease of uncertainties due to the increasing amount of data and information on the morphology which will be obtained through (amongst others) surveys. The best results can therefore be met by ‘learning by doing’ through observation, monitoring and (re-)calibration of modelling, which is characteristic for such a unique design.

At this point in time, the sand body of already reclaimed land is fully exposed to the incoming wave and current climate (see Figures 3 and 4). The offshore breakwater has not yet been constructed. The morphological activity around the reclaimed land is monitored regularly. With the known information (ref [1]) on the behaviour of the pre-construction conditions and situation (Figure 5), the changes in the morpho dynamic system can be identified, which in turn will improve the knowledge base.

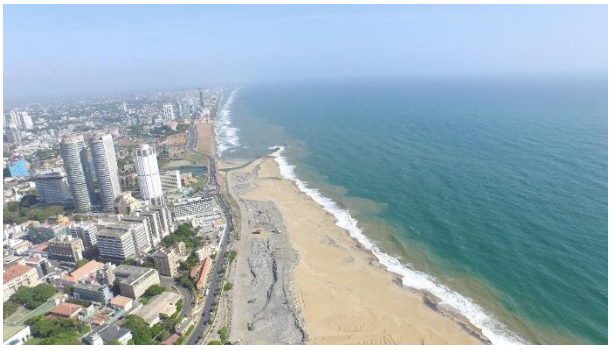

Figure 3: Sediment transport around the CPC development (photo September 2015)

Figure 4: Development of the beach profile of the newly reclaimed land (photo April 2015)

Figure 5: Base scenario along Galle Face

3. Beach design

The design of the beach will be obtained through the following steps:

1. Hydraulic data analysis and determination of governing hydraulic conditions (not part of this paper);
2. Beach stability analysis for the unprotected (= without groyne field) beach;
3. Design recommendations for the beach area.

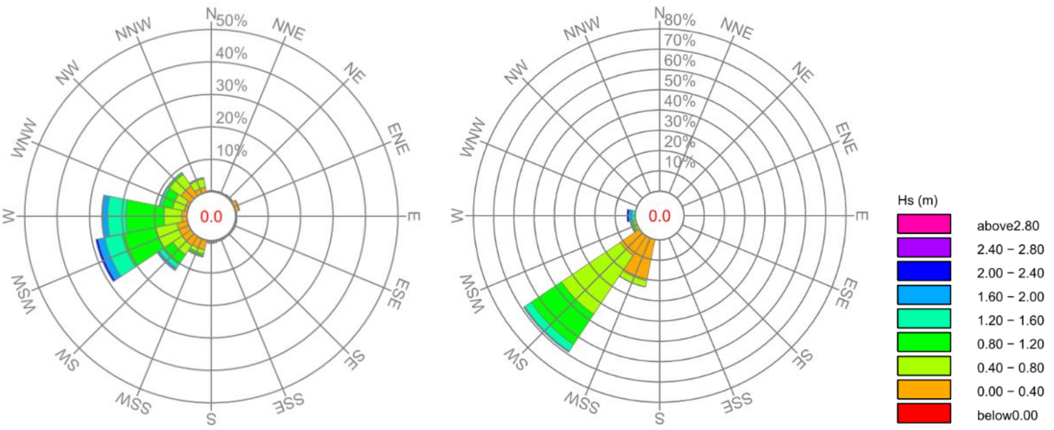

Figure 6: Average annual wave climate offshore the CPC development. Left: sea waves. Right: swell waves

3.1 Hydraulic conditions

Sediment transport along the convex shaped beach layout is mainly governed by the wave climate inside the lagoon. The hydraulic conditions inside the lagoon are obtained through numerical model simulations using the input of wave and wind statistics and 2D physical model test results, providing reliable data on wave overtopping and transmission of the offshore breakwater. The wave climate is a translation of the offshore wave climate disturbed by:

i) wave penetration through the northern and southern openings;
ii) wave transmission through the offshore breakwater;
iii) wave overtopping over the offshore breakwater and;
iv) locally generated waves inside the lagoon due to local winds.

The tidal currents (0.02 – 0.05 m/s), wind-driven currents (up to 0.2 m/s for normal conditions) and wave-driven currents (up to 0.2 m/s) are relative limited in the breaker zone of the beach area. Moreover, the currents will also be strongly varying over time and will have opposing directions. In spite of the moderate currents, practice shows that they may play a significant role in the transport of sediment (which is stirred up mainly by the wave action and transported along the beach by the currents). Due to the large timely and spatial variation of the currents and the moderate intensities, accurate assessment of the currents and their effect on the sediment transport is not practically attainable. Hence, this should be left to practice and monitoring after construction of the beach.

Figure 6 shows the governing sea wave conditions (left) and swell wave conditions (right). Sea waves are dominating from the WSW and W directions whereas swell waves are dominating from SW and show a more narrow sector of incidence. The governing overall direction of both sea waves and swell waves is within the sector W to SSW. The highest occurring waves also come from these directions. In addition, the NE monsoon from about October till April induces waves from NW direction with (generally) lower waves.

Upon reaching the offshore breakwater, the incoming wave energy will partly be absorbed, partly be reflected and partly be transmitted. The transmitted wave energy will subsequently propagate through the lagoon and reach the beach and the waves will have a different character there: the wave energy will be redistributed, resulting e.g. in a change in significant wave height. The quantification of the wave redistribution in the lagoon and actual wave transmission was a major subject of the 2D physical model tests (Figure 8). In addition to the wave energy deformation, the wave direction will change. For the predominant SW waves, the offshore breakwater will act as a “magnifying glass” and will cause the wave energy to converge, see Figure 7. Such a shift in wave angle has also been identified in literature studies (ref. [2])

To assess the wave climate in the lagoon in a 3D environment, the numerical wave model SWAN has been applied. This wave model enables to compute the wave energy through the openings into the lagoon with sufficient accuracy and uses the wave transmission relations through the breakwater as measured in the 2D physical model tests. Wave energy through the openings was also computed by a more accurate time-domain model, however such model cannot compute wave

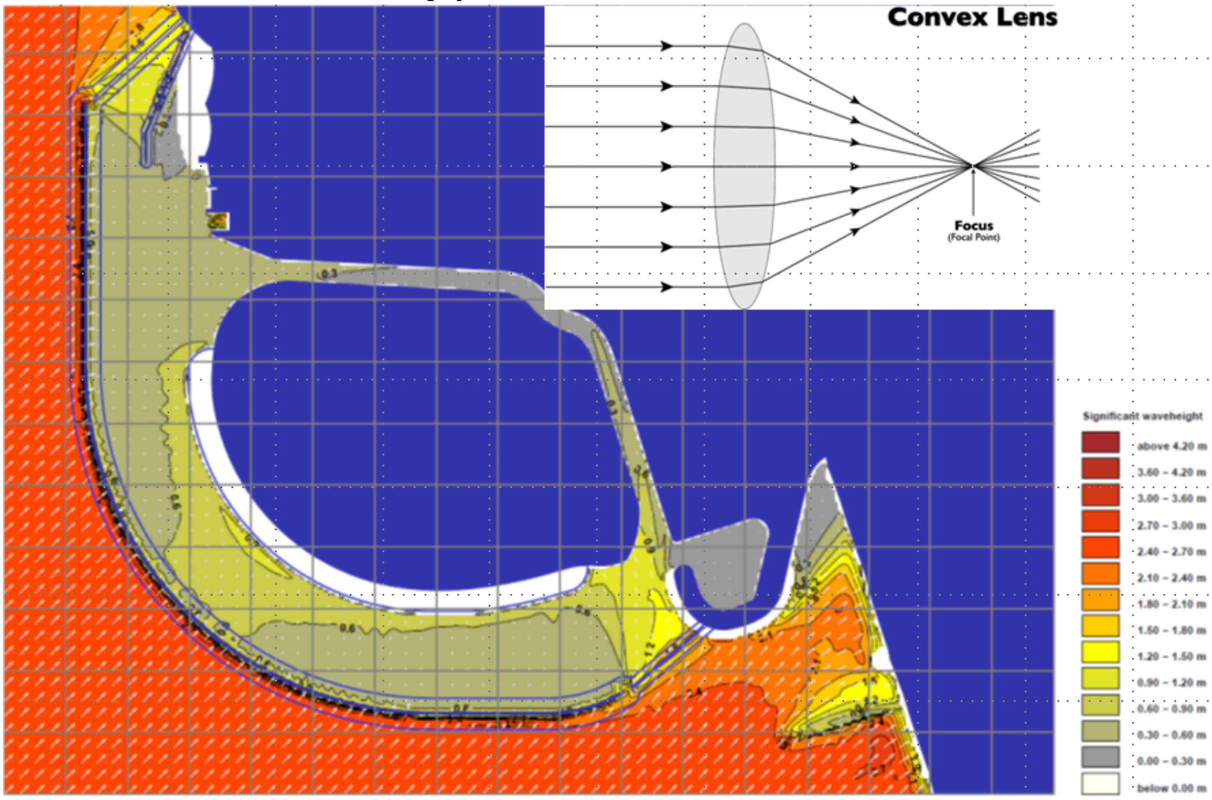

Figure 7: Model results of 'magnifying glass'principle of waves that overtop the Offshore breakwater.

transmission and computational times are very high. Although the spatial variation of the wave penetration was different between both models, the actual wave energy was very similar and for this reason the SWAN model provides sufficient accuracy. The computed change of wave direction follow the available theories in general and is in line with the theoretical expectations (Figure 7). Using this numerical wave model, the full offshore time series have been translated to a number of locations (12) along the beach profile within the lagoon area, which are further used in the beach stability analyses. The uncertainty within this modelling approach are known and considered in further analysis of the results, including sensitivity analysis of the morphological model,

From the 2D physical model tests the wave transmission – crest height correlation functions of the breakwater have been determined for the whole range of wave heights and periods (see Figure 9). These correlation functions have been used as main input parameters in the numerical modelling to determine the wave climate inside the lagoon area. For higher water levels (or lower relative crest levels) the wave transmission through the breakwater increases significantly, the future situation with respect to sea level rise should thus be taken into consideration. A high variability of the water level (e.g. by seasonal influence and uncertainties in sea level rise) makes it complex to accurately and reliably predict the wave climate. By taking different wave climates and different crest heights (i.e. water levels), uncertainties can be reduced as well as the sensitivity for the height of the breakwater and climate change effect on the water level can be accounted for.

In addition to the above, the following interesting and informative outcomes can be summarized from these wave transmission results:

- Measured wave transmission coefficients go up to 0.3 for high wave conditions (low relative crest height);
- Higher water levels (or lower relative crest heights) cause significant higher wave transmission, which will negatively affect wave disturbance and beach stability for future situations (including sea level rise or seasonal variations);

- For a relative high crest level the transmission coefficient is limited, but not negligible. This means that even for lower wave heights, a portion of the wave energy will constantly penetrate into the Lagoon area, always giving some waves at the beach;

Figure 8: Set up of 2D physical model test to determine the characteristics of the offshore breakwater

For longer wave periods (wave steepness = 0.005) the transmission is higher. This means that a minimum of approximately 20% of the swell wave height penetrates into the Lagoon. For sea waves this minimum is in between 5 to 10%.

3.2 Uncertainties

Particle size distribution

The exact sand characteristics of the available sand for the beach fill are yet uncertain. Also due to spatial variation in the borrow area and during and after construction of the beach, the particle size distribution along the beach may vary. Sieve curves from the currently constructed area of the CPC Project showed particle sizes ranging from 600 µm to 800 µm. To be on the safe side, a homogeneous particle size distribution with a mean particle diameter of 600 µm has been assumed for the stability analysis. To determine a band-width for the expected sediment transport, 300 µm and 900 µm particle size are investigated as well. In numerical modelling, the roughness factor to be applied normally is a calibration factor. As no calibration data is available here, based on similar studies this parameter is taken as k = 5 m, with a lower and upper limit of k = 2.5 m and k = 10 m.

Hydraulic Conditions

The beach stability analysis is performed for yearly average conditions. The impact of extreme storm conditions is not considered yet and should be included in a final design phase. Based on the computed annual average wave climate, a conservative and less conservative wave climate has been applied in the sediment transport computations.

Also the incoming wave direction at the beach has uncertainty as the change of wave direction in the numerical model cannot be affected. Although the results look similar to the available theories (ref. [2]), the local wave directions cannot be considered highly accurate. For this reason also the effect of the incoming wave direction on the beach stability is investigated by a sensitivity analysis.

Cross Shore Profile

In the sediment transport computation a constant beach slope of 1:20 is applied. This slope is considered stable according to generally accepted empirical slope stability formulae (refs 0, 0 and 0). Steeper slopes can be applied at depths beyond the depth of closure (refs. 0 and 0)where sediment in not influenced by waves and currents.

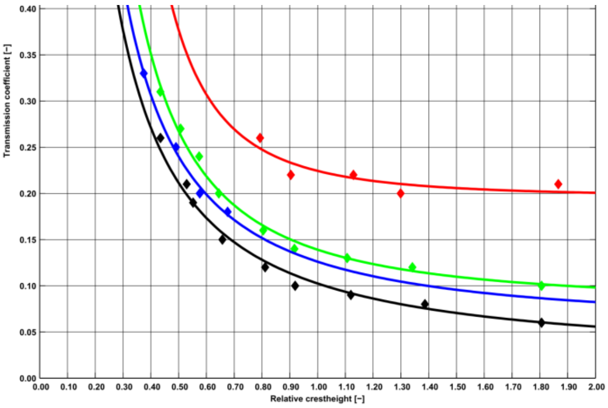

Figure 9: Transmission coefficient a function of the relative crest height as measured in the 2D physical model tests

3.3Beach stability

Wave-induced cross-shore redistribution of sediment will tend to shape the beach profile towards its own 'natural equilibrium'. Although the cross-shore beach stability may be important for safety reasons, and the cross-shore profile is relative stable, this paper focuses on long shore sediment transport, since the gradients of such transport are expected to be dominant for beach erosion and hence beach stability. The net sediment transport along the sand beach is computed while taking uncertainties in bathymetry, particle size distribution and wave climate into account. In the first design phase, an initial assessment is compared to the numerical model results.

The beach stability is done for an unprotected beach, viz. no additional beach stabilizing measures in addition to the offshore breakwater. Structural beach erosion is generated by long shore sediment transport gradients which require quantification to determine potential beach instability. Numerical modelling of long shore sediment transport and coastline evolution is carried out by using the LITPACK module of the MIKE21 software package from DHI (ref. 0). LITPACK is suitable for quasi-uniform (relative straight) coastlines or beaches. As the CPC beach-layout is in a strongly curved convex shape, the model results towards the edges of the beach (where curvature is strongest) should be considered cautiously. The model results are compared with generally accepted long shore sediment transport formulas as CERC (ref. 0) and Kamphuis (ref. 0). Model results are qualitatively in good comparison with the transport formulas. Sediment transport rates have been computed at 12 locations along the convex shaped beach layout.

Based on the transport gradient, the qualitative results of the sediment transport computations are shown in Figure 10 for the unprotected beach. The following conclusions are obtained:

- Points 578 and 584 are close to their own equilibrium (best estimate transport around 0), For the other points the order of magnitude does not change significantly for varying incoming wave directions;
- Along the west beach, a significant N directed sediment transport gradient is present (points 576 and 577). Depending on the detailed location of the beach end in the west, sediment transport toward the channel might occur. The channels are considered as sediment traps which may suffer from siltation if left unprotected;
- The overall computed sediment transport over the beach area is SE to E directed, transporting sand towards the SE canal/marine entrance/exit. This will potentially lead to structural erosion of the S beach and will cause siltation of the channel;
- The design orientation of the coastline at the W beach seems to be far from the equilibrium coastline orientation, which will lead on the long term to a significant reorientation of the W beach coastline, the extend of this effect also depends on the design of the transition between the beach and the canal boundaries;
- The design orientation of the coastline at the South beach is closer to its equilibrium orientation, which may lead on the long term to a minor reorientation of the S beach coastline;
- In terms of magnitude, sediment transport rates [m^3/yr] at the S beach are an order of magnitude smaller than the transport capacities of the W beach.

An example of the generated output at location 584 of a LITDRIFT (annual sediment drift) computation is presented in Figure 11. This location was selected as there was a clear sediment transport in both directions here, despite that the net transport is relative limited. This means that even when the net transport is relative low, it does not mean the total sand transport along the beach is low at all times. Especially near the canal entrances this may induce sand losses instead of sand being able to be transported back and forth in both directions along the coast.

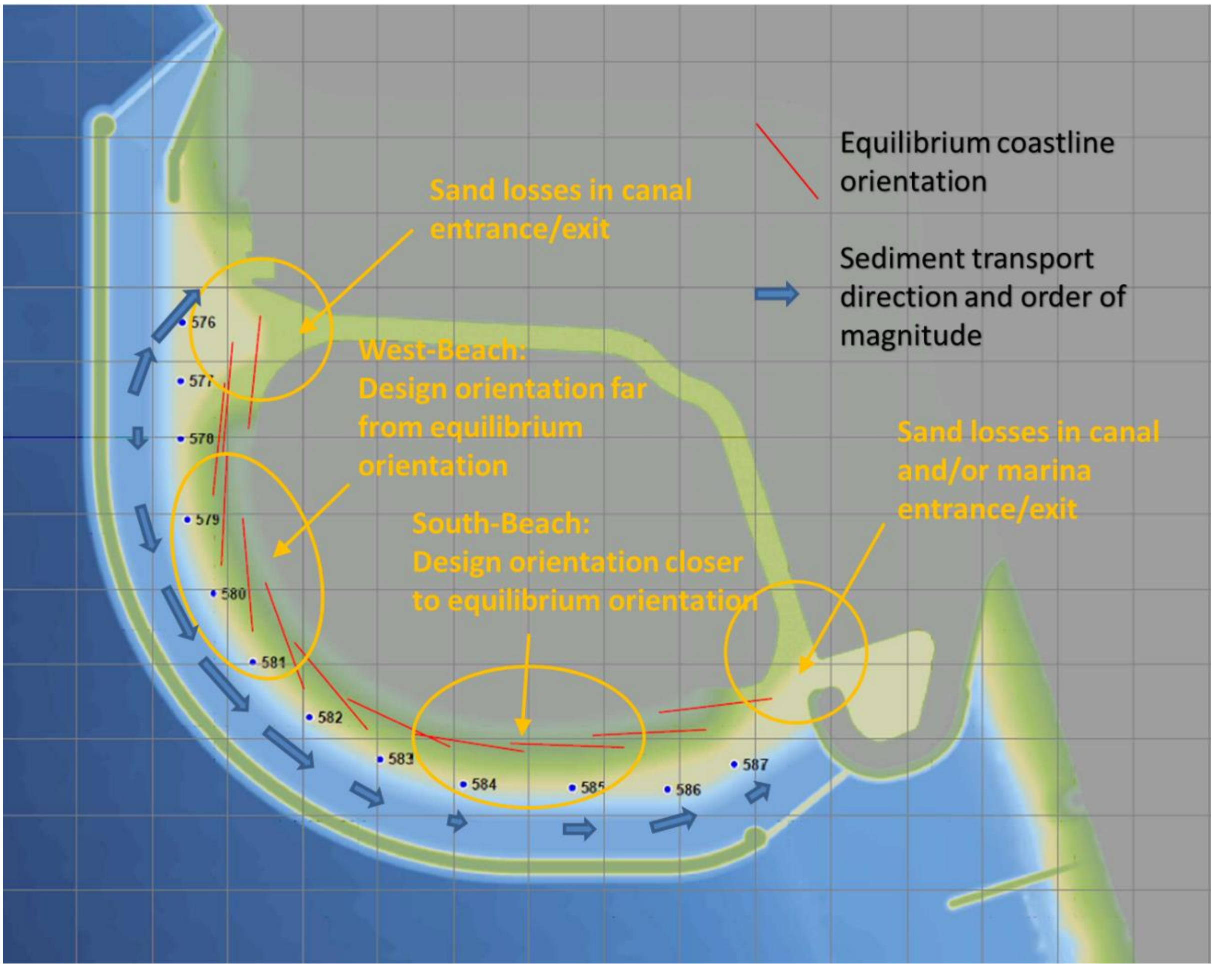

Figure 10: Qualitative transport directions and equilibrium orientation per output location

Results from the sensitivity analysis have shown the large impact of small changes in input parameters and lead to the following conclusions:

- The incoming wave height has a very large influence on the occurring sediment drift. This reveals that the actual magnitude of sediment transport is very complex to predict accurately and can strongly vary throughout the years. Also, sediment transport may increase further in the future due to sea level rise;
- The bed roughness has a pronounced influence on the occurring sediment drift. This confirms the uncertainty of sediment transport modelling, hence interpretation of computed values should be done with care;
- The grain size is important for the actual occurring sediment transport. If a larger grain size is used, the sediment drift can decrease by a factor 2. For this reason it is important to apply suitable larger grain sizes for construction of the beach;
- Overall, there is a large bandwidth in the actual sediment transport; the expected erosion volumes or meters coastline retreat thus indicate an uncertainty with factor 3 to 5. It is however clear that significant transport will be occurring, which will probably require beach stabilization measures if maintenance has to remain at practical levels.

Taking the model outputs into account, erosion rates in the range of 4-10 meters per year might occur. Without beach stabilization measures the required maintenance is in the order of 20,000 – 40,000 m^3/year. Averaged over time, erosion is expected at both the northern and eastern end and accretion in between locations 581 and 584 as shown in Figure 12. Temporary variations, e.g. due to storms, may change this pattern of erosion or accretion

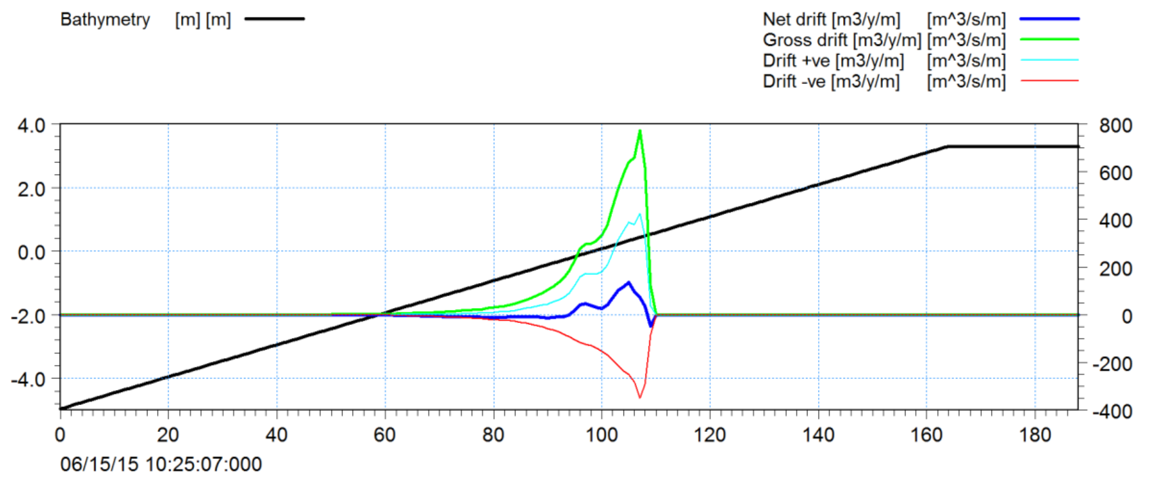

Figure 11: Computed net and gross transport along the south stretch of the beach profile

3.4 Design

Based on the preliminary model outcomes, we think it is highly advisable to install coastline stabilizing measures in order to reduce the long shore sediment transport capacity. As a feasible type of measures this stability can be improved by the construction of a well-tailored groyne field. The groyne field can be adjusted in such a way that it regulates or blocks the sediment drift. Per beach section, the groynes may be different. For adequate detailed design, more information on the hydrodynamic and morphologic system is required. Therefore, depending on the overall planning of the project, monitoring of the sediment transport processes and hydraulic regimes will lead to progressive insight into the expected beach stability. Moreover, with this information, the simulation models can be (re-)calibrated and updated. As an example: already during the construction of the offshore breakwater, the transmission parameters as used in the numerical model can be verified.

Depending on the design, adaptation of the cross-sectional beach profile towards a dynamic equilibrium profile will mostly take place in a relatively short period after completion of the beach. To take this adaptation into account, it is common practise to use a ‘buffer’ quantity of sand in the higher part of the beach, which may move downwards due to the initial changes or erode, ensuring that the minimum required width of the beach will remain over a certain (predetermined) period.

As a general approach, the best evidence for the performance (and thus for the design) of a groyne scheme may be sought in observation of similar groyne schemes in similar environments elsewhere. However, here it should be noted that this strongly convex-shaped beach design is unusual, so no similar artificial beach designs are present to use as reference. Therefore, all design considerations have to be based on the investigations carried out (physical and mathematical) and on experience. The accuracy of the numerical morphological modelling is a reflection of the accuracy of the input parameters, hence a large bandwidth in modelling results occurs. Results should thus be interpreted with care and require to be updated with progressive insight during design and construction. This is an important aspect of the engineering management process described in this paper. For this reason a flexible, costs efficient but effective design is proposed with the groyne field design.

The groyne scheme as designed based on the model calculations, consists of at 2 terminal groynes near the N and E beach boundary to prevent sediment losses into the canal. For the time being we foresee 8 intermediate relatively short groynes. The terminal groynes could be installed as rubble mound structure since they are of permanent character. For the intermediate groynes, impermeable wooden structures may be more suitable. The recommended preliminary layout design is presented in figure 14. The reason for choosing for the relatively short intermediate groynes is to remain as close as possible to the natural system. Advantage is e.g. that strong saw-tooth effects will be mitigated, as well as rip currents which are dangerous for bathing people. Further detailing of the beach stabilizing methods will follow from progressive insight. The groynes as indicated here, are considered no-regret and can be extended or adapted in future upon the increase of progressive knowledge, e.g. by monitoring and update simulation modelling. The detailed design should maximally allow for such extension or adaptation.

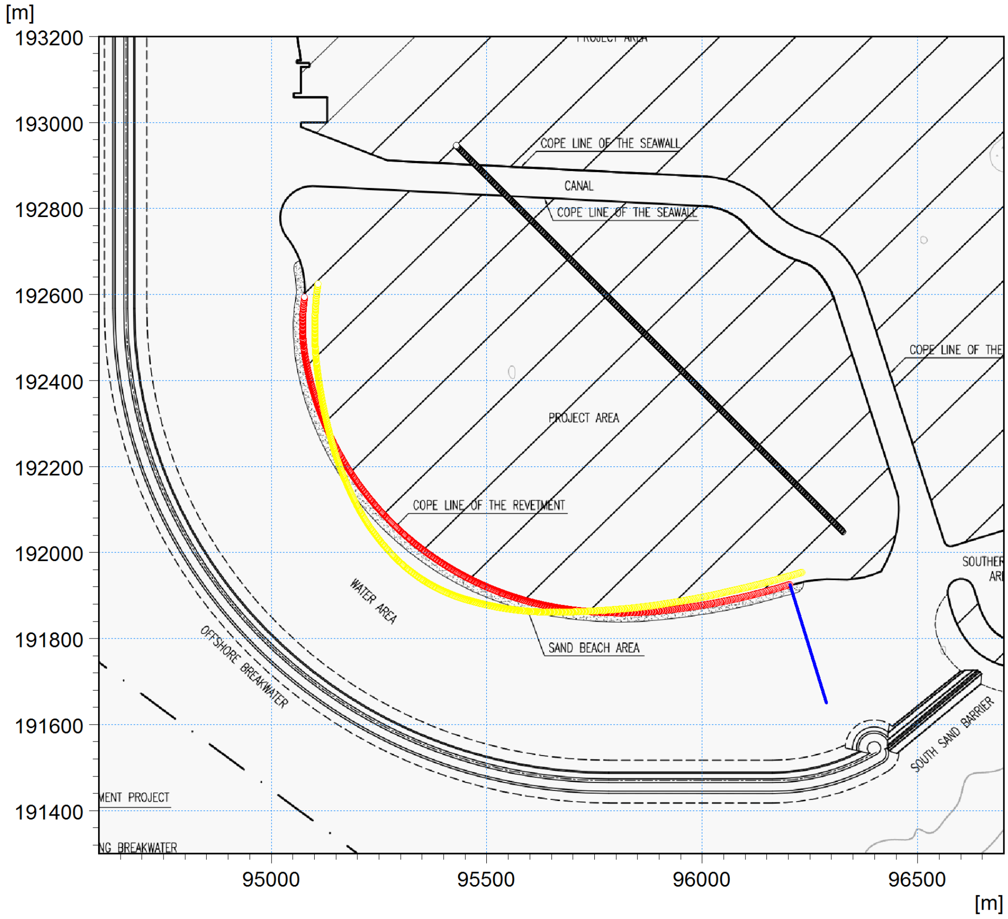

Figure 12: Model result: computed coastline evolution over 20 years (yellow line)

4. Conclusions and recommendations

4.1 Conclusions

The present paper illustrates the complex and iterative design process for the convex-shaped Colombo Port City (CPC) beach layout. To solve the complex problems involved, the sediment transport distribution along the beach has been determined for annual average wave conditions based on a 20 year time series. The hydraulic boundary conditions within the lagoon were determined by making use of the numerical wave model SWAN, in which wave transmission outcomes from 2D physical model testing have been used as input to determine the wave heights inside the lagoon area. Despite the use of state-of-the-art physical and numerical models, uncertainties remain. The model results are subject to uncertainties due to e.g. wave climate, offshore breakwater characteristics, beach profiles and sand properties.

We think that the largest uncertainty from this wave study is the prediction of the wave climate inside the lagoon, for which no tailor made combination of wave theories (transmission, reflection, diffraction, penetration) and calibration data are available. The complex layout of the development and the modelling assumptions lead to uncertainties in the output of the numerical wave models applied. Also for the main beach section, the input parameters for sediment transport computations (bed roughness, particle diameter) lead to uncertainties in the output results. Therefore, a sensitivity analysis has been carried out to determine the range of uncertainties in the computed sediment transport rates. In the sensitivity analysis the individual influence of i) particle size ii) bed roughness iii) wave climate and iv) coastline orientation on the sediment transport rates has been determined.

The modelling results indicate that without any beach stabilizing measures, the present convex beach layout could significantly erode due to gradients in the long shore sediment drift. The required maintenance is estimated to be in the order of 20,000 – 40,000 m^3/yr, however due to the uncertainties this estimation may vary up to a factor 3. In the SW area (middle area of the beach), the beach sand will start to accumulate. The source of this sand responsible for the accumulation are the N to S stretch in the W part of the beach and, to a lesser extent, the E to W stretch in the S part of the beach. Furthermore, sand may well be transported towards the canal entrances / exits near the end of the beaches and thus be 'lost'. The W and S parts of the beach are therefore subjected to structural erosion. The erosion rates at the N and S stretches of the W part of the beach may initially be around 10m per year and at the E to W stretches of the S part of the beach approximately 3m per year.

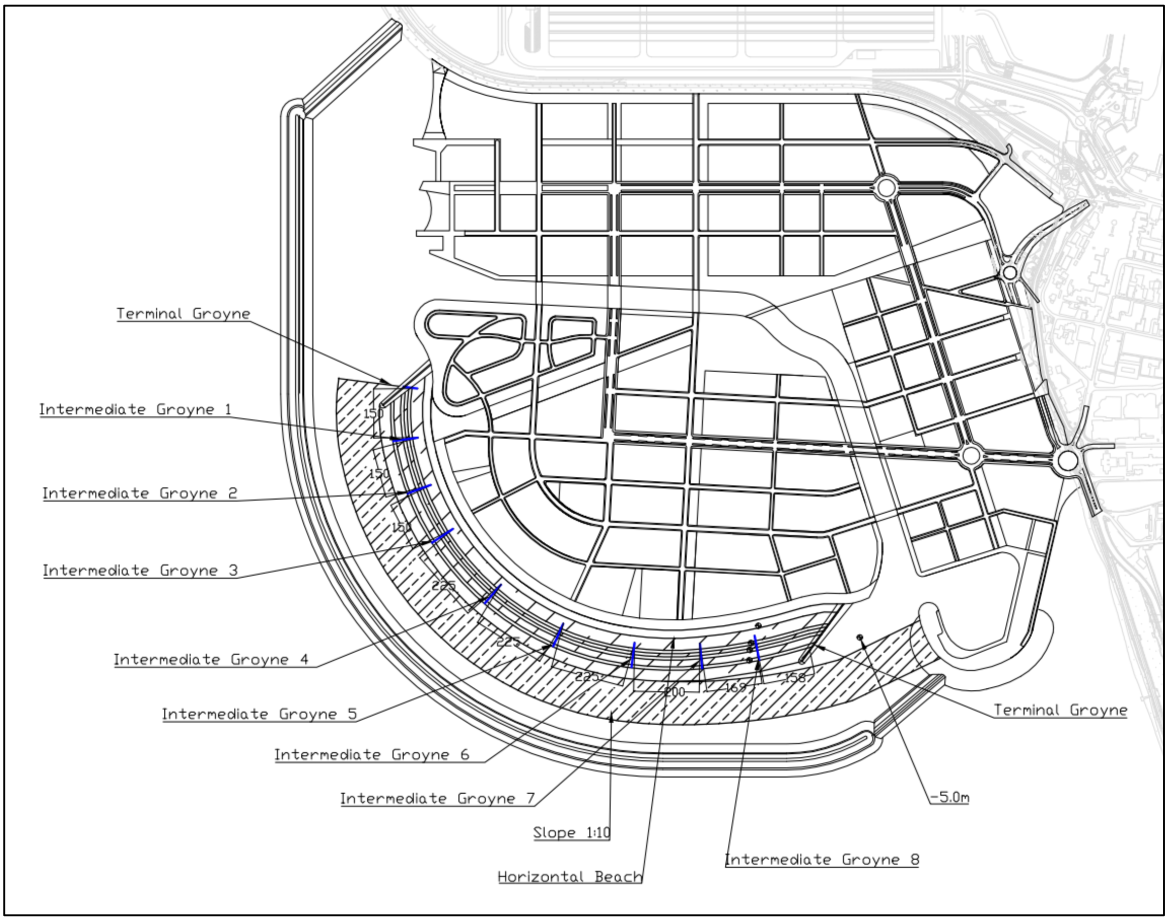

Figure 13: Layout groyne field with 2 terminal groynes and 8 intermediate groynes

The most NW tip of the main beach appears to be relative well sheltered by a protruding piece of land N of the main beach. This area is therefore less subjected to northerly directed sediment transport into the canal entrance. This is in contrast to the most SE tip of the main beach where there is a nett E directed sediment transport. On the long run this will results in erosion of the S main beach and accumulation of sand in the S Canal Entrance. This effect will be less in the N, but over time sand may bypass the land protrusion and be transported into the canal entrance there. Moreover, the overall sediment transport picture as shown in the above is an averaged picture. Individual periods (seasonality, storms) may give other distributions, so the sediment transport at the extremities will have to be arrested anyhow.

4.2 Recommendations

The sediment transport analysis along the beach of the CPC development indicates that at some locations significant erosion of the beach is likely to happen and that at other locations sedimentation may occur. If this would really occur and required maintenance is considered too high, the situation can be mitigated by a properly designed groyne system. At the other hand, we think that the uncertainties in the predicted erosion rates do not yet justify to propose a final solution at this stage of the project.

Instead, we propose to maximally utilize progressive insight on the breakwater characteristics, wave climate and particle size in next iterative design rounds. We therefore also recommend to start a monitoring program on hydraulics and the morphological behaviour of the beach and volumes of local erosion and accretion during construction. This may further indicate the necessity of the (interstitial) groynes. If such a groyne scheme is strongly indicated then, the monitoring results and subsequent increased knowledge on the local morphological behaviour of the beach can be used for the final design of the (initial) groyne field.

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Experimental Investigation of Performance of Reef Breakwaters

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Abstract: Reef breakwater is a low crested, rubble mound breakwater without a conventional multilayered cross section. It has been identified that transmission coefficient is one of the main parameters to quantify the performance of reef breakwaters and several parameters that influence transmission coefficient was identified. Accordingly, a comprehensive laboratory investigation was carried out and transmission of different reef breakwaters was studied by varying different influence parameters. It was observed that wave steepness, crest width and depth of crest submergence are the most influential parameters on transmission coefficient. Comparison between existing equations to calculate transmission coefficient was done using data from the present study. When using existing equations for the same input parameters, it can be seen that estimated transmission coefficient values differ from each other suggesting that their applicability to a real life problem is questionable. Therefore a new improved formula to estimate transmission coefficient was derived using dimensional analysis incorporating more influence parameters than existing equations. This formulation proved to be better than the previous equations.

Keywords: Physical model tests, Reef breakwaters, Transmission coefficient.

1. Introduction

Reef type breakwaters refer to a law crested rubble mound breakwater without the traditional multilayered cross section (see figure 1). These types of breakwaters are little more than a homogenous pile of stones with individual stone weight sufficient to resist wave attack. Because of the low crest height and using one armour layer these kinds of structures are relatively cheaper compared to conventional rubble mound structures.

These kinds of structures are mainly used at places where partial attenuation of the waves on the lee side of the structure is needed. Other purposes of reef breakwaters are,

- Protecting a beach or reduce the cost of beach maintenance
- Protecting the water intake for power plants
- Providing an alternative to revetments for stabilizing an eroding sea line

Reef breakwaters can be either submerged or partially submerged.

The idea of reef breakwaters first came up around the year 1976. Since then a number of reef breakwaters have been constructed. Reef breakwaters can be seen in places like, the lower central east coast of Florida, New Jersey coast, offshore of Grand Cayman Island etc... (Donald, et.al. 2003).

However, the amount of research done on reef breakwaters, specially submerged breakwaters is limited. Also the design criteria available for reef breakwaters are not well defined. Therefore, further researching is required to specify design parameters for reef breakwaters.

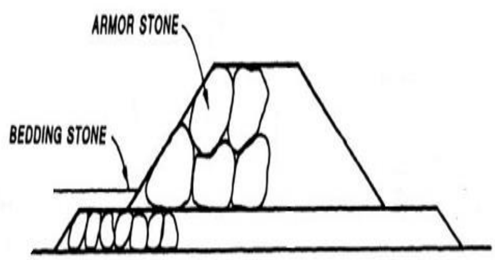

Figure 1: Typical cross section of a reef breakwater

Performance of reef breakwaters can be quantified using transmission coefficient ($K_t = H_t/H_i$), Reflection coefficient ($K_r = H_r/H_i$), Loss coefficient ($K_l = H_l/H_i$) and the stability of the breakwater where H_i is incident wave height H_t is transmitted wave height, H_r is reflected wave height and H_l is equivalent wave height corresponding to energy loss of the wave.

Among these parameters transmission coefficient was studied in this study. Figure 2 shows the most influential parameters on transmission coefficient.

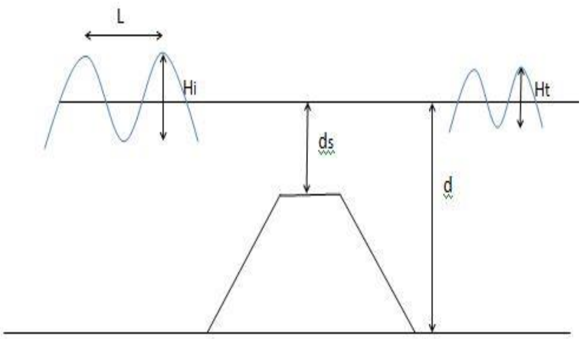

Figure 2: Influence parameters

where, d = water depth, d_s = depth of crest submergence, H_i = incident wave height and L = incident wave length.

2. Previous studies on transmission coefficient of reef breakwaters

Several research studies have been conducted and several formulae and methods to estimate transmission coefficient of reef breakwaters have been introduced in those studies. Most of these studies are based on physical model tests conducted under different conditions and others are from numerical model analysis. In this study, transmission coefficient was estimated using existing equations with present model parameters and those results were compared with model test results. These equations are from Coastal Engineering Manual (CEM) (2001) and from research studies done by Ahrens, et al. (1987), Van der Meer, et al. (2003), D'Agremond, et al. (1996).

The equation given in CEM (2001) for the calculation of transmission coefficient of reef breakwaters is a result of various model test results for rock armed law crested, submerged and reef breakwaters.

$$C_t = \left(0.031 * \frac{H_s}{D_{50}} - 0.24\right) * \frac{R_c}{D_{50}} + b \qquad (1)$$

For submerged breakwaters,

$$b = -2.6\, S_{op} - 0.05\frac{H_s}{D_{50}} + 0.85 \qquad (2)$$

where, C_t = Transmission coefficient, H_s = Significant wave height of incident waves, D_{50} = Median of nominal diameter of rocks, R_c = Free board (negative for submerged breakwaters) B = Width of the crest, S_{op} = Deep water steepness corresponding to peak period.

For reef type breakwaters transmission coefficient has been limited to maximum of 0.6 and minimum of 0.15. As mentioned earlier the equation is based on test data from several researchers under following test ranges.

$$1 < \frac{H_s}{D_{50}} < 6 \qquad 0.01 < S_{op} < 0.05 \qquad -2 < \frac{R_c}{D_{50}} < 6$$

It can be seen that influence of breakwater crest width has not been included in this equation.

The equation given by the Ahrens, et al. (1987) is given in equation 3. This equation has been given for breakwaters with relative free board $\left(\frac{F}{H_{mo}}\right)$ less than 1. The cross section area of the breakwater has been incorporated in this equation.

$$K_t = \frac{1}{1+\left(\frac{h_c}{d_s}\right)^{C_1}+\left(\frac{A_t}{d_s L_p}\right)^{C_2} .exp[C_3\left(\frac{F}{H_{mo}}\right)+C_4\left(\frac{A_t}{d_{50}^2}*\frac{1}{L_p}\right)]} \qquad (3)$$

where, C_1= 1.188, C_2= 0.261, C_3= 0.529, C_4= 0.00551, F = free board, h_c = water depth, L_p = incident wave length, H_{om} = zeroth moment incident wave height, A_t = cross section area of the reef and d_s = water depth.

Van der Meer, et al. (2003) proposed the following equation for the calculation of transition coefficient with a minimum of 0.075 and maximum of 0.8.

$$K_t = \left(-0.3\,\frac{R_c}{H_{om}} + 0.75[1-\exp(-0.5\varepsilon_{op})]\right) \qquad (4)$$

Seaward slope of the breakwater has been included in this equation with the introduction of surf similarity parameter. But the formula does not include the crest width and the armor gradation. R_c is the depth of crest submergence and this equation is valid under the range of $-1.66 < \frac{R_c}{H_{om}} < 1.66$.

D' Agremond, et al (1996) has proposed an empirical equation to calculate transmission coefficient from the experimental results using irregular wave conditions.

$$K_t = -0.4\frac{R_c}{H_{mo}} + \left(\frac{B}{H_{mo}}\right)^{-0.31}[1-\exp(-0.5\epsilon_{op}] * C \qquad (5)$$

In this equation the influence of the crest width (B) and the permeability have been incorporated with a constant C (0.64 for permeable structures and 0.84 for impermeable structures).

When using the above equations for the same input parameters, it can be seen that estimated transmission coefficient values differ from each other suggesting that their applicability to a real life problem is questionable. In addition, each of these equations to calculate transmission coefficient has limitations, constraining their applicability in problems which exceed those limitations. Therefore, to get a better understanding of wave transmission phenomena over reef breakwaters a series of physical model test runs were carried out.

3. Experimental procedure

3.1 Experimental set-up

The experiments were carried out in the wave channel in the Hydraulics Laboratory in Faculty of Engineering, University of Peradeniya, Sri Lanka. The wave channel was 12.75 m long, 0.52 m wide and 0.71 m deep. The side panels of the channel consisted of sixteen 12 mm thick Perspex sheets, which enabled visual observation such as the wave breaking and interactions of waves with models constructed in the channel.

The breakwater models were constructed using uniformly graded aggregates. Two uniform sizes of aggregates were used for the construction (D_{50}=41.5 mm and D_{50}=26 mm). For each stone class, three breakwaters were constructed with different crest width values (B) of 20 cm, 40 cm and 50 cm. Trapezoidal cross sections were used to construct the breakwater models. Each breakwater was constructed with a constant height of 25 cm and the seaward and the leeward slopes of the each and every breakwater model was maintained at a constant value of 1:1.5.

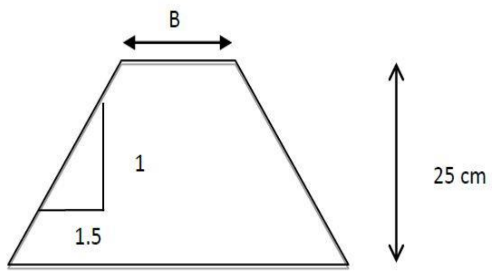

Figure 3: Cross section of a model breakwater

Figure 4: Breakwater model for 40 cm crest width using 26 mm armor units

One Armfield H40, resistant type, twin-wire wave probe was used to measure the wave parameters. Analogue to Digital converter with 16 analogue input channels was used to convert the analogue signals from the wave probes to digital signals. LAB VIEW data acquisition software was used to acquire the data from wave probes. A MATLAB program was used to obtain the required wave parameters. Figure 5 shows the experimental set-up used in this study.

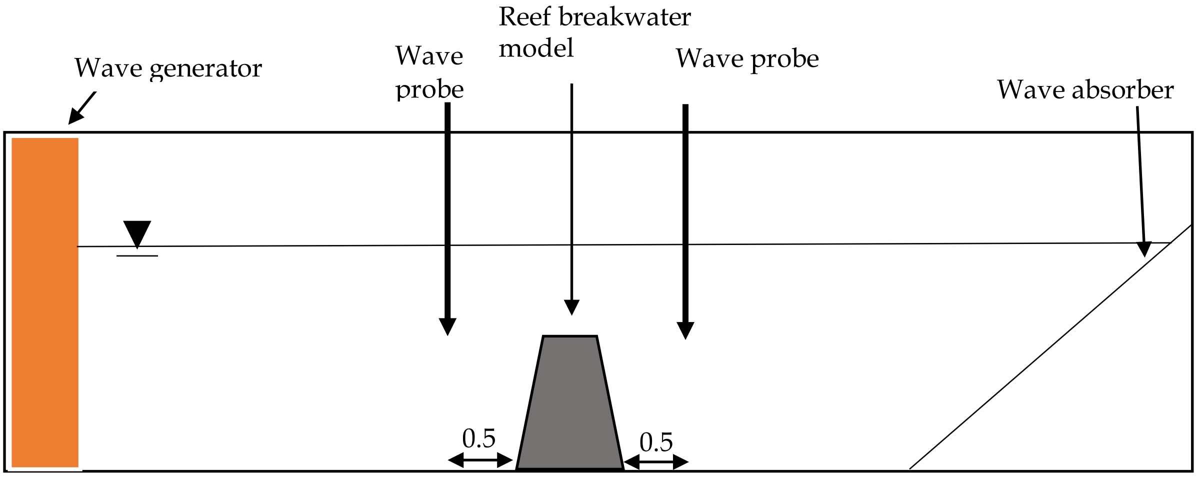

Figure 5: Experimental set-up

3.2 Model runs

Details of the experimental runs that were carried out for the varying parameters are shown in Table 1. Though 192 experimental runs were conducted, transmission wave height of some of the test runs could not be measured because wave energy transmitted was very low and transmitted wave was small. Rest of the experimental results were used in data analysis.

Table 1: Different sets of experimental runs

Variable	Values
Wave height (cm)	2 - 17
Crest width (cm)	20, 40, 50
Water depth (cm)	25, 30, 35, 40
Armour size (mm)	26, 41.5
Total no. of test runs 196	

4. Data analysis

4.1 Estimation of wave transmission coefficient using existing methods

Each of the existing formulae discussed in section 2 were used to calculate transmission coefficient inputting the data from this study and calculated transmission and measured transmission coefficients were compared. Figure 6 shows the variation of calculated and measured transmission coefficients.

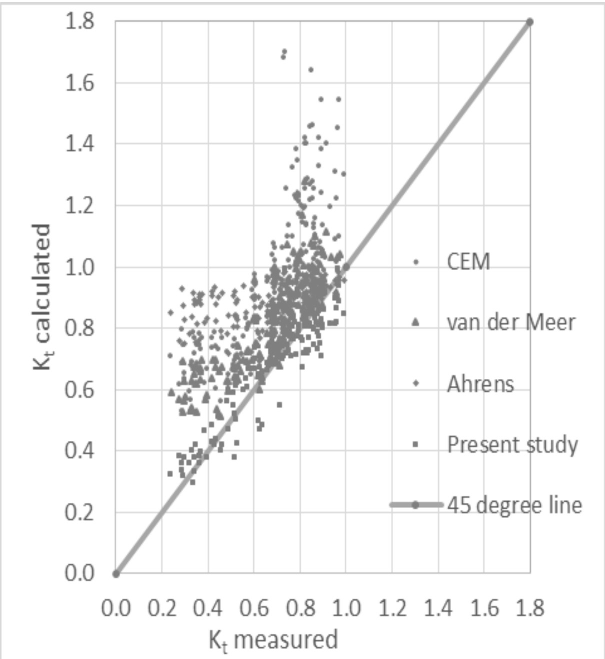

Figure 6: Comparison with previous studies

From the analysis it can be seen that data points are highly scattered. In addition there are some cases where estimated transmission coefficient is greater than one which is not acceptable. This seems there are major drawbacks and limitation in existing equations. Therefore a new formula was derived using an empirical method.

4.2 Development of a new formula to predict transmission coefficient

Dimensional analysis was carried out to formulate the dimensionless parameters which influence the transmission coefficient. Selection of the parameters affecting the transmission coefficient was done with the help of coastal engineering concepts and past studies. Following parameters were selected as the governing influence parameters associated with the performance of reef breakwaters considering the transmission of waves.

$$K_t = f(H_s, d_s, d, B, D_{50}, T, tan\alpha, g)$$

Where, d_s is the depth of crest submergence and $tan\ \alpha$ is the seaward slope of the breakwater. In the present study seaward slope was kept constant for every model test as 1:1.5. Therefore dimensionless parameter representing the seaward slope was neglected.

Using the dimensional analysis, following relationship was derived.

$$K_t = f\left(\frac{H_s}{d}, \frac{H_s}{d_s}, \frac{H_s}{B}, \frac{H_s}{D_{50}}, \frac{gT^2}{H_s}, tan\alpha\right)$$

Multi variable regression analysis was carried out to obtain the relationship between these dimensionless groups and the transmission coefficient. The derived formula is shown in equation 6 below.

$$K_t = 0.613 - 1.83\left(\frac{H_i}{d}\right)^2 + 0.02\left(\frac{H_i}{B}\right)^{-1} + 0.317\left(\frac{d_s}{H_s}\right)^{0.35} + 0.002\left(\frac{H_i}{D_{50}}\right)^2 + 5.32 * 10^{-7} * \left(\frac{gT^2}{H_i}\right)^2 \quad (6)$$

After obtaining the formula, K_t values calculated using it were plotted against the K_t values obtained from the experimental runs. Figure 7 shows those results. Compared to results from the existing equations, number of scattered data points are less and mean relative error between calculated transmission coefficient and measured transmission coefficient is less than 10%.

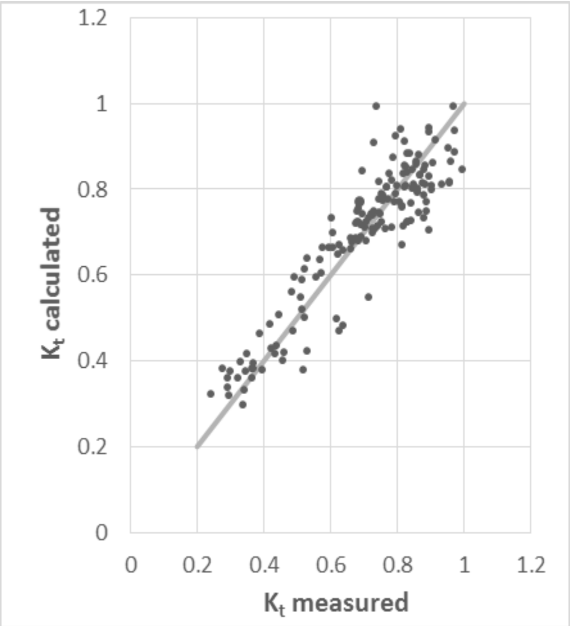

Figure 7: K_t measured Vs K_t calculated

5. Conclusion

The present formula derived for estimation of transmission coefficient contains more variables than the existing formulas. Existing equations are poor in estimating transmission coefficient when the depth of crest submergence is zero. But with equation derived from this study above shortcoming was eliminated.

Although the present study has arrived at the above conclusions, following limitations could be pointed out.

- Present study was done using regular wave conditions.
- Seaward slope of the model breakwaters was maintained at a constant value of 1:1.5 in the present study. Therefore validation of the obtained formula should be done if it is to be used for other seaward slope values.
- Wave periods used for the tests vary from 0.7 to 1.25 seconds. For higher wave periods, the present equation should be validated.
- Only two armor sizes were used for the construction of the breakwaters.

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Operational Behaviour of Hydraulic Structures in Irrigation Canals in Sri Lanka

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Abstract: The Hydraulic performance of structures provided for regulation in irrigation canals is influenced by many parameters such as type of flow control, type and frequency of adjustments, and topographical features of sub-systems that they are installed. The irrigation systems in Sri Lanka mostly designed for up-stream discharge control operation, with manually/mechanically adjusted gated structures, under steady state flow conditions. Further, topographical features of many gravity irrigation systems are varying at sub-system level due to presence of single and double bank reaches, inclusion of in-lined storage tanks. However, irrigation systems are operated under varying flow regime due to scheduled or unscheduled flows in water delivery. The adjustments of gates in structures in such situations are done by manually/mechanically by operators at different frequencies.
Therefore these situations are analyzed by hydraulic simulations using SIC hydraulic model for three different topographical sub-systems under different frequency of adjustments. The actual performance in water delivery is evaluated by effective volume of water delivered, timeliness in water delivery, at the final delivery location while maintaining on-line water adequacy through delivery points along the canal. The results will provide guidance for operation of regulation structures for effective conveyance of water under varying flow conditions.

Keywords: Canal Irrigation, Canal Operation, Canal Regulation

1. Introduction

The operational behaviour of hydraulic structures in irrigation canals in conveyance, regulation and delivering water is a very important aspect in operation of canals for irrigation water management. In the management of irrigation systems in Sri Lanka this aspect receives low priority due to lack of knowledge. In the implementation of more complex irrigation schedules, high operational efficiency could be achieved by use of canal operation techniques. As such in-depth study of technology used in system level and individual structures will provide the basis for study the operational behaviour of hydraulic structures mainly use for regulation. Generic typology has been developed (Renault and Godaliyadda, 1999) to improve Canal Operation techniques and applied selected Major Irrigation systems in Sri Lanka to study the canal operation.

2. Typology for irrigation systems in Sri Lanka

Major and Medium irrigation systems in Sri Lanka can be considered as homogeneous in technology at the system level. They are all designed for discharge control operations except where intermediate reservoirs can be used as volume control devices. They are all upstream controlled and consist of fully operated structures.

They are also mostly homogeneous at the sub-level of structures. A similar type of structures, i.e. undershot gated regulators with side weirs and under short gated offtakes is found in all the systems. All these structures are gradually adjustable, manually operated and in-situ controlled.

In conclusion, at the level of system and structures, only the storage criterion brings a partition of irrigation systems:

Low storage – Distributed Storage – Localized storage (see Table 1)

Table 1: Typology Matrix Application to Sri Lankan irrigation systems with partitioned classes.

Level of typology	Criterion of characterization	Identified Classes		
System & Structures	Controlled variable	Discharge		
	Type of control	Upstream		
	Degree of operation	Fully Manual		
	Adjustment(structure)	Gradual		
	Manipulation	Manual		
	Control	In situ controlled		
	Sensitivity	No information		
	Physical condition	Medium		
	Storage	Low Storage (DBK)	Distributed Storage (SBK)	Localized Storage (Intermediate Reservoir)
	Control	Variable		
	Bed material	Unlined		
Hydraulic Network	Type of supply(*)	Reservoir	River Diversion	
	Layout of lateral flows	Return Flow (RF)	Non Return Flow (NRF)	
		Single Bank Canal (SBK) With run-off	Double Bank Canal (DBK) without run-off	
		No run-off ditches		

2.1 Reservoir and diversion

In the case of a reservoir, freedom in selecting the input and stability, are generally high because, at least in the short term, water availability is not limited and the depth of water in the reservoir is steady. For river diversion, the freedom can be lower, because discharge to be diverted depends on instantaneous water availability. Furthermore, short-term changes in river level can lead to high variations in the discharge entering the system unless self- regulating structure equips the intake.

2.2 Single bank

The Single Bank Canal (SBK) has an increased free surface area compared to double bank canal. This reduces the speed of wave propagation. As a consequence, without operation the system, time-lags between the main sluice and targeted offtakes are greater.

A Single Bank Canal can also store more water per variation of water depth, in the typology it is considered as a “distributed storage”. Therefore proper management of its storage capacity can in some cases significantly reduce the time-lag between operation and delivery. Furthermore for unscheduled fluctuation, this additional storage capacity can be used to retain temporally surplus and to implement a reaction procedure.

2.3 Double bank

The double bank canal systems as a whole are not very common in Sri Lanka. However, there are numerous double bank canal subsystems as component of irrigation systems. The main characteristic of a double bank canal is that no flow enters the system during rains. Double bank canal has a limited distributed storage capacity. It is more dynamic than a single bank, i.e. fluctuations are generally propagated faster.

2.4 Intermediate reservoir

There is no clear criteria with respect to capacity allowing to declare whether or not a reservoir falls into this category. It is left to the concerned manager to categorize this feature.

Intermediate reservoir presents three important characteristics in relation to the operation. The first is the attenuation of upstream fluctuation which protects the downstream subsystem. The second is the possibility of issuing water in advance (before time leg). The third is the possibility of storing any positive unscheduled fluctuation occurring in the upstream subsystem. Therefore water savings can be achieved by controlling main supply through a feed-back volume control technique. Intermediate reservoirs are often called as regulating reservoirs.

3 . Flow fluctuation causes and their magnitudes

Fluctuations can be classified according to their causes and magnitude, and the degree of information available at management level.

A distinction is first made between scheduled and unscheduled fluctuations. Scheduled fluctuations concerns planned changes in the delivery and are assumed to be known by operation staff. Unscheduled fluctuations are the discharge variations that may occur along a canal and which cannot be predicted in advance. Unscheduled fluctuations can become known after a certain period of time if proper assessment of the canal flow status is carried out.

The second distinction is related to the magnitude of the change and leads to distinguish low and high fluctuation. At this point, only site specific considerations can decide where to place the threshold between low and high fluctuation. Low and High fluctuations may be distinguished on the basis of their cause or of their frequency of occurrence. They may be also distinguished with reference to conveyance and/or storage capacity to accommodate them. For example, a manager can classify any fluctuation that may lead to overtopping as HIGH and LOW otherwise.

3.1 Types of operation

In Sri Lanka the frequency of routine operation of gates is often twice per day. One operation takes place between 7 a.m. to 9 a.m. in the morning and the second between 4 p.m. to 6 p.m. in the evening. This pattern corresponds more or less to a 12 hours frequency operation- type. To investigate the relation between performance and operation, other types of operation are also considered in the study. The first one is the no-operation type (No). It corresponds to an “infinite” frequency and means that the system is left without changing the setting of the cross-regulator gates. Other alternatives are fixed frequency (FF) of 3 hours, 6 hours, and 12 hours during daytime. It must be pointed out that whatever the schedule, night- time is being kept without operation. A specific type of operation is studied for the purpose of scheduled delivery. It is the time-lag operation (TLO). The time-lag is the delay of the fluctuation propagation between the main sluice and the cross regulator gate. Operation is made approximately when the wave begins to pass through the regulator gates.

3.2 Evaluation of performance

For the purpose of evaluation and comparison, different performance indicators are used in relation to the water management targets.

3.2.1 On-line delivery adequacy

This indicator evaluates the deviation between actual and expected total volume delivered at the offtakes which are located between the main sluice and the tail delivery point.

3.2.2 Tail delivery adequacy

This indicator assesses the effectiveness in conveying the hydrogram to the delivery point. Specifically it represents the ratio between the effective volume reaching the tail-end of the subsystems and the targeted volume.

3.2.3 Timeliness at tail

This indicator evaluates the time deviation at tail, between actual and expected delivery.

4. Hydraulic simulations

In this section, hydraulic simulations carried out to analyze irrigation systems behavior as presented in Table 2.

Table 2: Matrix of the hydraulic simulation

Operation Mode	Simulated Fluctuation	Status of Delivery	Internal characteristics	Type of Operation
FSD (level and discharge control)	Low Increase (24hr)	Scheduled (Change in Delivery)	DB K	T.L.O 3hr 6hr
	High Increase (24hr)	Unscheduled (Return Flow, Run-off, Diversion Change)	SBK STO (Localized Tail Storage)	12 hr No Operation

4.1 Presentation of the studied subsystems.

To study the behavior of the main irrigation system types, hydraulic simulations have been performed in three different subsystems. The topography of these subsystems is derived from a real canal of Sri Lanka, the Kirindi Oya irrigation system. The DBK system considered in the study is the real (existing) main canal. The SBK canal has a twice bed width of DBK canal. The third system STO corresponds to a localized storage located at the tail-end of the DBK canal. The reservoir has a capacity of 600,000m$^{3.}$

4.2 Presentation of the hydraulic simulation model

The hydraulic software used in these simulations is SIC (Beaume et al., 1993), which has been recently equipped with additional regulation module. SIC, "Simulation of Irrigation Canals", is a mathematical flow simulation model developed by Cemagref,

France, to study the hydraulic behavior of irrigation canals under steady and unsteady flow conditions. Basically the model numerically solves the Saint-Venants equations of continuity and momentum for a given set of boundary conditions. The model comprised of three modules.

Topography module (Unit I):
This is designed to generate the topographic data of canals used by the computation programs of Units II and III. This module allows the user to input and verify the data obtained from a topographical survey of the canal

Steady flow module (Unit II):
This is designed to perform the steady flow computation. It allows to study the water surface profile for any given combination of offtake discharges and cross regulator gate openings. Unit II also allows to determine offtake gate openings and adjustable regulator gate settings required to satisfy a given water distribution plan whilst simultaneously maintaining a set of target water levels.

Unsteady flow module (Unit III):
This is designed to carry out the unsteady flow computation. It allows the user to test various scenarios of water demand schedules and operations at the head works and control structures. Starting from an initial steady flow regime, it will help the user to look for the best way to attain a new water distribution plan. The efficiency of the operational strategy may be evaluated via a set of water delivery indicators computed at the offtakes.

The regulation module attached to this module allows to select different operational options at cross regulators, when unsteady flow simulations are carried out.

Originally model has been calibrated under steady flow condition for main supply discharge of 8.21 m^3/sec and targeted offtake discharges in a real canal in Kirindi Oya irrigation system in Sri Lanka.

4.3 Fluctuation (Inputs) simulated

Fluctuations are of two types. The LOW positive fluctuation is an increase of 1 m^3/s occurring in the upstream part of the subsystem while the main supply is 8.21m^3/s. The HIGH positive fluctuation is an increase of 3 m^3/s at the same location with the same basic discharge of 8.21m^3/s.

The duration of the fluctuation is taken as 24 hours. This duration has been chosen as it is reasonable to assume that after 24 hours any unscheduled fluctuation can be detected and later on could be treated as a scheduled fluctuation if it continues.

5. Results and discussion

Results derived from the simulations are regrouped and displayed in the following manner: Performance indicators are plotted for types of subsystem under low magnitude fluctuation in Figure 1, for high magnitude fluctuation similar results are plotted in Figure2. The analysis will be carried out considering first the physical criteria of the typology, i.e. DBK, SBK and STO. The operation types tested here are: TLO, and NO, and FF (3 hr, 6hr, and 12 hr) during daytime (there is no night-time operation).

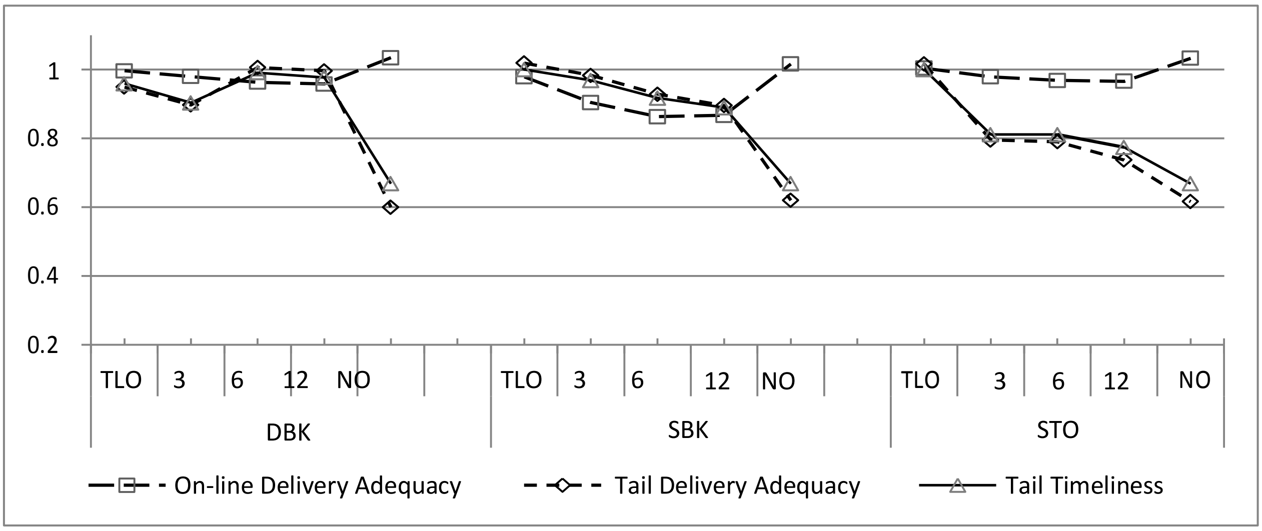

Figure 1. Performance *vs* type of operation for LOW positive fluctuation

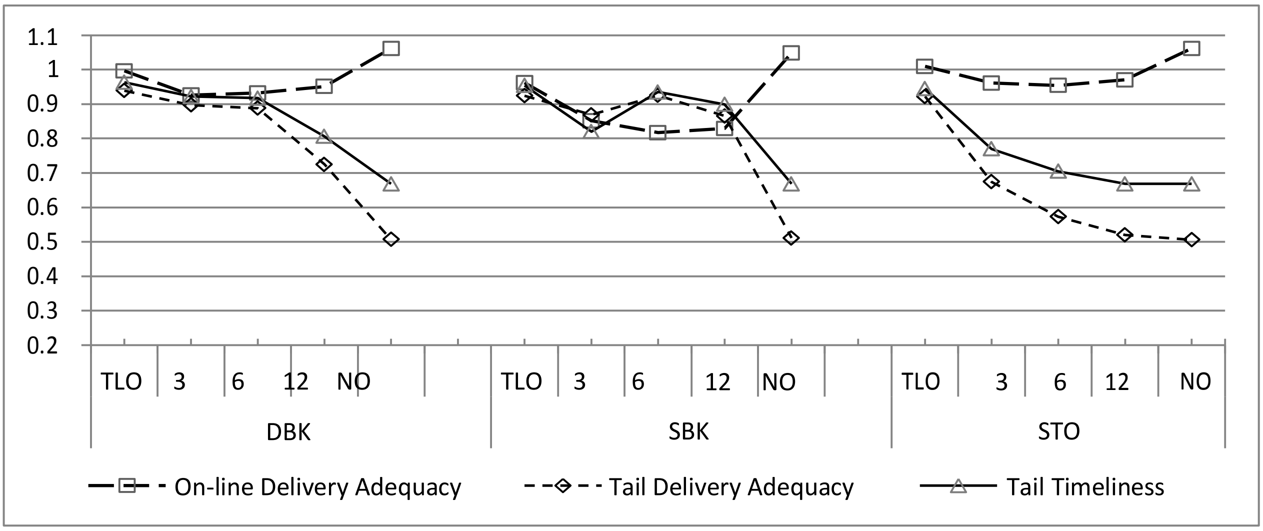

Figure 2. Performance *vs* type of operation for HIGH positive fluctuation

5.1 DBK Subsystem

Time-Lag Operation (TLO) appears to be the more effective operation type, as computed value of all indicators are very close to 1.

For scheduled change, in any case, the best solution is TLO. For fixed frequency type, 12hr is sufficient for low changes but 6hr is recommended for high changes. Furthermore, if the manager seeks a unique rule for operation, then 6hr operation during daytime period is recommended.

For unscheduled fluctuation same conclusion can be drawn except for TLO which is not relevant. In case of DBK system with Return Flow, where low magnitude fluctuations are expected, 12hr operation is recommended. In case of DBK with River Diversion, 6hr is recommended as the probability of having high fluctuations cannot be neglected.

5.2 SBK Subsystem

Similarly, for the SBK it is observed that Time Lag Operation (TLO) is the more effective type of operation, as all indicators are very close to 1. For fixed frequency operation, 3hr is better than 6hr and 12hr for low fluctuation. For high fluctuation, 6hr is the best option, whereas 3hr and 12 hr perform equally low.

For scheduled change, the best solution is TLO. For fixed frequency type, 3hr is performing little better than 6hr in case of low changes. However 6hr is better for high changes as stated for DBK. The manager will have to decide whether it is worthwhile to go for 3hr for low change instead of having a unique rule of operation. The 6hr operation during daytime period is recommended.

5.3 Storage Subsystem

The Time-Lag Operation (TLO) is the most effective operation type. Conversely to previous, poor performance is observed for fixed frequency type operations (3hr, 6hr, and 12hr). The fixed frequency type of operation, when applied to all the cross-regulators including the one downstream of the storage, is not effective for this type of subsystem.

For scheduled low fluctuation (change in delivery), storage can be used to issue in advance as long as the storage capacity permits it. For scheduled HIGH fluctuation, the most effective type is TLO but its performance does not reach a value of 1 as for Low fluctuation.

6 Conclusion

Time-Lag Operation (TLO) appears to be the more effective operation type, as computed value of all indicators are very close to 1 in all three topographical situations. The operational efficiency in delivering water of DBK canals are high if operated at fixed frequencies such as 6 hr and 12 hr. In case of Single Bank canals due to distributed storage, delivery performance are comparatively low but inversely such systems will attenuate unscheduled fluctuations in compared to DBK systems. The presence of SBK systems in River diversions with fluctuating flows and surface runoff from highland areas will not propagate the flows to tail end areas unlike in DBK systems. Especially when intermediate reservoirs are located in the system, it suggested to operate the gates when wave reaches by deploying gate operator at the location to store or release water.

Acknowledgement

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Broken Wave Loads on a Vertical Wall: Large Scale Experimental Investigations

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Abstract: Many storm protecting structures (eg. seawalls) are increasingly built at the back of the beach such that breaking waves are unlikely to reach them during the normal sea state. These structures are predominantly subjected to broken waves under most severe storm and tide conditions. Detailed studies relating broken wave forces to the incident wave parameters and beach slope are lacking. Therefore simplified assumptions are used to estimate the design loads due to broken waves. This knowledge gap has motivated to investigate the broken wave impact loads on coastal structures. A series of physical model experiments were carried out in the Large Wave Flume (GWK, Hannover, Germany) in order to measure the broken wave impact loads on a vertical wall. This paper describes the experimental results in detail. Based on the measured forces, a simple empirical formula is derived in terms of the wave parameters.

Keywords: Broken waves, Broken wave impact, Impact pressure, Vertical wall.

1. Introduction

Coastal structures such as seawalls, breakwaters, revetments, storm surge barriers etc. are built worldwide with the aim of protecting the hinterland from wave action, sheltering the harbour basin and for several other purposes. These structures are generally subjected to wave loading which may vary from slowly acting pulsating loads to more intense impulsive loads. Since 19th century there have been numerous experimental and numerical studies conducted on wave impact forces on coastal structures (eg. Bagnold [1]; Goda [2] and Blackmore and Hewson [3]). Much of the following researches have focused on the impact loading due to waves breaking directly at the structures as they produce impulsive loads which are high in magnitude and short in duration (eg. Oumeraci et al. [4]; Peregrine [5]; Bullock et al. [6] and Kisacik et al. [7]). Many studies have also concentrated on other types of wave loading such as forces due to tsunami waves, bores and surges (eg. Cross 1967 [8]; Ramsden and Raichlen[9]; Ramsden [10], [11] and; Yeh [12]). However, detailed studies related to broken wave loading (i.e waves are broken before reaching the structures) are lacking, thus information regarding this type of loading is very limited. Although the impact forces induced by the broken waves tend to be much lower than for other types of breaking waves, the broken wave loads act for longer durations and extended to larger distances, which lead to higher forces and impulses. Hence broken wave loading could be well engineering significance and needs to be given greater consideration in designing of coastal structures.

Figure 1: An example of a Seawall on the coast of Isle of Wight[13].

Existing coastal defences along many low lying coastlines are under increasing risk due to the rising sea-level and the increased intensity of the storm surges in the coming decades. Therefore storm protecting structures are increasingly built at the back of the beach. These structures are predominantly subjected to broken waves under the most severe storm and tide conditions. Examples of such structures would be storm walls on a dike, revetments and seawalls on shore, sheet pile walls on a beach face and run-up deflector on the shore revetment (Sorensen [14]). These structures are generally located where they are predominantly subjected to broken waves. An example of such situation is shown in Figure 1, where the seawall is exposed to broken waves. Although Coastal Engineering Manuel (CEM) [15] presents a method for broken wave force prediction, which is essentially based on number of simplified assumptions. Further CEM provides a method to calculate the total force induced by the broken waves and no estimation is given for the pressure distribution. Detailed investigations are therefore required not only to quantify the broken wave loading and pressure distributions but also to understand the process and mechanisms related to broken waves impacting the structures.

Therefore, preliminary investigation has been conducted in the present study in order to investigate the broken wave impact loading on a vertical wall. Series of physical model experiments were carried out with the regular waves in the Large Wave Flume (GWK), Hannover. The aim of this study is to analyse the loading characteristics and the pressure distribution of the broken wave impact on a vertical wall. An empirical relationship to estimate the broken wave impact loads is derived in terms of the wave parameters using the experimental data.

2. Physical model experiments

2.1 Experimental set-up

The experiments were carried out in the Large Wave Flume in Hannover, which has a length of about 330 m, a width of 5 m and a depth of 7 m. The wave flume is equipped with a piston type wave-maker and an active wave absorption system. Figure 2 shows a simplified sketch of the cross sectional view of the model setup. The structure consists of a vertical wall and a recurved wall on the top.

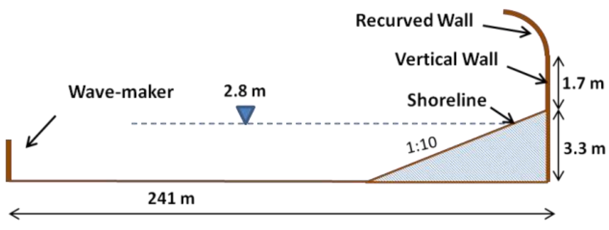

Figure 2: Cross-sectional view of the model set-up

The recurved wall is not considered in this study since the broken wave loads are expected to occur in the vicinity of the vertical wall. The vertical wall is 1.7 m height made out of steel frames embedded on the side walls of the flume, which is located 241 m away from the wave paddle. The bed slope in front of the wall is 1:10 and it was constructed with sand and geotextile. The most violent flows were expected in the vicinity of the toe of the structure and therefore it is protected with concreteblocks.

2.2 Instrumentation

The pressure transducers were flesh mounted as a vertical array in the middle of the vertical wall, sampled at a rate of 5 kHz. The positions of the pressure transducers are indicated in Figure 3. Since this study considers only the forces on a vertical wall, the other measurements made on the recurved wall are not shown.

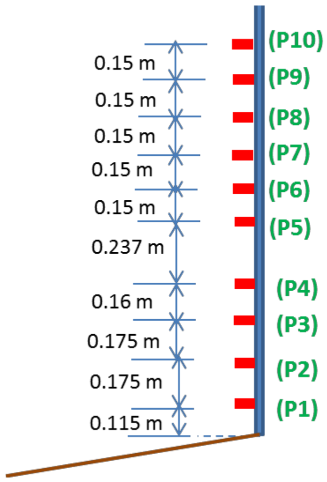

Figure 3: The locations of the pressure transducers along the vertical wall

The wave parameters were measured with 12 wave gauges placed along the flume in three groups, sampled at a rate of 100 Hz; one group is placed near the wave maker, which can be used for the reflection analysis, the second group is located near the toe of the slope, the third group is on the slope close to the structure.

2.3 Wave conditions

Experiments were performed with regular wave conditions as it is relatively easier to observe and record the physical processes related to breaking wave impact and many wave parameter combinations can be tested in relatively short period of time. Series of tests were carried out under four different water levels; 2.8 m, 3.1 m (where the shoreline is in front of the wall), 3.3 m (where the shoreline is just at the toe of the wall), 3.6 m (where the shoreline is behind the wall). Different wave conditions were tested for each water level, the wave height ranges from 0.6 m to 1.0 m and periods varies from 6 s to 12 s. Each test is limited to 15 regular waves. All the waves were broken before impacting on the vertical wall. The location of incipient breaking and the type of breaker were varied depending on the steepness of the incidentwaves.

3. Results and discussions

3.1 Pressure-time histories

As the broken wave impacts on the wall, the impact pressures were recorded by the pressure transducers located in the middle of the wall. A typical pressure time series recorded by a pressure transducer (P1, see the location in Figure 3) during a test (H=1.0m; T=8s; WL=2.8m) is shown in Figure 4. One could observe that the magnitude of the pressure peaks vary from one impact to other although the generated waves in one test are nominally identical. A similar trend was observed in the pressure-time series recorded by the other transducers as well, throughout the whole series of experiments. The highly stochastic nature of the impact pressures have already been reported by many authors (eg. Bagnold [1], Bullock et al. [6], Hattori et al. [16]). There can be several reasons for this variation. The amount of entrained air, which alters the density and compressibility of the impacting water mass, could influence in the magnitude of the impactpressures.

3.2 Impact processes

The broken wave impact processes were recorded by a high speed camera (300 fps). Different stages of a broken wave impact event are studied by synchronising the recorded video and the pressure time history obtained at the wall. Figure 5 illustrates the pressure-time history recorded along the wall. The locations of the pressure transducers P1, P2, P3 and P4 are as indicated in Figure 3. The black line indicates the corresponding force-time history obtained by pressure integration. The images are taken from the corresponding high speed video record at selected time moments of the pressure-time history. Although there was no direct synchronisation made between the pressure recording and the video (as they were started recording at different time instants), the video images are used for the demonstrating purpose.

The broken wave is associated with high turbulence and lots of entrained air as can be seen by the white patches in the images. Pressure-time history indicates that the pressure is zero just before the impact (t_1 =166.38 s). As the wave front impact the wall with a certain velocity (t_2 =166.46 s), the flow direction changes suddenly. This results in a sharp increase in the pressure (P1) to a peak value of 8 kPa. This stage is denoted as the initial impact. The magnitude of the pressure peak is mainly governed by the wave front velocity. The other influencing parameters could be the amount of entrained air, thickness of the wave front. Just after the initial impact, the pressure drops rapidly. During this stage, the kinetic energy is converted into potential energy. As a result, the foamy wave front starts rising up while the proceeding part of the wave impacts on the wall which is then deflected upwards (see the image at t_3 =166.60 s). At this moment, the proceeding part of the wave impact generates another peak in the pressure-time history. However, this impact is significantly dampened by the initial foamy front which is then deflected upwards.

As the wave continuously runs up along the wall to a maximum run-up height (t_4 =167.10 s), the pressure starts to increases on P2 and then P3 and P4. It is very difficult to capture the exact time from the video when the maximum run-up occurs, because the leading front is like water spray rather than run-up tongue. During this run-up stage, the pressure increases with lots of fluctuations, this can be observed in the pressure-time history. The reason for these kinds of fluctuations is not very clear. Chen et al. [17] reported that the measured pressures during the run-up (deflection) stage are smaller than the hydrostatic pressures computed by using the detected run-up surface elevation from the video. The same trend was also noted in most of the cases in this study. P4 indicates a negative pressure from the time when pressure starts to increase in P3 until around the time of maximum run-up. Such negative pressures were also observed by Hattori et al. [16] and they described the reason as extremely high velocity jet shooting up the wall face creates a lower pressure area around the pressure sensors located on upper wall.

Once the deflected water has reached to a maximum run-up level, it stats falling on to the remaining part of the incident flow. Then the reflection process takes place as the whole water mass gradually runs down and flows towards the sea. The second peak (t_5 =167.29 s) is generated during this stage. The second peak (pressure/ force) has always occurred after the instant of maximum run-up, which is in line with other reported studies by Ramsden and Raicheln [9] and Chen et al. [17]. During the run-down stage, the pressures along the wall are quite linearly distributed (Image at t_6 = 167.47), which also indicate the quasi-static nature.

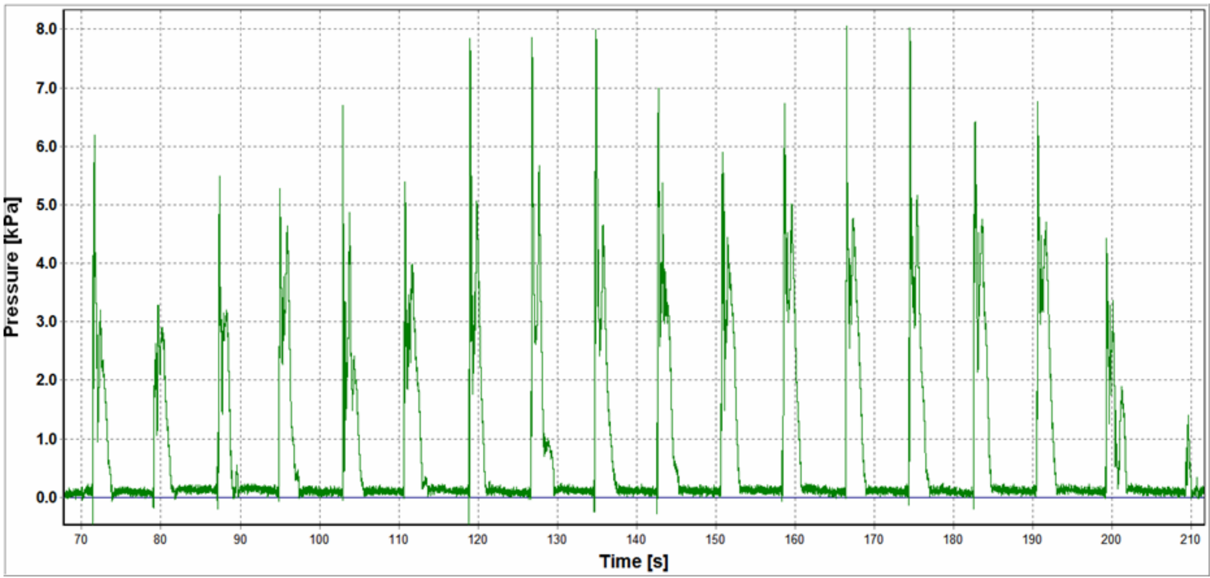

Figure 4: An Example of pressure-time series of broken wave impacts

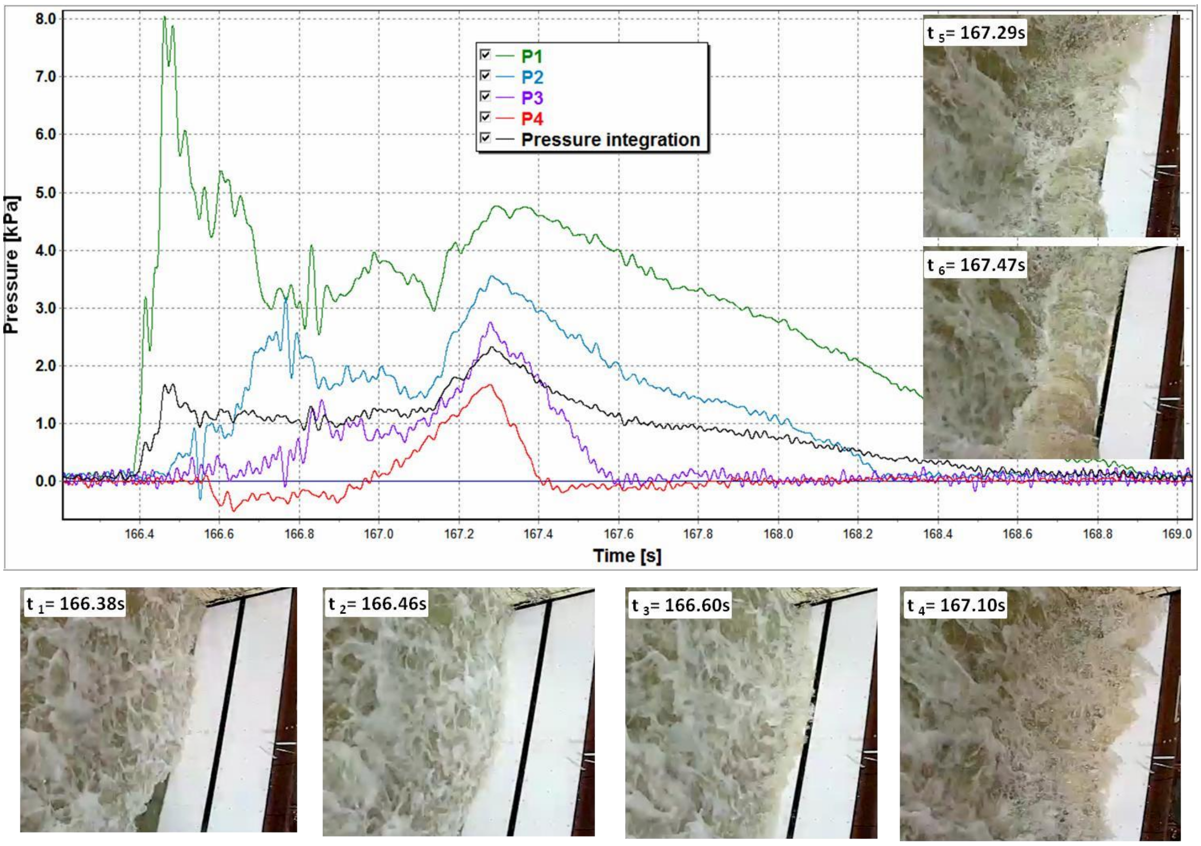

Figure 5: Pressure-time series of a typical impact event by broken wave and snapshots at selected time instants

The pressure integration (force) gives the second peak higher than the initial impact peak. This trend was noted in most of the cases, although the ratio between the first and the second peak vary from one test to other test depending on the wave conditions and water levels. Further, the duration of the second peak is higher and acting over larger area than the first peak. This suggests that although the initial impact produces higher pressure (first peak), which is highly local, the run-down process (after the wave front reached to a maximum run-up height) tends to generate higher forces over larger area. Therefore, the second force peak (quasi-static) needs to be taken into account in the structural design.

The kinematics of the broken waves was recorded by the high speed video. However, processing the high speed video is highly complicated due to the fact that broken waves are associated with high amount of entrained air and turbulence. More advanced techniques would be required on order to extract the information such as broken wave celerity, broken wave heights etc.

3.3 Impact pressure distribution

The instant pressure distribution along the wall height for the above impact event is given in Figure 6. The time t_1 to t_6 are the same instants in the pressure-time history and the video images in Figure 5. The pressure is zero everywhere at time t_1 just before the impact. There is a sudden increase in pressure on the bottom of the wall at t_2 when the initial impact occurs. As the wave front is deflected upwards in the next time step t_3, the initial pressure is dropped while increasing the pressure on the upper part of the wall. In this stage a negative pressure is generated as the wave front is shooting upwards with high velocity. As the wave runs up continuously, the pressure increases upwards until the wave front reach to a maximum level at time t_4. The pressure is zero above the wall height of 0.8 m for all the time instants. This is because the maximum run-up in this test has occurred around the location between P4 andP5.

When the run-down process starts (at t_5), the pressure becomes uniformly distributed along the wall height. At time t_6 when the reflected wave is fully formed, the instant pressure distribution tends approach the hydrostatic pressure. The hydrostatic pressure distributions are not shown here since in this study it was very difficult to extract the exact water surface levels along the wall from the video. Chen et al. [17] compared the instant pressure distribution with the hydrostatic pressure distribution computed by using the water surface elevation (from high speed images). Their results indicate that the measured pressures during the run-up process are always smaller than the hydrostatic pressures and then during the run-down process the measured pressures approach the hydrostatic pressures. Similar results were already reported by Ramsden and Raichlen [9]. They have further explained the reasons as follows: Large vertical accelerations associated with the run-up most likely cause the observed time lags between maximum run-up and maximum force and the differences between the force computed using a hydrostatic pressure distribution acting on the wall and that actually measured. Negative vertical accelerations in the flow decrease the pressure gradient and the force relative to those that would result if the pressure were distributed hydrostatically.

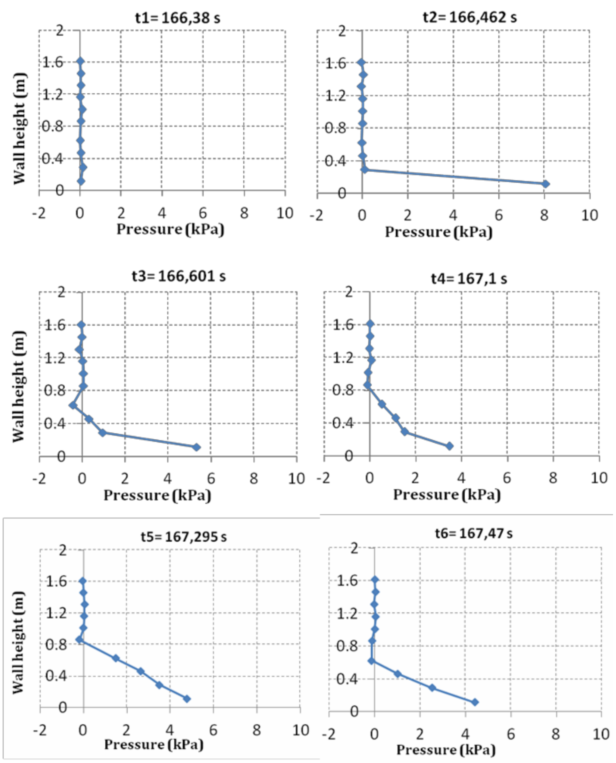

Figure 6: Instantaneous pressure distribution along the wall height (corresponding to the same time moments indicated in Figure 5)

3.4 Development of empirical relationship

In order to develop an empirical relationship between the broken wave forces and the wave parameters, Hughes's overtopping momentum flux concept [18] has been applied. This concept has been successfully applied recent study by Chen et al [17] for the overtopping bore impact.

A principle sketch of the typical situation of broken wave impact on a vertical wall which was investigated in this study is shown in Figure 7. The run-up surface slope is approximated as a straight line for gentle slope. Hughes [18] made a simple physical argument that the weight of the water contained in the hatched wedge area ABC is directly proportional to the maximum depth integrated wave momentum flux contained in the wave before it reaches the toe of the slope.

$$W_{ABC} = \frac{1}{2}\rho g \frac{R^2}{tan\theta}\left[\frac{tan\theta}{tan\beta} - 1\right] \quad (1)$$

$$(M_F)_{max} = \frac{K_M}{K_P} W_{ABC} \quad (2)$$

$$(M_F)_{max} = \frac{K_M}{K_P}\frac{\rho g}{2}\frac{R^2}{tan\theta}\left[\frac{tan\theta}{tan\beta} - 1\right] \quad (3)$$

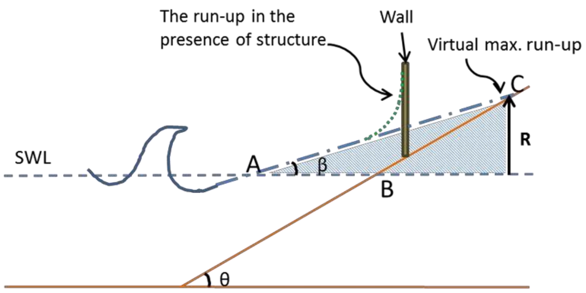

Figure 7: Principal sketch of broken wave impact on vertical wall

For regular wave run-up on an infinite beach slope Hunt's formula [19] can be used,

$$R = \xi_o H \quad (4)$$

$$\xi_o = \frac{tan\theta}{\sqrt{H/L_o}} \quad (5)$$

where L_o is the deep water wave length and is given by Eq. (6), H is the significant wave height at the toe of the slope.

$$L_o = \frac{g}{2\pi} T^2 \quad (6)$$

$$R = tan\theta\sqrt{H * L_o} \quad (7)$$

By substituting R (Eq. 7) in Eq. (3) gives,

$$(M_F)_{max} = \left(\frac{1}{2}\frac{K_M}{K_P}\right)\rho g H L_o \left[\frac{tan^2\theta}{tan\beta} - tan\theta\right] \quad (8)$$

$$(M_F)_{max} = c'\rho g * H * L_o * f(\theta) \quad (9)$$

The maximum depth integrated momentum flux $(M_F)_{max}$ is a physically relevant descriptor of the wave force on a structure with a force per unit crest length. Thus the maximum total impact force on the wall is given by,

$$F = c * \rho g * H * L_o * f(\theta) \quad (10)$$

where *c* is an empirical coefficient and $f(\theta)$ is function of bed slope and needs to be determined empirically.

Above equation can be written in a non-dimensional form, i.e.

$$\left[\frac{F}{\rho g {L_o}^2}\right] = c * f(\theta) * \left[\frac{H}{L_o}\right] \quad (11)$$

3.5 Fitting the empirical relation by using the experimental data

The formula given by Eq. (11) is then fitted with the experimental data. As reported by Chen et al [20], the quasi-static force (2nd peak) is more relevant for the large structures. Therefore, the analysis focuses on the second peak forces. Since this study has used only the regular waves, an average value of the peak force is determined by considering several impact events in each test. Therefore *F* in Eq. 11 represents the average peak force per unit length of the wall (N/m). *H* is the incident wave height (m), L_o is obtained by Eq. (6) in meters and ρ is the density of pure water (1000 Kg/m^3).

Figure 8 shows the results in four plots for water levels 2.8 m, 3.1 m, 3.3 m and 3.6 m. Linear relationships have been obtained with certain scatter, which is higher for the case of lower water levels (WL=2.8 m and WL= 3.1 m). The reasons for relatively large scatter observed for lower water levels can be explained as following. The forces are obtained indirectly by integrating the pressures, which were placed at certain intervals as shown in Figure 3.

In the case of low water level, the thickness of the impacting water layer and the run-up height along the wall would be relatively small. In such cases, the spatial resolution of the pressure transducers may not be sufficient to extract the actual pressure information. Hence, pressure integration would not provide very accurate force values. However, when the water level is relatively higher, the corresponding water layer thickness also would be higher. Thus the error from the pressure integration has very less influence on the calculated forces.

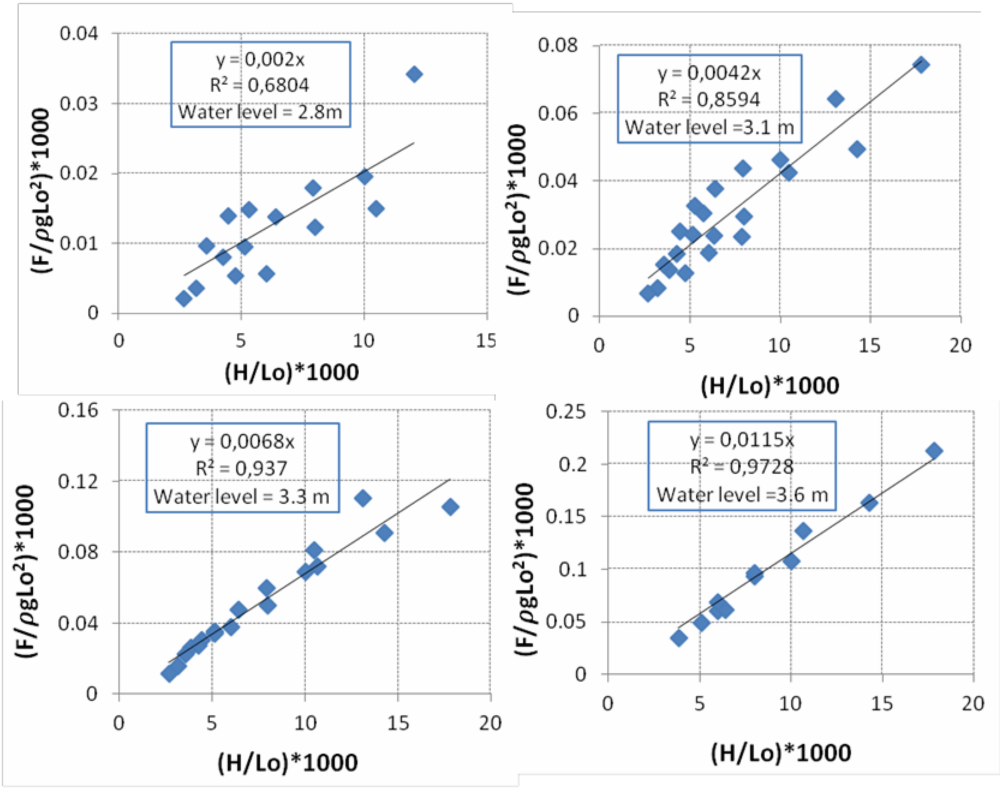

Figure 8: The wave parameters are related to the impact force for different water levels

An increasing trend of the dimensionless coefficient of the linear equations can also be seen with increasing water level from 2.8 m to 3.6 m. Therefore the non-dimensional coefficients of the linear equations are fitted with the free board Rc in order to include the water levels into the empirical formula. The free board Rc is the vertical distance from the toe of the wall to the Still Water Levels, measured in upward direction. Figure 9 shows that the non-dimensional coefficients are exponentially related with the water levels, which can be written as,

$$c = 0{,}0063 * \exp\,(2{,}2 * Rc) \qquad (12)$$

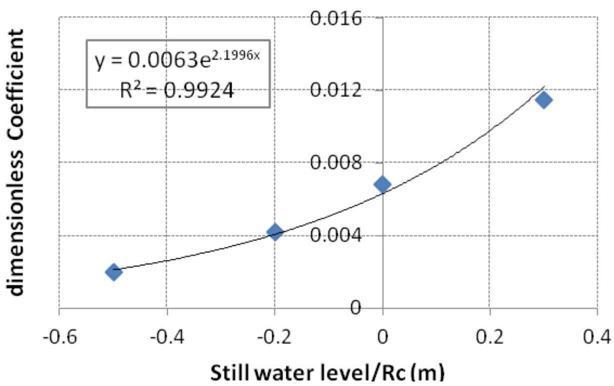

Figure 9: The dimensionless coefficients are linked to the water levels

Since all of these experiments were performed for only one bed slope (1:10), it is not possible to link the bed slope into the empirical relationship. Further data for different bed slop is required to study the effect of bed slope on the impact force on the vertical wall.

By substituting the non-dimensional coefficient c provided by Eq. (12) into Eq. (11) the empirical relationship can be written as,

$$\left[\frac{F}{\rho g {L_o}^2}\right] = 0{,}0063 * \exp\,(2{,}2 * Rc) * \left[\frac{H}{L_o}\right] \qquad (13)$$

It should be kept in mind that the above formula is only valid for regular waves with bed slope 1:10. A reliability check was performed by comparing the measured forces against the calculated forces using the proposed empirical formula. A good agreement was found between the predicted forces with the measured forces. However, additional experiments are required to incorporate the other influencing parameters (i.e bed slope, entrained air etc.) into the empirical formula. Moreover, the proposed relation for the broken wave force needs to be verified under irregular wave conditions as well.

4. Conclusions

Physical model experiments were carried out with regular waves in the Large Wave Flume, Hannover in order to investigate the broken wave impact loading on a vertical wall. The impact pressure peaks were observed to have highly stochastic nature even under regular wave conditions. The force-time history consists of two main peaks; the first peak of relatively shorter duration, is due to the initial impact and the second peak is the quasi-static force, which lasts relatively longer. When analysing the force-time history together with the high speed video of the impact, it was observed that the second peak is recorded after the instant of maximum run-up. This result is in line with similar studies by Chen et al. [17] and Ramsden and Raichen [9]. The whole processes related to broken wave impact on vertical wall are comparable to those of overtopping flow impact reported by Chen et al.[17].

Based on the works by Hughes [18] and Chen et al. [20], an empirical formula is derived for the broken wave forces in terms of the wave parameters and water levels. The proposed empirical relation for broken wave impact force requires only the wave parameters and the still water level. However, it should be kept in mind that the derived formula needs to be validated against irregular waves. The influence of the slope is not included in the formula since the experiments were carried out on a fixed slope. Further research is necessary to determine other influencing parameters and to incorporate them into theformula.

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Water Pollution in a Natural Stream and Its Impacts on Society and Environment: A Review of Studies on *Meda Ela*, Sri Lanka

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Abstract: *Meda Ela* which originates from *Kandy* Lake and runs through *Kandy* city is considered to be one of most polluted tributaries of *Mahaweli* River. The objective of the study was to critically review the published research findings related to *Meda Ela* pollution to existing problems, research gaps and the means to rectify the situation. The review was carried out under the categories of socio-economic background, land use changes, sources of pollution, solid and waste water disposal, water quality, cost due to water pollution, economic benefits and major stakeholders and their interactions of *Meda Ela*. Study identified a very high urbanization rate in the watershed during the last decade compared to previous 30 years. Major point sources include the hospital, bus stand, railway station, central market and the residences on either side of *Meda Ela*. According to the literature, elevated pollution levels are reported during wet season and NO^-_3, NH^+_4, PO^{3-}_4, suspended solids, heavy metals, DO, BOD and COD showed above threshold limits. This is an indication of non-point source pollution which is responsive to hydrological conditions. Impacts of water pollution include vector borne diseases such as *Dengu* and *Chickengunya*, bad odour, flash floods and contamination of shallow groundwater with heavy metals. The social and management setup in the watershed is very complex since many stakeholders are involved in polluting and managing *Meda Ela*. The analysis revealed that the relationship among different stakeholders is highly diverse and as a result, their contribution to control water pollution in *Meda Ela* is also very different. Hence, a strong connection should be established between community and other stakeholders to develop an efficient and effective management plan to safeguard *Meda Ela* stream and its watershed.

Keywords: Black water and Grey water, Biological indicators, Dissolves oxygen, Ethnic groups.

Introduction

Water, hitherto considered as an abundant resource, is nearing exhaustion due to increasing demands for irrigation, industry and domestic uses with continuously growing population. This paper deals with one of the widely studied water resource in the central hills (*Kandy*) of Sri Lanka. Available published research articles related to the study area were reviewed and organized under different topics focusing on proper watershed management as a mean of preserving its quality.

Kandy is recognized as a world heritage city by the United Nations Educational, Scientific and Cultural Organization (UNESCO) on account of its history and cultural treasures. A water canal called Mid-canal or "*Meda Ela*" originates from the overflow sluice of the *Kandy* Lake located next to the Temple of Tooth Relic, runs through the densely populated city, and thereafter drains into the *Mahawelli* River which is the largest river basin in Sri Lanka [1]. *Meda Ela* consists of seven major tributaries covering 1456.38 ha of land area (Figure 1), contributing to its inflow in addition to the main supply from the *Kandy* lake [2]. The length of the canal is about 8 km and the width varies from 10 m to 15 m along its course from the lake sluice to the confluence with the *Maliaweli* River at *Getambe* [3].

The *Meda Ela* is considered to be the most polluted running water system in the *Kandy* district [4]. This natural stream has been modified by constructing concrete banks and paving the bed with cement at certain places. However, a major part of the canal still flows as a natural course. The banks of the *Mada Ela* have been reinforced by concrete walls from the point of its origin (i.e.

Lake Sluice) to about 100 m downstream. The canal flows underground (<1 km) from the sluice up to the *Kandy* railway station. The *Meda Ela* then merges out and connects with a network of waste water canals draining from various parts of the *Kandy* city. The banks of certain parts of the *Meda Ela* have also been modified by cement walls from the railway station up to the *Mulgampola* area (up to 3 km). Beyond *Mulgampola*, up to *Getambe*, the canal flows along a more or less natural course.

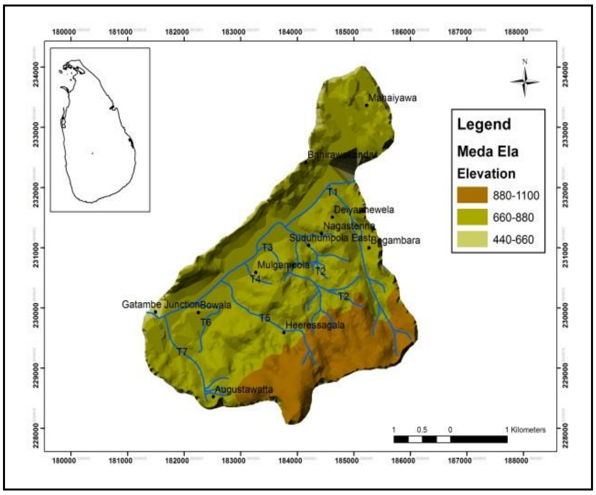

Figure 1: *Meda Ela* watershed, its tributaries and topography [2]

According to [2], the urbanization rate in *Meda Ela* watershed has drastically increased from 10.14 ha/year to 17.2 ha/year during the past decade (2003-2014) when compare to 1972-2003 periods. Therefore, amount of solid waste, sewerage, waste water and other disposal material generation also has increased drastically, but without enough facilities for proper disposal [5]. People who are living in these unplanned areas of the watershed directly or indirectly dispose wastes into natural environment which finally confluences with the *Meda Ela* since it is draining at the lowest part of the watershed [2].

Socio-economic background

The census of population and housing in 2001 revealed that the total population of the *Kandy* district was 1,279,028 of which 109,343 people permanently live within the *Kandy* Municipal Council (KMC) area [6]. *Kandy* city also caters to a floating population of 157,000 during day, 111,000 during night and a very high seasonal floating population during the *Perahera* (Buddhist festivals) season [7]. The population density of the *Kandy* district is over 660 people per km^2 and statistics from 1981-2001 indicate an increasing trend [8]. Nearly 350 houses are located along the *Meda Ela* canal bank [1]. There are different ethnic groups living in the area and the majority (66%) of them are Sinhalese, 23% Muslims and 10% Tamils [5]. The education level is relatively good and 94% of people sat for Ordinary Level Examination [1]. The average land extent owned by a household is 325 m^2 and the majority of people fall into the medium income level with a gross domestic product of US $2400 [1]. Of the total population, 19% are engaged in government employment and 37% work in the private sector and others have not permanent employment [5].

Approximately 87% of residents have potable water supply through connected pipes either by KMC or National Water Supply and Drainage Board (NWS&DB) and 9% of people have common pipe water supply while some of residents still use unprotected wells on the canal bank for domestic purposes. Average family size is five and average consumption of water per family is 995 L/day. About 91% of the families use piped water for bathing and washing and only 4% use common bathing places [1].

Land use change in the entire catchment

Unplanned urbanization is believed to be an important cause of destruction and degradation of natural water resources. Owing to rural-urban migration, it has been estimated that approximately 50% of the population will live in growing cities of less developed countries by the year 2025 [9]. Figure 2 displays the overall land use change in the *Meda Ela* catchment during the past 50 years. The urban area of the catchment has increased in recent years and yet the forest area remains mostly the same. The forest cover that contributes to the entire catchment consists of *Dunamadalawa* and *Udawatta kale* forest reserves and very small extents of forest areas are located in other places. Since two forest reserves are protected areas, it is not possible to encroach. The amounts of paddy lands have been drastically reduced during this period. On the other hands, home gardens show a remarkable increase during 1980 – 1992 time periods mainly owing to the construction of *William Gopallawa* road [2]. It is evident that the construction of this road has initiated the urbanization and development in the area resulting in increased land value.

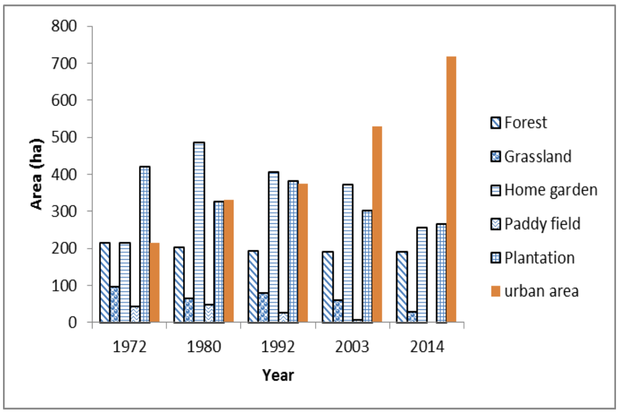

Figure 2: Land use changes of the whole *Meda Ela* catchment [2].

In 1972, only a small stretch of land was under urban use and much of the land was occupied by home gardens, paddy and plantations as shown in Figure 2. In 1980, much of the home gardens have transformed in to urban areas and by the year 1992 the paddy lands have begun to disappear from the land use [2]. In 2014, the majority of the catchment areas of 3rd, 4th and 7th tributaries have been converted from home garden to urbanized areas. With the improvement of the road network and due to the lack of land, the home gardens have been converted in to urban areas. The catchment area of the 3rd and 4th tributaries which have higher slopes and low time of concentration display the highest extent of land use change and both catchment areas are now completely under urban land use (buildings and houses). This is very dangerous for the local hydrology as more impervious lands have been formed from the natural pervious ground. With steep slopes and relatively high time of concentration of the catchment, it is possible to create flash floods in the downstream as well as landslides within the catchment area.

Source of pollution

Water quality of the *Kandy Lake* has been maintained to a certain standard due to its close proximity to the *Dalada Maligawa* (Temple of the tooth relic). There are many hotels, restaurants and private hospitals around the lake. Wastewater discharges from these locations into the lake are monitored in order to protect the lake. But it cannot be controlled and continues waste disposal is progressing. The contamination of the *Meda Ela* from downstream begins from *Kandy* market, which is maintained by the KMC. More than 20 meat and fish stalls are located in the complex [10]. Wastewater with very high organic loads from meat, fish and vegetable stalls and restaurants is discharged directly to relatively unpolluted water flowing in the *Meda Ela* from the *Kandy* Lake. The *Kandy Bogambera* prison, which is located adjacent to the market, also was releasing wastewater into the *Meda Ela* until the prison and slaughterhouse were closed recently.

Kandy is one of the focal points of public transport in Sri Lanka. According to the traffic police, more than 30,000 vehicles enter the city every day. The runoff from the bus stand, railway station and vehicle service stations carrying oils, grease and suspended particles attached with heavy metals, flows into the *Meda Ela*. The base hospital in *Kandy* is a large teaching hospital with all modern facilities for medical treatment. Wastewater of the hospital goes through a treatment facility within the hospital and is then discharged into the *Meda Ela*. Occasionally, the treatment plant does not function, and untreated wastewater is discharged into the *Meda Ela* [10] creating a situation which can cause infectious diseases.

Commercial laundries (*dobby* communities) located along the canal also discharge untreated wastewater into the *Meda Ela*. The *dobby* community has been living at this location for more than a hundred years. Clothes from operation theatres and wards of the hospital are also washed in these laundries resulting in a heavy pollution of water in the *Meda Ela* with pathogens and other microbes. The concentration of phosphate in the *Meda Ela* may be due to the detergent used in these laundries [10].

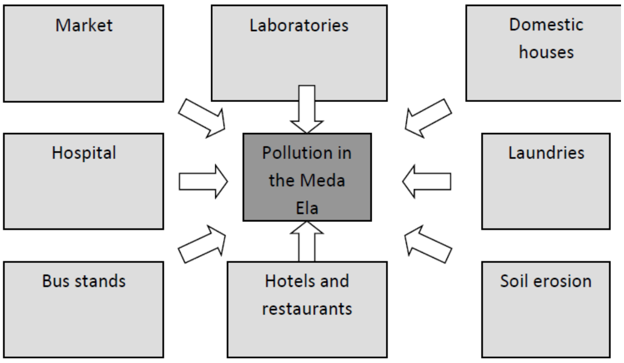

Figure 3: Multiple sources of pollution of the *Meda Ela* (Modified from [10]).

Figure 3 illustrates the possible point source of pollution of the *Meda Ela* watershed, but high number of non-point source of pollution is also involved in polluting the *Meda Ela* water during wet and dry seasons. As shown in Table 1, 90% of the respondents consulted express concern over untreated effluents generated by the market centre, general hospital, central bus stand, hotels, eating houses and soil erosion from building sites. Ninety present respondents commented on the untreated effluents generated by the railway station, slaughterhouse, storm-water runoff and soil erosion from construction sites. There were several other agents also responsible for the damage.

Table 1: Public opinion about source of pollution of the *Meda Ela* watershed

Cause of pollution	% Opinion
Garbage - resident population	63
Garbage - visitor population	68
Solid waste	59
Storm water run-off	89
Haphazard car-wash run-off	23
Soil erosion	92
Kitchen wastes	42
Toilet water	68
Untreated effluents	98
Street sweepings	82
Drain block out collections	94
Sewerage	29
Night soil	22

Source: [11]

Solid waste disposal

Rapid urbanization and lack of an organized system to collect and dispose solid and liquid waste is a serious risk factor related to water pollution in the *Meda Ela* watershed. Along its 8 km course, the canal collects massive quantities of effluent and domestic waste products. Topographically, being situated at a lower elevation, a large number of artificial and natural canals empty their waste loads into the *Meda Ela* canal. With respect to municipal solid waste, nearly 100 tonnes/day of solid waste is generated within the KMC area [8] with an average household solid waste generation of 1.5 kg/day [5]. There is obviously a variation of the amount of waste generated depending on the type of establishment. Commercial establishments generate more than 5 kg/day contributing to about 37% of the total generation. Pavement vendors, public eating-houses, restaurants, and groceries generate a high amount of waste (20%). Domestic households generate solid waste and contribute about 13% [11] and balance produced by schools, tea-kiosks, small hotels and wayside tearooms. Waste generation from families is mainly composed of by-products of day-to-day consumption with main contributions are from sweepings and kitchen wastes. According to the type of waste generated, categories are; Biodegradable - Short term (63%), Biodegradable - Long term (12%), Paper (11%), Polythene (6%), Metal (4%), Wood (3%) and Glass components (1%) [11].

There is a relationship between income levels, consumption pattern and ethnic groups to waste generation. Daily solid waste generation by an average Sinhala family is usually less than 2 kg while Muslim families generate more waste (>3kg) when compare to the other ethnic groups. With increasing income levels, the amount of waste generate is also increase [5]. The study done by [1] indicated that 71% of the *Mada Ela* community disposed their waste in municipal waste bins or collecting carts, but 29% of them disposed their waste directly into the *Mada Ela*. Also, the same study revealed that even some people who reside outside the *Meda Ela* catchment also dispose their solid waste into the canal. Solid waste management is a serious problem for the municipal corporation. The central collection system covers a limited number of areas due to the financial constraints. The land available for dumping is also limited. The KMC does not have a sanitary landfill site and open dumping along the streams is common. The dumping of solid waste into the *Meda Ela* becomes an easy way of solving the problem of solid waste, even though it is not a good way to dispose solid wastes.

Waste water disposal (black and grey water)

The colour of water in the *Meda Ela* is generally brown during dry weather flow conditions and looks severely polluted even to the naked eye. The discharge of the canal during the rainy season is substantial and perhaps water carries a heavy loading of particulate and suspended sediment. As a result, water becomes brownish yellow (murky) during most of the rainy seasons. It has also been reported as bad odour in several instances and the canal does not have visual attractiveness [12].

Methods of sewage disposal in urban catchments are also one of the most important factors in protecting water resources. If sewage is disposed without following a proper treatment and disposal methods, downstream people will be affected causing many health and environmental problems. In the *Meda Ela* catchment, pit type toilets (59%) are the most common and followed by septic tanks (30%) [5]). However, 5% of households dispose directly their sewage to the canal as an easy and cheaper method neglecting the adverse side effects. Approximately 1000 m^3/day of sewerage or black water flows into the *Meda Ela* [13] and most people living along the canal bank flush out their sewerage pits during heavy rains to avoid paying for gully trucks to empty their pits [5]. This is an ideal example to show the way of contaminating water in the *Meda Ela* as a result of a congested and unplanned urban environment and lack of community awareness on possible environmental and health consequences of contaminating water resources.

Water quality

The *Meda Ela* which is considered as an effluent canal has been hardly subjected to any systematic water quality assessment though it is a potential threat to health of people living in the vicinity. The general appearance of the stream is environmentally and aesthetically unacceptable. Several studies conducted time to time have reported basic physico-chemical parameters (e.g. pH, EC, NO_3-N, NH_4-N and DO) and presence of some heavy metals (i.e. Pb. Cd. V and Fe) in the canal. Those analyzed water samples were collected from the canal and several dug wells adjacent to the canal [14]; [15]; [6]; [5]; [1]; [16] and [2] were summarized below.

According to a study by [14], the nitrate (NO^-_3) concentration of the *Meda Ela* varied between 0.2 mg/L to 3.56 mg/L. The NO^-_3 loading to the canal was attributed to biogenic waste such as human and animal excreta, which accounts for a large percentage of the total nitrogen loading. However, extremely high nitrate concentrations were not reflected in the analytical data as some of the nitrogen species could have been incorporated in organic forms, particularly in the bottom sediments along the canal. A study conducted by [15] found a maximum nitrate value of 7.28 mg/L in a well adjacent to the *Meda Ela*. This value however is within the safe limit for drinking water. Although a large amount of human excreta is discharged into the canal, a complete nitrification does not occur within the system. Nitrogen loaded into the canal could retain as organic complexes, which can eventually be converted to nitrate. NH_4-N levels reported in the same study (i.e. 0.01 - 0.50 mg/L) fall within the acceptable range for running water systems. However, extremely high value of 4.0 mg/L was also reported. The total phosphorous level of the canal ranged from 0.15 mg/L to 15 mg/L. The upper level was extremely high and unacceptable for stream water according to Sri Lankan standards. In general, the total phosphorous levels were higher than the recommended levels compared to nitrate levels in the canal. Extremely high phosphorous concentration in this stream could he attributed to a massive input of phosphorous from human and animal excreta and organic garbage. In the case of heavy metals, the total Pb levels in the canal water varied between 20µg/L and 850µg/L while the average was 268µg/L. The Pb levels ranged from 20 µg/L to 640 µg/L in some wells, of which are higher than the upper limit (100 µg/L), recommended by the World Health Organization [17]. High emission of Pb from automobiles could be a contributory factor for high Pb levels in the canal and adjacent wells since the area is closer to the *Colombo-Kandy* main road. Municipal wastes are also highly contributed to Pb pollution in the *Meda Ela*. In addition, accumulation of Pb in the *Meda Ela* could be due to the waste petroleum products from garages and service stations. The total Cd concentration of the canal water ranged from 10µg/L to 310 µg/L with an average value of 138µg/L. Well water containing 10 µg/L of Cd is the maximum permissible limit recommended by the WHO. The total V concentration in the canal water ranged from 6.5µg/L to 45 µg/L with an average of 18µg/L. In the case of well water, the total V concentration ranged from 2 µg/L to 10.5 µg/L. Possible sources of V are the waste fluids from batik manufacturing factories, hospitals, sewage sludge, petroleum products and decaying plants [15]. The total Fe level of the canal water ranged from 0.1 mg/L to 8.5 mg/L with an average value of 4 mg/L. The well water had a total Fe level ranging from 0.12 mg/L to 2.8 mg/L, but few wells had exceed the recommended level of 1.0 mg/L.

The study done by [6] identified biological organisms to interpret water quality status in the *Meda Ela*. The presence of aquatic fauna is an indication of long term status of water quality providing an opportunity to understand the level of pollution. Measurements of biological parameters have shown that the *Meda Ela* water is polluted at different levels from upstream to downstream. The upstream of tributary 1 identified as having good quality water because of presence of Dragonfly and Damselfly. Both these species are very sensitive to polluted water. In lower part of all tributaries, there were crustacean species, Backswimmers, Water Whirligig Beetles, Water Measures, Water Digging Beetles, Water spiders; those indicate moderate water quality. At origin of the *Meda Ela*, Chironomid Midges (Blood Worms), Gastropods species, Coliform and E.coli were observed and could be categorized as a low water quality compared with other parts of the *Meda Ela*. Presence of pollution tolerant aquatic fauna such as Dipteran larvae, round worms and other annelids with high amount of Chironomid clusters in middle and end part of the *Meda Ela* indicate very low water quality.

According to the study done by [1] along the *Meda Ela,* the pollution level during the wet season was high. Dissolved oxygen was lower at end of the canal (2 mg/L) and especially in the wet season. The total suspended solids values (wet: 36-3073 mg/L and dry: 27-532 mg/L) had exceeded the discharge limits at all locations tested along the canal. The concentration of BOD_5 (wet: 3.9- 2646 mg/L and dry: 1.5-520 mg/L) and COD (wet: 4-2766 mg/L and dry: 19-741 mg/L) increased towards the end of the *Meda Ela*, and the highest BOD_5 values were much higher than the effluent BOD_5 discharge limit in Sri Lanka (i.e. 30 mg/L). Ammonia concentrations (wet: 2.5-40 mg/L and dry: 3.5-15 mg/L) were above the inhibitory limit for fish and no fish were observed in the canal. The total phosphorus (wet: 0.1-2.75 mg/L and dry: 0.25-1 mg/L) and phosphate concentrations (wet: 0.5-5 mg/L and dry: 0.5-3 mg/L) were high towards the end of the canal. Also, the same study shows that faecal coliform amount is very high in the *Meda Ela* in both wet (58-5400 counts/100 ml) and dry (0-3400 counts/100 ml) seasons.

[18] and [16] identified DO variation along the *Meda Ela* as a good indicator to monitor the impact of urban pollution in the *Meda Ela*. DO results for *Meda Ela* show how a natural stream lost its characters due to urban and domestic activities in the catchment area. At some sampling points, the DO concentrations have reduced to alarming rates. The critical area of pollution in the *Meda-Ela* can be identified from middle area where the DO concentration below 1 mg/L [16]. These areas are the most sensitive due to high urban activities that include the main bus stand, the base hospital and the market [17]. It is a very positive aspect to observe that turbulence and aeration through spills in latter part of the *Meda-Ela* will help in regaining the DO concentration of the stream before it reaches the *Mahaweli* River.

Cost due to water pollution in *Meda Ela*

According to the disease incidents, 13% of the families have got affected by dengue and *Chickengunya* fever during 2009 to 2010 period and the mean health cost was Rs.453.00 per incident per patient. People are aware that the canal provides an ideal breeding ground for mosquitoes due to stagnating water during dry weather flow conditions. This condition is further aggravated due to solid waste disposal and wastewater discharge. People in the area have got used to mosquito nets, repellent coils, repellent mats and fans to protect from mosquito bites. Monthly average cost for mosquito repellents was Rs.162.00 per family. Since the canal is the drainage path of the whole *Kandy* city area, flash floods after heavy rains are very common. It was revealed that 30% of the households that live closer to the canal are affected by intermittent floods and cost to protect from floods is Rs.450.00 family/year [5].

Problems to community due to *Meda Ela* pollution

Nearly 54% of the people, almost all the households near the bank of the canal, reported that they suffer from bad odour and another 28% said that they feel a strong odour during dry season [5]. Also 13% of people were affected by water born disease such as *Dengue* and *Chickengunya*. It was also revealed that 30% of the households that live closer to the canal are affected by intermittent floods.

Major stakeholders and their relationship with each other

[5] Identified KMC, Central Provincial Council (CPC), *Kandy* Divisional Secretariat (KDS), Gangawata Korale Pradeshiya Sabha (GKPS), UDA, Central Environmental Authority (CEA), NWS&DB, Irrigation Department (ID), commercial establishments, individuals and residents who live in the catchment of this canal as major stakeholders. In addition, GKPS, CPC, KDS and District Secretariats of *Kandy* are collaborating with the KMC in granting permission for constructing buildings, houses, etc. They are also providing approvals for management and development activities related to water bodies in the area. Other government related institutions such as ID, CEA and UDA are also involved in water related development activities in the area. Most of the residential population and the business community neglect their social responsibility towards proper waste disposal and wastewater management. Hence, these categories can be identified as key polluters.

The CEA and the HD of the KMC are responsible for monitoring wastewater discharge into the *Meda Ela* from the hospital in *Kandy*. The NWS&DB is responsible for supply of safe drinking water and proper drainage facilities. The HD is responsible for the cleanliness of all roads, drains, markets and the environment. Also implementing the health policy, monitoring effluent discharge of small scale operations (markets, hospitals and restaurants), creating public awareness regarding health issues and taking legal action are all responsibilities of the HD. PHI is the ground level implementing officers of all activities related to the HD. Legal actions for non-compliance can be taken by the PHI, but actions against polluters, particularly government establishments (ex. hospital) are a challenge for the PHI.

The KMC is responsible for the collection and disposal of solid waste and it consists of different departments responsible for maintaining different aspects of the *Kandy* city. Local government acts, municipal and urban councils' ordinances, the Pradesha Shaba act, Land Development Ordinances (No. 19 of 1935, No. 3 of 1946), the Land Acquisition Act and the Crown Land Ordinance (No.8 of 1947, 9 of 1947 and 13 of 1949) are prescribed in the government policy for land development and rehabilitation, and to maintain the reservation area of the *Meda Ela* [10].

The UDA Act, in Schedule IV (Form E-Regulation 18) also provides for the reservation of waterways. The reservation from the edge of the high water level of the *Meda Ela* from *Kandy* Lake to *Heerassagala* junction, and *Heerassagala* junction to *Mahawelli* river is three and six metres, respectively [10]. In spite of these limits and a large number of local and national government agencies responsible, houses have been constructed and some have expanded into the reservation of the *Meda Ela*. A reservation area is maintained to protect the canal and facilitate the flow of water, particularly during floods. It is also meant as protection from discharge of wastewater and dumping of solid waste.

As illustrates in Figure 4, high number of departments, institutes and authorities are responsible for maintaining and protecting all water bodies including the *Meda Ela*. Lack of good coordination, poor attitudes of offices, political influence, power relation, friendships etc. among those, govern continuous pollution of the *Meda Ela*.

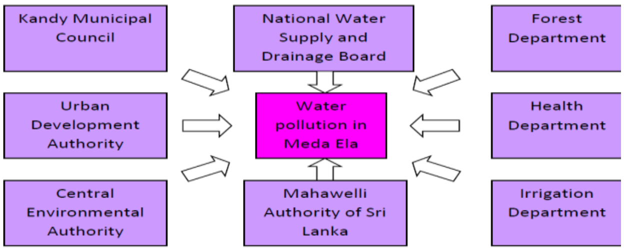

Figure 4: Institutions and departments linked to the *Meda Ela*.

As illustrates in Table 2, the KMC and residents live along the canal bank have a greater responsibility towards controlling water pollution. Though there are different social organizations in every Grama Niladhari (village level administrative unit) Division and high number of research have been carried out by Universities and government organizations, no collective actions have been taken to reduce the pollutant loading to the *Meda Ela*. The canal reservation has been encroached by people as a result of high land value in the area, political blessings and poor response of the law enforcing authorities to control these activities.

Table 2: Responsibility in controlling water pollution as ranked by the people in the *Meda Ela* Catchment [5]

Responsible institution/individual group	Group	Rank
KMC	399	1
People live along the canal	340	2
Kandy Central Market and hospitals	214	3
Other (Slaughterhouse, Hotels, etc.)	37	4
Visitors to Kandy city	34	5

Economic benefits of *Meda Ela*

The uppermost branch streams of the canal are being used by people for bathing, washing and even for drinking without purifying the water. Though the latter part of the canal is polluted, economic benefits are received by people who engage in laundry business and collect worms as fish feed.

Conclusion

The *Meda Ela* has a complex catchment area, which consists of urban centres, densely populated residential areas, hospitals, hotels, restaurants, market places, public transport destinations and a number of government and private sector institutions. As a result, the *Meda Ela* water is polluted due to solid and liquid waste generated and disposed without any treatment from point and non-point sources. According to water quality analysis along the *Meda Ela*, the pollution level in the wet season is high. DO is low at end of the canal and especially in the wet season. The concentration of BOD_5 and COD increase towards the end of the canal, and the highest BOD_5 values are much higher than the effluent BOD_5 discharge limit in Sri Lanka. Ammonia concentrations are also above the inhibitory limit for fish and no fish were observed in the canal. Total phosphorus and phosphate concentrations are high towards end of the canal and heavy loading of Phosphate promote algal blooms in downstream. The canal carries enamours amount of nitrogenous substances resulting from human and animal excreta. In spite of this, the nitrate and ammonium concentrations are found to be low indicating perhaps existing nitrogen in other forms. There is a possibility of moving nitrate through the alluvial layer of the floodplain of the *Meda Ela*. This is supported by the high concentration of nitrate found in adjacent dug wells.

The high concentration of Pb found in the canal water is a positive sign of lead pollution perhaps due to automobiles and waste petroleum products generated from service stations and garages. The concentration of Cd, V and Fe do not show a threat in the *Meda Ela* in 1987, but lack of a detailed recent study is serious barrier to understand the current situation. It has become a big barrier to identify the concentration of heavy metals in the *Meda Ela* under high and unplanned urbanization during past 30 year period. The ultimate result of this continuous pollution is vector borne diseases such as *Dengu* and *Chickengunya*, and mosquito breeding, bad odour, reduction in aesthetic appearance, flash floods and contamination of surrounding wells.

The awareness on water pollution due to waste disposal is high among the communities that live within the catchment even with good educational level. However, discharge of grey water as well as black water by people who live close to the canal is evident. Though their awareness on water pollution and possible consequences is high, they still continue to practice these activities due to lack of options for proper disposal of solid and liquid wastes. Lack of coordination among the stakeholders involved in the management of this water body has been identified as one of the major cause for unregulated activities that seriously affect the water quality. Hence, community and other stakeholders should get together in order to develop an efficient and effective management plan to safeguard the *Meda Ela* stream and its catchment area.

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Comprehensive Risk Assessment and Risk Management is Effective Tool of Consistently Ensuring the Safe Drinking Water in Colombo Metropolitan Area and Their Suburbs

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Abstract: Colombo is the largest economic and tourist attractive city in Sri Lanka, situated geographically at 127 degree 30' E longitude and 37 degree 00' N latitude. The city is spread over 37.3 km2 and its present population is 5.6 million in Colombo district .City administration is governed by Colombo Municipal Council with sub units in suburbs of Colombo.

The drinking water supply is managed by the National Water Supply & Drainage Board (NWS&DB). It's responsible for operating water treatment plants, safe storage and distribution of drinking water to consumers.

Quality of the drinking water is customer satisfaction, therefore to ensure the customer satisfaction, not only the treated water quality – SLS 614: 2013 – but also raw water quality has to be monitored within SLS 722: 1985.KelaniRiver is the main drinking water source for 80 % population from Colombo and this river provides 700,000 m3 /day raw water to the water treatment plants in Ambatale and Biyagama.

NWS&DB continue monitoring the raw water quality of Kelani river taking samples of selected locations to test selected water quality parameters.The purpose of testing the water quality in production centres and distribution system is to supply good quality drinking water achieving economic development with a healthy nation. Therefore NWS&DB has laboratory network in Colombo district to test physical, chemical and microbiological parameters in potable water according to SLS 614: 2013. Around 600 numbers of samples are taken monthly from distribution system to ensure microbiological quality of the drinking water.

Water quality monitoring is reactive attitude rather than preventive,field investigation revealed that any action and activity that is required to prevent or eliminate hazards.Therefore risk assessment, risk management and control measures are required to ensure the quality of drinking water to achieve health base targets..

Keywords: Control Measures, Hazards, Risk Assessment, Risk management

1. Introduction

Safe drinking water is a fundamental requirement and internationally accepted human right. The present approach for achieving safety of water is end product testing. This is to test water intake, after treatment at the water treatment plant, within the distribution system and at the consumers point. This traditional method has the shortcomings in a water supply system such as too little results and too late for preventive action and traditional method is reactive rather than preventive. The Water Safety Plan approach provides a management tool for improving safety of water and adopting multiple barriers risk management approaches using control measures. The final outcome is continuous safety and quality assurance of drinking water.

Colombo is the largest economic and tourist attractive city in Sri Lanka, the city is spread over 37.3 km2 and its present population is 5.6 million in Colombo district . The drinking water supply is managed by the National Water Supply & Drainage Board (NWS&DB). It's responsible for operating water treatment plants, safe storage and distribution of drinking water to consumers. Kelani river is the main drinking water source for 80% population in Colombo and the river provides 700,000 m3 / day raw water to the water treatment plants in Ambatale and Biyagama..

2. Hypothesis

2.1 Water Quality Monitoring

In order to maintain the raw water quality within the SLS 722: 1985 limits regular monitoring of water quality is carried out by NWSDB and Central Environmental Authority (CEA). As Kelani river is the main drinking water source for 80 % population from Colombo, NWSDB and CEA continue monitoring the water quality of Kelani river since 2003 taking samples of selected locations as elaborated table 1.

Table 1: Regular sampling locations of Kelani River Basin by NWSDB and CEA

Sampling Location NWS&DB under Pavithra Ganga programme	Sampling Location CEA under Pavithra Ganga programme
Victoria bridge	Japanese friendship bridge
RaggahawatteEla (Tributeriey)	RaggahawatteEla (Tributeriey)
MahaEla (Tributeriey)	MahaEla (Tributeriey)
PusseliOya (Tributeriey)	PusseliOya (Tributeriey)
Hanwella Bridge	Hanwella Bridge
PugodaEla (Tributeriey)	PugodaEla (Tributeriey)
Pugoda Ferry	Pugoda Ferry
WakOya (Tributeriey)	WakOya (Tributeriey)
Seethawake Ferry	Seethawake Ferry
Thalduwa Bridge	Thalduwa Bridge
Kaduwela bridge	Kaduwela bridge
Welivita	Welivita
Ambatale Intake	

Given below the water quality parameters tested by CEA and NWSDB monthly basis.
pH, Electrical Conductivity, Turbidity, Temperature
Dissolved Oxygen
Chemical Oxygen Demand
Biochemical Oxygen Demand
Phosphate, Nitrate, Chloride
Dissolved Lead and dissolved Chromium
Microbiological parameter such as Total Coliform and Feacal Coliform (E.coli)
Heavy metals Pb and Cr.
In addition, places where industrial zones discharges waste water such as downstream canal from Seethawaka Industrial Zone" and "Biyagama Industrial Zone" effluent discharge is checked by NWSDB.
Seetawaka – pH, Dissolve Oxygen (DO), colour on daily basis and Total Suspended Solid (TSS), Biochemical Oxygen Demand (BOD) & Chemical Oxygen Demand (COD) weekly basis.
Biyagama – TSS, BOD, COD &pH.monthly basis
Existing mechanism for monitoring Kelaniriver can be further strengthened ensuring maintaining the raw water quality SLS 722:1985 limits by restoring places where the water quality is deteriorated identified bywater quality monitoring.
Less than 300 words abstract should be provided as indicated.

2.2 Water quality monitoring in Water intakes

The purpose of testing the water quality in two intakes is to supply good quality drinking water achieving economic development with a healthy nation. Water we drink must satisfy criteria such as free from pathogens, injurious chemical and must be aesthetically satisfactory. Therefore NWSDB tests for Physical, microbiological and Chemical parameters in treated/potable water according to SLS 614: 2013 Ambathale and Biyagama are the main two water treatment plants in the Kelani river with capacities of 540,000 m^3/day and 180,00m^3/day respectively. Water quality testing of Kelaniriver in mentioned two intakes is carried out by the NWSDB laboratories on daily basis to ensure providing safe drinking water to more than 1 million people in the country.

At present Biyagama and Ambathale water treatment plants have online analysis of water quality parameters such as pH, turbidity, Dissolved Oxygen, Residual Chlorine (RCl), Aluminiumcolour and conductivity. Apart from that selected physical and chemical parameters were monitored hourly basis by the process control laboratories in our water treatment plant.

2.3 Random water quality monitoring by NWSDB

Four laboratories such as Kadawatha, Rathmalana(Western South), Ambathale and Kalatuwawa laboratories are dedicated to monitor water quality of the distribution network on the basis of population served.

Ambatale Laboratory	200 samples/month
Kalatuwawa Laboratory	50 samples/month
Western South Lab (Ratmalana)	80 samples /month
Western North Lab (Kadawatha)	80 samples/month

Apart from that heavy metal – Lead (Pb), Cadmium (Cd), Chromium (Cr) and Arsenic (As) analysis were carried out in our clear water reservoirs four times per month for Ambatale and the other places once in a month. Heavy metal contamination is checked in distribution system to ensure the quality of drinking water. Past analytical reports revealed that the heavy metals were not detected or below the detection limits in drinking water.

Monitoring is not the only tool to assure the safety of drinking water but also identification of hazards and assesses risks from catchment to consumer and prioritizes the risk and focuses on management those hazardous events within the highest risk and manage the risks by using control measures are very important to provide safe drinking water.

3. Methodology

Hazards and hazardous events are identified through visual observations, water sampling and testing and previous studies that are evaluated throughout the water supply systems. Possible hazards are identified from catchment to consumer and analyzed the variation of water quality parameters throughout the water supply system. These identified hazards are evaluated as low, medium and high risks using semi quantitative matrix and the same time possible control measures are implemented.

Table 2: Risk Assessment Table

Likelihood	Public health	Severity				
		Impact on public health	Major regulatory impact	Moderate aesthetic impact	Minor compliance impact	No impact or not detect
		5	4	3	2	1
Once per day or more	5	E	E	VH	H	M
Once per week	4	E	VH	H	M	L
Once per month	3	VH	H	M	M	L
Once per year	2	H	M	M	M	L
Once every 5 years or less	1	M	L	L	L	L

4. Results

All identified hazardous events are mapped in the catchment of the Kelani river and same time industries are categorized as type A, B and C. The type A industries are high polluting industries will not be permitted to be located upstream of drinking water abstraction point. The effluents from type A industries should not be discharged upstream directly or indirectly. The effluents from these industries should be treated to the designated national effluent standards and discharged to water bodies, which are not used as drinking water source or downstream of the last drinking water abstraction point or to the marine environment. Type B industries will be permitted to be located upstream of drinking water abstraction point. Industries and their distribution in Kelani river basin are given below.

Table 03: Industries in Kelani river bas

District	DSD	A	B	C	Total
Colombo	Homagama	213	205	311	729
	Hanwella	72	193	111	376
	Colombo	138	84	15	237
	Kolonnawa	87	59	90	236
	Kaduwela	268	255	136	659
	Thibirigasyaya	51	41	3	95
	Kotte	45	63	3	111
	Maharagama	77	85	111	273
	Padukka	111	82	131	324
	Kesbewa	113	139	331	583
Gampaha	Mahara	79	132	110	321
	Wattala	131	53	55	239
	Biyagama	116	130	178	424
	Dompe	94	73	129	296
	Kelaniya	83	50	75	208
	Ja- Ela	163	104	63	330
	Gampola	86	79	121	286
	Attanagalla	70	103	21	194
Kegalle	Ruwanwella	53	80	73	206
	Dehiovita	35	41	74	150
	Yatiyantota	31	29	42	102
	Deraniyagala	15	20	15	50
	Warakapola	54	70	105	229
	Galigamuwa	44	52	104	200
	Aranayaka	16	49	52	117
	Bulathkohupitiya	11	26	24	61
	Kegalle	47	70	114	231
Ratnapura	Eheliyagoda	31	49	84	164
	Kuruwita	49	51	137	237
	Ibulpe	17	32	113	162
	Ratnapura	30	53	117	200
NuwraEliya	Ambagamuwa	37	14	113	164
	NuwraEliya	61	13	59	133
Kandy	Gaga IhalaKorale	15	14	41	70
	PasbageKorale	25	27	23	75
Grand Total		2705	2827	3473	9005

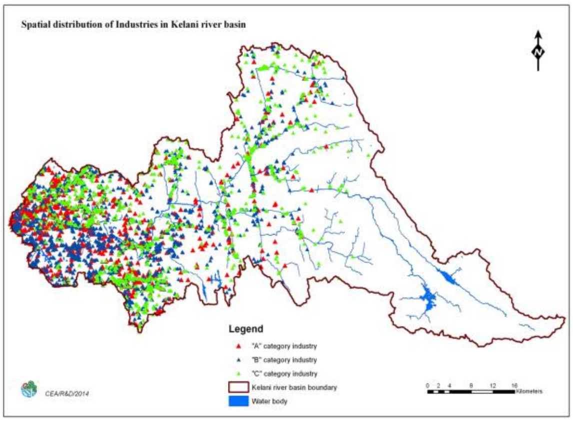

Figure 1: Distribution of type A, B and C industries in Kelani river basin

Table 4: Hazard Analysis

Process Step	Hazard	Hazardous Event	Risk Band	Control Measures	Risk Band
Kelani river catchment	Chemical	Oil & Grease contamination through Pattiwila canal	Medium	No current control measure / Propose to construct Wetland	Medium
Kelani river catchment	Microbiological	Contamination through storm water runoff during monsoon	High	No current control measure	High
Kelani river catchment	Physical, Chemical and Microbiological	Industrial wastewater discharge from two main industrial zones	High	Improvement of the existing treatment plants and interceptor sewerage line	High
Secondary Chlorination in Maharagama	Microbiological	Under dosing / No proper dosing system	Very High	Installation of new chlorinator	Medium
Clear water reservoir in Church Hill	Microbiological	Contamination through air vent	Medium	Fix the suitable mesh for the air vent	Low
Distribution system in Elvitigala Flats	Microbiological	Cross contamination with sewerage line	Very High	Replacing the pipeline	Low
Fecal contamination in Mattegoda distribution system	Microbiological	Stagnation of water	Very High	Installation and proper operation of the washout	Medium

5. Conclusion

Water quality monitoring and water quality surveillances are required to conform the product is to fit for human consumption but comprehensive risk assessment and risk management is required to assure the safety of the drinking water. Control measures are placed at each process step for each hazardous event. These control measures taken to prevent further contamination, removing hazards or inactivating the pathogens and monitoring the quality of water during distribution.

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A Study on Water Management Strategies Practiced in Healthcare Facilities: A Literature Review

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Abstract: Water management in facilities can be simply explained as consuming water effectively without obstructing the functions of the facility. Healthcare facilities are one of the major types of facilities which consume a huge amount of water for their daily operations. Therefore, water management in healthcare facilities should be given a special attention in order to reduce the operational cost of the facility while contributing to sustainable development of the country. Various strategies can be practiced for water management in healthcare facilities and the understanding on current water management practices is important to take necessary measures to improve the current practices. The aim of this research was to investigate the current water management practices of healthcare facilities. Hence, a comprehensive literature review was carried out to identify the water management practices of healthcare facilities. The identified strategies could be categorised in to two as general water management strategies and strategies which are specific to healthcare facilities. Application of these strategies enable the management of healthcare facilities to minimize the drawbacks of their current water management practices and reduce the water consumption of their facilities by a considerable amount.

Keywords: Healthcare Facilities, Strategies, Water Management

1.0 Introduction

Water is essential for human life while being an indispensable resource for the economy, and also plays a fundamental role in the climate regulation cycle (Euro stat, 2012). However, less than 3% of the globe's water is fresh, but 2.5% out of that is frozen. As a result humanity has had to rely on remaining 0.5% to fulfil its fresh water needs (World Business Council for Sustainable Development [WBCSD], 2005).

Agriculture is the main user of water by far and irrigated agriculture accounts for 70% of water withdrawals in the world (World Water Assessment Programme [WWAP], 2009). Industries are the second largest user of water after the agriculture and anyway the amount of water used varies widely from one type of industry to another (World Business Council for Sustainable Development [WBCSD], 2005).Therefore, domestic demand for pipe born water (city water) is improving significantly creating tight competition for city water between industries and domestic consumption.

Freshwater abstractions from public water supplies, irrigation, industrial processes and hydro power plants apply a major pressure on water resources with significant implications for issues of quantity and quality of water resources (Organization for Economic Co-operation and Development [OECD], 2005).

Healthcare sector is one of the industries that demands higher amount of water for their daily operations. And total water consumption of a hospital is divided among different functions of hospitals (US Department of Energy [DOE], 2011). Hospitals have to focus on water efficiency and conservation measures in order to ensure the sustainable use of water in the hospital (Smith, 2007). Accordingly, the aim of this paper is to investigate the water management strategies currently practiced in healthcare facilities in the world.

2.0 Water Management

Buildings can be classified into major five categories as residential, commercial, public, industrial and agricultural buildings and each type of building has different water consumption levels (Bio Intelligent Service of European Commission [BIS], 2009). However, there are primary water use purposes such as toilets, showers, wash basins, kitchens, HVAC systems and landscaping that are common to any type of building (Arab Forum for Environment & Development [AFED], 2006). Although water consuming activities often remain similar, overall water consumption is greatly influenced by various factors such as building features/infrastructure, geographical location, appliances installed and human activities in the building (Groundfos Commercial Building Services [GCBS], 2012).

Water management can be described as the process of planning, monitoring and controlling of facility water use and water quality in order to obtain an optimum use from available water. According to the National Cleaner Production Centre [NCPC] of Sri Lanka (2012), water management concept has major two aspects as water conservation and water efficiency. It further states that water efficiency focuses on achieving the same result with the minimum amount of water usage while water conservation directs towards reducing the wastage of water. Effective water management in the Commercial, Industrial and Institutional (CII) sector can have a tremendous impact on overall water consumption and deliver a range of economic and environmental benefits (Cohen Cohen, Ortez & Pinkstaff, 2009).

There are few major principles that should be considered when implementing a water management program. They can be explained briefly as follows;

- Basic principle of a good water management is to manage available water resources among the functions effectively satisfying the water demand and the water quality (DOE, 2011).
- Successful water management consider the technical side such as installing efficient fixtures and making maintenance modifications as well as the human side such as changing long standing behaviors and expectations (United States General Services Administration [GSA], 1999).
- According to GSA (1999) water management strategies can be categorized into major three general areas as;
    - Reducing losses
    - Reducing the overall amount of water used
    - Reusing water that would otherwise be discarded
- Water efficiency implementation at work starts with understanding a facility's water using processes and it is important to understand the facility water use and water use patterns before developing a water management plan (EPA, 2012).
- The management of the building must be committed to water management if they want to convince occupants that their actions make a positive difference (GSA, 1999).

3.0 Water Management in Healthcare Facilities

Healthcare facilities can be categorized into public buildings or commercial buildings. However, several core and non-core functions are performed in healthcare facilities and water is used in different amounts for all those functions. As common to any facility, domestic water uses such as drinking, washing and personal hygiene can be identified healthcare facilities as well. Moreover, there are number of unique water-using activities in healthcare facilities such as;

- Vacuum pump systems
- Medical air and compressor equipment
- Sterilizers and central sterile operations
- Laboratory hood scrubbers
- X-ray equipment and film developers
- Water-treatment systems for kidney dialysis and laboratory water
- Therapeutic baths and treatments

According to New Hampshire Department of Environmental Services [DES] (2013) these unique functions including laundry and kitchen account for a large percentage of total water consumption of healthcare facilities.

Since large hospitals employ several water use functions, water use effectively and efficiently is a major requirement in medical facilities and water management practices become more important. Hospitals and medical facilities have obtained significant operating cost and energy savings by instituting such water saving measures (North Carolina Department of Environment and Natural Resources [NCDENR], 2002).

But there is a major constrain when applying water saving measures to hospitals and it is that need for maintaining quality of water. In hospitals there is a higher risk for water to get contaminated and people to get infected by water (Angelbeck, Ortolano, Canonica, & Cervia, 2006). Therefore management need to be careful much when implementing water reuse and recycling strategies. However there is no single approach or solution for dealing with water quality issues within healthcare facilities (Noonan & Garnys, 2014). Therefore, simply available quality water should be managed properly among functions that demand quality water while distributing low quality water among functions that do not demand quality water (Australian Government Department of the Environment and Heritage [DEH], 2006).

3.1 Common Water Management Strategies

There is a wide variety of common water management strategies available for facility managers to choose from, in relation to every use of water in a building. Some strategies simply alter the water use habits of building occupants while some others change the way of water systems are operated and maintained (GSA, 1999).

3.1.1 Water Management Plan (WMP)

A water management plan (WMP) is an essential tool to achieve an effective and sustainable outcome in relation to water efficiency (DEH, 2006). Facility water management plan should consider conducting facility surveys, water use monitoring, determining performance targets, identifying saving options, engaging building users and allocating resources (AFED, 2006). General outline of a WMP is given in table 1.

Table 1: General Outline of a WMP

Source: Water efficiency guide - Department of Environment and Heritage (2006)

Key areas	Details
Initial plan	▪ Purpose and scope ▪ Policy and principles ▪ Goals and objectives
Baseline data and performance targets	▪ Major water systems and equipment ▪ Water supply – metering and tariffs ▪ Consumption history ▪ End users and users ▪ Consumption drivers and KPIs ▪ Benchmarks and targets
Water saving measures	▪ Technical efficiency measures ▪ Behavioral measure ▪ Conservation measures
Management performance	▪ Management team / committee ▪ Management action plan ▪ Assessment criteria and scoring ▪ Risk assessment
Performance reporting	▪ Recipients ▪ Responsibility and accountabilities ▪ Scope, measures and targets ▪ Frequency

3.1.2 Water Use Monitoring and Education

Building operators can understand and manage facility water use by routinely monitoring facility water use through existing water meters and metering allows a facility to quickly find and fix leaks or other water wastages (EPA, 2012). This strategy has several functions to ensure an efficient water use in a facility and just installing meters at the required locations may not give a successful outcome. Those functions are;

- Choosing what to meter / sub-meter
- Installing meters
- Maintaining meters
- Take meter readings
- Recording metered data to track water use

Rectifying such leaks quite often provides the best return on investment of all water saving measures and it should be performed before going for any other water efficiency measures (DEH, 2006). Following methods can be used to detect the leaks and to avoid leaks;

1) Maintain records of water meters readings and pay close attention to variations (EPA, 2012).
2) Implement preventive maintenance to make sure the replacement of problematic items (DEH, 2006).
3) Perform a water assessment or audit of the facility at least once every four years (EPA, 2012).
4) Make water leak reporting as a responsibility of facility staff and encourage visitors as well (DEH, 2006).

User education is a cost-effective way to enhance a facility's water-efficiency efforts which can result in significant water savings (EPA, 2012). There are several measures that can be taken to educate employees and other building occupants on water savings;

1) Communicate the water management program of the facility to employees.
2) Notice monthly water use figures to building occupants so that are informed about the facility's progress.
3) Create point-of-use reminders to encourage positive behaviours.
4) Train and instruct relevant staffs to ensure proper implementation of any new or revised procedures involving water management.
5) Provide water efficiency tips to regular water consumers or consuming functions.

3.1.3 Water Management in Different Functional Areas

3.1.3.1 Washrooms and toilets

Every washrooms and toilets of buildings have at least some sanitary fixtures or equipment including water closets, urinals, faucets and showerheads etc... Following main water management strategies can be identified as commonly in relation to sanitary accessories.

1) Install dual or variable flush systems for water closets and commodes (AFED, 2006).
2) Improve the flush systems with modern low volume cisterns and flush systems (Environment Agency of UK, 2007).
3) Install manual flush or sensor operated flush for urinals (AFED, 2006).
4) Improve the faucets and showers with high efficiency models with aerators (Cohen, Ortez, & Pinkstaff, 2009).
5) Install sensor operated faucets to avoid the water wastage when opening and closing a manual faucet (Texas Water Development Board, 2011).

3.1.3.2 Commercial kitchens

Several other commercial or institutional sectors including hospitals, offices, schools, and hotels also have substantial kitchen water use that accounts for as much as 10% to 15% of the facility's total water use (EPA, 2012).

Hot water boilers supply hot water to different equipments and accessories of the kitchen such as faucets, pre-rinse spray valve and dishwasher and conservation mainly should be established at the hot water using equipment and accessories (Fisher-Nickel Incorporated [FNI], 2010). Further continuous recirculation system is water efficient where hot water is circulated through the system and unused water is returned back to the heater.

Install a pre-rinse spray valve in order to reduce water and chemical consumption of dishwashers in commercial kitchens. Pre rinsing dishes with the use of high-pressure nozzles with a hand-held trigger can result in substantial water savings of the dishwasher (AFED, 2006). Further it states that using a dishwasher whenever possible for washing dishes itself is far more water efficient than manual washing.

3.1.3.3 Laundries

Laundries are another high water use area especially for hospitals, hotels and commercial linen services (Water Use and Conservation Bureau [WUCB], 1999). There are both technical and behavioural measures to effectively manage the water consumption of commercial laundries;

1) Wash full loads only by adjusting laundry schedules or washing clothes only when it is necessary (AFED, 2006).
2) Change the existing laundry methods / chemicals into new methods / chemicals that require fewer wash and rinse steps (WUCB, 1999).
3) Install a rinse water reclamation system to reuse discharged rinse water or Use batch-washers that use less water since they reuse rinse water for the first rinse (WUCB, 1999).

3.1.3.4 Mechanical systems

HVAC system, boilers and fire protection systems can be identified as mechanical systems that consume water for their operations.

Cooling towers are the major consumer of water of a HVAC system which usually account for up to 30-40% of a commercial or public building's water use (Chandrathilaka & Fernando, 2011). As the major consumer of water, the building HVAC system should be an obvious target for water conservation efforts where significant water savings can be obtained

(Weimar & Browning, 2010). Measures that can be taken to ensure a proper management of water in the HVAC system as follows;

1) Install water meters on the makeup water and blow down line of cooling tower systems and monitor (Weimar & Browning, 2010).
2) Continuously treat cooling tower water to prevent forming of scaling or use softened makeup water to control bleeding rate (Weimar & Browning, 2010).
3) Use treated air handler condensate water, grey water or rain water as cooling tower makeup water.

Water consumption rates of boiler systems differs depending upon the size of the system, the amount of steam used and the amount of condensate return (WUCB, 1999). There are few major water efficiency and conservation measures that can be applied to boilers;

1) Take condensate water return to the boiler and modify the system if this option is not available with the system.
2) Install an automatic controller to turn off the unit when steam is not in use or not required.
3) Install an automatic blow down control for boilers to better manage the boiler makeup water requirement.

Fire hydrant system and sprinkler system can be considered as water using fire protection systems. As major water conservation strategy, responsible personnel should maintain a proper procedure for the immediate detection of leakages and fix them. In addition to that rain water or raw water can be used to feed the fire protection systems instead of using pipe born water.

3.1.3.5 Pools, spas and ponds

Properly managing evaporation, leakages, presence of a fountain or waterfall and maintenance requirements will result in considerable water savings of pools, spas and ponds. Several major strategies to manage water consumption can be identified as follows;

- Sub-metering the water supply / make up line especially for the pools and spas.
- Using rain water or raw water for water ponds in the hospital premises.
- Taking immediate steps to the leakages of pools, spas or ponds.
- Shutting down or removing unnecessary fountains or waterfalls that causes for aeration lose and significant amount of vaporization.
- Using the pool water for several cycles by cleaning the water through a proper filtering system.
- Reusing drain water from pools, spas and ponds for different purposes such as landscaping.

3.1.4 Rain Water Harvesting, Waste Water Reuse and Recycling

Gray water discharged from bathroom sinks, showers and clothes washing machines is possible to reuse and recycle for some other purposes (GSA, 1999). Before reusing grey water, it requires a treatment including biological trickle filters, clarifier, self-cleaning filtration, UV filter disinfection and chemical treatment via a water monitoring system (DEH, 2006). Such treated grey water can be used for flushing in toilets, cooling tower makeup, floor or toilet cleaning and gardening (AFED, 2006).

Further some amount of overall water consumption may potentially be replaced by rainwater in commercial buildings. Systematically treated rain water can also be used for above mentioned water use functions.

3.1.5 Outdoor Water Use

If the facility maintains any landscaping, then exterior water use management should be an important part of the overall water management programme (WUCB, 1999). Landscape water use is largely dependent upon climate, plant type, and an irrigation system's efficiency (EPA, 2012). Therefore those factors should be concerned when effectively managing landscaping water use.

Following strategies and guidelines on efficient landscape irrigation can provide significant and immediate water savings (WUCB, 1999);

1) Watering plants early in the morning in order to minimize the evaporation of water.
2) Making sure sprinklers have been adjusted accurately towards landscape plants.
3) Adjusting sprinklers and other water delivery devices to concentrate water at the root area of plants.
4) Refraining from watering when it's windy or raining.

5) Watering deeply and less frequently instead of lightly every day.
6) Using a hose with an attached nozzle or spray head with an automatic shutoff option to avoid water waste.

Following maintenance, retrofit, and replacement options can provide additional landscape water savings (WUCB, 1999);

1) Planting low water use trees, shrubs and ground covers instead of high water use turf grass.
2) Ensuring that each type of plant material receives only the amount of water it needs.
3) Use water wise plants and a drip irrigation system instead of sprinklers for small turf (less than 10 feet wide) areas.
4) Separately monitor the volume of water applied to the landscape and carefully regulate the water delivered to each zone of the irrigation system.
5) Maintain a schedule for watering times and durations to feed plants with water only when required.
6) Inspect irrigation systems regularly and replace or repair broken sprinkler heads, broken or damaged components.

3.2 Specific Water Management Strategies for Hospitals

There are water saving technologies unique to medical facilities which should be considered for managing water, along with the water saving practices common to any facility (Arizona Department of Water Resources [ADWR], 2009). Between 4 – 15% of the water consumption of a hospital is used for medical equipment that is all vital to a well-functioning healthcare facility. Many of these machines run throughout the day and night and use large quantities of water (Cohen et al., 2009).

3.2.1 Medical Equipment with Single Pass Cooling

There are several medical equipments with single pass cooling systems as well which are inefficient. Such equipments are ice machines, film processing X-ray machines, degreasers, hydraulic equipment, condensers, air compressors and vacuum pumps (NCDENR, 2002). Water management strategies for such equipment are as follows;

1) Every time shut off water of single passing cooling systems when they are not in use (NCDENR, 2002).
2) Install automatic valves on single pass cooling water film processing or X-ray equipment to stop water flow when equipment is not in use. E.g. Use temperature control valves (NCDENR, 2002).
3) Improve existing single pass cooling water film processing equipment with closed loop cooling systems (Cohen et al., 2009)
4) Replace original liquid ring vacuum pumps which continuously discharge and replace fresh water, with water recirculation system.
5) Replace old technological equipment and machines with new technological machines that do not consume water for cooling or film processing.

3.2.2 Steam Sterilizers

Steam sterilizers are commonly used in laboratory or medical settings to disinfect containers, trays and other instruments. Steam is applied under pressure to destroy bacteria and other impurities in the sterilizer (DOE, 2011).
In order to save water in sterilizers mainly;

1) Retrofitting steam sterilizer with a water saving device which monitors the drain water temperature and applies cold water only when necessary (Cohen et al., 2009 & DOE, 2011).
2) Collect the steam released by the steam trap of the large steam sterilizers and send it back to the boiler through the condensate return line to the boiler (East Bay Municipal Utility District [EBMUD], 2008).

3.2.3 Distil Water Plant

Distil water plants / distillers produce non-contaminant and non-infectious for laboratory usages (Acmas Technocracy Limited, 2010). A distil water plant has a process as in the chamber water boiled into water vapour and condensate the vapour in order to return back into liquid state. However a considerable amount of cool water is spent on condensation activity of the plant. If it is a single pass cooling system, need to change that to a closed loop system or cooled by passing through a chilled water coil (NCDENR, 2002).

4.0 Conclusion and Future Research Agenda

Healthcare facilities have a greater responsibility for properly managing their own water consumption and contributing to minimize the crisis for water. Therefore, water management strategies are required for healthcare facilities in order to ensure an effective and efficient usage of water. Water management strategies can be identified in few major categories as developing and acting on a water management plan, water use monitoring and education, water management in different functional areas, water use by mechanical equipment and water management in medical activities. The identified factors were categorised in to general water management strategies that can be applied to any type of facility and strategies which are specific to healthcare facilities.

According to Sri Lanka National Water Partnership [SLNWP] (2010), Sri Lanka is seemingly having adequate water resources, but there are seasonal and regional variations that lead to periodic water crises. Further, as per the Annual Report of Central Bank of Sri Lanka (2012), the demand for pipe born water has increased significantly in line with rapid expansion in commercial activities, industrial activities and urbanization. Hence, water management practices should be incorporated in such sectors in Sri Lanka as well.

Though adequate literature on water management practices could be found in global context, the available literature on water management in healthcare facilities is hardly found. Therefore, the article motivates an agenda for future research that advocates the evaluation of current water management practices of healthcare facilities in Sri Lanka.

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SECM/15/041

Concept for Separation of Different Wastewater Streams in order to Minimize Emerging Contaminants in Drinking Water

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Abstract: In the recent past trace levels of many contaminants were reported in drinking water. Substances that are resistant to bacterial degradation will flow along with treated effluent and will end up in inland surface waters from which raw water is extracted for public water supply systems. In certain extreme situations, presence of excessive concentrations of these contaminants can inhibit bacterial degradation of biological wastewater treatment processes. Therefore there is a natural tendency for build-up of these contaminants in water sources. The emerging contaminants found in drinking water are heavy metals and hazardous substances that flow along with industrial effluent, hospital effluent and agricultural runoff.

In order to remove these substances that are resistant to biological wastewater treatment, it is recommended to separate them in concentrated form by a separate collection system without allowing to mix with other wastewater streams.

For wastewater other than domestic nature containing heavy metals, residual dies etc. and hazardous wastewater generated from hospitals such as radioactive iodine treatment for cancer patients, chemicals used for X Ray processing, Amalgam used by Dentists to fill up cavities , Antibiotics, Laboratory chemicals such as Salicylic Acid, Benzoic Acid, Ethidium Bromide (used for molecular biology research), Xylene, Formalin for preserving biological specimens, etc. must be separated in concentrated form in a separate collection system without mixing with wastewater of domestic nature and disposed after treatment as hazardous wastewater. As per the "Policy on siting of high polluting Industries " it is not possible to locate industries that are categorized as Type "A" high polluting industries upstream of intakes that extract raw water to produce potable water supply as it is very difficult remove these substances by conventional water treatment techniques

Key words: Emerging contaminants, hazardous wastewater, wastewater separation

1. Introduction

In the recent past trace levels of many contaminates were reported in drinking water. Substances that are resistant to bacterial degradation will flow along with treated effluent and will end up in inland surface waters which extract raw water for public water supply. In some extreme situations, presence of excessive concentration of these contaminants can inhibit bacterial degradation of biological wastewater treatment processes. Therefore there is a natural tendency for build-up of these contaminants in water sources as treated effluent is discharged in to inland surface waters.

Substances such as methyl mercury can lead to bio magnification in the food chain. In Minamata Bay in Kiyushu Island in Japan people experienced the Minamata disease as a result of discharge of organic mercury which lead eating of contaminated sea food. DDT is also banned in most countries because it has the ability of bio magnification along the food web.

The emerging contaminants found in drinking water in Sri Lanka are heavy metals and hazardous substances that flow along with industrial effluent, hospital effluent as point sources while agricultural runoff as non-point sources.

As per the "Policy on siting of high polluting Industries" the Joint Cabinet Paper on No 3(i)/1/23(Xiii) dated 29th April 2009 by the Ministry of Water Supply & Drainage and Ministry of Environment which will not allow to locate industries that are categorized as Type "A" high polluting industries upstream of intakes that extract raw water to produce potable water supply as it is very difficult remove these substances by biological wastewater treatment processes.

In order to carryout appropriate treatment/disposal for wastewater having different characteristics, NWSDB's "Concept for wastewater separation in major institutions" was developed and adopted in year 2009.

2. Hypothesis

2.1 Waste minimization

Wastewater characteristics differ dramatically from industry to industry hence industry specific wastewater treatment plants are different from industry to industry. Significant amount of wastewater generated from many industries of domestic nature without heavy metals and toxic substances. However when wastewater of domestic nature mixed with heavy polluting streams of the industry it is very difficult to remove these substances. In higher concentrations, heavy metals and toxic substances present in wastewater may hinder the biological treatment processes. Therefore it is of paramount importance that these heavy polluting streams from the industry must be separated in concentrated form and treated by appropriate technique in order to recover, reuse these substances back in the industry or dispose as necessary without allowing to mix with low polluting wastewater streams. During retrofitting or building new industries, the industrialist should incorporate machinery and equipment that will minimize heavy polluting streams. Selection of proper raw material having low residual contaminants can also minimize pollutants in the effluent. By applying cleaner production techniques such as cascade washing or counter current washing in plating industry will minimize fresh water usage.

Once heavy polluting streams are separated in concentrated form and carryout appropriate pre-treatment is done, major portion of the wastewater generation from industries can be treated using biological wastewater treatment techniques.

2.2 Equalization/Neutralization

In order to ensure uniform wastewater characteristics entering industrial wastewater treatment plants, it is necessary to have an equalization/neutralization tank with mixer/bubble aeration to prevent settlement and to control odour. In case of some industrial wastewater of acidic or alkaline in nature require NaOH or HCl dosing to address pH correction at Equalization/Neutralisation tank.

2.3 Grease and Oil Interceptors

In order to prevent accumulation of grease/oil/fat in the sewer network and to reduce sewer cleaning frequency it is necessary to have a grease interceptors especially for Restaurants, Hotels, and Takeaway shops. However for sizing of the grease interceptors it is possible to separate wastewater from food processing area, to satisfy the requirement of grease trap needed only for that particular area and the remaining wastewater can be mixed with the effluent of the grease trap/oil interceptor.

In case of industrial wastewater having grease and oil, it may be necessary incorporate API (American Petroleum Institute), CPI (Corrugated Plate Interceptors) or DAF (Dissolved Air Floatation) or any other suitable method depending on size of oil droplet diameter [1].

Transformer coolant oils such as PCB (Polychlorinated Biphenyl) are carcinogenic hence should not be discharged in to water bodies under any circumstance.

2.4 Separation of Toxic/Inhibiting substances

It is very much important to separate toxic/radioactive/inhibiting substances being mixed with wastewater of domestic nature in order to carryout appropriate specific treatment. In hospitals the waste streams from radioactive iodine therapy, pharmaceutical wastes/antibiotics, residuals of Amalga and effluent from X Ray process consisting with heavy metals, Laboratory wastewater such as Xylene, Formalin, Ethidium Bromide, Salicylic etc. should be separated at the source as concentrated form as possible with separate collection system so that the wastewater having domestic nature can be treated by biological treatment methods without any difficulty. The heavy polluting wastewater streams have to be treated separately as hazardous wastewater and disposed appropriately.

For the wastewater emanating from the National Institute of Cancer, Maharagama, NWSDB obtained concurrence from Atomic Energy Authority (AEA) about requirements for retention time in the delay tanks in order to achieve radiation levels for alpha and beta to meet inland surface discharge standards. A battery of 6months delay tanks were constructed incorporating 3mm stainless steel jacket inside of 250mm thick concrete walls. Once capacity of these tanks reach full level, officer from AEA will close the inlet and leave for 6month, after which radiation level will be checked and effluent is directed to the sewage pumping station which intern discharge to Biological Nutrient Removal Wastewater Treatment Plant at Moratuwa/Ratmalana and final effluent is discharged vial 550m long sea outfall which is equipped with diffusers to enhance instantaneous dilution of treated effluent which has already treated up to meets inland surface discharge standards at the treatment plant.

Large quantities of antibiotics are administered to humans and animals to treat diseases and infections every year. Antibiotics are also commonly used at subtherapeutic level to livestock to prevent diseases and promote growth. Antibiotics are among the emerging micro contaminants in water because of concerns of their potential adverse effects on the ecosystem and possibly on human health. Antibiotics are likely to be released into the aquatic environment via wastewater effluent and agricultural runoff as a result of incomplete metabolism, ineffective treatment removal or improper disposal. Ching et al [2].

2.5 Wastewater treatment process selection criterion

The wastewater treatment process has to be chosen with due consideration for following aspects but not limited to;

- Influent wastewater characteristics
- Requirements of the disposal standards or ultimate end use (discharged to inland surface water body, sea outfall, irrigation, effluent re use for non-portable uses, ground water recharge etc.)
- Availability of land
- Energy consumption/recovery
- Capital cost
- Operation and maintenance cost
- Availability of skilled personal
- Tariff Structure

3. Verification

Antibiotics were categorized according to their chemical and structural properties. members of the same class of antibiotics have similar structure, act by similar mechanisms, and are likely to behave similar in the environment. More than ten antibiotic classes (aminoglycoside, ionosphere, β-lactam, macrolide, polypeptide, quinolones, sulphonamide, tetracycline, streptogramin and other) are currently in use. Among the antibiotic classes six are important in both human medicine and animal husbandry (amminoglycoside, , β-lactam, macrolide, quinolones, sulfonamide, tetracycline). Forcus of on these six antibiotic classes because of the high risks for human health if they present in water as contaminants. The cross use of these antibiotics classes both in human and animals may lead to a more rapid development of bacteria resistance towards these drugs. Ching et al [2]

According to Ching et al [2], among the β-lactam, macrolide, sulfonamide and fluoroquinolone classes, the estimated concentrations for these antibiotics in USA in untreated municipal wastewater are listed in Table 1.

Table 1: Predicted concentration of antibiotics in untreated municipal wastewater.

Antibiotic	Class	Predicted wastewater con. (ng/L)- excluding metabolism	Predicted wastewater con. (ng/L)- including metabolism
amoxicillin	β-lactam	27,000	16,000
azithromycin	macrolide	9,200	NA
sulfamethoxazole	sulfonamide	3,800	3,200
ciprofloxacin	fluoroquinolone	3,100	1,400

NA = not available

Table 2-13 to Table 2-15 of Metcalf & Eddy [4] specifies "Metal concentrations threshold of inhibitory effect on heterotrophic organisms", "Typical waste compounds produced by commercial, industrial and agricultural activities that have been classified as priority pollutants" and "Typical discharge limits for toxic constituents found in secondary effluent" respectively.

According to Wikipedia [3] dental amalgam is a liquid mercury and metal alloy mixture used to fill cavities caused by tooth decay. Low-copper amalgam commonly consists of mercury (50%), silver (~22-32%), tin (~14%), copper (~8%) and other trace metals. According to Water World Magazine [7], USEPA study shows that half the mercury enters water treatment facilities from Amalgam which is mixture of mercury and other metals. USEPA propose common sense of rules to cut down the metal discharges to water treatment facilities by 8.8 tons per year and is open for public comments and expects to finalize the standards by September 2015.

According to Water World Magazine [7], a new survey conducted by NSF International find 82% of the consumers are concerned about emerging contaminates in drinking water. Emerging contaminates refers to prescription drugs, over the counter medication, detergents, pesticides, herbicides. Although health risks associated with trace levels of these contaminates are not yet well understood, their presence in drinking water have many consumers concerns. Despite their concerns. Interesting to note though study shows only 28% correctly bring medication to pharmacists or clinic for proper disposal while others say throwing them to trash or flushing them down the toilet.

According to Wijesinghe et. al. [8] indicates that mean concentration of Cadmium and Led in Kelani river for year 2001 to 2004 have exceeded limit specified by SLS 722, "Raw water extracted for drinking purpose" as shown below,

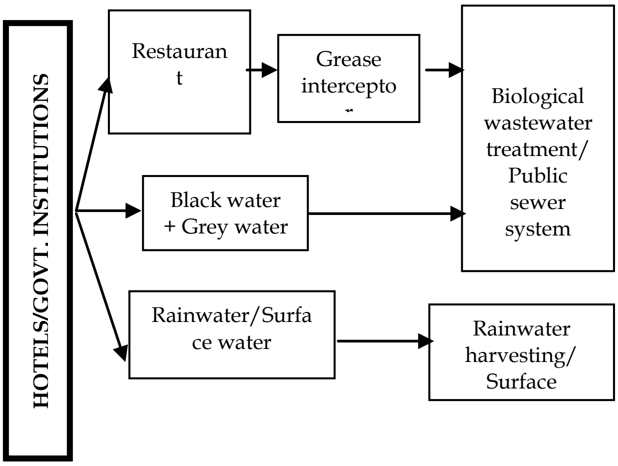
Table 2: Heavy metal concentration in Kelani River 2001 to 2004

Year	Pb		Cd	
	Mean Concentration in Kelani River (mg/l)	As per SLS 722 (mg/l)	Mean Concentration in Kelani River (mg/l)	As per SLS 722 (mg/l)
2001	0.086	0.05	0.005	0.005
2002	0.027	0.05	0.008	0.005
2003	0.155	0.05	0.004	0.005
2004	0.080	0.05	0.009	0.005

4. Conclusions

As per the NWSDB's Concept for Wastewater Separation in Major Institutions [5], wastewater having different characteristics should separated in concentrated form in separate plumbing system as follows and carryout appropriate treatment/disposal in order to control emerging contaminants in drinking water;

4.1 Wastewater from hotels and Government Institutions that producer's wastewater of domestic nature can follow concepts similar to the following;



4.2 Wastewater from hospitals could carry out following in line with the concept for wastewater separation;

- **wastewater of domestic nature, rainwater/storm water and food processing** should be carried out in line with 4.1

- **Wastewater resulting from radiation therapy treatment** should be kept for 6 months in delay tanks having 250mm thick concrete walls comprised with 3mm thick stainless steel jacket, 250mm thick concrete slab as per AEA recommendations

- **Wastewater from X Ray Processing** can be collected in containers in concentrated form in separate collection system (silver can be precipitated and separated from water phase)

- **Dental Wastewater resulting due to Amalgam** used to fill cavities. Amalgam consist of 50% Hg, 22-32% Ag, 32% Sn, 8% Cu (Ref. Wikipedia). This should be collected in containers in concentrated form in separate collecting system. This should be treat/dispose as hazardous wastewater.

- **Antibiotics/pharmaceutical wastewater** should be collected in containers in concentrated form in separate collection system. Suitable deactivation of antibiotic should be carried out (thermal, alkali/acidic, chemicals such as peroxides)

  Human exposure to pharmaceuticals through drinking water can be reduced through a combination of preventive measures, such as take-back programmes, regulations, public guidance and consumer education to encourage the proper disposal of unwanted pharmaceuticals and minimize the introduction of pharmaceuticals to the environment [9].

  With respect to conventional drinking water treatment, bench scale studies showed that coagulation (with or without chemical softening) is largely ineffective in removing pharmaceuticals [9].

- **Ethidium Bromide, Salicilic, Xylene, Formalin and other laboratory wastewater** should be separated in concentrated form in separate plumbing system, treat and dispose as hazardous wastewater (scheduled waste)

4.3 Industrial wastewater consisting with heavy metals

- Heavy polluting industries categorized as type "A" [6] should not be established upstream or downstream (up to stretch that can back flow of pollutants to the intake along with salinity wedge during high tide period where intakes are closer to the sea) of major rivers that extract raw water to produce potable water.

- Implement interceptor sewer to bring the treated effluent from Biyagama Export Processing Zone (BEPZ) and the effluent from other heavy polluting industries located upstream of Ambatale and Kelany Right Bank (KRB) intake downstream of proposed salinity barrier.

- For wastewater having Cr^{6+} to be first converted to Cr^{3+} by mixing with waste Fe^{2+} while controlling the oxygen reduction potential between 250 and 350mv at desired pH

- Dewatered sludge should be disposed in compliance with the requirements stipulated in EU Council Directive 86/278/EEC for land application of Sludge until a local standard is established

- pH requirement for different metals [1] should be controlled in the desired range depending on level of removal desired for different metals based on Fig. 1, followed by coagulation, flocculation and sedimentation

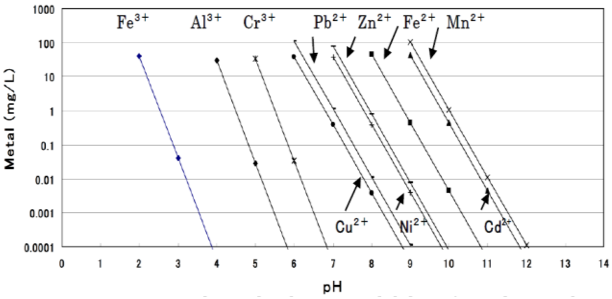


Fig.1 Relationship between solubility of metal ion and pH

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SECM/15/054

VIRAL HEPERTITIES OUT BREAK IN ELLA, UVA PROVINCE, SRI LANKA

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Abstract:
The study based on Epidemic outbreak of Viral hepatitis in Aluthgama-Kalugalpathana in Ella area. Aluthgama; the small village, five kilometer away from Bandarawela town and it is located in Dova GND in Ella D.S. in Badulla District. The population of Aluthgama village around 750 in 215 number of families. In the month of April 2015 it was reported outbreak of viral hepatitis in Aluthgama area. The investigation study was carried out by National water supply and drainage board with help of health sector of Badulla district. Through the several investigations finally identified the reason for this critical situation as unsafe drinking water supply. Villagers consume water by pipe borne water supply scheme; named kalugalpathana Water supply scheme. The maintains of scheme has been taken over by Ella pradeshiyasabawa since 2013. The small stream flow down from hill-top is based for water project has no proper disinfection process. Study path gave the conclusion as raw water in catchment was contaminated by the people who are deforesting the catchment by living the upper catchment area during that short period. The one of that group was the dieses carrier.

Keywords: safety, hygiene, quality, satisfactory

1. Introduction

Sri Lanka is still under developing country which is having more features in geographically. The highest mountains in the central area whereas the lot of water bodies' start from there and goes towards the sea making several water streams around the country. For serve the nation with treated drinking water, water treatment plants are constructed in several areas according to the geographically and location of crowded areas. Hence nearly 1000 small and big treatment plants are proceeding to achieved the our target of customer satisfaction. Among that Ella region is the one of water treatment area in Badulla District. Out of 18 small water treatment plant in Ella, Aluthgama- Kalugalpathana is serving 20 cubic meters per day of water quantity. The study based on Epidemic outbreak of viral hepatitis in Aluthgama-Kalugalpathana in Ella. Villagers consume water by pipe borne water supply scheme; named kalugalpathana Water supply scheme. The scheme has been taken over by Ella pradeshiyasabawa since 2013.

2. Objectives of the study

The key objective of this study to identify the epidemic outbreak of viral hepatitis in Aluthgama-Kalugalpathana which is important case study of human heath based water treatment process.

3. Methodology

Aluthgama; the small village, five kilometer away from Bandarawela town and it is located in Dova GND in Ella Divisional Secretory in Badulla District. The population of Aluthgama village around 750 in 215 number of families. In the month of April 2015 it was reported outbreak of viral hepatitis in Aluthgama area. The investigation study was carried out by National water supply and drainage board with help of health sector of Badulla district. Through the several investigations finally identified the reason for this critical situation as unsafe drinking water supply.

The case study conduct in few steps

- Demarcate and map the catmint treatment and water distribution areas.
- Water samples were collected and tested throughout the process including customer drinking point
- Point out and concentrated the houses of reported patients.
- To catch up all the data from Physical health instructor, Grama Niladari and other relevant

supporting services conducted the meeting in the village.

4. Results and Discussion

Water quality data of considered area is in table 1.

Table 1: Water quality of Aluthgama water Supply Scheme

Place	Date	Coliforms/ 100 ml	E-coli / 100 ml	E.C. us/cm	Chloride mg/l	Alkalinity mg/l	Hardness mg/l	TDS mg/l
Aluthgama intake 01	03.04. 2015	950	105	78	14	38	36	39
Aluthgama intake 02	03.04. 2015	10	2	103	16	50	48	51
Stock tank	03.04. 2015	750	200	73	14	36	44	36
section A, Chandrapala.No. 31	03.04. 2018	200	48	75	10	36	40	37
section B, Damayanthi, No.399	03.04. 2015	70	15	75	10	36	40	37
section C, Chamila Warnasooriya	03.04 .2015	12	15	76	12	38	40	38
section D, Ramanayaka,No. 309	03.04 .2015	10	12	84	12	42	42	42
section E, Lalitha Kumar, No. 265	03.04. 2015	15	13	76	12	36	42	38

According to the above results the initiation of the case identified as contamination of catchment.
Therefore the case study turns towards the identifying of the pollution source of catchment. The visual observation of particular field seems the temporary settlers as used woodcutters. Upper catchment land contents the important trees of Department of Forest and they have given the subcontract to deforest to get timber. This subcontract people used the above mentioned huts. Small stream flow down from hill-top is based for water project has no proper disinfection process. Study path gave the conclusion as raw water in catchment was contaminated by the people who are deforesting the catchment by living the upper catchment area during that short period. The one of that group was the dieses carrier.

Problems enclose; The existing water supply scheme has 137 connections and the component of the scheme content 3 km main, 15m^{3}capacity ferrocement storage tank and distribution system. Proper laying of pipes was not observed. There is no proper disinfection system or good understanding of safe water. Only one lady care taker works in pradeshiyasabawa for maintains and she has no capacity to overcome the water supply problems. Water delivers as intermittence supply and may cause re-contamination within the pipe line.The area belongs to Catchment is 5 acres which do not sufficient to get enough quantity of water. Villagers are very poor and no good hygiene practices.

Problem overcomes; Awareness programs among the villagers done with the help of P.H.I of the area. Apply the Chlorination in proper manner to overcome the fecal contamination of raw water. Water quality testing is carrying out continuously for verify the absence of Bacteriological contamination. Catchment protection program will be introduces.

5. Action taken

- With the help of PHI and the Caretaker proper disinfection process started.
- Follow-up the sample testing continuously.
- Awareness program carried out
- Start to develop the water safety plane for this water supply scheme.
- Under the rules and regulations of NATIONAL WATER SUPPLY AND DRAINAGE BOARD to protect and maintain the above scheme.

6. Recommendation

In order to promote the water safety plane (WSP) staring committee was appoint for wider applications, some key factors shall be addressed. Firstly, proper planning of WSP is important with reference to the specific needs and conditions. This could be facilitated by WSP in local planes for water managements. However, planning needs to be taken with care for all sensitive issues including public health, stakeholders and viability of operation and maintenance. Secondly, economic and financial requirements are also needed to be studied in depth, as less viable schemes for WSP will create a social burden and will not last for long. Thirdly, local capacities including human resources and legal frame work are also very important in achieving sustainable targets of water safety planes.

Acknowledgment

The authors are grateful to the villagers of Aluthgama for their full contribution of the case study. Further extent the gratitude to the Public health inspector(PHI) Ella division, Department of Health and the Gramasewaka of Aluthgama for their guidence. Further appreciate to the National water supply and drainage board, Laboratory staff for their assistant and support.

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Development and Implementation of Water Safety Plan in Kondawatuwana Water Supply Scheme

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Abstract: A Water Safety Plan (WSP) for Kondawatuwana Water Supply Scheme (KWWSS) was developed by National Water Supply and Drainage Board (NWSDB). Possible hazards were identified through field visits. The risks were assessed using a semi-quantity approach. The existing control measures and the verification of control systems were used to assess the residual risks and form a prioritized standard risk matrix. Based on this, an improvement plan was drawn up to suggest corrective actions and a time frame for implementation. The findings of this study are being used to modify or repair components of the water supply system and upgrade management procedures. Water Safety Index is introduced as a bench marking management tool in order to monitor the progress of implementation of WSP. This paper highlights the lesson learned during the development and implementation of the WSP and the key challenges faced.

Keywords: Drinking Improvement Plan, Risk Assessment, Water Quality Monitoring, Water Safety Index, Water Safety Plan

1. Introduction

According to the World Health Organization (WHO) Guidelines for Drinking-Water Quality, water safety plans (WSPs) is the most effective means of ensuring the safety of a drinking water supply (WHO, 2004). This concept draws from the traditional multiple-barrier risk management techniques and the Hazard Analysis and Critical Control Point (HACCP) approach, which is applied in the food manufacturing industry. The HACCP-concept has been successfully implemented and refined in several water supply systems to improve drinking water quality and security. Several cases studies from industrialized and from developing countries document increased compliances with drinking water quality regulations and reduced cost of operation as result of implementing HACCP. The WSP has been developed based on these experiences.

Developing a WSP involves undertaking risk assessment and identifying hazards at each step of drinking water supply from catchment to consumer (WHO 2004). Over the last decade, WSPs have gained acceptance as an important framework for achieving water quality and health-based targets. WSP can be developed for any type of water supply system, piped- or community-based. Public water utilities in Australia, UK, Latin America and the Caribbean (Bartram et.al, 2009), Bangladesh (Mahmud et.al, 2007), Belgium, Switzerland and the Netherlands have successfully developed and implemented WSPs for their water supply system.

The experiences with WSPs for small, community-managed water supplies are limited and being developed. This has been because of the difficulty in implementing water quality management in situations with limited technical expertise which are often remote. The use of model or guided WSPs may also help to significantly reduce the costs and complexity of implementing WSPs within utility supplies in developing countries (Howard et al. 2005). Godfrey and Howard (2004) documented the WSP approach with certain alteration to overcome the challenges likely to be faced in developing countries, which is limited to small systems.

A case study of implementing a WSP for a large piped-water supply in a developing country is presented in this paper. NWSDB is more concerned in implementing WSP in Kondawatuwana water supply scheme, due to following site specific reasons:

1. Raw water have been contaminating by waste water, solid waste disposal, sewerage disposal, manure disposal and agriculture chemical in the catchment area;
2. The algae concentration is higher in the raw water and can be controlled by limiting

nutrition from point and Non-point pollution sources;
3. The treatment is less effective in removing Iron (Fe) and Manganese (Mn) in terms of aesthetic quality;
4. The water contains silica micro particles and the treatment plant is not equipped for removing silica;
5. The water is consumed by more than 325,000 people from this area and considerably high rates of water loss experienced.

2. Methodology

2.1 Study Area

KWWSS is an organized urban piped water supply system established under the Eastern Coastal Towns of Ampara District (ECTAD) project and implemented by NWSDB in year 2000. NWSDB is responsible for operating intake, water treatment plant, storage tanks and distribution systems.

Water was extracted from intake in the irrigation tank, and the treatment plant is located closed to the source. The treatment processes includes lime and alum feeding, pre-chlorination, Dissolved Air Floatation (DAF), rapid sand filters with air and water backwashing system, post chlorination and neutralization system. The Water treatment plant is designed to produce 72,000m3/day of drinking water complying with the SLS standards. The water is distributed to Ampara town, adjacent towns, the coastal towns of Ampara district and two towns in Batticaloa district as shown in Figure 1. It is expected to serve 436, 400 numbers of people in the design horizon of year 2030. The current water production is only about 60% of its designed capacity.

2.2 Development Plan

The approach for developing the WSP was based on the guidance of WHO expert and the water safety plans manual (Bartram et.al, 2009). The approach was modified to deal with the specific problems found in this drinking water supply system such as higher water loss, limited data availability, unplanned development, and lack of consumer awareness. These issues are common to several developing countries and the information presented in this paper could be useful to practitioners facing similar challenges.

2.3 Formation of the WSP team

The WSP team consisted of staff from NWSDB and stakeholders. The strategy in developing, and implementing the WSP was carried out through working together as teams for catchment and treatment, distribution and consumer. The support of local authorities and stakeholders were required for collection of information related to the catchments. Presentations were arranged to NWSDB staff and the stakeholders to understand the important of the WSP and the benefits of implementing in Kondawatuwana water supply scheme.

2.4 System Assessment

Figure 1 shows the schematic of the KWWSS including the source, treatment plant and coverage areas. This scheme has 9 intermediate sumps with pump houses and 16 elevated towers, connected with 20 water transmission mains and two direct pumping systems to distribution system using variable speed drive. The water is supplied through 1,500 km of pipeline to about 90,000 metered connections.

Water samples were tested and analyzed from source, component of each unit process, storage locations, and tap of selected consumers for physical, chemical and bacteriological parameters according to standard methods. Water quality data records for the previous 3 years were also analyzed.

A household sanitary survey was conducted in the catchment in 200 household in three Grama Sevaka Niladhari Divisions (Himuthurawa, Paragahakale, and Abayapura). The survey study was designed especially to identify the causes of pollutant in the catchment area.

2.5 Hazard Identification

Hazard identification was done for the catchment area, raw water source, water treatment plant, storage locations, distribution networks and selected household storages. Hazards within the catchment area and in the cascade arrangements of reservoirs were identified by the household sanitary surveys, visit by the WSP team, land use pattern and satellite images. Hazards within the water treatment plant were identified with the help of treatment plant operators and NWSDB officers.

Within the storage and distribution system, the hazards were identified by reviewing the condition of structures and pipes based on the maintenance schedule and experience of operation and maintenance staff. Few selected household storages were inspected to identify the hazards.

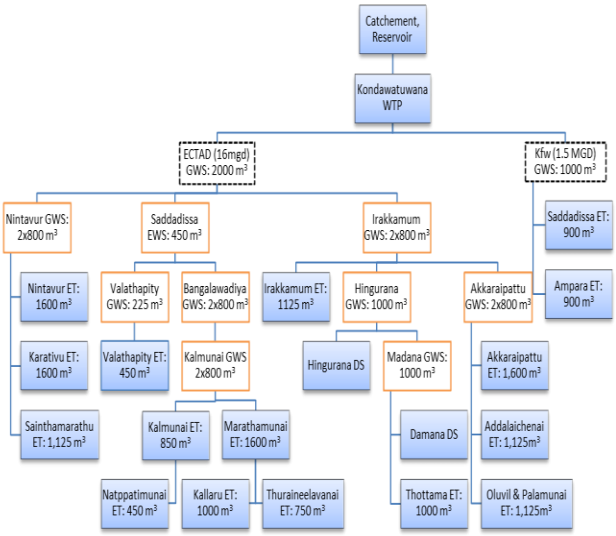


Figure 1: Basic elements for describing the water supply system

The transmission lines were mapped including the locations of flow meters in order to improve the calculation of actual water loss in the system. The likelihood and consequence for each hazard was identified for risk assessment.

2.6 Risk Assessment

The risks were assessed using a semi-quantity approach (Deere et.al. 2001). The likelihood and consequence of each hazard was identified based on the field visit, expert opinion, available data, previous studies, and experience of Operation and Maintenance (O&M) staff. Each hazard was scored according to the likelihood of occurrence and consequence in terms of health-based targets and compliance with regulations. Residual risk score was generated based on individual scores and existing validated control measures.

2.7 Identification and validation of control measures

Control measures are steps taken to ensure that the quality of water supplied consistently meets targets. Site visits were conducted to identify existing control measures and to assess the effectiveness of the control measures. The validation of the control measures was done by reviewing the maintenance and monitoring records and field inspection.

The residual risk score was calculated based on the findings from the validation of the control measures. In the final risk assessment matrix, risks were categorized in terms of their potential impact on the performance of the water supply system and its capacity to deliver safe drinking water to consumers. Workshop discussions were held for WSP teams and O&M staffs including treatment plant operators to finalize risk assessment matrix and prioritization of residual risk.

2.8 Improvement plan

An improvement plan was drawn up to address the all uncontrolled and prioritized residual risks. The plan included risks that scored very high (score >15) and high (score 10-15) on the risks band. The less significant risks (medium and low risk) were not specifically addressed in the plan as these would be eliminated once the control measures for more significant risk are implemented or risk with less priority.

3. Results and Discussions

Hazard identification and risk assessment for each component within the water supply system is presented in the following sections.

3.1 Catchment and Raw Water Source

Table 1 shows the list of potential hazard from the catchment to consumer. These hazards include existing as well as potential hazard based on future growth and development of the areas in and around the raw water source. The hazards exist due to the prevailing anthropogenic activities around the vicinity of the raw water source. Rainfall runoff is contaminated due to these activities, resulting in the potential contamination of the source water.

Other anthropogenic sources include discharge of wastewater from human habitations including animal farm and horticulture in domestic level and discharge of drainage water containing agrochemicals from paddy fields. This is a result of inadequate urban development planning in allocating settlement areas and catchment for drinking water source.

The challenge in controlling contamination of raw water sources is enhanced due to social and cultural aspects prevalent in the region. Enforcing control measures or corrective actions to restrict the use of water bodies for such activities is a challenge. Implementing any such measure without involving the local community in the complete decision process would be ineffective.

3.2 Water Treatment Plant

Selected hazards identified in the water treatment plants by the WSP team and the operators are described in Table 1. The WSP team reviewed the procedures in the treatment plant and it was found to be operating as per standard operating procedure (SOP) and some risks were identified. However, the online monitoring system was not operational. This increases the risk of not identifying changes in the quality of water being supplied to the community.

The maintenance of water treatment plants poses a key challenge. This was observed, due to lack of local agent readily available in purchasing the spares for maintenance. This also leads to lack of proper monitoring of the water quality being supplied. The efficiency of the water treatment plant was calculated based on the available monitoring parameters. Figure 2 depicts the boxplot with quartile spread of treatment efficiencies for each parameter.

Checking of the risks of protozoan & Disinfection-by-Products (DBPs), replacing the defective online monitoring systems, checking the water treatment efficiency of unit processes, following standard operation procedure for normal operation, preparing emergency plan for incidents and emergency situations, maintaining proper asset management systems including maintaining minimum spares for M&E equipment and laboratory items are suggested improvements.

3.3 Distribution network

The low number of uncontrolled risks was identified in the distribution network as shown in Table 1. Prior records have shown deterioration in water quality in the distribution network with sudden exposure of colored water due to disturbance of iron and manganese settled in the pipelines.

There is no extreme risk identified in the distribution network because of non-availability of sewer lines in the vicinity of the drinking water pipeline which may leads to cross connections. However, preparedness for corrective maintenance works, monitoring and maintaining positive pressure throughout the distribution network; written hygienic procedures for repairing burst mains and laying new mains, including disinfection before return to service; avoiding disturbance of deposits by avoiding sudden increases in flow and flow reversals and a programme to routine flushing and maintenance; are to be followed to maintain the quality of supply in distribution system.

Regular internal and external inspection of service reservoirs/water towers to make sure there are no structural defects and that access hatches, vents and other openings are either locked or covered to prevent ingress are proposed to ensure re-contamination in storage.

3.4 Households

Findings of the household sanitary survey and the hazards are shown in Table 1. The most common risk of recontamination comes from the water handling and storage practices of the users themselves. Even if the tap water supplied is free from contamination, it may get recontaminated in few places as a result of people dipping their hands into the stored water, lack of hygiene in the household and the storage container being accessible to children.

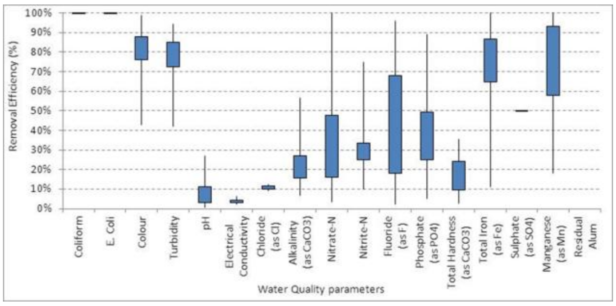


Figure 2: The efficiency of the water treatment plant

Hazards within the households completely depend on the users, awareness within the community regarding these hazards and willingness to adopt safe sanitary practices. NWSDB has established a call center (a tall free number of 1939) that operates 24 hours to address customers' complaints. This is important to receive feedbacks from consumers to further improve the water supply services and to provide assurance to the customers that NWSDB strives to provide good quality water that meets the local potable water standards. NWSDB also is introducing consumer guide manuals and trained plumbers for establishing best practices in internal plumping

3.5 Water quality monitoring

Water quality monitoring was done at all levels of the supply including raw water sources, water treatment plants, distribution network and households. No microbial contamination was observed in any part of the water treatment plant transmissions and consumer’s tap. Recontamination was observed within the distribution network. Further, residual chlorine was more than the required 0.2 mg/l in 100%of the samples collected from household taps.

3.6 Control measures

No control measures were present within the catchment which is not a protected zone. This is under forest cover, certain anthropogenic activities were found to exist. Water samples were analyzed to validate controls at WTP. Studying water quality records, visual inspection and the experience of the operator also helped in validation. Visual inspection showed that several unit operations were found to be working effectively. Even though it removes Fe, Mn below guideline value they are settled in pipelines.

Certain control measures were found in the distribution network. Booster chlorination is done at WTP, intermediate sumps and few elevated tanks and residual chlorine levels are checked at consumer premises. It is in the range of 0.1–0.3 mg/l which confirmed that chlorination was being done effectively.

3.7 Improvement plan

The improvement plan includes corrective actions such as capital works such as repair/ rehabilitation and maintenance, community awareness programs for consumers, training programs for operators enhanced communication among various stakeholders etc. The implementation of the improvement plan is in progress and the post-implementation risk scores from the potential risks to be calculated after implementation of corrective actions.

In the case of catchment, WTP and distribution network, a significant reduction in risk has been assumed as there are construction and repair works. However, in households, corrective actions involve behavior change. Therefore, marginal risk reduction is expected as consumer response cannot be accurately predicted. Implementation of corrective actions within a timeline is the responsibility of WTP operators.

Table 1: Risk Matrix from Catchment to Consumer

ID-No	Hazardous event	Associated hazards (and issues to consider)	Hazard type	Risk L	Risk C	Risk S	Risk rating	Controls	Possible validation	Risk L	Risk C	Risk S	Risk rating
A.1	Seasonal variations	Changes in source water quality (e.g. Algae)	C	4	5	20	**Very High**	Sonication at intake, Selective withdrawal from multi-level intake Powdered activated carbon dosing at the treatment plant,	Historical algae monitoring data (catchment and after treatment) Historical data on algal blooms	3	5	15	**High**
A.2	Geology	Iron, Manganese etc. into water.	M	3	4	12	**High**	Selective withdrawal from multi-level intake not operated	Historical water quality monitoring results and Dredging records	3	4	12	**High**
A.3	Agriculture	Agricultural waste, Slurry and dung spreading,	C,M	3	4	12	**High**	Codes of practice on agricultural	Historical water quality monitoring	3	4	12	**High**

		Disposal of dead animals						chemical use and slurry spreading, Public awareness to agrochemicals when needed only	results				
A.4	Forestry	Pesticides, PAHs - polyaromatic hydrocarbons (fires),	P,M	3	4	12	**High**	Awareness to Forest & Wild life Department	Historical sanitary inspection records	3	4	12	**High**
A.5	Farming	Cattle defecation in the catchment (a source of potential pathogen like Cryptosporidium)	M	3	4	12	**High**	Preventing livestock grazing, Moving farm operations away from sensitive locations	Historical microbial source water quality data, sanitary survey results for buffer distances, Historical catchment inspection results	3	4	12	**High**
A.6	Army Camp & Houses	Effluents, contaminants polluting source water	P,C,M	4	5	15	**Very High**	Awareness Program, Introduce toilet for dwellers in catchment, Reduce soil erosion & runoff, Buffer zone vegetation.	Collaboration with Environmental Authority to obtain historical reports on effluent discharge	3	5	15	**High**
B.1	Operation failure of stop logs	Entering of High turbidity and sediments	P,C	3	4	12	**High**	Repairing of online monitoring of Turbidity in Raw water line	Equipment maintenance and replacement records	3	4	12	**High**
B.2	Instrumentation failure	Loss of control	M	3	4	12	**High**	Continuous monitoring with alarms	Equipment maintenance and replacement records	2	4	12	**High**
D.1	Cross connections / unhygienic practices	Contamination	P,M	2	5	10	**High**	Property inspections, Public awareness, Approved quality of materials in customer plumping	Historical water quality monitoring results Historical incident records	2	4	8	**High**

3.8 Management programs

SOPs are a critical focus during the implementation of a WSP and to be developed. This requires documenting procedures to be followed during normal operating conditions and in specific 'incident' situations. An incident is a situation where water being supplied for drinking purposes might become unsafe (WHO 2004).

Investigation should be undertaken involving all staff to discuss the current performance, assess inadequacy of current procedures and address any issues or concerns during documentation of procedure. An annual review protocol has also been prepared, which includes testing water quality at each step of the water supply system and comparing this to critical limits based on the drinking water guidelines.

3.9 Supporting programs

A key finding of this study is the importance of supporting programs. It was found that the water supply utility alone cannot ensure the provision of safe drinking water and protection against water-borne diseases. As shown in “Households”, improper storage and handling also cause recontamination. Therefore, it becomes necessary to create awareness about point-of-use water treatment. Tools such as locality meetings and informative posters and use of mass media have to be deployed to promote safe water practices and hygiene.

Local dispensaries and primary health centers should report cases of water-borne diseases to the local government hospitals. Training and awareness workshops have been carried out to enhance the skills of WSP team members and public health inspectors and to increase their capacity to employee training to disseminate the concept of WSP, monitoring employee performance through indicators such as response time, % customer complaint solved and supply restoration time after cleaning or repair.

3.10 Challenges faced during implementation

One of the challenges faced during this study is related to the feasibility of catchment protection within the water supply. Since agricultural drainage is directed to the reservoir, it might not be feasible to change and implement policies regarding land use and catchment protection. It is also not possible to delineate a timeline for each corrective action. Therefore, instead of substantially restricting activity within the catchment, stakeholders should be sensitized about the need to restrict on activities causing the drinking water source polluted.

In the context of water supplying in developing country, there are some limitation in the allocation of funds for rehabilitation activities, although there is full commitment of the operator and top level management of NWSDB. The coordination of framework for the WSP and the communication between various stakeholders are required a motivation mechanism.

Public health surveillance is of great importance in order to meet health-based targets. Sufficient data regarding water-borne diseases were not available at primary health care centers. More efficient record keeping is needed to identify incidents of disease and quantify the effects of WSP.

4 Conclusion and Recommendation

4.1 Conclusion

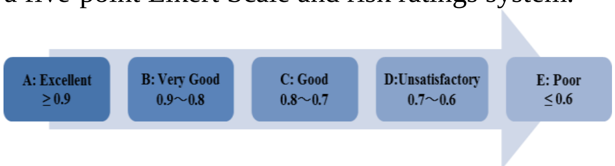
This study describes the process of developing and implementing a WSP for one of the large-piped water supply in Sri Lanka. To introduce the concept of WSP, it was essential to have a key resource person with prior experience in developing WSPs. This experience came from WHO and NWSDB who have formed a group of experts to guide future WSP development in Sri Lanka and steps have been taken to identify and train water supply officials from selected regions.

The findings of the WSP study are being used to prioritize the interventions planned as a part of the activities of operation and maintenance staff. The principles of WSP have been used to make day-to-day operations more efficient and employees more accountable. It has resulted in better communication and incident reporting.

This study revealed certain vulnerabilities in the water supply system, especially within the catchment areas and the support from stakeholders are highly valuable. Further, it was found that there is a lack of consumer awareness regarding point-of-use water treatment and limited availability of health data, which is not conducive to achieving health-based targets. Supporting programs are, therefore, critical to address these issues.

4.2 Recommendation

The primary operational characteristic of the WSP is the provision of a water safety index (WSI) which identifies drinking-water safety levels using a five-point Likert Scale and risk ratings system.



The closer a value in the WSI is to 1, the higher its safety grade. The WSI is considered to be a very useful indicator with its advantage of quick understanding of the drinking-water safety level of a water plant and an objective and structured system to identify its weak points or vulnerabilities.

Equations to calculate WSI is as follows:

WSI = (safety rating ratio) - (risk rating ratio)

$$\text{Safety rating ratio} = \frac{\text{total no. of low rating item}}{N}$$

N = total number of assessed WSP check-list items)
Risk rating ratio = (medium ratio) + (high ratio) + (very high ratio)

$$\text{Medium ratio} = \frac{\text{total no. of medium rating item}}{3N}$$

$$\text{High ratio} = \frac{\text{total no. of high rating item}}{2N}$$

$$\text{Very high ratio} = \frac{\text{total no. of very high rating item}}{N}$$

Acknowledgements

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Potential of Different Biochars for Glyphosate Removal in Water; Implications for Water Safety

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Abstract: We investigated the potential of two different types of biochars (BCs), a waste by product from a Dendro bioenergy industry (DBC) and a steam activated rice husk derived biochar (SRBC) to remove glyphosate from aqueous media. Equilibrium isotherms and kinetics experiments were conducted to study the adsorption behaviour and postulate potential mechanisms. Glyphosate adsorption on both BCs was strongly pH dependent, exhibiting maximum on DBC and SRBC at 5-6 and 2-4 pH, respectively. Isotherm data obtained for DBC adsorption was best fitted to Freundlich and Temkin models indicating a multilayer adsorption, whereas glyphosate adsorption on SRBC was well described by Freundlich and Langmuir models suggesting both physisorption and chemisorption mechanisms for the adsorption process. The Langmuir maximum adsorption capacity of DBC and SRBC was 44.00 and 123.03 mg/g, respectively. The kinetics of glyphosate adsorption on DBC were best described by pseudo-second order mode indicating that the rate limiting step can possibly be a chemical adsorption, while pseudo-first order model described kinetics of glyphosate adsorption on SRBC indicating physisorption interactions for glyphosate adsorption. Pore diffusion, π*-π electron donor-acceptor interactions and H-bonding were postulated to be involved in physisorption, whereas electrophilic interactions led to chemisorption type of adsorption for both DBC and SRBC. Overall, both DBC and SRBC could be a promising remedy of glyphosate removal from aqueous solution.

Keywords: Pesticides, physisorption, chemisorption, kinetics, Adsorption capacity.

1. Introduction

The presence of pesticides in water at elevated concentrations has become a worldwide environmental problem in the recent decades due to its serious consequences on human health as well as the surrounding ecosystems. Glyphosate ($C_3H_8NO_5P$) is an organophosphorous herbicide that is widely applied in controlling unnecessary growth of grasses, sedges, weeds and plants in farming, forestry, play grounds, parks and roads [1]. Glyphosate was introduced into the global market in 1974 and it has currently become one of the most widely used herbicides with the total global consumption of over 70,0000 tons per year [2]. So that, its high demand and excessive usage have posed widespread environmental problems due to persistent glyphosate residues.

The fate of glyphosate is mainly associated with the soil. Due to its high water solubility (12,000 mg/L), it can easily runoff into surrounding water reservoirs leading to significant glyphosate contamination in surface as well as groundwater systems [3]. Consequently, humans may be subjected to numerous health problems of glyphosate exposure, including eye and skin irritation, contact dermatitis, eczema, cardiac and respiratory problems and allergic reactions [4]. It has been recently hypothesized the association of glyphosate exposure for chronic kidney disease of uncertain Aetiology (CKDu) in the rice paddy farming areas in Sri Lanka [3]. Hence, the use of glyphosate as a herbicide has been recently banned in Sri Lanka.

Adsorption is an efficient and promising strategy for decontaminating wastewaters. Although glyphosate has been currently recognised as a critical pollutant in the environment, no many studies have been documented for the removal of glyphosate from aqueous solution. In a previous study, activated carbon derived from waste newspapers has been successfully used as an adsorbent to remove glyphosate from aqueous solution [1]. Nevertheless, biochar (BC), a carbon-rich product of waste materials such as rice husk, tea refuse, etc. has been recently used as an alternative and economically viable adsorbent to remove hazardous inorganic and organic contaminants [5-7]. The use of BC is therefore a good option not only for environmental remediation, but also for waste management in the environment.

Up to date, many studies have focused on the application of non-activated BC in the remediation of pollutants in water and soil systems [7]. More recently, it has been reported that birch wood BC is capable of controlling the fate of glyphosate in the soil by reducing its mobility [8]. However, the activation of BC via steam is capable of escalating its adsorption capacity [1]. Only limited studies have been found to be focusing on the use of engineered type of BC for the removal of toxic contaminant [5, 9, 10]. In a recent study, steam activated BC produced from tea waste was proposed as an assured treatment for the removal of sulfamethazine from water with an adsorption capacity of 33.81 mg/g [1]. However, to our knowledge, the adsorption behaviour of glyphosate onto engineered BC has not previously been evaluated. Similarly, the identification, characterization and application of some waste by-products of bioenergy industries for environmental remediation aspects have also been limited [6, 11]. Hence, this is the first time reporting on a comparison of adsorption capacities of two different types of BCs; steam activated rice husk derived BC (SRBC) and a waste by product of a dendro bioenergy industry in Sri Lanka (DBC), for the removal of glyphosate from aqueous solution.

2. Materials and Methodology

2.1 Biochar Production and characterization

The DBC was obtained as a waste by-product from a bioenergy industry (Dendro) at Thirappane, Anuradhapura District, Sri Lanka, where biomass of *Gliricidia sepium* is gasified at 700-900 ºC in order to generate electricity. The collected dendro biochar was air dried and ground to less than 1 mm prior to use. Physico-chemical characterization of biochar was done in our earlier study using standard procedures (Herath et al., 2015).

The SRBC was produced from rice husk collected from Sri Lankan rice mills. Rice husk was then washed several times with distilled water and air dried. The dried biomass was crushed and ground to <1.0 mm in particle size. Rice husk was pyrolyzed at 700 ºC with a heating rate of 7 ºC min-1 for 2 h under limited O_2 in a modified N11/H Nabertherm (Germany) furnace. Char samples were then treated with 5 mL/min of steam for an additional 45 min under the peak temperature after the 2 h pyrolysis period had elapsed.

2.2 Analysis of glyphosate

Analytical grade herbicide glyphosate (N-(phosphonomethyl) glycine) was used for this study. All the reagents used were obtained from Sigma Aldrich and were of analytical reagent grade. Residual aqueous glyphosate concentration was measured following the method described by Tzaskos et al. 2012 [12]. This is a colorimetric method, in which a purple colored complex is developed due to the reaction of glyphosate with ninhydrin and sodium molybdate. Standard glyphosate solutions including 4, 6, 10 and 14 mg/L were prepared from a stock solution of 1000 mg/L, followed by the addition of 0.5 mL of 5% ninhydrin and sodium molybdate solutions. The colour intensity of samples was measured at 570 nm (λ_{max}) using a UV-visible spectrophotometer (UV-160A, Shimadzu, Japan).

2.3 Effect of initial Ph

The effect of pH on glyphosate adsorption on SRBC and DBC was studied by adjusting the pH of glyphosate solutions with 1 M HNO_3 or NaOH solution in the range of 2.0 to 10.0 and glyphosate concentration of 20 mg/L and BC dosage of 0.5 g/L.

2.4 Batch adsorption kinetic experiment

Batch adsorption experiments were carried out at 20 mg/L of initial concentration of glyphosate and a sorbent dose of 0.5 g/L. The initial pH was chosen based on the optimum pH value obtained from the edge experiment. Teflon centrifuge tubes containing 10 ml of 20 mg/L of glyphosate solution. The mixtures were shaken for 30, 45, 60, 90, 120, 180, 240 min of reaction time at 100 rpm of shaking speed at room temperature (25 ºC). At each reaction time, three centrifuge tubes were taken out and centrifuged at 3000 rpm for 15 min.

2.5 Adsorption isotherm

Batch isotherm studies were carried out in the concentration range of 0-100 mg/L at pH 5-6 and 3-4 for DBC and SRBC, respectively for an adsorbent dose of 0.5 g/L. An equilibrium time of 3 h was chosen based on preliminary kinetic experiments.

2.6 Experimental data modelling

In this study, non-linear isotherm and kinetic models were applied to the experimental data due to discrepancies of linear models as reported in recent studies [13]. In order to investigate the mechanism of adsorption process, five different non-linear kinetic models namely, the pseudo-first order, pseudo-second order, Elovich, Parabolic diffusion and Power function were applied to the experimental data [13]. The isotherm experimental data were analysed using four non-linear isotherm models including, the Langmuir, Freundlich, Redlich-Peterson and Dubinin-Radushkevish [14].

3. Results and discussion

3.1 Characterization of biochars

Proximate and surface characterization data for SRBC and DBC are summarized in Table 1.

Table 1: Proximate and surface characterization data for SRBC and DBC.

Parameter	Values	
	SRBC	DBC
pH	9.53	10.10
Moisture (%)	3.03	6.5
Mobile matter (%)	15.96	9.9
Ash (%)	45.62	19.7
Resident matter (%)	35.39	63.8
BET Surface area (m^2 g^{-1})	230	774
Pore volume (cm^3 g^{-1})	0.08	0.89
Pore diameter (nm)	4.79	4.08

3.2 Effect of initial pH

The pH of solution is one of the key parameters that can have an impact on the adsorption process [1]. The optimum pH for maximum adsorption of glyphosate for SRBC (16.4 mg/g) and DBC (21.6 mg/g) was observed at 3-4 and 5-6, respectively. The glyphosate adsorption on both biosorbents decreased significantly with increasing solution pH. It is clear that the adsorption of glyphosate on both SRBC and DBC is highly dependent on solution pH, since it affects the surface charge of the adsorbent, as well as the degree of ionization and speciation of the adsorbate [1].

The point of zero charge (pHpzc) can be used to explain the effect of pH on the adsorption process. It has been previously proven that when the pH of solution is below the pHpzc, the surface of the adsorbent is positively charged and it becomes negative if the pH is above the pHpzc [1]. The pHpzc values of SRBC and DBC were 6.65 and 7.30, respectively. Since the pH value of the solution is below the pHpzc of adsorbents, the surface of both SRBC and DBC becomes positively charged, exhibiting predominantly the strong electrostatic interactions with negatively charged groups of glyphosate molecules. When pH of the solution is decreased, the adsorption of glyphosate onto BC surface increases significantly due to increase in positive surface charge of BC that would encourage the electrostatic forces between the BC surface and negatively charged glyphosate species. When the pH of solution is increased the density of positive charge sites of BC surface decreases and the adsorption of glyphosate decreases due to the repulsive force between adsorbent and negatively charged adsorbate (9<pH). Hence, strong electrostatic interactions of positively charged BC surface and anionic groups of the glyphosate molecule are believed to be the major mechanism, resulting in the highest adsorption of glyphosate at acidic pH values.

3.3 Adsorption kinetics

Figure 1 shows the effects of shaking time on the adsorption of glyphosate and non-linear kinetic model fittings for SRBC and DBC. In the adsorption process of SRBC, a rapid adsorption of glyphosate was observed within first 60 min of contact time resulting in an adsorption of 29.3 mg/g (73.0%), and it was then followed by a slow adsorption rate reaching the equilibrium after 90 min standing 29.0-30.5 mg/g (74.0-76.0%) of maximum glyphosate adsorption (Figure 1a). Whereas the adsorption of glyphosate occurred very rapidly in DBC with an apparent equilibrium reached around 50-60 min at a maximum adsorption of 18.5-20 mg/g (Figure b).

The two phase adsorption is a commonly possible phenomena, pre-dominated by a rapid phase and a relatively slow phase [4]. This could be attributed to the fact that available active sites on the SRBC and DBC tend to get progressively saturated with time, resulting in a slow adsorption of the solute ions onto the bulk of the adsorbent. The rapid adsorption at the initial contact time is due to the availability of the positively charged surface sites of biosorbents for glyphosate interaction and the decrease in adsorption with time is perhaps due to the electrostatic hindrance between the adsorbed negatively charged adsorbate species onto the surface of BCs.

It can be noted that kinetics of glyphosate adsorption on SRBC were described well by the pseudo-first order model than other kinetic models applied in this study (Figure 1a). Fitting experimental data best to the pseudo-first order model suggested that the adsorption of glyphosate onto the steam activated RHBC would be more inclined towards physisorption type interactions, and also the adsorption process depends on the initial concentration of glyphosate. In contrast to the adsorption behaviour of SRBC, the pseudo second order model described well the adsorption kinetics of DBC, suggesting that chemisorption mechanisms can trigger the glyphosate adsorption on DBC.

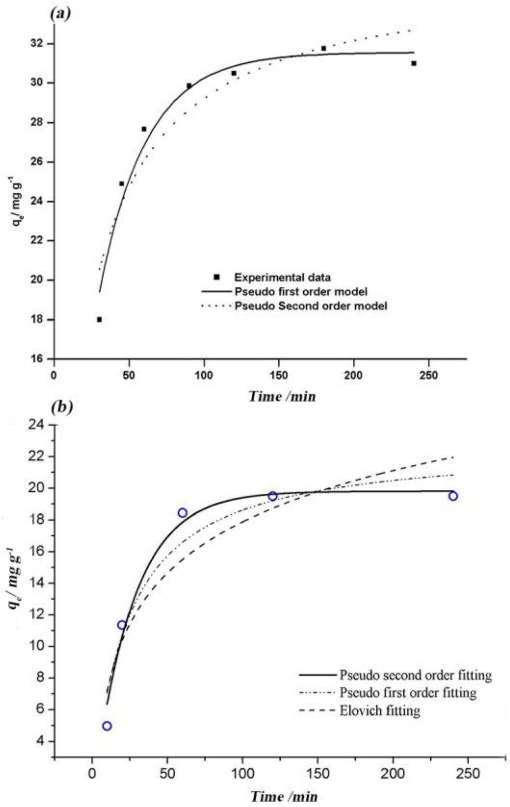

Figure 1: Non-linear kinetic model fittings for glyphosate adsorption on (a) SRBC (b) DBC

3.4 Adsorption isotherm

Adsorption isotherm models are generally used to describe the relationship between the amount of glyphosate adsorbed by a unit weight of adsorbent and the remaining amount of substance in the solution at equilibrium. Figure 2 shows the non-linear isotherm model fittings for the adsorption of glyphosate on SRBC and DBC at 25 ºC.

The adsorption isotherm data of SRBC fitted well to both Freundlich and Langmuir isotherm models best with high regression coefficients (R2>0.93), compared to other isotherm models (Figure 2a). Freundlich isotherm model suggested that glyphosate adsorption is a multilayer physical type of adsorption onto heterogeneous surfaces of the RHBC having different adsorption energies [14]. Glyphosate adsorption on SRBC was also well described by the Langmuir model, proposing that the glyphosate adsorption can also be governed by chemisorption mechanisms. The maximum adsorption capacity of SRBC calculated from the Langmuir model was 123.03 mg/g. Hence, it seems judicious to assume that glyphosate is adsorbed on the SRBC at 4.0 pH via both physical and chemical interactions.

The adsorption isotherm data of DBC was also best fitted with Freundlich model (R2= 0.96) (Figure 2b), suggesting physisorption interactions via multilayer adsorption on heterogeneous and amorphous BC surface. Moreover, fitting experimental isotherm data of DBC (R2 = 0.92) to Temkin equilibrium model indicated the involvement of chemisorption type interactions for glyphosate adsorption on DBC. Overall, isotherm modelling data was greatly evident with the association of both physisorption and chemisorption mechanisms for the adsorption of glyphosate on SRBC as well as DBC.

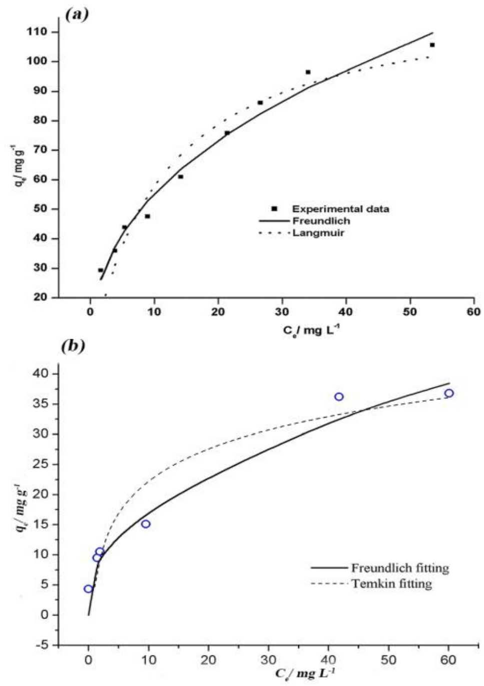

Figure 2: Non-linear isotherm model fittings for glyphosate adsorption on (a) SRBC (b) DBC

3.5 Glyphosate adsorption mechanisms

In this study, mechanistic modelling of isotherm and kinetics data suggested that both physisorption and chemisorption mechanisms can govern the glyphosate adsorption on SRBC and DBC. The diffusion of glyphosate through the pores of BCs would likely to be the primary physical type mechanism. The SRBC and DBC normally consist of a plenty of micro-, meso- and macro- pores with high pore volumes. So that, the glyphosate molecule may easily diffuse into micro-, meso-, and macro-pores in BCs. High surface area of BCs greatly influenced the glyphosate adsorption. The π^+-π electron donor-acceptor interaction is considered as another possible mechanism for glyphosate adsorption on both adsorbents at low pHs. In acidic pHs, the glyphosate molecule can exhibit strong cation-π bonding. Electron rich carbonized surface of SRBC and DBC can be bonded with protonated amino group of the glyphosate molecule forming strong π^+-π electron donor-acceptor interactions. Strong H-bonding between BC surface carboxylic and phenolic groups and glyphosate molecule can also govern the adsorption process. In acidic pHs, surface phenolic and carboxylic groups of SRBC and DBC show great tendency to act as H-donor and acceptor, thus resulting in strong H-bonding with glyphosate molecules.

Chemisorption mechanisms may occur via electrostatic and electrophilic reactions. In acidic pHs, the phosphate group of glyphosate molecule easily tends to be protonated, which is capable of acting as a strong electrophile. This protonated phosphate group has high tendency to attack on either ortho or para positions of aromatic phenolic derivatives present in the SRBC surface, thereby leading chemisorption mechanisms via strong chemical bonding between glyphosate molecule and the surface of BC. Moreover, in acidic medium, negatively charged groups of ionized glyphosate molecule can be attracted by positively charged BC surface leading strong electrostatic forces between BC surface and glyphosate molecule.

4. Conclusions

The present study was conducted to investigate the potential of two different types of BCs; SRBC and DBC to remove glyphosate from aqueous solution. The SRBC showed the highest adsorption capacity for glyphosate compared to DBC. The adsorption of glyphosate on both adsorbents was highly pH dependent and the maximum adsorption was occurred under low pH conditions. Isotherm data obtained for DBC adsorption was best fitted to Freundlich and Temkin models indicating a multilayer adsorption of glyphosate, whereas glyphosate adsorption on SRBC was well described by Freundlich and Langmuir models suggesting both physisorption and chemisorption mechanisms can govern the adsorption process. The kinetics of glyphosate adsorption onto DBC were best described by pseudo-second order mode indicating that the rate limiting step can possibly be a chemical adsorption, while pseudo-first order model described kinetics of glyphosate adsorption on SRBC, indicating physisorption interactions for glyphosate adsorption onto SRBC. Pore diffusion, π^*-π electron donor-acceptor interactions and H-bonding were postulated to be involved in physisorption, whereas electrophilic interactions led to chemisorption type of adsorption for both DBC and SRBC. Overall, results concluded that both DBC and SRBC are highly effective in removing glyphosate in aqueous solution, thereby providing a distinct advantage in the remediation of glyphosate contaminated wastewaters.

Acknowledgements

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SECM/15/105

Catchment Protection of Gin Ganga (River) as part of Water Safety Plan (WSP) in Greater Galle Water Supply Scheme (GGWSS)

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Abstract: Gin Ganga (River) is the main raw water source to Greater Galle water Supply Scheme (GGWSS). Managing drinking water quality from catchment to consumer is the main objective of the Water Safety Plan (WSP) of GGWSS. There are three Water Treatment Plants (WTP) purifying and delivering 80,000m³ of treated water per day to approximately 450,000 people through 15 distributions centers. The Gin River has a total length of 113km and a catchment area of about 932 km² Catchment of the Gin River includes Galle, Matara, Ratnapura and Kalutara administrative districts. The Gin River originates from the Gongala mountains in Deniyaya and flows to the Indian Ocean at Gintota in Galle. Gin River annually discharges about 1268 million m³ of water to the sea. Rainfall pattern in the catchment is bimodal, falling between May and September and again between November and February. Rapid changing of land use pattern and high rate of application of agrochemicals and fertilizers has significantly affected the raw water quality. A quantitative and continuous assessment of water quality along Gin River is necessary to identify the trends and to develop sustainable remedial actions. Content of heavy metals in raw water is also an important parameter to be tested due to lack of previous data. The WSP Team is already established and the Greater Galle WSS WSP has been completed. The External Stakeholder Team consists of relevant stakeholders; the Galle District Secretary, relevant Divisional Secretaries, Health Authorities, Irrigation Officials, Agriculture Officials, Central Environmental Authority etc. The initiatives and the encouragement of the External Stakeholder Team to implement the catchment protection program for Gin River is commendable as the group has already carried out a Sanitary Survey and pollution source identification exercise along the Gin River. The objective of this paper is to present the Water Quality Modeling System prepared for Gin River and its effectiveness in protecting the catchment to effectively implement the Greater Galle WSS Water Safety Plan.

Key words: Catchment, Consumer, Gin River, Water Safety Plan

1. Introduction and Background

The water quality of the Gin River is of vital importance since Gin River is the major source of the potable water supply in the Galle district. The deterioration of water quality of river created adverse impact on human health and hence the socio economic development of the entire District. Most of individual water source (dug wells, surface water sources) are being continuously polluted by different source of waste such as domestic, agrochemical, industrial and electronic wastes. The sources mainly include point sources, industrial discharge and uncontrolled sewerage discharge and non – point sources of pollution which primarily include the storm water runoff from residential, industrial, commercial and agricultural land use. Regulatory bodies have already paid increasing attention to safeguard the water quality in Gin River and other related branch streams connected with the main river from Gongala kanda at Deniyaya to Gintota at Galle. Multiple stakeholders contribute their important service to accomplish water quality in Gin River. Provincial Water Quality Surveillance meeting (PWQSM) headed by the Chief Secretory of the Southern Province and District Water Quality

Surveillance Meeting (DWQSM) headed by District Secretary of Galle are the two most important meetings conducted by every three months intervals. These two meetings mainly focused on pollution of water sources by various activities of communities and changing of climate patterns. National Water Supply and Drainage Board, Irrigation Department, Agrarian Service Department, Agriculture Department, Central Environmental Authority, Coastal Conservation Department, Assistant Government Agents, Regional Health Officers, Public Health Inspectors, Department of Sri Lanka Police and several other direct and indirect stake holders are participated both meetings and discuss the current issues and implement various mitigation actions. Some of them are such as structural and non-structural best management practices and. however the effectiveness of this mitigation action is still limited due to the time, cost and political influence. In this context, in depth understanding on the trends of pollution, spatial and temporal variability of river water quality, interrelationship between key water quality indicators are of crucial importance to increase the effectiveness of mitigation action and water quality monitoring program. Consequently this research study to be conducted to understand the water quality statues of Gin River, trends of pollution and identify the relationships between key water quality indicators.

2. Study Area

Gin River originates from the Gongala Mountains and flow to the Indian Ocean at Gintota. Rainfall pattern is mainly depending on Northeast monsoon between November and February and Southwest Monsoon between May and September followed by inter-monsoon rains in remaining months. Annual rainfall is less than 2500mm in downstream and above 3500mm in the upstream. Gin catchment consists of mainly natural and plantation forest, agriculture and settlements of communities. Cultivation includes paddy, tea, rubber palm oil and cinnamon.

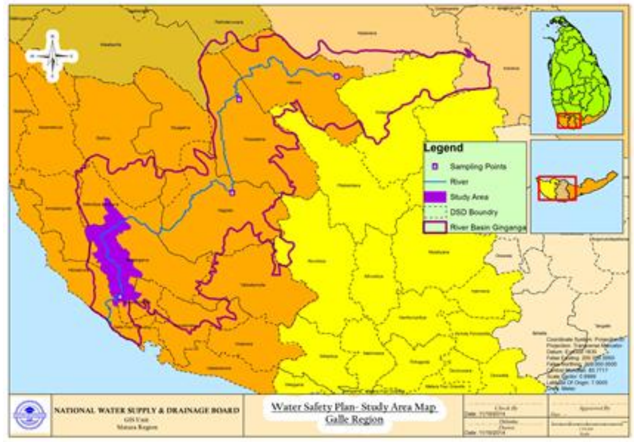

Figure 1: Gin River Basin

Nearly 83% of the catchment are belongs to Galle district and balance shared by Matara, Kalutara and Ratnapura District. Catchment consist of 932 km^2 and lies between coordinates 80° 08” E to 80° 40” E and 6° 04” N to 6° 30” N. Galle districts’ water supply systems mainly depend on the water resources in Gin River basin and presently covered by 34% of district population and planned expand 60% of district population in year 2025. Present extraction quantity of row water is about 80,000 m^3 per day and it will be increased by 100% in year 2025 to achieve planning goals. Studies are mainly focused to identify point –source pollution, non-point source pollution, changing of land use pattern, quantity and quality of fertilizer application for cultivations. The major industries in Gin River basin included relative to tea, rubber, cinnamon and palm oil. As the first stage of WSP concentrate area is limited up to upstream of catchment covered with Baddegama, Bope-Poddala and Velivitiya - Divitura divisional secretaries’ limits and monitoring of water quality parameters is covered upstream up to Lankagama Neluwa Divisional secretory limit.

3. Methodology

Water Quality is a function of chemical, physical, and biological characteristics. Changing of concentration of chemical, physical or biological parameters due to the human and natural actions is directly affected to the water treatment process. As Gin River is an invaluable water resource, NWS&DB has been monitoring water quality since 1972 and but it was limited to physical parameters and it was developed from year 2000 onwards by implementing laboratory with a Chemist at Wakwella Water Treatment Plant. The analyses of available water quality data in this paper mostly on Wakwella and Baddegama water intakes. Water samples have been analyzed in Galle and Matara Regional laboratories of NWS&DB. These laboratories of NWS&DB are well equipped to carry out the required water quality tests (physical, chemical and biological) in water supply schemes operated and maintained by NWS&DB to maintain Sri Lanka standards for drinking water quality. Water and Environmental laboratory of University of Ruhuna provide facilitate to test the heavy metals as needed. Row water qualities of the Gin River are regularly measured at once or twice a month. The specific test methods employed in the laboratories are summarized in Table 1.

The water quality data for Color, pH, Turbidity, Electrical Conductivity (EC), Chemical Oxygen Demand (COD), Dissolved Oxygen (DO), Biochemical Oxygen Demand (BOD), Total Alkalinity is the main parameters were studied to find significant variation of row water quality. In addition to these parameters it is needed to pay more attention on Chemical and Heavy metal concentration of row water with parallel to changing of land use pattern, community settlement and industrialization of the catchment area. These parameters are still under observation level

Table 1: Water quality test method

Parameters	Test Methods
Colour	Calibrated colored disks
Turbidity	Nephelometry
PH	PH meter
Electrical Conductivity	Conductivity meter
Chloride (Cl)	Titrimetric
Alkalinity	Titrimetric
NO3-N	sectrophotometric
Fluoride (F)	colorimetric
Total Suspended Solids (TDS)	Gravimetric
Hardness	Titrimetric
Calcium ($Ca^{2}+$)	Titrimetric
Iron	colorimetric
SO^{2}- 4	sectrophotometric
No of Coliform	Membrane filtration
No of E Coli	Membrane filtration
Biological Oxygen Demand (BOD)	Titrimetric

4. Physical Observations

Sand mining is livelihood of communities living close to the river bank. Erosion of river bank and unbalance ecosystem is created due to the excessive sand mining from downstream at Wakwella intake to up to Neluwa divisional secretary limit. Branch streams which are connected with Gin River also polluted with agrochemicals residual and waste dumped by Tea factories, communities living in river bank, hotels and vehicles’ service centers. Irrigation Canal (Ship block) which was constructed for collecting excess water of paddy field was connected with Gin River at Wakwella just upstream of intake. Buffalos and oxen are freely living most areas of river bank in down streams. Death of these animals due to various diseases, bodies were dumped in to the river and it was increased during the flood period. It was observed that the several agrochemical empty containers also were dumped to river by the farmers due to non-awareness. Little amount of e- waste also was visible in downstream of the river in semi urbanize area near to Udugama Nagoda Agaliya, Baddegama and Galle. These wastes are collected across the bridge crossings near to intake structure. People along Gin River use water for their daily water consumption without proper disinfection system.

Figure 1: Irrigation canal with high concentration of agrochemical just upstream of the Wakwella intake

Available data of testing row water at Wakwella and Baddegama WTPs is also used for analysis and find any trend of variation with time. Water quality will be affected by flow volumes, and affecting both concentration and total loads. Regional Chemist of NWS&DB attends periodical testing of required parameters and Water resources and Environmental laboratory of University of Ruhuna facilitates testing for heavy metals. Research studies mainly focused to formulate a model for predict the concentration of physical, chemical and biological water quality parameters with changing of concentration in various location of upstream in Gin River. Data to be collected with related to land use pattern, quantity of fertilizer application and its contents, quantity of pesticides and weedicides applications, identify the point pollution sources and non-point pollution areas and content and concentration of discharge waste are the main activities to be carryout under data collection and analysis. Ongoing research predicted that land use drop of cultivated areas from 1983 in 51% to 2020 to 34% [1]. But data is not available to prove the reduction of application in pesticides and fertilizers even though the reduction of cultivated land. It is needed to collect the data to do analysis to predict the affecting to row water quality

Figure 2: Point source of pollution in Gin River Near to Nagoda

5. Results and Discussion

Figure 3 shows the monthly variation of colour during 2010 to 2014 at water quality intake point at Wakwella. Colour is refers to aesthetic appearance of water and provide indicator to quality of row water for which level it has to be treated to make best for human usage without suspicious.

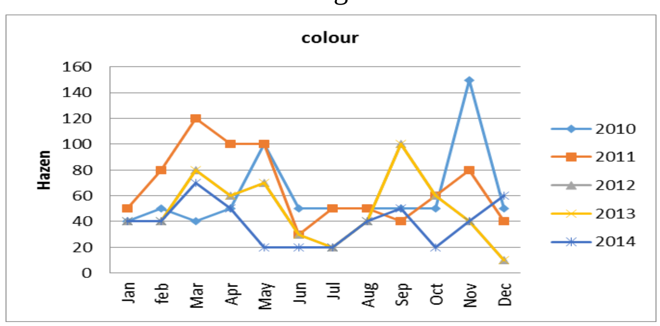

Figure 3: Monthly variation of average color

High colour values were observed during the Northeast monsoon between November and February and Southwest Monsoon between May and September.

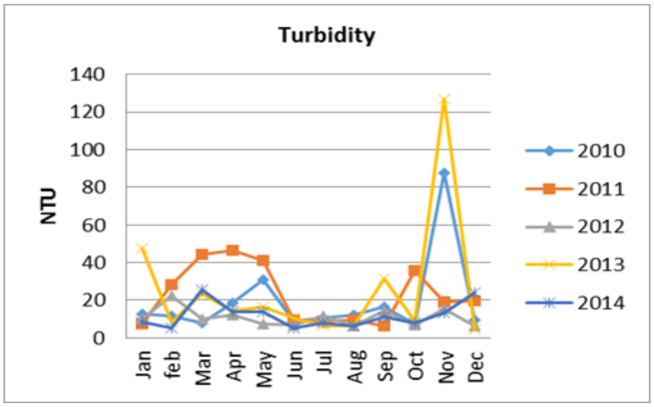

Figure 4: Monthly variation of average turbidity

Figure 4 shows that the monthly variation of turbidity during the same period of colour was monitored. Turbidity is the indicator a measure of water clarity. Turbidity is increased as a resulted of suspended solid particles. Measured results of average colour and turbidity shows that turbidity values were increased the color values also proportionally increased. It was indicated that some amount of suspended particles were dissolved in water and caused to increase turbidity value. Turbidity degrades drinking water quality, aesthetically displeasing opaqueness, producing colloidal material provide adsorption sites for taste and odor, producing chemicals and harmful organism and water treatment may increase.

Figure 5 shows the monthly average conductivity of the row water from 2010 to 2014 and shows that well below the maximum allowable limit even though the sampling point about 2.0km from the coast. There is a salinity barrier across the Gin River constructed in 2004 about 500m downstream of the Wakwella Intake. It was almost closed during high drought period and was affected to maintain the low values of conductivity.

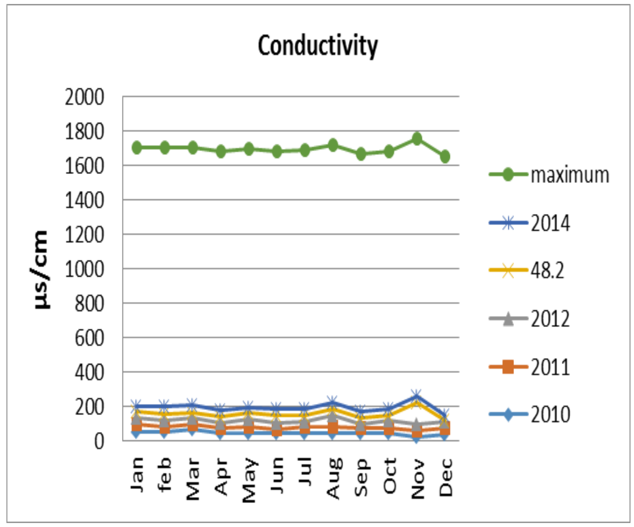

Figure 5: Monthly variation of average conductivity

pH value of row water is a most important chemical parameter with relative to water treatment. Flocculation and sedimentation process is depending on pH value of Row Water. Neutralized pH value should be maintained both consumer taste and in transmission and distribution systems to avoid scaling or corrosion of pipeline. Figure 6 shows average monthly PH value variation in past five years.

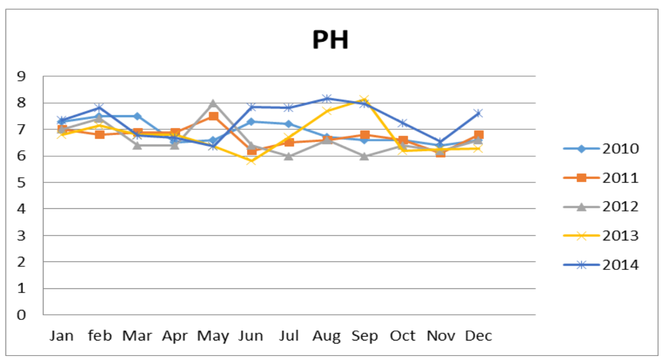

Figure 6: Monthly variation average pH value

It lies between 6 and 8. It is perfect range for row water and can be corrected pH value for optimum dosage of chemical need for water treatment.

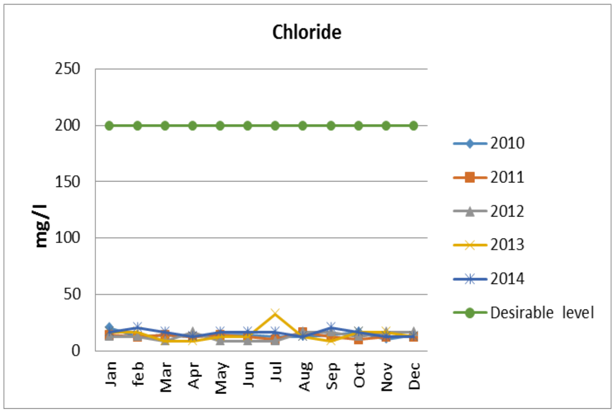

Figure 7: Monthly variation of average chloride

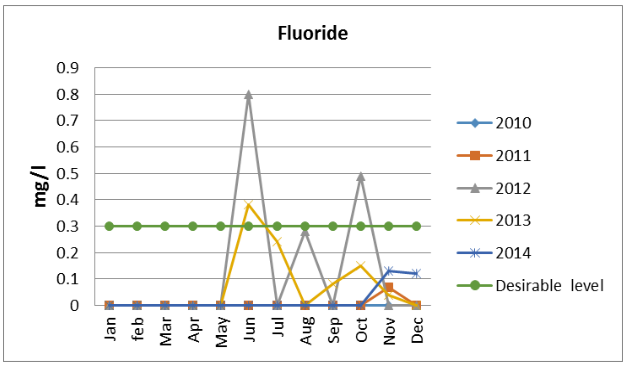

Figure 8: Monthly variation of average fluoride

Figure 7 and Figure 8 shows the monthly average variation of chloride and fluoride from 2010 to 2014. Chloride is well below the desired level. Fluoride is considerably increased than desirable level in year 2012 and 2013 in month of June, August and October.

Figure 9 shows the Total Dissolved Solid (TDS) during the period of 2010 to 2014. TDS refer to suspended matter dissolved in water. Solids may affect water or

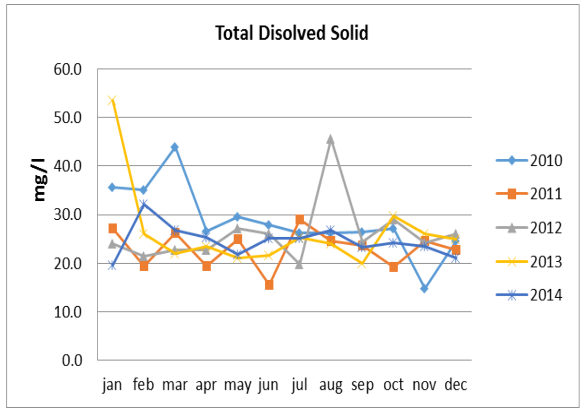

Figure 9: Monthly variation of average Total Dissolved Solids (TDS)

effluent quality adverse in a number of ways. Increased TDS may impart a bad odour or taste to drinking water as well as cause scaling of pipe and corrosion. High TDS level indicates water hardness in respective sampling station. Desirable value of TDS is 500 mg/l and results reveal that far below from desirable level and there is no considerable variation during the observed period. TDS is also considered as general indicator of overall water quality (Tambekar etal 2012). It is a measure of organic and inorganic materials dissolved in water.

Figure 10 shows the variation of monthly average Hardness values and figure 11 shows the variation of monthly average of Sulphates values and they are well below the desired level from 2010 to 2014.

Figure 12 shows the variation of monthly average of Calcium values and they are also well below the monitoring periods.

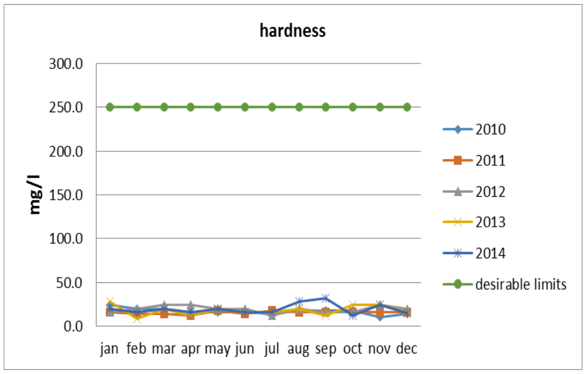

Figure 10: Monthly variation of average hardness

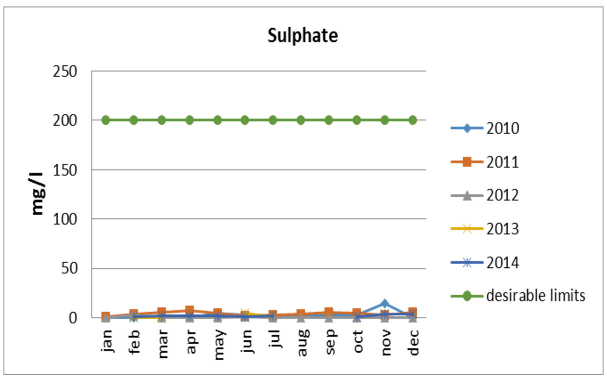

Figure 11: Monthly variation of average Sulphates

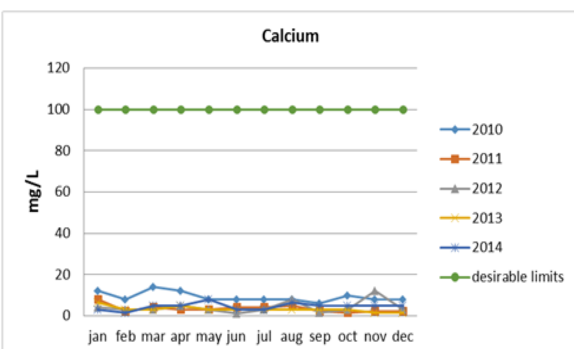

Figure 12: Monthly variation of average calcium

Figure 13 shows the monthly variation of average iron values and it was shown that is exceeded the desirable level. It was also shown that it increase to maximum level during February to June and September to December. That is the North East and South West Monsoon rain periods of the particular years. Excess iron should be removed through the Aeration, Flocculation, Sedimentation and Filtration. Cost of chemical consumption may increase during rainy periods.

Figure 14 shows Coliform values in row water and it is excessively increased than desirable limits. That is Gin River is highly polluted with fecal. Therefore proper disinfection method is to be applied in water treatment process.

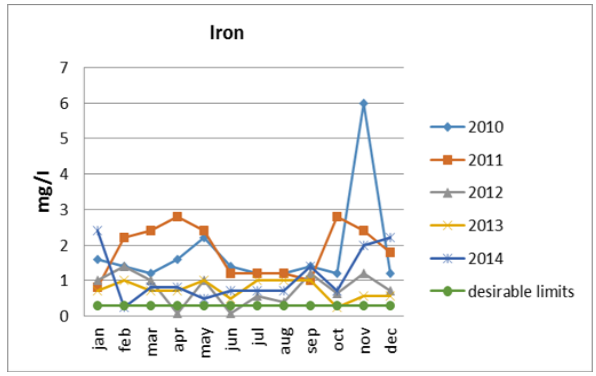

Figure 13: Monthly variation of average iron

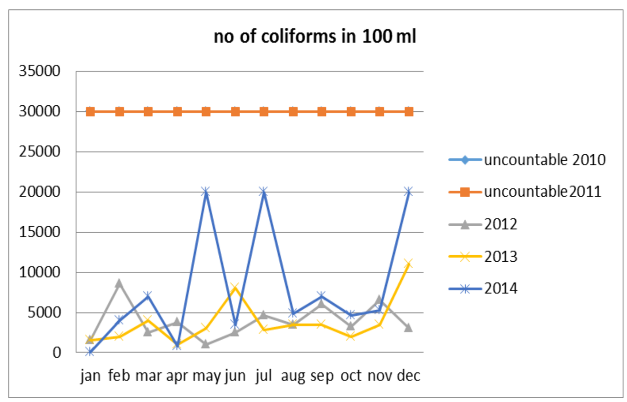

Figure 14: Monthly variation of average coliforms

Dissolved Oxygen (DO) and Biochemical Oxygen Demand (BOD) are measured in year 2015 at Wakwella and Baddegama intakes and shown in figure 15 and figure 16. Dissolved Oxygen is higher at Baddegama intake than Wakwella intake. Main reason for this variation is flow pattern of Gin River. Baddegama intake is situated up stream and river flow is little turbulent and Wakwella intake is situated in downstream and laminar flow is more experience. Monitoring oxygen concentration is a convenient way to feel the clause of aquatic ecosystem. Dissolved Oxygen is an important water quality parameter in assessing water pollution. The change in oxygen content leads to undesirable odor, under anaerobic conditions. Both intakes are shown that from January 2015 to August 2015 decrease DO by 8mg/l to 6mg/l. Desired level of DO is 4mg/l. It is needed to continue continuous monitoring of DO in both intakes.

Figure 16 shows the BOD variation in year 2015 in both Baddegama and Wakwella intakes. It was not shown any tendency to increase and range of variation is1mg/l to 2 mg/l during the year 2015. Monitoring of BOD and COD are to be further assessing of row water quality as a part of WSP.

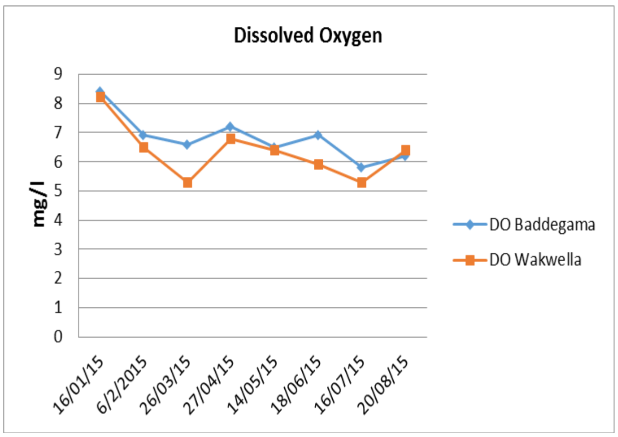

Figure 15: Average Dissolved Oxygen variation at Intakes Wakwella and Baddegama

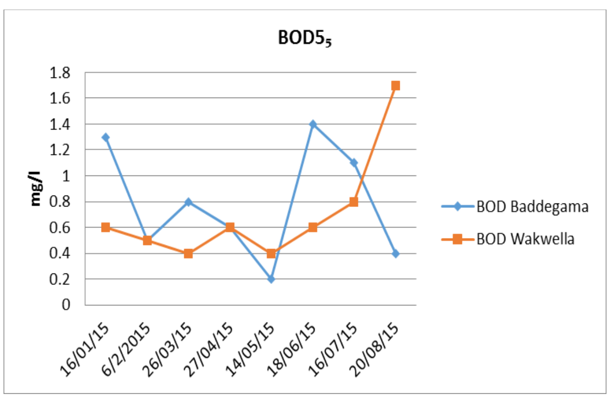

Figure 16: Average Biochemical Oxygen demand in Intakes Wakwella and Baddegama

6 Conclusion

It was noted that some water quality parameters at both intakes in Wakwella and Baddegama were in acceptable level for Sri Lankan standard inland surface water quality standard. Water contaminated from fecal pollution is more obvious than other agrochemical or industrial pollution. Progressive monitoring of other water quality parameters such as heavy metal and agrochemical related element is vital important to long term sustainability of WSP in GGWSS. Direct users of Gin River water for domestic and other related activities are in questionable due to presence of high uncountable coliform counts. Water contamination will increase in future increasing water stress further by considering industrial and agricultural demand. The main purpose of this paper is to initiate water quality modeling as a tool to assist in analyzing various scenarios and developing suitable water quality management option in the Gin River from upstream to two intakes points.

7 Acknowledgements

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SECM/15/118

Application of Water Quality Simulation for Water Safety Plan at Mahaweli River Basin, Kandy

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Abstract: Water Safety Plan (WSP) is an effective risk assessment framework to elaborate possible risks for water supply systems. However, risk quantification of "severity" and "likelihood" of specific incidents is challenging especially for the risks at water sources. Water quality simulation is an essential tool for predicting the risks and applying effective countermeasures, while data availability and usability for practitioners remains as obstacles for implementation. The objective of this study is to develop appropriate simulation methodology for the risks at water source of all the water supply systems in Sri Lanka. In this report, we focused on the Mahaweli river basin in Kandy. Reviewing the existing WSP at Greater Kandy WTP, the contamination of intake water by the leachate from neighbouring solid waste damping site was considered as a significant risk. We developed dynamic hydraulic model and water quality model which can configure input and output data on Microsoft Excel interface. As a result of the simulation, it was implied that the contaminant from the leachate can flow back to the intake of the WTP due to density flow. Re-risk assessment for possible countermeasures showed the structure modification and leachate treatment are effective to mitigate the risk for water quality hazards.

Keywords: Risk Assessment, Raw Water Quality, Water Quality Simulation, Water Safety Plan

1. Introduction

1.1 Background and Objectives

The Security of water safety for piped water is the important issue in Sri Lanka since the piped water coverage has been dramatically improved recently all over the country. Facing the end year of Millennium Development Goals (MDGs), In 2015, access to piped water on premises is 12% while it has been 34 in 1990, that is to say that the served population has been 3.5 times increased in only 25 year [1]. The water supply expansion also encompassed the change of water sources especially from groundwater to surface water. Population increase, urbanization and groundwater contamination such as fluoride and arsenic accelerated the surface water use as water sources. Thus the water safety of surface water source is increasingly important as the countries drinking water sources.

Surface water contamination is common and historically indigenous in Sri Lanka. The country has been historically doing indirect water reuse as typically described as "Tank Cascade System". There are many lakes (Wewa) in midst of the rivers to store and diverge water to use for lives and irrigation [2]. The system is efficient in terms of the utilization of limited fresh water resources while it also definitely causes the risk of contamination. It is still common in Sri Lanka that untreated sewage and irrigation drainages inflows at the upstream river from the intake of downstream regions. As a result, algal bloom is reported in Kandy Lake which produces toxic substances which may not properly be dealt with solely by conventional water treatment [3]. Due to high population growth and economic development, this contamination issues are being important all over the country.

There are also increasing concern for contamination from more artificial point sources such as wastewater from factories and leachate from garbage dumping sites. Especially for wastewater from factories, it is of large public concern because of the serious accident at the Kelani River where large volume of oil and greases were discharged and affected 680,000 m3/day of water supply mainly to capital city, Colombo [4].

Facing these problems which could affect water safety of piped water sources, Water Safety Plan (WSP) has been currently introduced at a high speed. WSP is a participatory management approach for water supply systems which includes comprehensive risk assessment and management [5]. WSP has been widely implemented among water utilities all over the world. Despite the importance for practitioners, limited research publications are available. More scientific researches are needed for WSP because risk assessment may contain critical uncertainties without proper scientific approaches are applied.

Model studies are one of the possible tools to solve these problems on water safety plan because it gives scientific evaluation to the risk assessment and outputs visual results which could be understandable for the non-technical stakeholders. There is few previous study of water quality simulation for Sri Lankan water environment. One study in Kandy Lake mainly focused on the water environmental improvement for the lake [6]. The issue of previous simulation model was the complexity of the modelling procedure and updating practices. Since the water safety plan needs continuous review and update, it is critical that the model can be easily handled by the practitioners with familiar interfaces. Thus, simple, easy for handling and technically established methodology should be applied to the water quality modelling.

The objective of the study is (i) to review water quality issues in Sri Lanka from actual water safety plan and (ii) to establish appropriate simulation methodology for the risks at water source of all the water supply systems in Sri Lanka. We selected the Mahaweli River basin as a model case to utilize water quality model for WSPs.

1.2 Site Description

Mahaweli River is a longest river (335km) in Sri Lanka which also has the largest drainage basin in the country [7]. The river water is also a major water source of Kandy which is second largest city in the country. There are two major intakes for Water Treatment Plants (Kandy South WTP and Greater Kandy WTP) and two intakes for small WTP (Polgolla WTP and Kandy Municipal WTP). Both major WTPs formulated WSPs for each water supply system under the facilitation of National Water Supply and Drainage Board (NWSDB). Since there is no proper wastewater disposal system, untreated domestic sewage is released directly into the Mid-canal [8] and the Mahaweli River.

Gohagoda dumping site is one of the main dumping site of Kandy Municipal Council (**Photo 1**). The site started to dump waste since 1970s. The waste includes household, commercial, market, healthcare and industrial wastes. According to a previous research, “there are no environmental protections measures taken for solid waste disposal” [9]. The dumping site is located just adjacent to the intake site. There is the discharge point of the leachate from the dumping site at approx. 100 m downstream of the intake station for raw water (Figure 1).

1.3 Review of existing Water Safety Plan

We have reviewed the Water Safety Plan for Greater Kandy Water Supply System to identify the risk for water safety. In the identification of

Figure1: Gohagoda Dumping Site

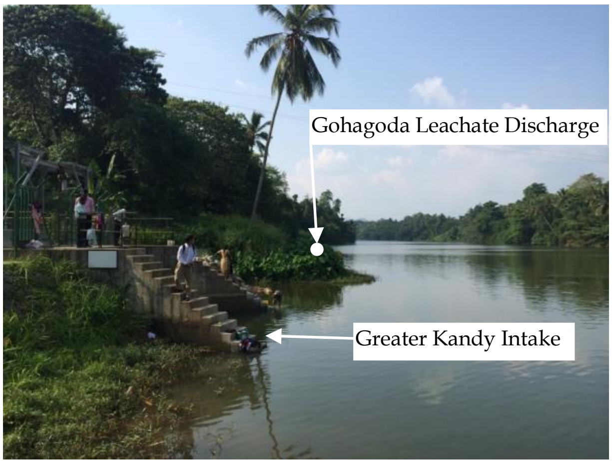

Figure 2: Intake of Greater Kandy Water Treatment Plant and Leachate Discharge Point

Table 1: Hazards of Greater Kandy WTP assessed as "Very High" Risks by assessment of raw risks

Location /Process step	Hazardous event	Hazard type	Residual Risk after Control Measures
Source	Pathogenic contamination from septic tanks and waste from Kandy city through Middle canal (Meda-Ela)	Physical/Chemical/microbiological	Very High
Source	Leachate from Kandy city garbage dumping site entering intake	Physical/Chemical/microbiological	Very High
Source	Pollution by agrochemicals during spraying season	Chemical	Very High
Distribution chamber	Power failure at WTP	Chemical & Microbial	Very High
Service reservoirs	Unauthorized personnel entering premises	Chemical & Microbial	Low
Pipe network	Contamination of treated water	Microbial	Very High

hazards and hazardous events and assessing the risks (module 3), six major risks are rated as "Very High" risks as raw risks (without control measures) (Table 1). Three of six risk incidents are of source problems such as the pollution from contaminated distributaries (Mid-canal), leachate from Kandy city dumping site (Gohagoda dumping site) and possible agrochemical contamination. The risks were evaluated by water quality record or observation. In this study, we focus on the leachate from Gohagoda dumping site, because the problem is likely to be most unknown matter in terms of possibility of incidents.

2. Materials and Methods

2.1 Model Framework

<u>1) Overall Model Framework</u>

The study area is Mahaweli River including Mid-canal and Kandy Lake as shown in Figure 3. After the confluence with the Mid-canal in the Mahaweli River, there is the reservoir created by Polgolla Barrage for the purpose of hydropower, irrigation and water supply. In Polgolla Reservoir, there is the intake station for Grater Kandy WTP, which raw water quality may be affected by leachate from Gohagoda dumping site.

Figure 3: Study Area

The overall model framework in this study is shown in Figure 4.

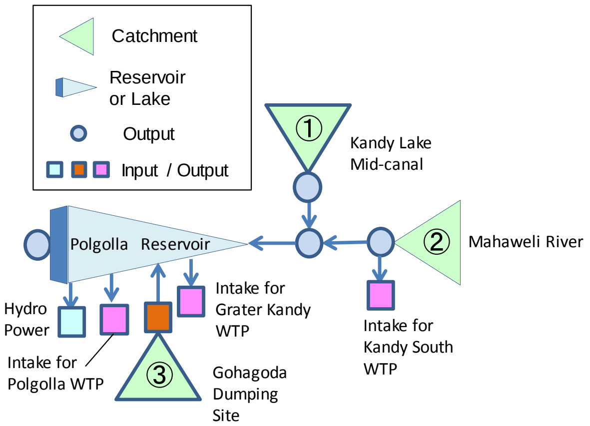

Figure 4: Overall Model Framework

2) Vertical Two Dimension Model for Reservoir

To analyze hydraulic and water quality situation in reservoir, the vertical two dimension model was applied under assumption of hydrostatic pressure distribution (Figure 5). This simulation model can calculate the distribution of water temperature and water quality from hydraulic variables in vertical and flow direction. The vertical two dimension model consists of hydraulic model and water quality model.

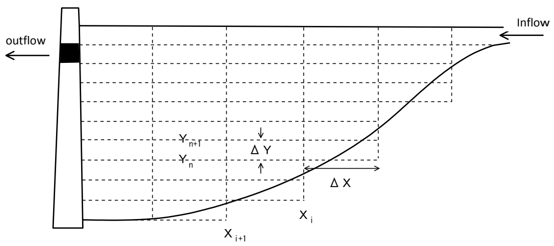

Figure 5: Mesh Division for Reservoir

a. Hydraulic Model

As for hydraulic model, hydraulic variables (e.g. flow velocity) are calculated from equation for continuity and equation for conservation of momentum.

i) Equation for Continuity

$$\frac{\partial \bar{u}}{\partial x}+\frac{\partial \bar{v}}{\partial y}=0 \qquad (1)$$

ii) Equation for Conservation of Momentum

$$\frac{\partial \bar{u}}{\partial t}+\frac{\partial}{\partial x}\left(\overline{uu}\right)+\frac{\partial}{\partial y}\left(\overline{uv}\right)=-\frac{\partial}{\partial x}\left(\frac{P}{\rho}\right)+\frac{\partial}{\partial x}\left(A_x\frac{\partial \bar{u}}{\partial x}\right)+\frac{\partial}{\partial y}\left(A_y\frac{\partial \bar{u}}{\partial y}\right) \qquad (2)$$

$$\frac{\partial P}{\partial y}=-\rho g \qquad (3)$$

Where x and y: coordinate in flow and vertical direction, u and v: flow velocity in x and y direction, ρ: water density ($\rho = \alpha T^2+\beta T+\gamma$; α, β, γ: constant, T: water temperature), P: pressure, t: time, A_x and A_y: coefficient of eddy viscosity in x and y direction.

The density flow is analyzed through calculating of density depending on water temperature.

b. Water Quality Model

As for water quality model, water temperature and water quality are calculated from equation for concentration balance including advection and diffusion.

i) Equation for Temperature Balance

$$\frac{\partial T}{\partial t}+\bar{u}\frac{\partial T}{\partial x}+\bar{v}\frac{\partial T}{\partial y}=\frac{\partial}{\partial x}\left(K_x\frac{\partial T}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_y\frac{\partial T}{\partial y}\right)+\frac{H}{\rho C_w} \qquad (4)$$

ii) Equation for Water Quality Balance

$$\frac{\partial C}{\partial t}+\bar{u}\frac{\partial C}{\partial x}+\bar{v}\frac{\partial C}{\partial y}=\frac{\partial}{\partial x}\left(D_x\frac{\partial C}{\partial x}\right)+\frac{\partial}{\partial y}\left(D_y\frac{\partial C}{\partial y}\right)+S \qquad (5)$$

Where T: water temperature, H: unit volume, C_w: absorbed heat by solar insolation, t: time, K_x, and K_y: diffusion coefficient for temperature in x and y direction, D_x and D_y: diffusion coefficient for water quality in x and y direction, S: term for water quality variation in ecological model.

Term for water quality variation is calculated from the ecological model as shown in Figure 6 including growth of phytoplankton by intake of nutrient (nitrogen and phosphorus).

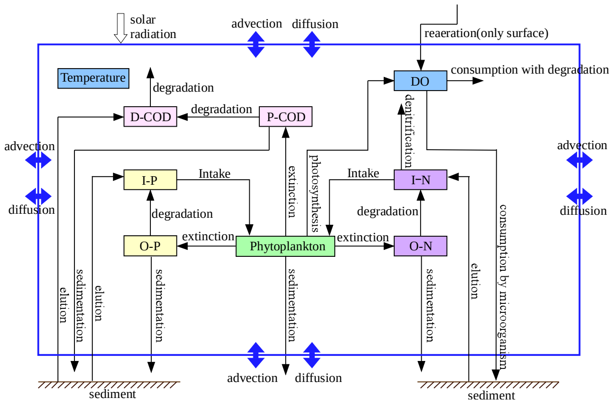

Figure 6: Ecological Model

3) Catchment Model

The discharged water quality (COD, nitrogen and phosphorus) from catchment is calculated from the annual pollution load from nom-point and point sources (Figure 7). The annual pollution load is calculated from residential and tourist population, and land use. The daily discharged water quality is calculated through L-Q (Load-Quantity of flow) equation which parameters are identified by annual pollution load and observed water quality. The impact of load reduction measures in catchment including installation of sewerage system can be evaluated by this catchment model.

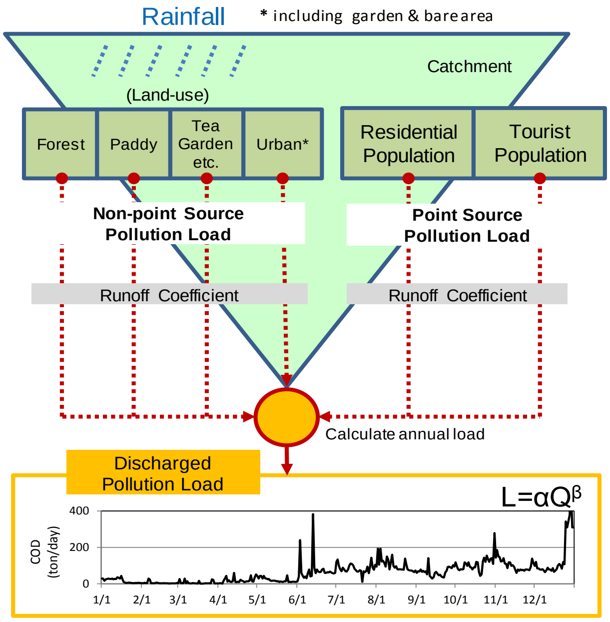

Figure 7: Catchment Model

(2) Datasets

The calculation conditions for Polgolla Reservoir are shown as follows. All the datasets were provided by courtesy of the water environment laboratory of the Department of Civil Engineering, University of Peradeniya.

1) Topographical Condition

Polgolla Reservoir which length is 7.5 km was divided into meshes at intervals of 0.5 km in flow direction and 1.0 m in vertical direction (Figure 8).

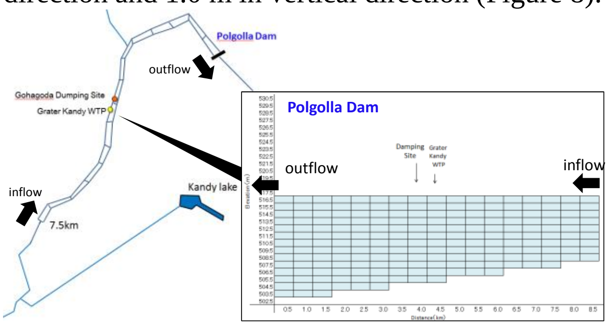

Figure 6: Mesh Distribution

2) Meteorological condition

The average meteorological condition (e.g. temperature, solar insolation, relative humidity, wind velocity and cloudiness) was set according to monthly meteorological data observed in Kandy and nearby cities.

3) Hydrological Condition

The inflow condition for Polgolla Reservoir was set according to the actual record of inflow volume in year 2014. The hydrological condition in 2014 corresponds to dry year in recent 7 years.

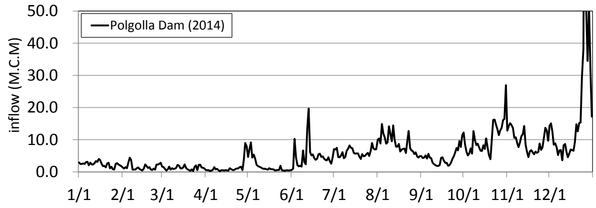

Figure 9: Inflow Condition

4) Water Quality Condition

The target items for water quality simulation are water quality items calculated by ecological model shown in Figure 6 (COD, nitrogen, phosphorus, DO and etc.) and heavy metals (lead (Pb) and zinc (Zn)). The water quality condition was set according to discharged water quality calculated through the catchment model. The concentration of Pb and Zn was set according to the observed data of Mahaweli River and leachate water of Gohagoda dumping site. It is necessary to set more accurate and detailed water quality condition through water quality monitoring and data accumulation.

Table 2: Water Quality Condition

	①Inflow from Kandy Lake	②Inflow from Mahaweli River	③Leachate from Gohagoda Damping Site
Catchment Area (km^2)	255	1063	0.06
Discharge Volume (m^3/s)	13.1	54.6	0.006
COD (mg/L)	22.4	8.0	700
T-N (mg/L)	5.9	1.9	700
T-P (mg/L)	1.1	0.4	8.0
Pb (mg/L)	0.005	0.005	12.9
Zn (mg/L)	0.145	0.145	700

2.3 Interface of Simulation Model

We developed hydraulic model and water quality model for Polgolla Reservoir, which can configure input and output data on Microsoft Excel interface. Users can easy to handle the simulation model (e.g. change of input data and evaluation of simulation results).

3. Results and Discussion

3.1 Simulation Results for Water Intake Quality

The current water intake quality in 2014 was calculated through the water quality simulation model developed by this study. Figure 10(A) and (B) show the simulation results in Polgolla Reservoir. The simulation result shows that substances in leachate discharged from dumping site can flow back to upstream by approx. 2 km.

The phenomena can be explained by the density flow due to water temperature differences in the reservoir. The water temperature between surface and bottom layer in the reservoir is different due to sunshine and low flow volume especially in the dry season. Then the difference in temperature makes density flow including following flow in middle layer and backflow in surface and bottom layer. As a result, the contaminated substances discharged from dumping site can flow back to upstream mainly by density flow.

Figure 11 shows the annual variation result of Pb for water intake quality at Grater Kandy WTP. The result shows Pb concentration at water intake is higher than background value in the Mahaweli River throughout the year. The concentration in the dry season (January to April) is especially higher than that in the rainy season. The result also shows that seasonal variability of the contamination level results from the backflow of contaminated substances in leachate from dumping site.

As a result of the simulation, it was implied that the contaminant from the leachate can flow back to the intake of the WTP due to density flow. The simulation results of Pb and Zn don’t exceed the WHO guideline values. However, there is a possibility that the actual contaminant concentration of intake water is higher than simulation results, because the water quality conditions in this study were set by constant values due to lack of detailed monitoring data.

It is necessary to conduct detailed water quality monitoring in reservoir and leachate of dumping site for not only contaminated substances but also water temperature in vertical direction for evaluation of density flow. Through the water quality monitoring, accuracy of simulation model can be improved and more detailed impact on intake water quality can be evaluated.

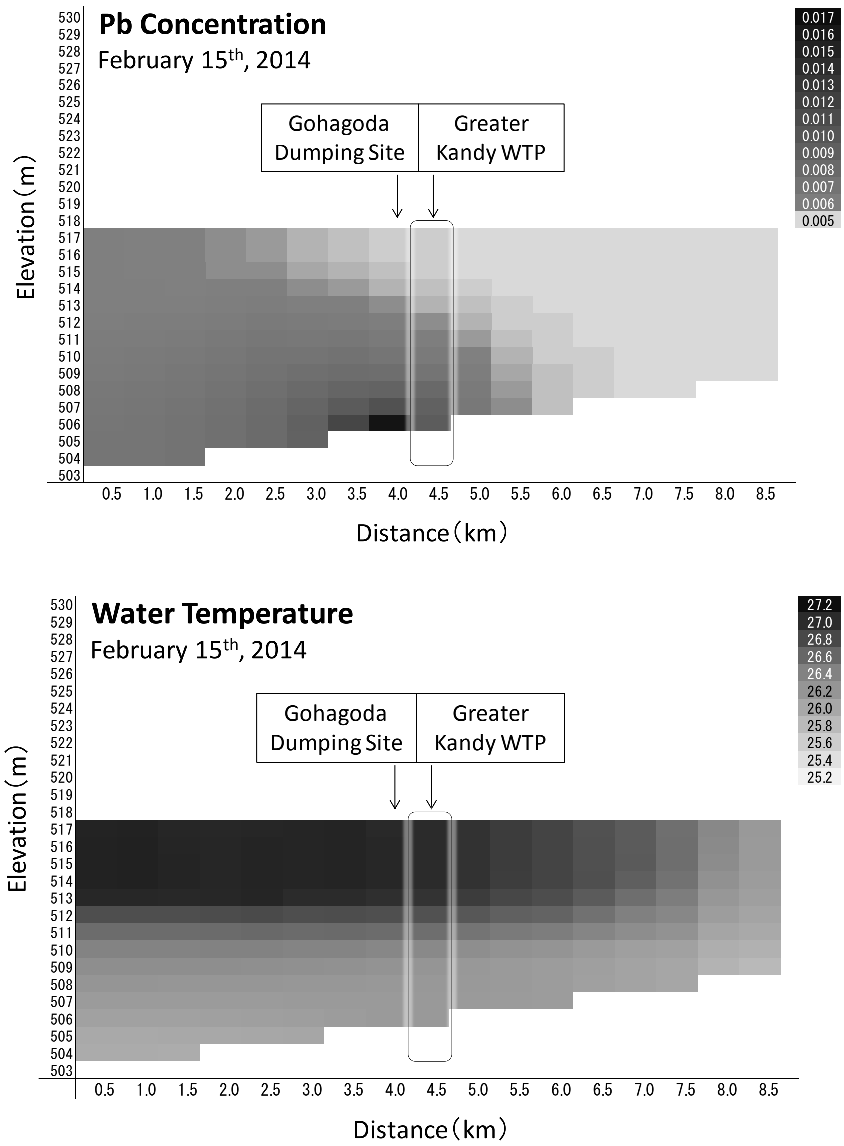

Figure 10: (A) Simulation Result of Water Quality Distribution,
(B) Simulation Result of Water Temperature Distribution

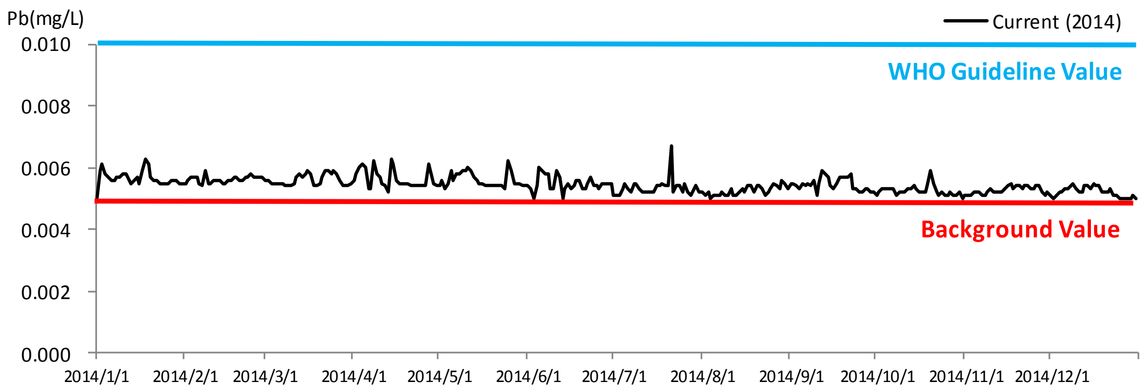

Figure 11: Simulation Result of Pb Annual Variation

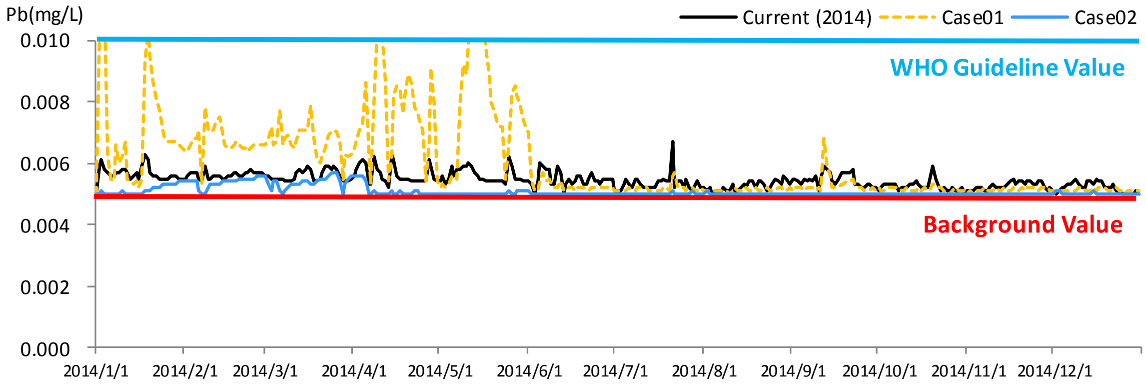

Figure 10: Simulation Result of Pb for Control Measures

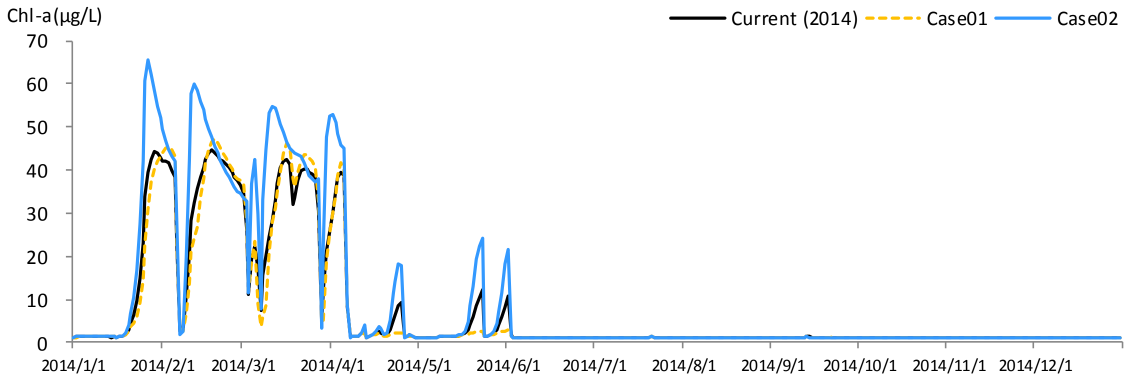

Figure 13: Simulation Result of Chl-a for Control Measures

3.2 Control Measure Identification and Re-risk Assessment

To assess possible control measures for mitigation of the risk for intake water quality hazards, the impact on improvement of intake water quality by the following 2 control measures was calculated through the developed simulation model.

Table 3: Control Measures

Case	Control Measure
Case 01	Installation of fence in bottom layer
Case 02	Intake of raw water from surface layer

As for Case 01 (installation of fence), the intake water quality of Pb goes over 0.01 mg/L while the maximum concentration of the baseline case (without any countermeasure) is about 0.006 mg/L (Figure 12). The results shows the water quality could be worsen by the countermeasure in terms of heavy metals. The possible explanation is that the backflow in surface layer increases and contaminated substances in surface stay longer in bottom layer at intake location.

As for Case 02 (intake from surface), the intake water quality of Pb is lower than that of baseline case throughout the year. Thus it is implied that intake of raw water from surface is effective control measures to mitigate the risk for intake water quality hazards. The improvement can be explained by the fact that the contaminant concentration in surface layer is lower than that in bottom layer due to density difference. However, the intake water quality of Chl-a (chlorophyll-a) in Case 2 was increased in the dry season (Figure 13) due to growth of phytoplankton. Thus it is necessary to care the impact on water treatment by increase of phytoplankton.

In these two assessments of countermeasures, Case 02 was implied to be effective in terms of improving heavy metals water quality while it may affect the risk of toxic substances formulated by phytoplankton in raw water. We could not attain the conclusion that some realistic countermeasures can surely reduce the risk. Possible other effective alternative for the countermeasures is the installation of leachate treatment facility in the dumping site. The method is more direct and effective control measure while there are many stakeholders and complicated process for implementation. The flashing discharge from lower gate at Polgolla Barrage is also effective control measure. In that case, it is necessary to prepare the proper operation rules for gates in the barrage.

In this research, we could not reach quantitative re-risk assessment utilizing the results because the concentration of modelled substances (Pb) did not go beyond the WHO guideline level and thus it is less likely harmful for human body. More research is needed for identifying the possible harmful substances such as agro-chemicals, carcinogenic substances and other unknown substances. Re-risk assessment for possible countermeasures through water quality simulation could show the structure modification and leachate treatment might be effective to mitigate the risk for water quality hazards.

4. Conclusions

The challenge for water safety was reviewed and some practical tools to assess the risks of water safety were developed. We have attained following conclusions in this study:

1) From a review of Water Safety Plan of a water supply system (Greater Kandy WTP), water source issues are evaluated as "Very High" because of the unavailability of observed data. Contamination backflow of the leachate from the adjoining solid waste dumping site was identified one of the critical and unknown risks.

2) Water Quality Simulation model for Pollgolla Reservoir was developed to evaluate the risks to water safety. The simulation model was proven to be effective to show the result visually and logically. It was implied that;

- Leachate contamination can flow back by 2km due to density flow.
- Fences at Intake (Case 01) have implied to have adverse impact on water quality and Intake at surface (Case 02) found to be effective while it has the concern to be more affected by toxic phytoplankton.

Acknowledgement

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CLIMATE CHANGE IMPACT PREDICTION IN UPPER MAHAWELI BASIN

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Abstract: Upper Mahaweli basin is the origination of the main water source of Sri Lanka which is the Mahaweli River. Therefore it is a timely requirement to identify the future climate trends on the basin, to take suitable adaptation strategies. Statistical Downscaling model (SDSM) was used to predict future rainfall patterns of the study area. Observed point rainfall data of ten gauging stations within the study area and Global Climate Model (GCM) data of Hadley Centre Coupled Model, Version 3 (HadCM3) were used for model calibration and validation processes. A representative data set for the study area was generated using Thiessen polygon method from the observed rainfall data of selected gauging stations. Quality of the input data was checked prior to the model calibration. Daily rainfall was forecasted from 1961 to 2099 under A2 (high emission scenario) & B2 (low emission scenario) defined by Intergovernmental Panel on Climate Change (IPCC). Under A2 scenario the total annual rainfall, maximum annual rainfall and annual averaged daily rainfall show an increasing trends and under B2 scenario all the above mentioned parameters show decreasing trends. But the recorded decreasing trends are insignificant.

Keywords: Global Climate Models, Statistical Downscaling model, Emission scenarios

1. Introduction

The climate affects to mankind in a wide variety of ways mainly through precipitation and solar radiation. The climate change phenomenon has become a major concern in modern world due to its adverse effects.Climate change prediction is important to understand accompanied impacts and necessary adaptation to minimize adverse impacts. Simulating the natural atmosphere by using computer models is the main tool that is used for climate prediction.

General Circulation Models or Global Climate Models (GCMs) are mathematical models used to simulate the natural atmosphere. GCM outputs cannot be used directly due to the mismatch in the spatial resolution between GCMs and hydrological models. Then the process of downscaling is required to match those spatial resolutions. In order to understand climate change impacts at basin scale GCMs data are downscaled using standard downscaling methods.

There are two standard downscaling methods namely Dynamical Downscaling (DD) and Statistical Downscaling (SD). Dynamical Downscaling (DD) generates regional scale information using Regional Climate Models (RCM) with coarse GCM data used as boundary conditions and Statistical Downscaling (SD) develops quantitative relationships between large scale atmospheric variables (predictors) and local surface variables (predictands) by using statistical methods [1].

Statistical downscaling methodologies have several practical advantages over dynamical downscaling approaches. In situations where low–cost, rapid assessments of localized climate change impacts are required, statistical downscaling represents the more promising option [2].

Statistical Downscaling model (SDSM) is a combination of Multiple Linear Regression (MLR) and the Stochastic Weather Generator (SWG). Quality control, Transform data, Screen variables, Calibrate model, Weather generator, Summary statics, Frequency analysis, Scenario generator, Compare results and Time series analysis are the key functions of the SDSM model.

SDSM model was used by Dharmarathna [3] to select adaptation measures to sustain rice production in Kurunagala District under the impacts of climate change. Minimum and maximum temperature and rainfall data were downscaled from GCM outputs and the forecasted daily maximum and minimum temperatures showed an increasing trend under both A2 and B2 scenarios while annual rainfall did not show a significant increasing or decreasing trend.

De Silva [4] used the SDSM model to downscale past and future GCM data available at a coarse resolution to the Kelani basin. In this analysis the Kelani basin was divided into two sub basins as lower basin and upper basin. Total annual rainfall was shown an increasing trend for both upper and lower basins under high and low emission scenarios.

2. Study Area and Methodology

The area up to the Polgolla barrage of Mahaweli basin which covers an area of 788 km^2 was selected as the study area. Observed rainfall data of ten gauging stations were used for models calibration and validation. Figure 1 illustrates the study area and selected gauging stations.

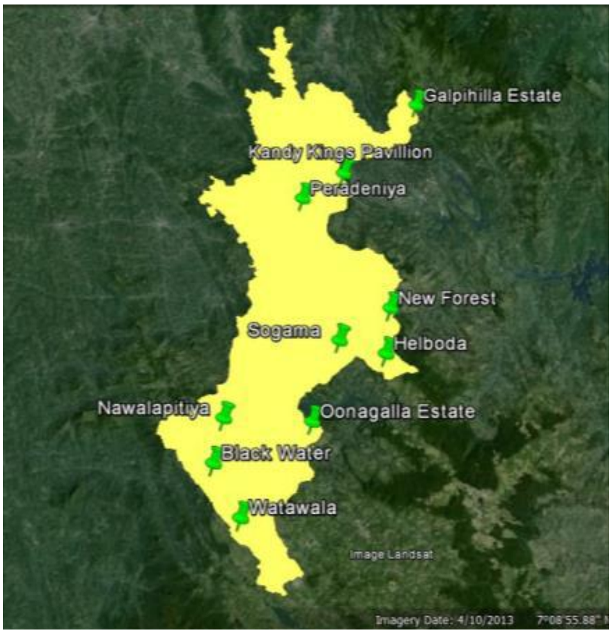

Figure 1: Study area and gauging stations

Observed point rainfall data were spatially distributed over the basin area by Thiessen polygon method and one representative data set for whole study area was generated to feed to the SDSM model.

Statistical Downscaling Model (SDSM) version 4.2.9 was used for climate modelling and rainfall was forecasted up to year 2099 using GCM data**.** GCM data were downloaded from Canadian Climate Scenarios Network [5]. National Centers for Environmental Prediction (NCEP_1961-2001) data set was used to calibrate and validate the model. Then for future rainfall predictions, Hadley Centre Coupled Model, Version 3 (HadCM3) data sets for A2 and B2 scenarios (H3a2a_1961-2099 and H3b2b_1961-2099) were used.

Future forecasts of annual averaged daily rainfall, annual maximum daily rainfall and monthly averaged total precipitations were made under to different emission scenarios namely A2 scenario (high emission case) and B2 scenario (low emission case) of IPCC.

2.1 Model Preparation

Modelling process was set as conditional which assumes an intermediate process between regional forcing and local weather. Forth root transformation was used to convert the skewed rainfall distribution into a normal distribution. The value of variance inflation, which controls the magnitude of variance inflation in downscaled daily weather variables, was set as 18 and the value of bias correction, which compensates for any tendency to over– or under–estimate the mean of conditional processes by the downscaling mode was set as 0.8.

2.2 Model Calibration

Rainfall data from 1971 to 1986 were used for model calibration. The Screening Variable option was used in the choice of appropriate downscaling predictor variables for model calibration. Table 1 illustrates the selected predictor variables among the available 26 variables in SDSM model for the Upper Mahaweli basin.

Table 1: Selected predictor variables for model calibration

Predictor Variable	Description
ncepp_fas.dat	Surface airflow strength
ncepp_zhas.dat	Surface vorticity
ncepp5_fas.dat	500 hPa airflow strength
ncepp5_uas.dat	500 hPa zonal velocity
ncep5_zas.dat	500 hPa vorticity
ncepr850as.dat	850 hPa geopotential height
nceprhumas.dat	Near surface relative humidity
ncepshmas.dat	Surface specific humidity

The simulated values of annual averaged daily rainfall, annual maximum daily rainfall and monthly averaged total precipitations were used to compare with observed rainfall data. Further, the number of dry days of simulated and observed were compared for the above time period. Figure 2 illustrates the comparison done for annual averaged daily rainfall.

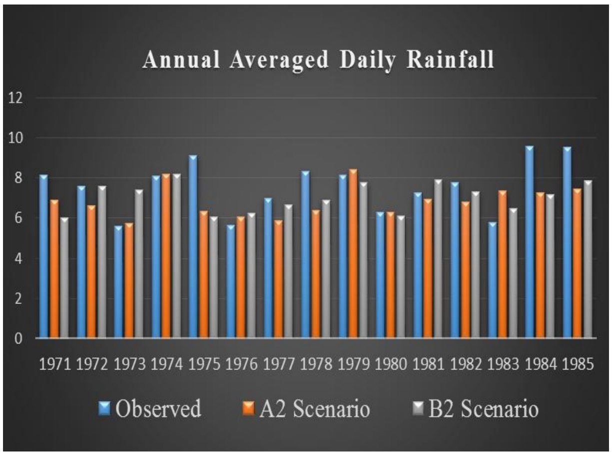

Figure 2: Variation of annual average daily rainfall

The Mean Model Error Percentages (MME %) were calculated to compare the simulated (SR) and observed (OR) rainfall values.

$$\text{MME \%} = (SR - OR) / OR \; x \; 100 \; \% \qquad (1)$$

Figure 3 illustrates the calculated MME percentages for annual averaged daily rainfall under two emission scenarios.

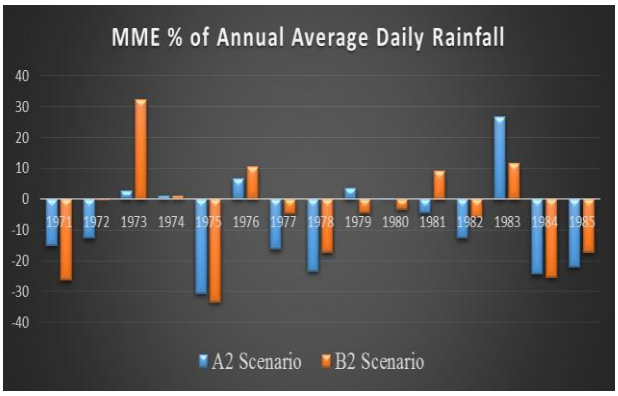

Figure 3: MME% for annual averaged daily rainfall

When calibration period is considered averaged mean model error % values were 13, 13 and 18 for averaged annual daily rainfall, monthly rainfall and number of dry days respectively under A2 emission scenario. Under B2 scenario respective values were 12, 13 and 15.

3.3 Model Validation

Observed rainfall data from year 1986 to 1993 (8 years) were used to validate the model by keeping the same values for variance inflation and bias correction which were used in model calibration stage. Figure 4 shows the comparison between observed and simulated rainfall values for validation period.

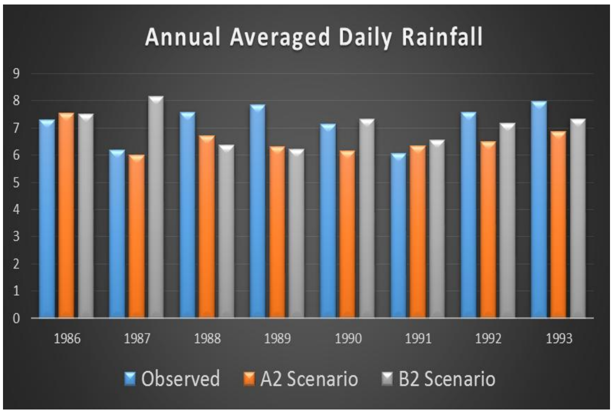

Figure 4: Model validation – Annual averaged daily rainfall

When validation period is considered averaged MME % values were 10, 17 and 10 for averaged annual daily rainfall, monthly rainfall and number of dry days respectively under A2 emission scenario. Under B2 scenario respective values were 12, 13 and 8.

3. Results and Discussion

Calibrated model was used to forecast daily rainfall from 1961 to 2099 under both A2 and B2 scenarios. Following figures illustrate the Time Series Graphs (TSG) derived for annual average daily rainfall, annual total rainfall and annual maximum rainfall under both A2 and B2 scenarios for the study area.

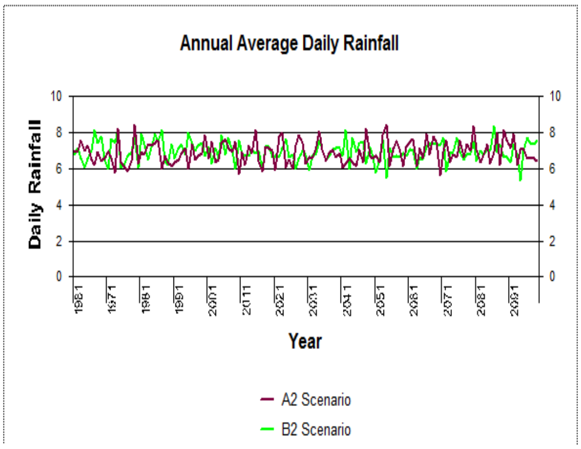

Figure 5: TSG of annual average daily rainfall

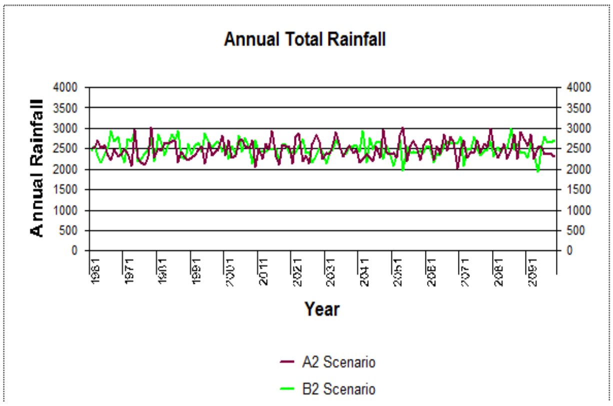

Figure 6: TSG of annual total rainfall

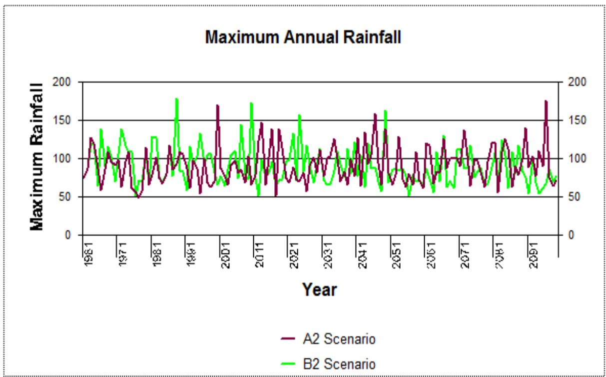

Figure 7: TSG of maximum annual rainfall

Best fit lines for each time series plot under both A2 and B2 scenarios were generated to identify the trend of each variation. Figure 8 and Figure 9 illustrate the best fit lines generated for annual total precipitation under A2 & B2 scenarios.

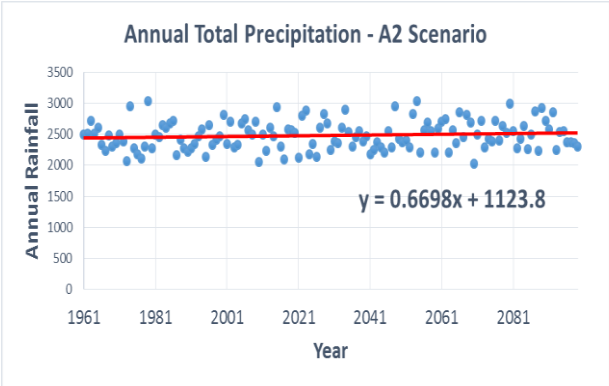

Figure 8: Best fit line for annual total precipitation under A2 scenario

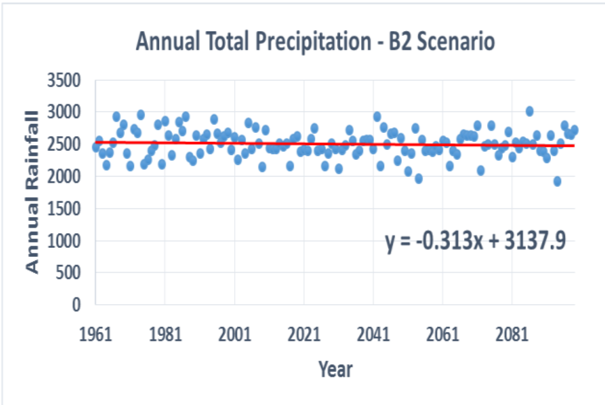

Figure 9: Best fit line for annual total precipitation under B2 scenario

Annual total precipitation, maximum annual rainfall and annual averaged daily rainfall show an increasing trends of 0.7 mm per year, 0.1 mm per year and 0.002 mm per year respectively under A2 scenario. Under B2 scenario all above mentioned parameters illustrate decreasing trends of 0.3 mm per year, 0.15 mm per year and 0.001 mm per year respectively.

4. Conclusions

Annual total precipitation, maximum annual rainfall and annual averaged daily rainfall show increasing trends under A2 scenario and decreasing trends under B2 scenario. But the recorded decreasing trends are insignificant.

References

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