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Welcome by Conference Chair

It is my great pleasure to see that the 1st International Conference on Engineering Research and Practice is being held during 4-5 February 2017 in Dhaka, the capital city of Bangladesh.

As the General Co-Chair of the 1st International Conference on Engineering Research and Practice and Chair of the Global Circle for Scientific, Technological and Management Research (GCSTMR), I wish every success with such an important international event.

I very much appreciate all the hosts of this conference: Bangladesh Open University, Dhaka University, Islamic University of Technology and Dhaka University of Engineering and Technology for supporting such an international event.

I would like to thank all the GCSTMR Board Members, sponsors, secretaries, IT team, Conference Committee Members, reviewers and local Organizing Committee for making this conference as a success.

I am confident that this conference will provide a unique platform for effective exchange of ideas, reaffirming the existing collegial contacts, provide opportunities for establishing new ones as well as providing a forum for academics and researchers to present and share the results and findings of their latest research and practice on a wide range of topics relevant to this conference.

Associate Professor Ataur Rahman, PhD, FIE Aust, MASCE, MAGU, MIWA
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Welcome by Global Circle for Scientific, Technological and Management Research (GCSTMR)

I am particularly pleased that the 1st International Conference on Engineering Research and Practice is going to be held during 4-5 Feb 2017 in Dhaka, Bangladesh.

As a Board Member of the Global Circle for Scientific, Technological and Management Research (GCSTMR), I wish you every success with such an important international event.

In particular, I thank the hosts of this conference: Bangladesh Open University, Dhaka University, Islamic University of Technology and Dhaka University of Engineering and Technology for jointly providing the necessary support as well as the splendid venue for the Conference.

I also thank the local sponsors, the reviewers and all numerous others who have made a contribution towards this conference – as your help has been essential for this important event to take place.

I am sure this conference will provide a unique platform for a fruitful exchange of ideas, reaffirming the existing collegial contacts, provide opportunities for establishing new ones as well as providing a forum for academics and researchers to present and share the results and findings of their latest research on a wide range of topics covering all the conference themes.

It is my sincere wish and hope that this event will set a precedent for many more yet to come!

Dr Vojislav Ilic, Fellow American Society of Mechanical Engineers
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Stress Concentration Factors in Circular Hollow Section T-joints with Concrete-Filled Chords

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Abstract

The use of welded trusses made up of concrete filled circular hollow sections (CFCHS) is currently being increased in highway bridges in many countries. Most bridge failures are due to fatigue problems in welded joints. Therefore, many research studies are paying attention to the improvement of fatigue strength in these types of joints. Cyclic loading has direct impact on increasing the chance of fatigue to occur, especially at hot spot locations, where cracks are likely to initiate and propagate. In this paper, experiments on a welded concrete filled circular hollow sections (CFCHS) T-joint with a small non-dimensional parameter, β (ratio of diameter of brace, d, to diameter of chord, D) subjected to axial loads and in-plane bending are presented to determine the strain concentration factors (SNCF) on hot spots in a welded joint.

Keywords: Tubular section, tubular joints, concrete-filled steel tube, stress concentration factor, fatigue.

1. INTRODUCTION

Cyclic loading is a repeated loading or stress applied to a structure. Due to cyclic loading, fatigue will develop in the structure in general and in the welded parts in particular. Fatigue is the weakening of the material caused by cyclic loading (Fisher & Roy, 2010). The behavior of steel under cyclic loading has been studied for a long time; therefore, the allowable fatigue stress has been determined to be a function of type of loading, strength of steel, and the ratio of minimum stress to maximum stress.

In recent years, fatigue failure has been one of the most common concerns that requires special focus and a good understanding of its mechanisms and risk factors to minimize possibilities of failure. To study fatigue life of steel structures, design engineers should examine fatigue strength under realistic loading conditions such as, traffic, wind and wave loading, in addition to the structure dead load (Mann, 2010). The stages of the fatigue process are; crack initiation, crack propagation, and sudden fracture or brittle fracture. Fatigue cracks always initiate at the welded joints due to high stress concentration at the intersections (the hot spot region).

Therefore, a welded steel tubular T-joint has been tested to determine the location of high strain concentration. Strain concentration factors have been determined based on the hot spot stress method for T-joints subjected to axial tension, compression and in-plane bending, and the strain concentration factors (SNCF) for concrete filled circular hollow section (CFCHS) T-joints have been compared to those for empty tubular joints based on current design guidelines.

2. TEST SPECIMEN

A specimen of concrete filled circular hollow section (CFCHS) T-joints was prepared for testing. The specimen has a concrete filled circular hollow section chord member welded with an empty circular

hollow section brace. The chord and the brace have different geometric parameters. The chord is of size 165.1x5.4 mm with a length of 1000 mm and the brace is 48.3x5.4 mm with a length of 572 mm, Figure 1. The specimen is made of grade C250LO steel, whose minimum yield strength is 250MPa and minimum tensile strength is 320MPa (Standards Australia 2009). Steel tensile testing showed that the mean yield stress for the steel was 300MPa, the ultimate tensile strength was 370MPa, and the elongation percentage was 32%. The chord has been filled with concrete of grade 32MPa and the compressive strength test showed that the compressive strength for the concrete was 36MPa.

A jack was used to apply tension and compression axial load as well as in-plane bending at the end of the brace. The magnitude of the applied loads was 3kN, 6kN, 9kN, and 12kN respectively in ten cycles of loading. The specimen was loaded under linear elastic condition. This test was carried out to determine the differences in joint behavior under tension, compression and in-plane bending. The specimen was simply supported at the two ends of the chord member. The dimensions of the specimen are shown in Figure 1.

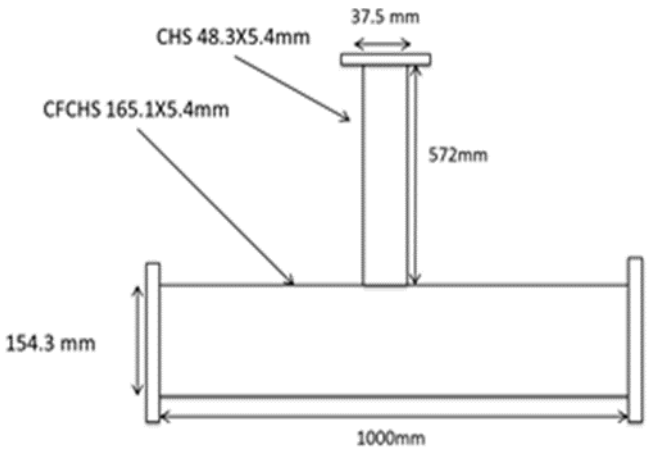

Figure 1. Specimen and dimensions

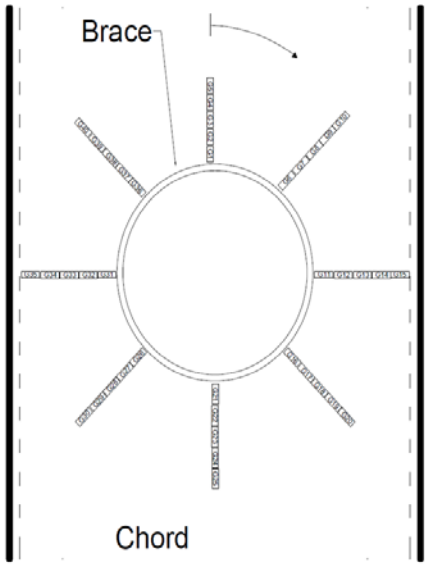

Figure 2. Strain gauge locations in the chord (Chen & Roy, 2011)

Strain gauges were placed around the entire chord-brace intersection in lines on both the brace and chord at 45˚ intervals as shown in Figure 2. A set of strip strain gauges with five strain gauges each was located in each line, as shown in Figure 2. The minimum distance between the strain gauges and the welded toe was 4 mm, and the maximum distance was 8 mm based on recommended extrapolation region from CIDECT Design Guide (Zhao et al 2001). In order to measure the nominal strain, four strain gauges were placed at the midpoint of the brace around the outer surface at 90˚ intervals.

2.1. T-joint under axial Tension

Axial tension loads were applied to the end of the brace starting with a minimum value of 0kN and ending with maximum load value of 12kN. By recording strain values around the intersection of the chord and the brace, hot spot strain can be calculated by using the linear extrapolation method after plotting the strain with respect of strain gauge distance from the welded toe on the crown, middle, and saddle position of the chord and the brace. Figures 3 and 4 show the strain distribution on the chord crown and brace crown respectively, under axial tension for consecutive load increases.

2.2. Strain Concentration Factors (SNCF)

The strain concentration factors (SNCFs), fatigue life would be easy to assess at welded joints. The SNCF can be determined experimentally by measuring strain at the hot spots using strain gauges.

SNCF is the ratio between the hot spot strain at a welded joint and the nominal strain in the member that causes this hot spot strain (Tong et al 2007).

$$SNCF = HSS / Nominal\ strain = \varepsilon_{hssn} / \varepsilon_{nom} \quad (1)$$

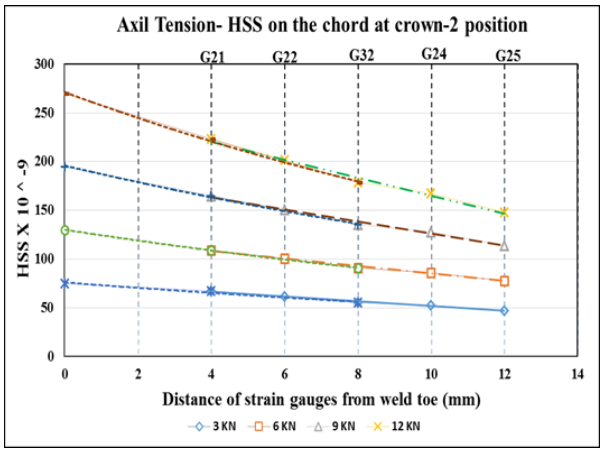

Figure 3. Hot spot strain (HSS) values on the chord crown position - tension

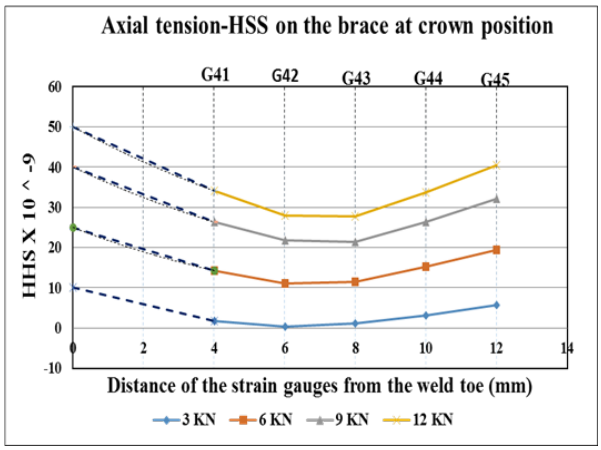

Figure 4. Hot spot strain (HSS) values on the brace crown position - tension

2.3. Experimental results and discussion

The experimental strain concentration factors at the crown, middle, and saddle locations of the chord and brace members of concrete filled circular hollow section T-joint (CFCHS) under axial tensile forces on the brace were determined based on the hot spot strain and the nominal strain. By calculating the strain concentration factor for empty T-joints using the equations given in the Design Guide for Circular and Rectangular Hollow Section Welded Joints under Fatigue Loading-8, (Zhao et al 2001) and comparing them with the experimental strain concentration factors, a significant difference can be noted. Maximum values for SNCF in the empty circular hollow section T-joint are on the saddle. The average of the experimental strain concentration factors (SNCFs) distributions on the chord and brace of the specimen and the calculated (SNCFs) for empty CHS joints are plotted in Figure 5 and Figure 6, respectively.

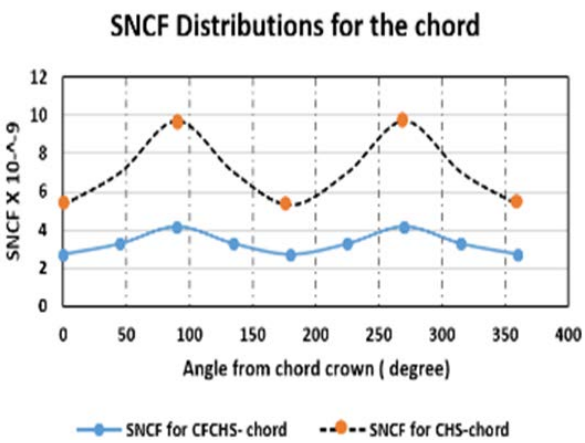

Figure 5. Axial tension-SNCFs distributions for the chord

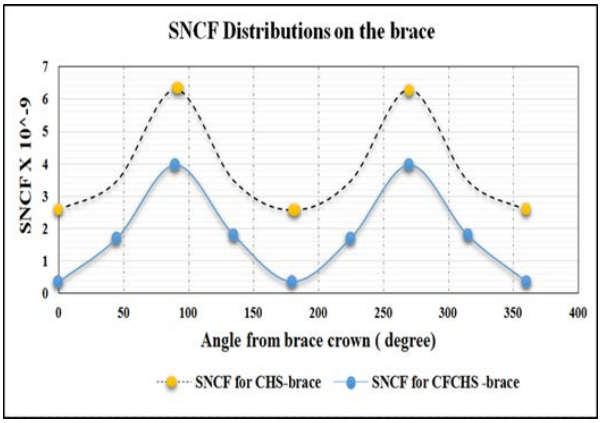

Figure 6. Axial tension-SNCFs distribution for the brace

2.4. T-joint under axial compression

Four different axial compression loads, 3kN, 6kN, 9kN, and 12kN, were applied to the end of the brace in ten cycles, starting with a minimum value of 0kN, and ending with maximum load value of

12kN. By recording strain values around the intersection of the chord and the brace, hot spot strain (HSS) can be calculated by using the linear extrapolation methods after plotting the strain with respect of strain gauge distance from the welded toe on the crown, middle, and saddle position of the chord and the brace. Figures 7 and 8 show the strain distribution on the chord crown and brace crown respectively, under axial compression.

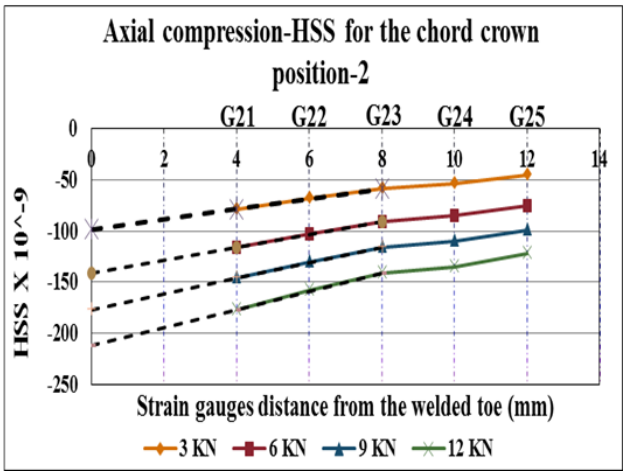

Figure 7. Hot spot strain (HSS) values on the chord crown position - compression

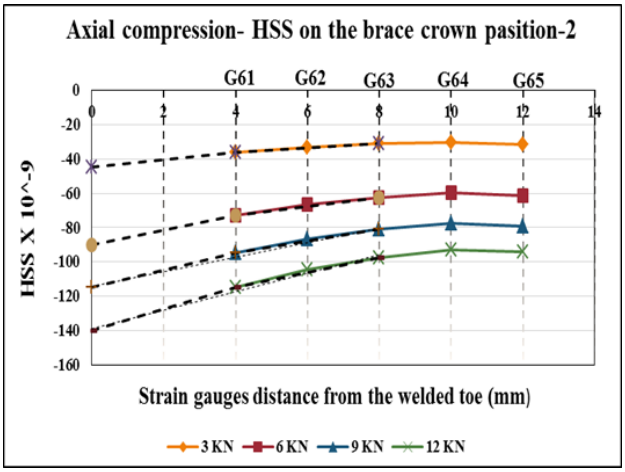

Figure 8. Hot spot strain (HSS) values on the brace crown position - compression

2.5. Experimental results and discussion

The experimental strain concentration factors (SNCF) at the crown, middle, and saddle locations of the chord and brace members of concrete filled circular hollow section T-joint(CFCHS) under axial compression forces on the brace were determined. By comparing the SNCFs for the axial tension and the SNCFs for axial compression, the magnitude of the maximum SNCF for tension in the chord is comparable to that for compression. The average of the experimental strain concentration factors (SNCFs) distributions on the chord and brace of the specimen are plotted in Figure 9 and Figure 10, respectively. Figures 9 and 10 show that the maximum SNCFs, which occur in the chord, moves from the saddle under tension to the crown location under compression.

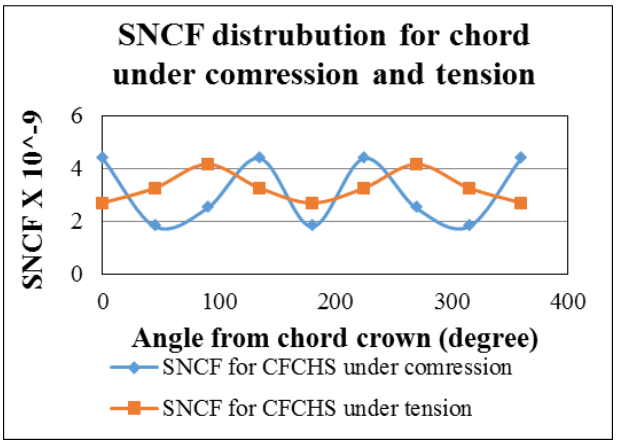

Figure 9. SNCFs distributions for the chord under compression and tension

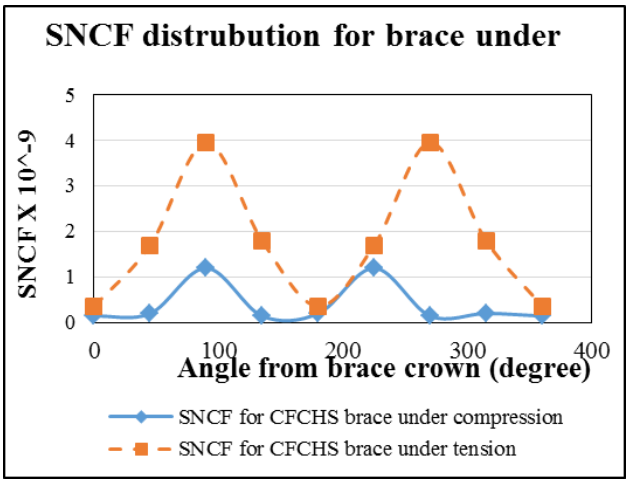

Figure 10. SNCFs distribution for the brace under compression and tension

2.6. T-joint under in-plane bending

Two different in-plane bending moments, 0.245kNm and 0.36kNm were applied to the end of the brace in ten cycles, starting with a minimum value of 0kNm, and ending with maximum load value of 0.36kNm. By recording strain values around the intersection of the chord and the brace, hot spot strain (HSS) can be calculated by using the linear extrapolation methods after plotting the strain with respect of strain gauge distance from the welded toe on the crown, middle, and saddle position of the chord and the brace. Figures 11 and 12 show the strain distribution on the chord crown and brace crown respectively, under in-plane bending in the brace.

2.7. Experimental results and discussion

The experimental strain concentration factors (SNCFs) at the crown, middle, and saddle locations of the chord and brace members of concrete filled circular hollow section T-joint(CFCHS) under in-plane bending moment on the brace were determined. The strain concentration factors for empty CHS which have been calculated using equations given in Design Guide for Circular and Rectangular Hollow Section Welded Joints under Fatigue Loading No. 8, (Zhao et al 2001). By comparing the experimental in-plane SNCFs for CFCHS and the calculated in-plane bending SNCFs for CHS, a significant difference can be noticed. The average of the experimental strain concentration factors (SNCFs) distributions on the chord and brace for the specimen and the strain concentration factors (SNCFs) distributions on the chord and brace for CHS are plotted in Figure 13 and Figure 14, respectively. SNCFs under in-plane bending show a slight decrease due to concrete filling at the crown position. However, the brace location does not seem to benefit due to concrete-filling under in-plane bending as shown in Figure 14, with SNCFs slightly higher than those in empty joints. However, the maximum SNCF for the CFCHS T-joint, which occurs in the chord, is still lower than that for the empty CHS T-joint.

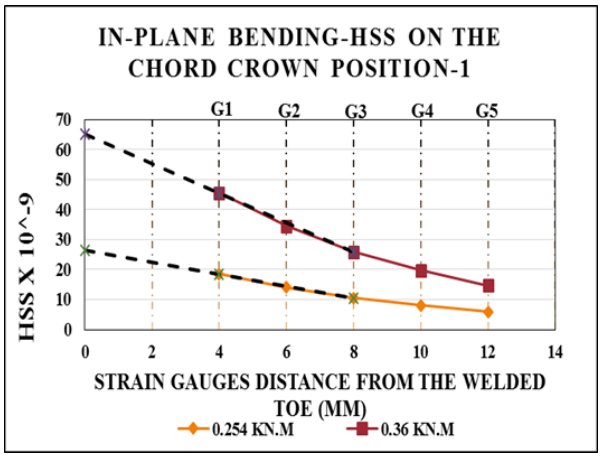

Figure 11. Hot spot strain (HSS) values on the chord crown position – In-plane Bending

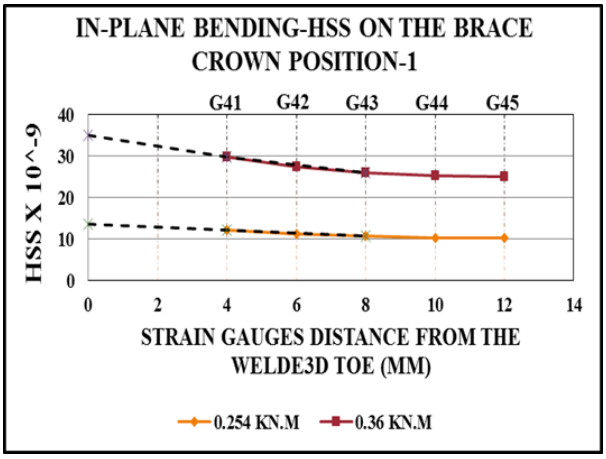

Figure 12. Hot spot strain (HSS) values on the brace crown position – In-plane Bending

3. CONCLUSIONS

An experimental investigation has been carried out at Western Sydney University laboratory on a specimen made of concrete filled circular hollow section chord welded to an empty circular hollow section brace subjected to axial tension and compression forces and in-plane bending. Material properties of the steel and concrete used in the test specimen were measured, and the following conclusions have been emerged from the study. After measuring the strain on the chord and the brace, by recording the strain data, hot spots strain (HSS) has been determined to calculate strain concentration factors (SNCF) in both the chord and the brace. The results were compared with the

existing research results of welded empty T-joints made of circular hollow section (CHS) from the Design guide 8 (Zhao et al 2000).

1. The CFCHS joint has lower SNCF in its chord and brace when its brace is loaded in tension compared to empty CHS T-joints.
2. The CFCHS joint has lower SNCF in its brace when its brace is subjected to axial compression forces compared to empty T-joints. Maximum SNCFs for axial compression forces are of the same magnitude as maximum SNCFs for axial tension forces but occur in different locations.
3. The SNCFs for the CFCHS T-joints in the chord are lower when it's subjected to in-plane bending in the brace compared to the SNCFs for an equivalent empty CHS T-joint. The maximum SNCF for the empty CHS T-joint is at the chord crown position, while the SNCF for the CFCHS T-joint is minimum at the chord crown position of 180° degrees due to compression at this location.

The concrete filling effectively reduces the peak SNCF under both axial loading and in-plane bending. The concrete in-fill in the CHS T-joint also improves the stiffness and the strength of the joint significantly.

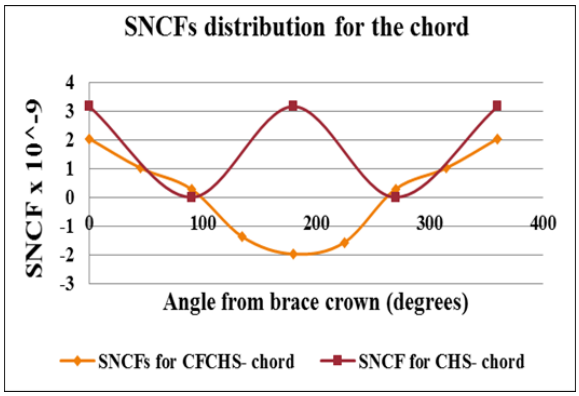

Figure 12. SNCFs distributions for the chord under In-plane Bending

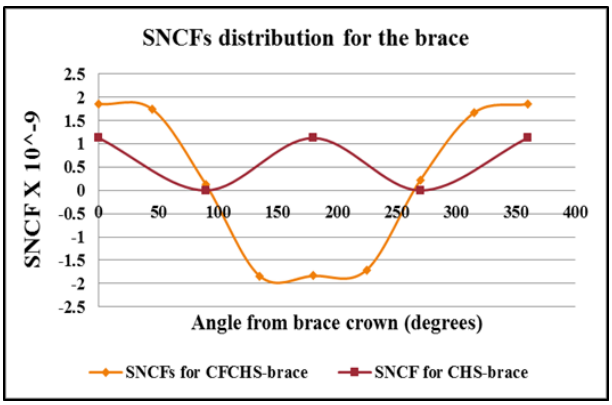

Figure 13. SNCFs distribution for the brace under In-plane Bending

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Social Media and Crowd Sourcing to Evaluate and Compare Priorities and Preferences for Sustainable Transportation System

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Abstract

To ensure that decisions are made reflecting public needs, public participation has become an indispensable part of transportation planning process. Social media and crowd sourcing are gaining popularity day by day to interact and engage with the general public in various sectors. Though this approach has high potential, its application has been so far been quite limited in the field of transportation planning. This research aims to evaluate the possibility to conduct public participation through the use of crowd sourcing and social media in transportation planning. For this, it uses social media to engage with general people to identify their views on sustainable transportation system. Based on this, it designs an online questionnaire survey to ascertain and prioritize various aspects of sustainable modes of transport, e.g., walking, cycling and public transport environment through Analytic Hierarchy Process (AHP). Through this, the study identifies and compares the perception of general road users and transportation experts regarding sustainable transportation. The questionnaire was spread through social media. A total of 80 general public and 10 transport experts responded to the questionnaire. The survey responses show variations between general public and expert's opinions in prioritizing components of transport systems. Besides, as the most feasible sustainable transportation, public transport was prioritized by highest number of respondents with the provision of facilities for women and children, surveillance for enhanced security and convenience, etc. The information is expected to be highly beneficial for both the academicians and practitioners from relevant backgrounds.

Keywords: Social Media, Public Participation, Sustainable Transportation, Public Transport, Analytic Hierarchy Process (AHP).

1. INTRODUCTION

Transportation systems should be developed in such a manner so that it contributes to provide mobility to people without compromising the needs of future generation. Black (2010) attempted to define a sustainable transportation system as one that provides transport and mobility with renewable fuels while minimizing emissions detrimental to the local and global environment and preventing needless fatalities, injuries and congestion. The definition highlights that sustainability is not just being environmentally responsible but transport still needs to achieve its role of providing mobility, safety and comfort. The inadequacy of transport facilities are one of the major bottlenecks to socio-economic development of the major cities and national integration (Mannan et al, 2001). A sustainable transportation system of a city assists in economic and social development of the city life. Economists have argued that for assisting overall economic development an appropriate transport planning is needed. Communities which are successfully improving the sustainability of their transport networks are doing so as a part of a wider program of creating more vibrant, liveable, sustainable cities.

Public participation in transport planning is a recent trend though there is an increasing number of

cases in Europe where the public is involved in the decision-making process. Citizens should be involved in transportation planning phases such as in the identification of transport and mobility problems, in specifying the vision and objectives, in the strategy development process, in suggesting possible solutions and also during the identification and evaluation of those solutions (Rupprecht Consult, 2013). Public participation is based on the belief that people whose lives are affected by transportation planning and investment decisions have a right to be involved in the decision-making process and influence choices that are made. Directly engaging citizens in this process promotes successful problem solving, yields diverse voices and new ideas, and gives the public a sense of ownership of the developed solutions (MARC, 2013).

The context of transport planning has changed dramatically in recent years, raising some difficult challenges but also creating new opportunities for public involvement (Krätzig and Warren-kretzschmar, 2014). Introduction of new media provide new opportunities to involve most citizens and civic organizations in the transportation planning process. Many transit agencies have begun to incorporate social media into their marketing and communications strategies. Though it is being practiced in small scale by some authorities, very few research studies explored the possibility to use social media and crowd sourcing to encourage public participation. Purpose of this research is to evaluate the possibility to promote public participation through the use of crowd sourcing and social media in transportation planning. Finally to identify and compare the perception of general road users and transportation experts regarding sustainable transportation system.

2. METHODOLOGY

2.1. Social Media Activist Group and Preparation of Questionnaire

The concept of sustainable transportation system of a city expects most trips being made by walking, bicycle or public transport in a safe, secured, convenient, affordable and timely manner with leaving very few trips for car. This study encourages public participation through social media to understand the concept of sustainable transport held in mind of transportation professionals and the general road users. For this, the sustainable transportation system concept was built around three modes – walking, bicycling and public transport (bus), and their various criteria were identified through the participation in social network (Facebook). Finally, the weights of each criterion were evaluated by applying AHP where the data was collected through an online survey with general road users and transportation professionals as respondents.

The online link to the Facebook group: https://www.facebook.com/groups/855287761258451/

AHP was used as an appropriate instrument to model the survey questionnaire. AHP implies to Analytic Hierarchy Process which was developed by Thomas L. Saaty in early 1970's. It is a quantitative multi-criteria decision making approach. It assists complex decision making by using a set of pairwise comparison matrix. It also determines the relative importance and gives a ranking to the criterions. From feedbacks and opinions of the members of the social media group 17 criteria were derived to reach the goal of positive sidewalk environment. Similarly 22 criteria were deduced to achieve improved public transportation and 9 criteria for achieving proper cycling environment as a sustainable transportation system of Dhaka city.

The link to the online survey questionnaire: https://ahpsurvey.untappedideas.com/

2.2. Using AHP Methodology to Prioritize Road User Preferences

The online questionnaire survey was considered as the most appropriate instrument to elicit

comparison and prioritization of road user needs and environmental attributes. The survey inquired about (1) the respondent's socio-demographic profile, (2) their preference between two paired comparisons of criteria and (3) their preference between three proposed sustainable transport systems of Dhaka city. In point 2 and 3 survey participants were asked to choose between paired comparisons (e.g. A and B) using a scale value from 1 to 9. A choice of 1 meant that the survey participant expressed an equal sense of preference between A and B while a choice of 9 proximate to B meant that the participant expressed an extreme sense of preference for B over A and vice versa. For example, choose the relative importance of Safety consideration over Mobility concerns in evaluating Cycling as a sustainable transportation system of Dhaka. If Safety consideration should be strongly prioritized then assign a value of 5, which stands for strong importance in Saaty's (2008) scale, by encircling the number or placing a tick mark on it. The comparison is shown if Figure 1.

Mobility	9 8 7 6 5 4 3 2 1 2 3 4 (5) 6 7 8 9	Safety

Figure 1. Pairwise comparison between two criteria in Analytic Hierarchy Process (AHP)

Saaty's (2008) scale with explanations for AHP comparison is shown in Table 1.

Table 1. Intensity of importance with explanations for AHP comparison (Saaty, 2008)

Intensity of importance	Definition	Explanation
1	Equal importance	Two items contribute equally to the objective
3	Moderate importance	Experience suggests that one be slightly favored over the other
5	Strong importance	Experience suggests that one be strongly favored over the other
7	Very strong importance	Item strongly favored and its priority demonstrated in practice
9	Absolute importance	Importance of one over another affirmed on highest possible order
2,4,6,8	Intermediate values	Used to represent compromise between priorities listed above

By analyzing their feedbacks, the preferences of sustainable transportation system was determined and evaluated. However, it provided important and crucial insights about citizen needs, preference, and prioritization. Before the questionnaire survey was distributed, the investigator conducted a pilot test to determine the degree of difficulty of the questions being asked, establish the length of time to answer the questionnaire and determine level of response of respondents so as to ensure effectiveness, reliability and validity of the questions. The questionnaire was spread through the social media. Total of 80 general public and 10 transportation experts participated in the survey questionnaire.

3. RESULTS AND DISCUSSION

This section explains the results of the survey, which include the overall socio-demographic characteristics of respondents and comparison of average weightage values of criterions. The weightage values are assigned based on the survey feedback analysis of general public and transportation experts. Table 2 shows the socio demographic characteristics of general public respondents where it shows that majority of the survey participants were male (male to female proportions 87 to 13). There was also a significantly young population cohort with more than half of the participants belonging to the 18–25 age group and 80% were still studying.

Table 2. Socio demographic characteristics of public respondents

Attributes	Categories	City sample N= 80 (%)
Gender	Male	87
	Female	13
Age range	<18	4
	18-25	76
	25-40	14
	40+	4
Education	<College	7
	College	5
	Undergrad	75
	Graduated	17
Employment	Employed in office	19
	Self employed	5
	Unemployed	76
Car ownership	Family car	36
	Personal car	8
	Do not own car	56

According to survey responses of both general public and experts, it appears that public transportation is the most prioritized transport (55.5% by experts and 54.7% by general public) among three followed by walking (30.5% by experts and 26.1% by general public) and cycling (14.1% by experts and 19.1% by general public). The priorities of weights for the three mode of transport are given in Figure 2.

In case of sidewalk, the results of public respondents present the criterions with the highest priority values include: toilets and dustbins provision (8.1%), no hawkers on footpath (7.7%), facilities for disabled people (6.6%), traffic signals provision (6.4%), provision of evening lights (6.2%) where necessary. Where the experts think that evening lights provision (13.5%), seating facilities at stations (11.3%), facilities for disabled people (8.9%) and shed providing roads (5.9%) should be given highest priority. The comparison of the global priority of weights for all positive sidewalk environment criterions are shown in Figure 3.

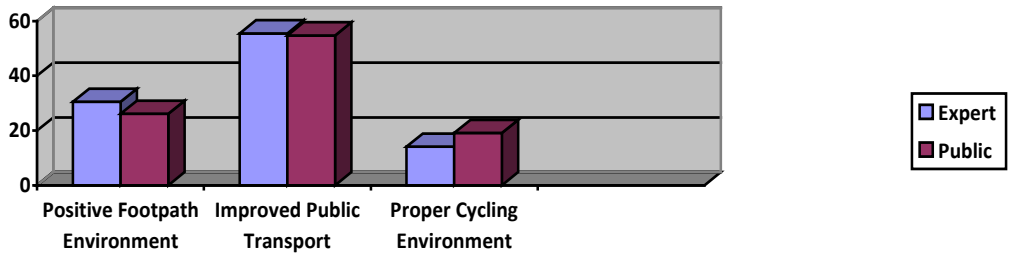

Figure 2. The global priorities of weights for the three sustainable modes of transport

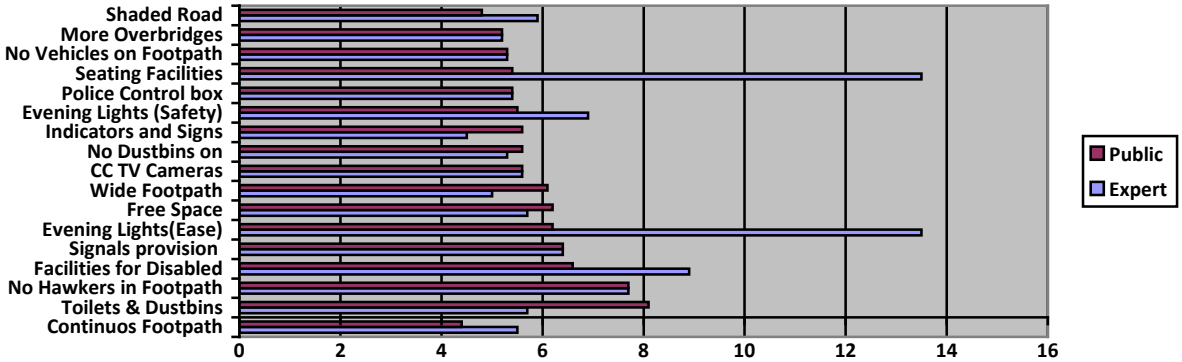

Figure 3. The global priorities of weights for positive sidewalk environment criterions

The results of the global priority of weights for proper cycling environment criterions are given in Figure 4. The criterions with the highest priority values include: separate crossing system (19.9%), provision of evening lights (19.1%), speed monitoring (15.7%), parking facility (13.2%) and provision of traffic signals (12.0%). On the other hand, the experts thought is also the same as the general citizens as they have given priority to the same factors. So, here we find consensus between the experts and public.

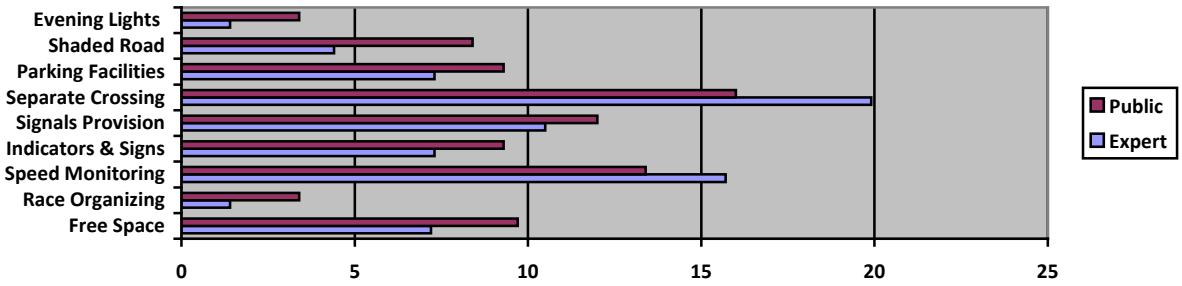

Figure 4. The global priorities of weights for proper cycling environment criterions

The results of the global priority of weights for public transport criterions are given in Figure 5. From the results it appears that the criterions with the highest priority values assigned by public are: facilities for women and children (9.93%), CC TV Camera provision (8.97%), reduced student fare (7.3%), proper license giving system (6.48%). Here a major difference was found between the results of public and experts is that experts think that behavior of drivers and conductors (12.5%), neat and clean transport (11.8%), facilities for women and children (10.5%), bus stoppage fix and monitoring (6.6%) are most important for a sustainable public transport.

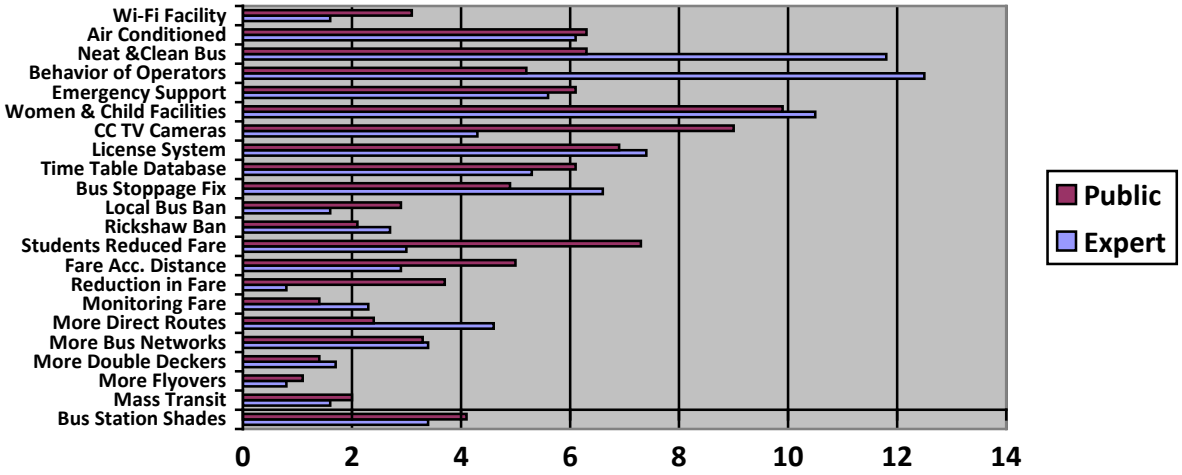

Figure 5. The global priorities of weights for an improved public transport criteria

4. CONCLUSION

From the feedback results it appears that there exist a difference in perception between transportation experts and general road users about sustainable transportation system. For certain criteria it is found that the opinion of experts and general public varies a lot while ranking criterions. In fact variations are also found between expert and public individual's feedback results. This study evaluates the role of social media as a public participation tool in the decision making process of transportation planning of a country. It will also provide a way to make ranking and compare between different criteria and alternatives of a transportation plan or a project. For planners in future, this study will assist to incorporate necessities of users as public input in the planning process not limited to only transportation sector.

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Advanced Agglomerative Clustering Technique for Phylogenetic Classification

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Abstract

Trivial agglomerative hierarchical clustering technique has very high computational complexity O(n3) and requires more number of iterations to prepare the phylogenetic classification. Recent advancement in this field has already developed some improved techniques to reduce the time complexity. To improve this further, an Advanced Agglomerative Clustering Technique (AACT) has been proposed here. The proposed method mainly aims to identify required number of distinct clusters over vast dataset with lower complexity and thus reducing the time complexity than existing methods. The proposed technique AACT consists of three phases. In the first phase, the idea is to partition the entire input dataset into required number of clusters using the traditional K-Means clustering technique using Manhattan distance, which is very much faster than trivial hierarchical agglomerative clustering technique. In second phase, the proposed technique computes centroids over each individual cluster from the result of K-Means clustering and selects the representative genes closest to the centroids. In the final phase, the proposed method uses S-Link technique over the result of second phase generating the final phylogenetic tree. Experimental result shows that the proposed technique is by far very faster than traditional agglomerative approach and even slightly faster than some techniques that were developed recently for faster analysis.

Keywords: *S-Link, Manhattan Distance, Advanced Agglomerative Clustering Technique (AACT), Phylogenetic Classification.*

1. INTRODUCTION

Bioinformatics is an interdisciplinary field that develops methods and software tools for understanding biological data. As an interdisciplinary field of science, bioinformatics combines computer science, statistics, mathematics, and engineering to analyze and interpret biological data. Bioinformatics is both an umbrella term for the body of biological studies that use computer programming as part of their methodology, as well as a reference to specific analysis "pipelines" that are repeatedly used, particularly in the field of genomics. Traditional methods for studying individual genomes are well developed. However, they are not appropriate for studying microbial samples from the environment because traditional methods rely upon cultivated clonal cultures while more than 99% of bacteria are unknown and cannot be cultivated and isolated. Metagenomics use technologies that sequence uncultured bacterial genomes in an environment sample directly and thus makes it possible to study organisms which cannot be isolated or are difficult to grow in lab. Recent development in phylogenetic classification has resulted in methods like improved agglomerative clustering technique which takes into account the power of K-means clustering to shrink the vast data set and later on use the hierarchical clustering to derive the final result. In short, it takes the output of K-means clustering and puts it as input of agglomerative clustering technique to obtain the final result.

2. PROPOSED METHOD

2.1. Skeleton of Proposed Method

Initially DNA is extracted from the environment directly and it is known as metagenomics. Metagenomes are manipulated using enzyme called "Restriction Endonucleases". After that a library of metagenomics is constructed and finally DNA analysis is performed.

2.1.1. Sequence Analysis

The metagenome or the DNA sequence generally consists of a large number of nucleotides. A new approach to analysing gene functions has emerged. DNA arrays allow one to analyse the expression levels (amount of mRNA produced in the cell) of many genes under different time points and conditions to reveal which genes are switched on and switched off in the cell. The outcome of the study is an **n** by **m** expression matrix, I with the **n** rows corresponding to genes, and the **m** columns corresponding to different time points & conditions. Clustering algorithms group genes with similar expression patters into clusters with the hope that these clusters correspond to group of functionally related genes. To cluster the expression data, the **n** by **m** expression matrix is often transformed into an **n** by **n** distance matrix **d** where $\mathbf{d_{ij}}$ reflects how similar the expression patters of genes **i** and **j** are.

2.1.2. Clustering and Cluster Manipulation

We run the K-Means Algorithm on the values of the distance matrix and cluster the different values. We find a marker gene for each of the cluster from which the distances of other members of that cluster is minimum. Then considering each marker gene as single entity, we apply S-LINK Agglomerative Hierarchical Clustering to produce the phylogenetic tree.

2.2. Proposed Algorithm

The **A**dvanced **A**gglomerative **C**lustering **T**echnique(AACT) using Manhattan Distance works in the following way.

2.2.1. K-means Stage

In this stage the traditional k-mean technique is applied and identified **l** distinct clusters over the input dataset **X**. Generally, the traditional k-means technique consists of three steps. In the first step, to fix the **l** centroids values $\overline{K} = \overline{K}_0,.., \overline{K}_{l-1}$ over the input dataset **X** as defined $\mathbf{X} = X_0,.., X_{n-1}$, where **X** represents input dataset, n denotes number of objects that belong to input dataset **X** and $\overline{K}$ represents the number of centroid values identified in **X**. In the second step, it maps the **l** clusters in $\overline{K}$ over the input dataset **X** through the process of measuring Manhattan distance between dataset **X** and **l** centroid values as defined in the equation (1).

$$C_j = min\{D(X_i, \overline{K}_j) \mid \forall X_i \in X, \forall \overline{K}_j \in \overline{K}_l\} \quad (1)$$

Where $\mathbf{D}(X_i, \overline{K}_j)$ represents the Manhattan distance between $\mathbf{i}^{th}$ object in **X** and $\mathbf{j}^{th}$ centroid in $\overline{K}$ and is defined as equation (2).

$$D(X_i, \overline{K}_j) = \{ (X_i - \overline{K}_j) \mid \forall X_i \in X, \forall \overline{K}_j \in \overline{K} \} \quad (2)$$

Where $\mathbf{X}_i$ denotes the dataset **X** and $\overline{\mathbf{K}}_{\mathbf{j}}$ is centroid value of $\mathbf{j}^{th}$ cluster. In the next step, it partitions the input dataset **X** into **l** distinct clusters $\mathbf{C} = \{C_0, ..., C_{l-1}\}$ in $\overline{\mathbf{K}}_{\mathbf{j}}$ as defined in equation (3).

$$\overline{K}_j = \{ \frac{1}{N_j} \sum_{l=0}^{n_j} C_{jl} \mid \forall C_{il} \in C_j,\ \forall C_j \in C\} \quad (3)$$

Where $\mathbf{C}_{ij}$ represents the $\mathbf{i}^{th}$ object in the $\mathbf{j}^{th}$ cluster that belongs to the C. Repeat the steps from step 2 to step 3 until the result of the current iteration equal to previous iteration. This modified K-means algorithm is described in the below subsection.

2.2.2. Algorithm for K-means Clustering

Input: $\mathbf{X} = \{X_0, ..., X_{n-1}\}$
Output: **l**-clusters = $\{C_0, C_1, ..., C_{l-1}\}$

Begin:

1. Fix the l centroids values $\overline{K} = \{\overline{K}_0, ..., \overline{K}_{n-1}\}$ over the input dataset **X**.
2. Map **l** clusters in $\overline{K}$ over the input dataset X by using the equation (1) and (2).
3. Partition the input dataset **X** into **l** distinct clusters $\mathbf{C} = \{C_0, ..., C_l\}$ using the equation (3).

End

2.2.3. S-LINK Stage

In this stage, the S-LINK (Single Linkage) technique is applied to identify 'm' clusters over the result of k-means technique **C**. *S-LINK* technique consists of four steps. In the first step, it computes centroid over each individual cluster in the result of k-means, **C** for **i = 0,1, ..., l-1** using the equation (4).

$$\overline{C} = \sum_{i=0}^{l-1} \sum_{j=0}^{n_j} C_{ij} \quad (4)$$

Where $\mathbf{C_{ij}}$ denotes the $\mathbf{j^{th}}$ object in the $\mathbf{i^{th}}$ cluster. **l** denotes the number of clusters and $\mathbf{n_i}$ denotes number of objects in the $\mathbf{i^{th}}$ cluster. In the second step, it constructs the distance matrix $\mathbf{D_{ij}}$ over the result of $\overline{C}$ based on Manhattan distance and is defined in equation (5).

$$D_{ij} = \{_{i=0,1,..,k-1;j=i+1,..,k} d(\overline{C}_i, \overline{C}_j) \mid \forall \overline{C}_i \in \overline{C}, \forall \overline{C}_j \in \overline{C}\} \quad (5)$$

Where $d(\overline{C}_i, \overline{C}_j)$ represents the distance between $\mathbf{i^{th}}$ and $\mathbf{j^{th}}$ cluster belonging in $\overline{C}$ and is defined in equation(6).

$$d(\overline{C}_i, \overline{C}_j) = |\overline{C}_i - \overline{C}_j| \quad (6)$$

If $\mathbf{i^{th}}$ and $\mathbf{j^{th}}$ cluster are containing more than one objects, then compute the distance of set of objects then compute the distance of set of object pairs between $\mathbf{i^{th}}$ and $\mathbf{j^{th}}$ clusters and then consider the minimum distance of object pair as a distance of $\mathbf{i^{th}}$ and $\mathbf{j^{th}}$ cluster as defined in equaion (7).

$$d(\overline{C}_i, \overline{C}_j) = min\{d(\overline{C}_i, \overline{C}_j)\} \quad (7)$$

Where $\overline{C}_i, \overline{C}_j$ denotes object pairs of $\mathbf{i^{th}}$ and $\mathbf{j^{th}}$ clusters and $\overline{C}$. In the fourth step, it finds the closest cluster pair with minimum distance Δd over the distance matrix $\mathbf{D_{ij}}$ as defined in equation (8).

$$\Delta d = min\{D_{ij} \mid \forall D_{ij} \in D\} \quad (8)$$

In the next step, merge the closest cluster pair ($\overline{C}_i$, $\overline{C}_j$) into a single cluster $\overline{C}_{ij}$. Then delete the **j**th and compute the centroid of new cluster $\overline{C}_i$. Repeat the step two, until the number of iterations is satisfying **(l-m)** where **m** is the number of clusters. This modified S-LINK algorithm is described in the below section.

2.2.4. Algorithm for Agglomerative Clustering

Input: $\mathbf{C} = \{\mathbf{C_0}, ..., \mathbf{C_{l-1}}\}$
Output: $\mathbf{G} = \{\mathbf{G_0}, ..., \mathbf{G_{l-1}}\}$

Begin:

1. Compute centroid over the each individual clusters in the result of K-means, **C** for **i = 0,1, ..., l-1** using the equation (4).
2. Construct distance matrix $\mathbf{D_{ij}}$ over the result of $\overline{C}$ based on Manhattan distance in equation (5) and (6).
3. If **i**th and **j**th clusters are containing more than one objects, then compute the distance of set of object pairs between **i**th and **j**th clusters and consider the minimum distance of object pairs as a distance of **i**th and **j**th cluster using equation (7).
4. Find the closest cluster pair with minimum distance $\Delta\mathbf{d}$ over the distance matrix $\mathbf{D_{ij}}$ using the equation (8).
5. Merge the closest pair ($\overline{C}_i$, $\overline{C}_j$) into single cluster $\overline{C}_{ij}$. Delete the **j**th cluster and compute centroid of new cluster $\overline{C}_i$. Repeat the steps, until the number of iterations is satisfying **(l-m)**.

End

2.3. Simulation Technique

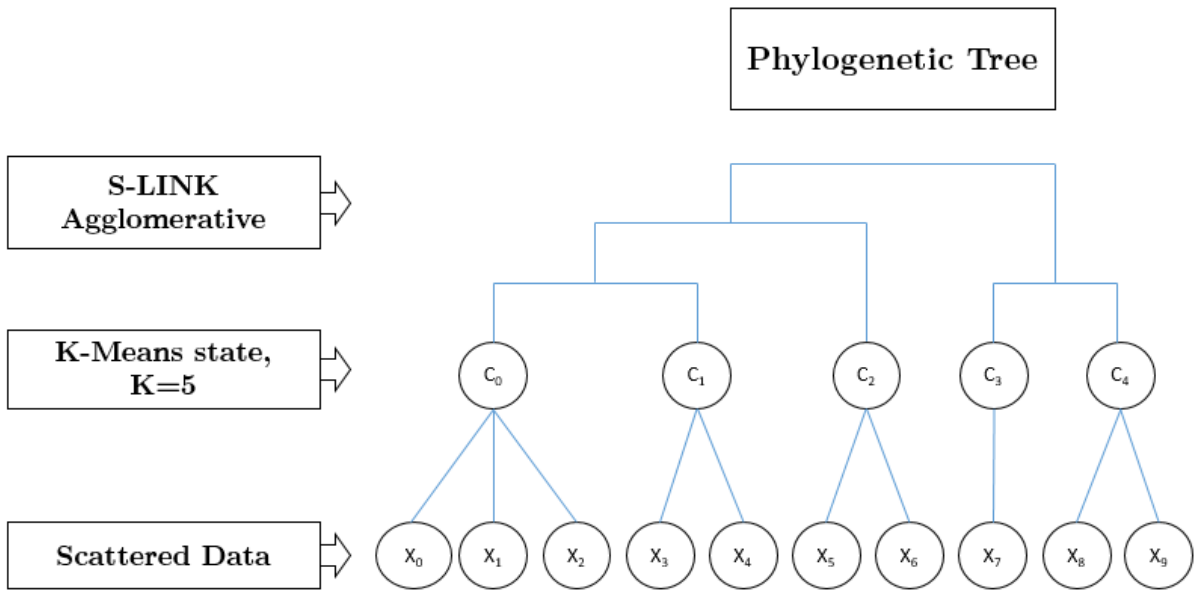

Figure 1. Simulation Technique of the Proposed Method

Here let us consider $\mathbf{X_0}$, ..., $\mathbf{X_9}$ are the preliminary dataset which is found out by exponential of the intensity values in the dataset. In the first step, the number of clusters is specified and $\mathbf{X_0}$, ..., $\mathbf{X_5}$ are the random clusters of K-means method. Then rest of the data is assigned to the clusters based on the closest distance. Then from each of the clusters of K-means, one representative data is selected from each of the cluster. Now, in agglomerative stage, each pair of the closest cluster is merged together in

each of the iteration and ultimately, they merge to one cluster. If we want to get finally **m** number of clusters, then we have to stop **m** iterations before the iteration loop ends. Thus, we get the phylogenetic tree.

3. RESULT & DISCUSSION

3.1. Complexity Analysis

The proposed **A**dvanced **A**gglomerative **C**lustering **T**echnique(AACT) is better suitable to identify **m** distinct clusters over the large dataset with lesser computational complexity and finite number of iterations. In stage one **(k-means)** dataset of size n is reduced to **l** distinct number of clusters with **l** number of iterations and computational complexity of **O(nl)** where, **n** is the size of dataset and **l** is distinct number of clusters. In second stage **(S-LINK) l** distinct number of clusters obtained from stage one **k-means** is reduced to m number of groups with **(l-m)** number of iterations and computational complexity of $\mathbf{O(nl+l^2)}$, where **nl** is the computational complexity of stage one and $\mathbf{l^2}$ is the computational complexity obtained due to the construction of distance matrix $\mathbf{D_{ij}}$ in the S-LINK stage. In this method, in case of using the distance function for distance matrix generation, using Manhattan distance rather than the Euclidian distance gives a consistent improvement in performance. Here also Minkowski distance can be used but they will give higher computational complexity due to calculation of higher power distance and subsequent higher square root.

3.2. Experimental Result

In this section, we compare among the trivial agglomerative method, most recently developed Improved Agglomerative Clustering Technique and our proposed method. As we can see from the table that our proposed method works faster than the **IACT** as Manhattan Distance was used to calculate distance among clusters instead of Euclidean Distance, which reduced the time required to compute the phylogenetic classification. So, our proposed method works faster than the **IACT**.

Table 1. Time complexity comparison

Sample Count	AACT (Proposed) (sec)	IACT (sec)	Trial Approach (sec)
4000	19.063	20.4776	40.7660
8000	38.4779	40.5365	80.8364
12000	56.6029	62.3473	120.7839
16000	76.1390	80.9419	163.9884
20000	97.3996	103.3372	203.4975
24000	115.2550	124.1642	243.6152

4. CONCLUSION

The size of dataset in Metagenomics is humorously large. In order to manipulate this vast and increasing dataset we need very efficient algorithms. Next, each of the clusters of Advanced Agglomerative Clustering Technique should be annotated from the databank of NCBI (National Center for Biotechnology Information). So far, all the clustering algorithms run in sequential execution but for further improvement these algorithms can be optimized for parallel execution. Even in this age of distributed computing, this system can easily be optimized for distributed systems.

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Water Quality in Landsat OLI Images

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Abstract: *Remote sensing images are representation of the earth's surface as seen from space without any physical contact. It generally detects the surface feature through reflectance of matter and represents various Digital Number (DN) values. In the context of water it also represent DN values according to reflectance of water properties, whatever it can detect from the space. Landsat Operational Land Image (OLI) is more effective to detect the surface features. It has been used to detect water body. The surface water quality is not same all over the surface so the energy absorption and relaxation is not the same. The DN value of surface water is changed due to the change of chemical properties of water. It will be a fundamental study to explore the relationship between DN value and water quality. This research explores a complete integration of DN values of Landsat OLI satellite image with same surface water quality, which are collected from several points. The chemical properties of collected surface water, which are analyzed through lab and is integrated with DN value of the Landsat OLI satellite images. The variations among the chemical properties of collected surface water and the DN values of the exact Landsat OLI images are categorized within an index. The correlations between surface water quality and DN values of Landsat OLI are investigated in this research.*

Keywords: Remote sensing, Satellite image, Landsat OLI, DN values, Water quality

1. INTRODUCTION

Remote sensing is a technology to acquire data of any object without physical contact. Remote sensing is used to land use pattern and change detection of temporal variations. Remote sensing techniques have widely been used in water body assessment (Alparslan et al., 2007, Brando and Dekker, 2003, Chen et al., 2007, Giardino et al., 2007, Hadjimitsis and Clayton, Kondratye et al., 1998, Koponen et al., 2002, Pozdnyakov et al., 2005, Ritchie et al., 2003, Seyhan and Dekker, 1986, Wang and Ma, 2001). The landsat OLI (operational land image) have high resolution band those are reflected by the water and its substance materials. The water quality depends on the dissolve and substance materials (Dekker et al., 1995). There is a variation between the reflection of fresh water and polluted water. The presence of dissolve materials represents water quality. Water color depends on the absorption and scattering of light by organic and inorganic constituents present in the water (Bukata, 2005). Suspended materials absorb the energy. The absorption and reflection depends on the amount of suspended materials and other components of water. Suspended sediments increase the radiance emergent from surface waters in the visible and near infrared proportion of the electromagnetic spectrum (Ritchie et al., 1976). The water body is more reflected in band 3 (green band) and less reflected in band 5 (NIR band). By using this band, the water depth is detected but the water quality can also be detected by using this band. The reflectance of polluted water and the reflection of depth water are not same, as a result the DN value or signature also not same. The variation of DN value is estimated by the variation of ground water quality.

The main aim of this research is to explore the relation between the surface water quality and Landsat OLI image of different locations. The specific objectives are:

1. To detect the DN value or signature of a particular point from Landsat OLI bands.
2. To identify the surface water quality and
3. To explore the relation between water quality variation and DN value.

2. STUDY AREA

Seven particular locations have been selected for collecting water samples. This area of interest has been selected by satellite image investigation and Google earth observation. The surface locations of these points have been identified by using Global Positioning System (GPS). The Landsat OLI satellite image and locational references of water samples have been shown in Figure 1 and Table 1.

Table 1: Sample references

Satellite Image Reference: LC81370442016351LGN00 / 16 December 2016					
Id	**Longitude**	**Latitude**	**Id**	**Longitude**	**Latitude**
A	90°19'53.466"E	23°54'29.996"N	E	90°14'21.658"E	23°50'43.233"N
B	90°20'34.77"E	23°50'40.622"N	F	90°15'1.721"E	23°54'16.119"N
C	90°20'33.084"E	23°49'36.422"N	G	90°14'29.126"E	23°54'10.053"N
D	90°15'33.169"E	23°47'51.585"N			

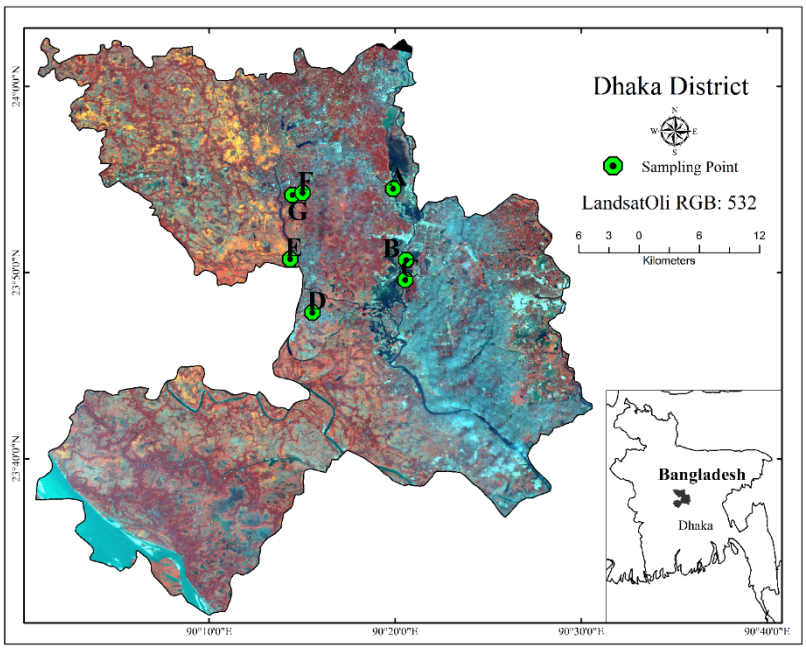

Figure 1. The locational of water sample

3. DATA AND METHODS

The water quality data has been collected from area of interest at the same time and the fixed selected coordinates. This collected water sample has been processed by lab analysis. The DN value of Landsat OLI has been collected by using normal band detection and NDWI (Normalized Deference of Water Index) model. The DN values of Landsat OLI have been created by the reflection and absorption intensity of water. The green band is more reflected by water and dissolved materials in water. The NIR infrared band is more absorbed in water and dissolved materials (McFeeters 1993). A model stands on (Green band -NIR band) / (Green band + NIR band), that is known as NDWI model. This equation has been used to identify the water quality in this research. The collected DN values and parameters of the water quality have been analyzed by statistical analysis (Figure 2).

4. RESULTS AND DISCUSSION

The integration of DN values with the water quality of the selected water samples are the focus of this research. Several parameters of water have been identified in laboratory analysis, such as pH, TDS, DO, Turbidity and EC. The parameters of different types of water depend on the sources of water such as sea water, ground water and surface water.

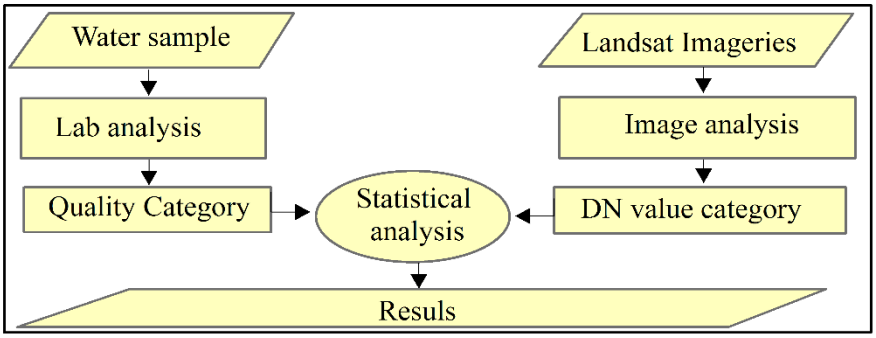

Figure 2: Data processing approach

Optimum reflectance: Water quality is affected by dissolved materials and substances in water body (Dekker et al., 1995). Suspended sediments and materials increase the radiance of surface water in the electromagnetic spectrum (Ritchie et al., 1976). Satellite sensor creates the DN value according to the electromagnetic spectrum or surface water radiance. The results of DN value depend on the target objective and band of satellite image. The reflectance of each band is not equal or optimum. The DN value of particular band has been distinguished by target. The blue, green, red and NIR bands of Landsat OLI reflectance are minimum for water body, but NDVI reflectance is optimum for quality exploration (Table 2).

Table 2: Satellite image properties

Id	Blue Band	Green Band	Red Band	Mean of True Color	NIR	NDWI	Temperature (°C)
A	75	47	59	60	15	214	20.12
B	69	71	52	64	28	192	20.2
C	4	21	33	19	18	224	20.32
D	7	57	47	37	26	187	20.91
E	29	25	49	34	28	184	20.45
F	66	77	75	73	92	142	20.98
G	4	8	15	9	11	229	20.11

Water Quality: The water quality of the collected samples is determined in lab analysis. The pH, TDS, DO, Turbidity, EC and temperature have been used to determine physical properties of water. The temperature has been detected by using band 10 and DN value is converted to radiance value. The radiance value represents the kelvin and centigrade (Table 3).

Table 3: Water quality

ID	pH	TDS (mg/l)	DO (ppm)	Turbidity (FTU)	EC (µs/cm)	Temperature (°C)
A	8.3	656	3.6	21.16	95.3	20.12
B	7.81	209	3.56	18.35	65	20.2
C	7.41	742	0.1	28.04	92.7	20.32
D	7.43	303	0.21	8.28	51.2	20.91
E	7.36	498	2.11	35.58	64.7	20.45
F	7.13	223	3.7	6.09	64.3	20.98
G	7.24	1183	3.42	27.82	131.8	20.11

Water Quality and DN value: The DN values are affected by sediments, physical properties and dissolve materials (Potes et al., 2012). The electromagnetic energy is influenced by water quality.

There is a relation between DN values and the physical properties of water. The NDWI values and water parameters have been categorized according to minimum to mean-1σ, mean-1σ to mean, and mean to maximum. The A, B, C, D, E, F and G geospatial points are represented by the frequency of occurrence (Table 4). The frequency of occurrence has focused the relation between water quality and Satellite Image.

Table 4: Category of water parameter

Parameter	Minimum to Mean-1σ	Mean-1σ to Mean	Mean to Maximum
NDWI	F	B,D,F	A,C,G
pH	E,F,G	C,D	A,B
TDS	D,F	E	A,C,G
DO	E,D	F,G	A,B,C
Turbidity	B,D,F	A	C,E,G
EC	D,F	B,E,F	A,C,G
Temperature	Detected by band 10 of Landsat OLI		

The NDWI is more fruitful in TDS, EC and Turbidity, here the correlations are always strongly positive. The NDWI and pH have positive correlation and DO have negative correlation (Table 5).

Table 5: Correlation of NDWI and water parameter

Parameter	**TDS**	**pH**	**DO**	**Turbidity**	**EC**
NDWI	0.8	0.34	-0.13	0.43	0.77

The trend analysis of NDWI and others parameters have shown that the NDWI values have more optimum results for TDS, Turbidity and EC. The temperatures of satellite image have thermal band (Band 10 or 11 of Landsat OLI). In this analysis, the pH and temperature is interrelated so the thermal band of Landsat OLI is more effective for pH (Figure 3).

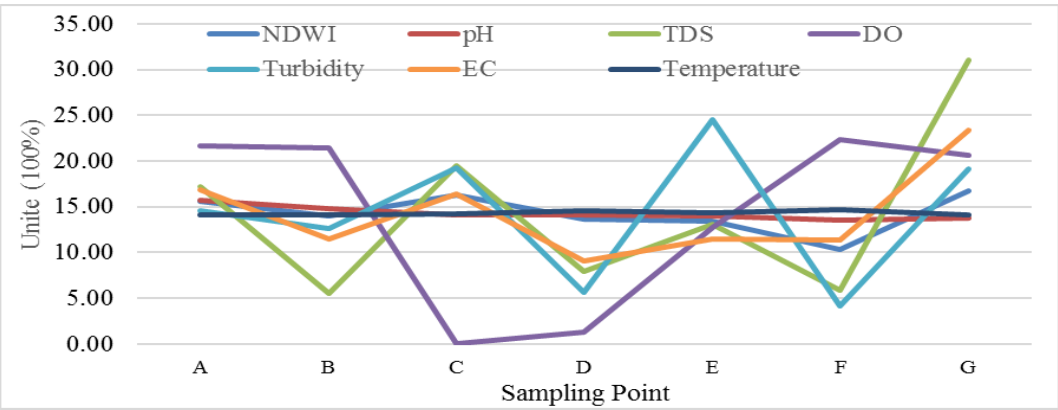

Figure 3: NDWI and water parameter

5. CONCLUSIONS

We have found very strong relations between water quality and satellite image. The water quality parameters are categorized into three types and they are highly related with the DN values of Landsat OLI images. The strong positive correlations have been found among the DN values of NDWI, TDS, Turbidity and EC. The pH is related with thermal band of Landsat OLI. The temperature is related with the radiance value of thermal band. This research indicates that some parameter of water quality is highly interrelated with the reflected sensors

of Landsat OLI. Mainly TDS and Turbidity of water serve as major factors for the interpretation of Landsat OLI imageries.

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Water Resources Management in Bangladesh: Past, Present and Future

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Abstract: *Water Management and Water Governance in Ganges-Brahmaputra-Meghna (GBM) Delta especially in Bangladesh is a crucial issue. Water Management practice and initiative started in ancient times. During ancient rulers, there was some form of water management and governance structure, while in the modern periods, state controlled water management was there for decades and attempts for decentralization of water management and governance were made which is growing gradually towards pro-people through adoption of peoples' participation in management and governance as well. Water management in Bengal in the early stages was better than in other parts of the world. About 3000 years ago, the rulers of the then Bengal introduced overflow irrigation system that was practiced till 1200. From 1200 there were various periods / segment of water management like, the medieval period (1200-1757), colonial Period (1757-1947) and modern period 1947 onward. In the modern period, Government of Bangladesh has taken many initiatives on Water Resource Management (WRM) system. These include nation owned Large Scale projects (50s & 60s) to extensive minor irrigation in winter using Low Lift Pump (LLP) and tube wells and Flood Control and Drainage (FCD) small scale, low cost, quick-gestation flood control and drainage projects (70s-90s) to Integrated Water Resources Management (IWRM). WRM projects included structural and non-structural measures (FF&W – Flood Forecasting and Warning & FP-Flood Proofing). At the onset of 21st Century, adoption of National Water Policy (NWPo) and Guidelines for Participatory Water Management (GPWM) by the Government, Stakeholder participation at all levels of the project cycle, Socio-economic Impact Assessment (SIA) and Environmental Impact Assessment (EIA) and Environmental Management Plan (EMP) have been made mandatory in planning practice. Recent day's WRM initiatives pay preference to participatory approach and capacity development to build a resilient nation. Climate change and its impact on economy, livelihood, infrastructure and environment have become a great concern for adaptation and to establish resilience with changes of scenario. Now on, WRM planning will have to consider climate change and add provision of adaptation and resilience to climate change and gradual decentralization of management. This study will help finding out the integration gap in approach, effective participation of stakeholder/community in the project cycle, understanding between agencies and community/stakeholders, focusing on limitation of water resources and optimization of use of the scarce resource and ensure minimum impact on nature and environment.*

Keywords: Water resources, Management, Governance, Climate Change, Adaptation, Resilience, Community, Stakeholder, Decentralization, Participatory.

1. INTRODUCTION

In the ancient period, the human habitation grew around Water Sources and usually natural plantation, the source of food for their survival. With elapse of time and population growth, source of survival water and plants/fruits/crops started getting limited; human community had to find more resources. Initially they found new resources by moving to new areas, but at later stage, they started growing food with additional efforts like cultivation and irrigation. So they have to consider the water source, its quality, distance from farm, transport of water, optimized use etc. All these activities they learn by doing, is termed Water Resources Management in course of time, which now a day is a combination of many activities, lot of considerations such as Technology, Nature, Society and Economy; and involvement of many, States, Stakeholder, Beneficiary, affected community and last not least the anticipated changes of Bio-Physical systems. **Modern Water resource management** is the activity of planning, developing, distributing and managing the optimum use of water **resources**. It is a sub-set of water cycle management. While **water resources** are sources of **water** that are potentially useful. Uses of **water** include agricultural, industrial, household, recreational and environmental activities. The majority of human uses require fresh **water** which is gradually decreasing and additional efforts are needed to get it.

Bangladesh, the delta of the Ganges-Brahmaputra-Meghna (GBM) river system is one of the largest deltas in the world. Totalling approximately 24 thousand kilometres, the country is traversed by a vast network of huge rivers, their tributaries and distributaries, mountain streams, winding seasonal creeks and canals. Some 405 rivers crisscross the country, of which 57 are trans-boundary. Bangladesh has a monsoonal climate with a hot, humid wet season, a cooler dry season and a hot dry pre-monsoon season. In January, the daily maximum temperature averages about 26ºC and in April is about 35ºC. The annual average rainfall varies from 1927 mm in the north west (NW) region, 1950 in the southwest – south central (SW-SC), 2133 in the north central (NC), 2447 in the south east (SE) and 3091 mm in the north east (NE). The corresponding figures for potential evapotranspiration are 1309, 1327, 1275, 1276, and 1261 mm for these regions. Thus, the two most westerly regions are the driest and have the greatest potential evapotranspiration, with rainfall the greatest and potential evapotranspiration least in the northeast. The strong monsoon peak of rainfall happens from May to September or October, with very little rainfall in December and January. The potential evapotranspiration is more evenly distributed, though it is generally higher towards the end of the dry season from March to May. Rainfall exceeds potential evapotranspiration in the wet season, but is less than potential evapotranspiration during the dry season (CSIRO, 2014).

Much of Bangladesh is a low lying, flat flood plain at the confluence of three major rivers and is vulnerable to seasonal (river & flash) and coastal floods & storm surges. About 20% of the country is flooded annually, during extreme events; this can increase up to 60% or more (. The GBM river system has the largest total sediment load (2.4 billion tons) (Anwar, 1988) in the world, derived principally from upstream areas. Coupled with a dynamic hydraulic regime, the main rivers are subject to active erosion and sedimentation processes. On average, some 6000 ha of river bank erosion occurs in the country annually, leading to the displacement of about 50,000 people.

2. METHODOLOGY

In order to draw a picture of WRM in this particular part of the World, review of literature on Water Resources Management Practices, Water Policies, Plans, Achievements of Implementing Agencies, Books, Documents, Reports, Papers on Governance and role of Community in Water Management have been searched in the BWDB, LGED, IWM libraries, notable resource persons have been contacted, and discussed to collect information in different form. Collected reading materials have been reviewed to extract views, comments, critics and consensus of different authors /persons on the subject matter and compiled to prepare a comprehensive overview on Water Resources Management, development, practice, progress, technology, adaptation with ever changing multi-dimensional drivers those influence WRM; from indigenous knowledge exercise to adoption of latest technology, concepts, consideration of socio-economic and biophysical changes

in the regional and global scale.

3. WATER RESOURCES

The water resources available to Bangladesh consist of both internally generated surface water resources (rainfall and runoff) and trans-boundary inflows, and groundwater. According to FAO (2013), annual total renewable water resources amount to approximately 1211 (bcm) (SW-1190 bcm., GW-21 bcm.) Internal renewable water resources are estimated at 105 bcm (SW-84 bcm and GW-21 bcm). Externally renewable water resources total 1106 bcm, of which 0.03 bcm from groundwater and the remainder from trans-boundary river flows. Water resource is the key to the countries' sustainable development. The National Sustainable Development Strategy (NSDS), (Bangladesh Planning Commission, 2013), identifies the following five priority development sectors for the country: Agriculture, Industry, Energy, Transport and Human Resource Development. Human Resource Development is strongly affected by the availability and quality of water resources and a safe sanitation system, with particular poverty, gender and health dimensions.

The SW resources available in Bangladesh include main and regional rivers including trans-boundary and a vast network of wetlands. Water quality is a growing concern for the country, with 32 rivers and many of the wetlands are at serious environmental risk due to pollution, encroachment, and disconnection between wetlands and the river system. Groundwater, the major source of water for irrigation and (safe) drinking water and industry is at threat for arsenic pollution of shallow aquifers and intrusion of salt water, industrial pollution, declining of GWT due to overexploitation etc.

4. WATER VULNERABILITY AND CHALLENGES

Challenges are natural as well as man-made, including alternating floods and droughts, cyclones, expanding water needs of a growing population, large scale sedimentation and river bank erosion, rapid urbanization and industrialization, global warming and deforestation. An additional and growing challenge is the deterioration of surface and groundwater water quality, the decline of natural wetlands and water bodies and the maintenance of healthy aquatic ecosystems. Climate change, the expected upstream development and abstraction of water and the lack of a sustainable financing of water resource infrastructure operation & maintenance further exacerbate these challenges (BDP2100, 2015). With enormous crisscross river systems, having mostly low lying deltaic landforms, receiving maximum runoff from Hindu Kush Himalayan (HKH) region and being close proximity to the Bay of Bengal, the country is most vulnerable to climate related disasters. Bangladesh is already experiencing increased temperature, changes in rainfall pattern and distribution, sea level rise and salinity intrusion at an accelerated rate and increased disaster intensity, which will become greater issues in the future (Rahman, LinkedIn. /face book, December 13, 2016).

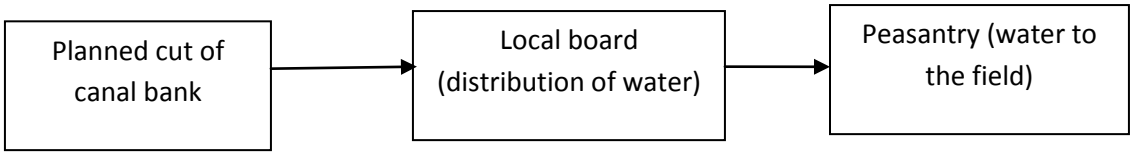
Water Resources Management in Bangladesh

Historical development of water related activity of human kind reached this modern definition "Water Resources Management" after a long way back from Overflow Irrigation-Canal Digging-Pond/Dighi digging-Water Management-Water Resource Management (WRM) to Integrated Water Resource Management (IWRM) and now on future Water Management shall have to be Resilient and Adaptive to Climate Change to ensure Water Governance by Stakeholder participation.

According to Ali (2002) Water Management in Bengal-East Pakistan-Bangladesh can be divided into four distinct Period(s), namely Ancient Period, Medieval Period, Colonial Period and Modern Period. The periodic development is discussed here briefly.

Ancient Period

Irrigation (Water Management) in Bengal in the early stages was better than in other parts of the world. About 3000 year ago, the rulers of the then Bengal introduced overflow irrigation system that was practiced till 1200. Ancient ruler's representative used to planned

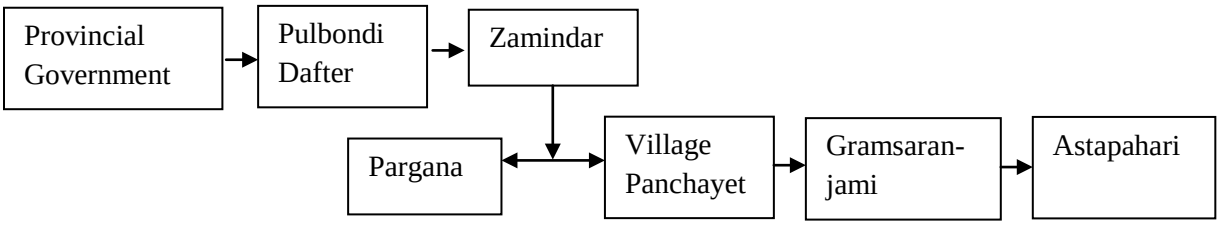


cutting of the canal bank, handover the responsibility of distribution of water to the local boards. The boards working through the peasantry ensured water distribution in the field. The management of overflow irrigation was systematic (as shown above) and somehow involved peasantry participation (Ali, 2002).

The medieval period (1200-1757) (Sultani , Turkish and Mughals Period)

During this period, the irrigation practice improved from overflow to tank irrigation, Flood control through construction of embankments and improvement of drainage facilities began, which continued till the end of the Mughal Empire in 1757.

The Mughal rulers maintained the previous systems and added the concept of additional canal along the river channels (for better irrigation, drainage and navigation) and embankments besides the rivers. The Mughals also added dredging of rivers (Ali, 2002). The Mughals maintained an independent "Pulbandi Daftar" for looking after embankments, roads, bridges and river dredging. The Pulbandi Daftar delegated functions to the local Zamindars and Government allocated budget in the form of a deduction from land revenue collected by them from the Parganas. Thus, the Zamindars took responsibility of FCDI management and maintenance by deploying day and night workers named 'Astaprahari". Astapaharis worked under supervision of village officials of 'Gramsaranjami', the gramsanjami worked under the direction of Village Panchayet. Everyone works of FCDIM within the limits of a mouza were looked after by Village Panchayet. Similarly, these works within a Zamindari were looked after by the Zamindars. The hierarchy is presented below (Ali, 2002).



Colonial Period (1757-1947)

The British regime covering 1757-1947 is called the Colonial Period. Colonial British rulers abolished the Pulbandi Daftar. Zamindars were relieved of their traditional duties. The state support to the Village Panchayet was withdrawn and the gramsaranjami was dissolved. As a result, the maintenance of canals and embankments were neglected. Unofficially, the local Zamindars continued their maintenance job for a long period of time until the Zamindari system was abolished by the British ruler. The end results were frequent flooding and subsequent crop damage.

Modern period 1947 onward

Recent Past 1947-71 - East Bengal got independence as a part of Pakistan and was named East Pakistan in 1947, and the modern period starts. Negligence of the British Rulers in WM system maintenance for long 200 years, conditions of the WM system badly deteriorated resulting frequent flood and damage of crops. Prior to 1947 there had not been any Government-led national scale water sector development Policy, plan or program. Recent past 1947-71 historical events towards WRM include appointment of Krug Mission, 1956/7 and formulation of first ever 20 years Master Plan (IECO), 1964, which emphasised on large scale FCDI projects development of WR of the country to achieve the goal of increasing agriculture production to achieve

national self-sufficiency (Ali, 2002), examples are implementation of CEP, KIP, GKIP and CIP projects. Implementation of Master Plan 1964 yielded immediate results. After two decades, reviews and evaluations indicated that the performance of large scale projects had deteriorated considerably, especially in terms of O & M (Rahman et al., 2007), everything under state ownership.

Present Period 1972 onwards: Key events and key development features are discussed below.

(A) Land and Water Resources Sector Study (World Bank), 1972: Creation of the BWDB in 1972, adoption of a strategy of extensive minor irrigation in winter using LLP and tube wells and for flood control small scale, low cost, quick-gestation flood control projects in shallow flooded area (DFID, WRM in Bangladesh). This was a major deviation in the strategy followed since 1964. Examples include implementation of EIP, LRP, DDP and SRP. The main aims of the project were to increase in agricultural production and generation of employment for unskilled laborers during slack season and "increased participation of the beneficiaries in planning, operation and maintenance" through allocating earthwork to Landless Contracting Societies (LCS) (Datta, 1999), which was internalized within BWDB and GoB by including in GPWM.

(B) **Establishment Master Planning Organization, 1980 and WARPO in 1991; National Water Plan I, 1986 and II, 1991; Flood Action Plan, 1989; Merger of WARPO and FPCO, 1999:** In the 1980s, focus was shifted from mono-sector (agriculture) to multi-sector approach, Master Plan Organization (MPO) was created to undertake national water plan that prepared National Water Plan-I in 1986, NWP-I and National Water plan (NWP-II) in 1991 and Bangladesh Water and Flood Management Strategy 1995.These national plans assembled substantial information and used wide range of planning models and analytical tools for public sector strategies and programs, many of which were adopted by the Government and endorsed by donors (FPCO, BWFMS, 1995).

(C) National Water Policy, 1999; Guidelines for Participatory Water Management, 2001; National Water Management Plan, 2004 (completed in 2001); Coastal Zone Policy, 2005:

The strategy BWFMS recommended preparing a **National Water Policy** and a **National Water Management Plan.** The Government of Bangladesh Finalized the National Water Policy in 1999 (Ministry of Water Resources, 1999). The declaration of the National Water Policy is a bold step towards Good Governance in Bangladesh. (Honourable Prime Minister Sheikh Hasina, Ministry of Water Resources, 1999, forward). It is the policy of the Government that all necessary means and measures will be taken to manage the water resources of the country in a comprehensive, integrated and equitable manner (**Ministry of Water Resources, 1999, p-2).** The policy is bringing order and discipline in the exploration, management and use of water resources in Bangladesh with stakeholders /people's participation at all stages of project cycle in sustainable WRM**. Guidelines for Participatory Water Management, 2001**, followed by **Participatory Water Management Rules, 2014** was formulated for wise implementation of the National Policy and Strategy. Socio-economic Impact Assessment (SIA) and Environmental Impact Assessment (EIA) and Environmental Management Plan (EMP) have been made mandatory in planning practice. National Water Management Plan, 2004 outlined 84 programs under 8 clusters for integrated, comprehensive and sustainable WRM of the Country**. Haor Master Plan**, 2013 and **National Water Act, 2013** added opportunities for efficient WRM and water governance as well.

5. FUTURE WRM

Integrated Water Resource Assessment (CSIRO, 2014); The project studied water resources in Bangladesh from both a physical and a socio-economic perspective, examined both water supply and water demand issues, where demand has both physical components (irrigation water demand) and socio- economic components (urban and industrial demand, demand for food and hence irrigation). It examined historical water use and crop production, and the likely future water use influenced by climate change, population growth and a growing economy. This also examined the impacts of changing water on the national economy and on the vulnerability of individuals and households.

Update National Water Management Plan (WARPO), is on-going that will take into account findings of IWRA (CSIRO), Policy, Strategy, Guidelines, Acts, Rules and Regulations. It will pay preference to participatory approach and capacity development to build a resilient nation. Climate change and its impact on economy, livelihood, infrastructure and environment has become a great concern for adaptation and to establish resilience with changes of scenario. Now on, WRM planning will have to consider climate change and add provision of adaptation and resilience to climate change and gradual decentralization of management. The SDG, Bangladesh Vision 2021, Bangladesh Vision 2041 will be guiding future WRM planning. The ongoing Bangladesh Delta Plan 2100 will help us to find out the integration gap in approach, effective participation of stakeholder/community in the project cycle, understanding between agencies and community/stakeholders, capacity building of implementing agencies, focusing on limitation of water resources and optimization of use of the scarce resource and ensure minimum impact on nature and environment.

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Failures of Steel Bridge Structures due to Cyclic Loading – A Review

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Abstract

Previous research on failures of steel bridges due to cyclic loading have been examined and studied. The most common reasons for bridge failures are due to force majeure, accidental overload and impact, structural and design deficiencies, construction and supervision mistakes, as well as lack of inspection, maintenance and repairing. Furthermore, fatigue failure is another main cause of bridge collapse. Previous experimental determinations of Stress Concentration Factors (SCFs) on empty tube-to-tube T-joints have been investigated. This investigation resulted in the development of design guidelines for fatigue of empty CHS uniplanar and multiplanar joints. However, little research has been carried out on the determination of the SCFs of T-joints with concrete-filled chords. The aim of this paper is to highlight the research gaps for future research in order to reduce failures of steel bridges under cyclic loading.

Keywords: fatigue design, steel structures, Stress Concentration Factors (SCFs), concrete-filled joints

1. INTRODUCTION

Chen et al (2015) stated that the use of tubular structures has increased considerably because of their light weight, easy fabrication, rapid erection and pleasing appearance. Currently, in steel tubular structures, concrete-filled steel tubes (CFST) are widely used as main members. CFST have high stiffness, high strength and high ductility (Han, cited in Chen et al 2015). Zhou and Zhu (1997) stated that numerous research achievements of CFST structures were transferred to construction practices. As a result, CFST have been widely used in numerous structures for arch bridges, electricity transmission masts, industrial buildings and platform columns in underground railways. Zhao et al (2010) stated that there is an increasing trend in using CFST in structural frames and support, industrial buildings, spatial construction and transmitting poles. In bridge and high-rise buildings structures, such composite columns have become very popular.

Due to the increased application of CFST structures, CFST arch bridges have become one of the competitive styles in moderate span or long span bridges. Two types of long-span concrete-filled tubular arch bridges are listed in Table 1 and Table 2.

Table 1. Recent long-span concrete-filled tubular arch bridges (Zhou & Zhu 1997)

Bridge	Main span (m)	Service date	Type
Sanshanxi bridge	200	September 1995	Lohse arch
Huangbei River Bridge	160	November 1996	Upper loading fixed arch
Xialao River Bridge	160	November 1996	Upper loading fixed arch
Xiamenkou Wujiang Bridge	140	September 1996	Fixed arch

Table 2. Recent Reinforced Concrete (RC) arch bridges with concrete-filled tubular stiffened skeletons (Zhou & Zhu 1997)

Bridge	Main span (m)	Service date	Type
Wanxian Yangtze River Bridge	420	July 1997	Upper loading fixed arch
Shuanglong Bridge	168	October 1994	Fixed arch
Luonguo Zinshajiang Bridge	160	1995	Fixed arch
Taibai Bridge	130	1993	Rigid RC frame

The Huangbei River Bridge and Xialao River Bridge are two typical examples of concrete-filled tubular arch bridges. The Huangbei River Bridge shown in Figure 1 and Xialao River Bridge shown in Figure 2 were built in China and completed in 1996 to carry traffic (Zhou & Zhu 1997). The concrete-filled tubular arch bridges were confirmed to be economical and efficient. The concrete-filled tubular arches generate a composite structure of high load capacity that is light weight. The hollow tubular structure serves as formwork, reinforcement and false work during erection (Zhou & Zhu 1997).

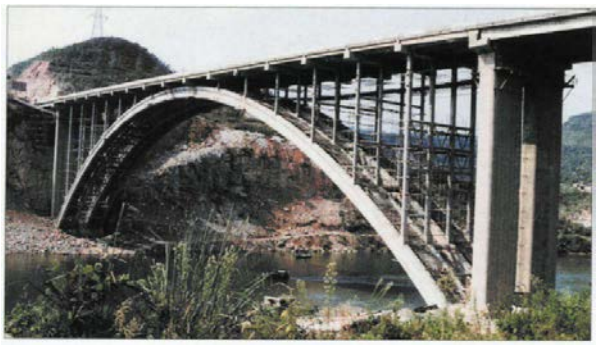

Figure 1. Huangbei River Bridge (Zhou & Zhu 1997)

Figure 2. Xialao River Bridge (Zhou & Zhu 1997)

A 370 m electric transmission line tower was constructed as shown in Figure 3. The lattice structures' joints connect the steel tube brace and concrete-filled steel tube chord (Chen et al 2015).

Figure 3. Electric transmission line tower in Zhejiang Province (Chen et al 2015)

2. LITERATURE REVIEW

Previous research on failures of steel bridges due to cyclic loading have been examined and studied. The main causes of bridge collapse are identified and examples of bridges that have failed are given. Previous experimental determinations of SCFs on concrete-filled tubular T-joints, tube-to-tube T-joints, CHS N-joints and tube-to-plate T-joints are provided. Existing design guidelines for fatigue are also reviewed.

2.1. Main Causes of Bridge Collapse

The most common reasons of bridge failures are due to force majeure, accidental overload and impact, structural and design deficiencies, construction and supervision errors, as well as lack of inspection, maintenance and repairing (Biezma & Schanack 2007). Bridges are susceptible to damage by aggressive environmental conditions, human actions and natural disasters. Furthermore, fatigue failure is another main cause of bridge collapse. Table 3 provides some examples of bridges that have failed due to the causes mentioned above.

Table 3. Summary of bridge failures (Biezma & Schanack 2007)

Date	Bridge Name	Location	Fatalities	Main Causes
06/12/1825	Bridge above the River Saale	Germany	55	Overload
28/12/1879	Tay Bridge	United Kingdom	75	Construction and supervision mistakes Design deficiencies
14/06/1891	Railway bridge above the River Birz	Switzerland	>70	Structural and design deficiencies
15/12/1967	Silver Bridge Pt.	Pleasant, New Jersey	46	Construction mistakes and lack of maintenance or inspection
15/10/1970	West gate Bridge	Melbourne, Australia	35	Construction, design deficiencies and supervision mistakes
10/11/1971	Rhine Bridge	Koblenz, Germany	13	Design deficiencies
14/04/2003	Sgt. Aubrey Cosens VC Memorial Bridge	Latchford, Canada	None	Design deficiencies, lack of maintenance and inspection
01/08/2007	I35W Bridge	Minneapolis, United States	13	Fatigue cracks in structural members (Hao 2010)

2.2. Experimental Determination of SCFs

2.2.1. Tube-to-tube T-Joints

The main girder for truss arch bridges and cable stayed bridges can be made of circular hollow sections (CHSs) and Concrete-filled Circular Hollow Sections (CFCHSs) in large span bridges (Wang et al 2013). The use of welded connections in bridges under cyclic loading determines the fatigue strength which controls the life of steel bridges. A CHS to CHS joint plus ten CHS to CFCHS joints were designed to consider the effect of concrete strength grades on SCFs at joints as well as considering the effects of different non-dimensional geometric parameters. The CHS brace members subjected to axial tensile or compressive force were known to be fully welded. The quality of welds influences the fatigue strength of welded joint. High quality welding is necessary to avoid physical defects such as porosity and cracks. Wang et al (2013) noted that a crack is the worst type of defect. Figure 4 displays the set-up of the fatigue test for each T-joint specimen. The T-joint's brace is under a cyclic axial load.

2.2.2. Experimental Investigation of CHS N-Joints

Chen et al (2016) carried out the SCF testing of 4 large eccentricity N-joints. These N-joints were subjected to axial compression load in the vertical CHS brace, axial tension loading in the inclined CHS brace and without additional axial loading in the horizontal CHS chord (Chen et al 2016). Figure 5 shows a typical joint specimen. The welding that is located at the intersections was complete

penetration groove weld. The ends of the inclined brace and chord, in the test specimen's set-up, were welded onto the end of the flat plates (10mm thickness) and bolted to create a pin support of inclined chord as well as brace.

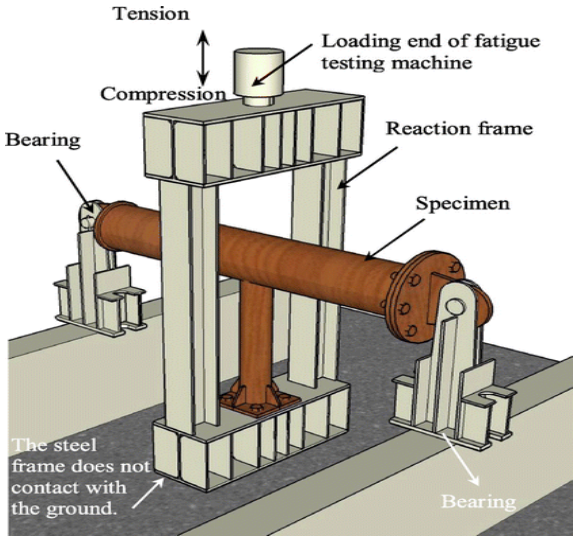

Figure 4. Fatigue test set-up (Wang et al 2013)

Figure 5. N-joints specimen subjected to axial loading (Chen et al 2016)

2.2.3. Fatigue Tests of Tube-to-plate T-Joints

Jiao et al (2013) investigated fatigue behaviour of very high strength (VHS) circular steel tube (t<2mm) to plate T-joints under cyclic in-plane loading. They carried out fatigue tests through the use of a MTS-810 testing machine. This machine had a loading capacity of 100KN. Figure 6 shows the set-up of the fatigue test. SHOWA strip strain gauges were used on the specimens' tension side in the bending plane to measure SCFs based on the hot spot stress method and to determine S-N fatigue design equations.

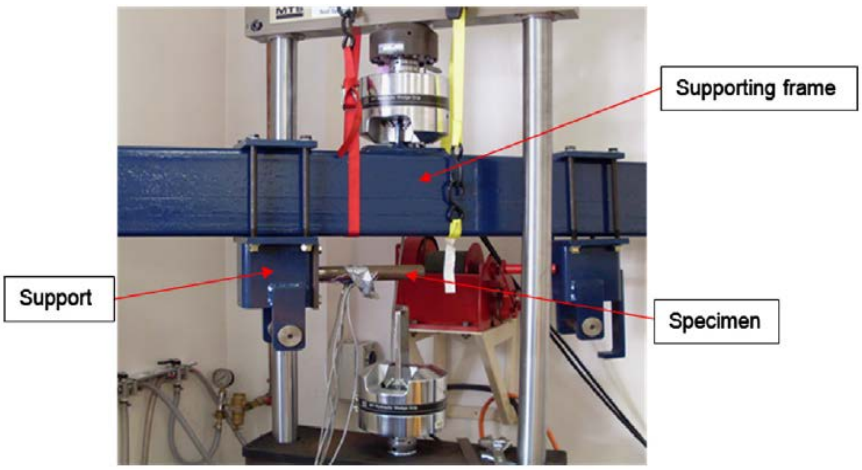

Figure 6. Fatigue test set-up (Jiao et al 2013)

2.2.4. Concrete-filled Tubular T-Joints

The SCFs of concrete-filled tubular T joints subjected to in-plane bending and axial loading were studied. The hot spot distribution was investigated through performing experimental investigation. In the research, five tubular T-joint specimens with concrete-filled chords and three specimens of hollow steel tubular T-joints were used. Chen et al (2010) stated that concrete filling decreases the peak SCFs. The test set-up for in-plane bending as well as axial loading is displayed in
Figure **7** and Figure 8. The test rig was used to test the tubular joints.

Figure 7. Test set-up for in-plane bending Chen et al (2010)

Figure 8. Test set-up for axial loading Chen et al (2010)

2.3. Fatigue design of CHS Uniplanar T or Y-joints using Design Guidelines

The CIDECT Design Guide No. 8 recommends that the SCF for uniplanar CHS T or Y-joints should be at least 2.0 (Zhao 2001). Figure 9 shows a CHS uniplanar T or Y-joint where the locations of the crown and saddle points as well as the geometric parameters are defined.

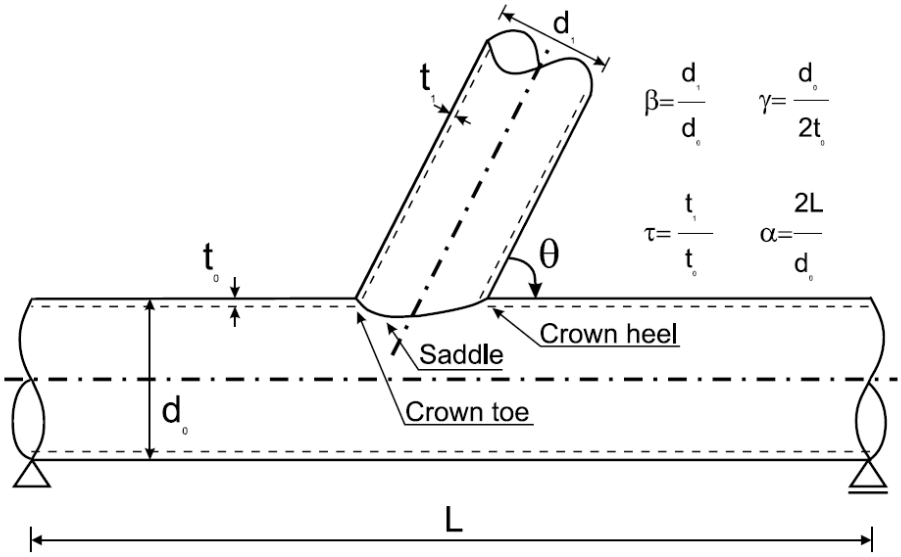

Figure 9. A uniplanar CHS T-joint or Y-joint (Zhao 2001)

The calculations for SCFs in CHS uniplanar T or Y-joint are based on the work of (Durkin, Ethymiou & Efthymiou, cited in Zhao 2001). An equation of SCF for CHS uniplanar Y-joint for the location of chord crown under axial loading is:

$$SCF = \gamma^{0.2}\tau[2.65 + 5(\beta - 0.65)^2] + \tau\beta(0.5C\alpha - 3)\sin\theta \quad (1)$$

- Where C = Chord-end fixity
  - C = 0.5 (for fully fixed chord ends)
  - C = 1 (for pinned chord ends)

Efthymiou cited in Zhao (2001) found a typical value for C to be equal to 0.7. The formulas of the SCFs provided by Zhao (2001) including Equation 1 are only for empty T or Y-joints.

3. CONCLUSION

In summary, extensive experimental determination of SCFs on concrete-filled tubular T-joints, tube-to-tube T-joints, CHS N-joints and tube-to-plate T-joints were carried out. The literature review showed that there is limited research on the fatigue behaviour of concrete-filled T-joints. As a result, there are no standards for the design of tubular joints with concrete-filled chords. Therefore, T-joint specimens with concrete-filled chord should be tested under axial tension, axial compression and in-plane bending in order to determine the distribution of the SCFs around the brace-chord intersection and to find out if it is beneficial to have concrete-filled T-joints for fatigue design.

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Influence of Chemical Admixtures on Fresh and Hardened Properties of Ready Mix Concrete

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Abstract

An experimental investigation was carried out to study the effects of types of chemical admixture on fresh and hardened properties of ready mix concrete. For conducting the investigation different concrete mixtures were prepared using six different types of chemical admixture (one water reducer based on lignosulfonate; and five superplasticizers based on organic polymer, second generation polycarboxylic ether polymer, polycarboxylic ether, sulfonated naphthalene polymer and synthetic polymer). Chemical admixtures were collected from the local market. Each mixture was subjected to prolonged mixing; slump tests were performed at 15 minutes intervals to assess the fresh behavior of concrete. Cylindrical specimens of diameter 100 mm and length 200 mm were made with the concrete mixtures for assessing the hardened properties of concrete. The specimens were tested for compressive strength, splitting tensile strength and Young's modulus at the age of 28 days.
Results indicate that sulfonated naphthalene polymer based superplasticizer and second generation polycarboxylic ether based superplasticizer show best performances in both fresh and hardened states of concrete. Lignosulfonate based water reducer exhibits poor performance in comparison to the superplasticizers.

Keywords: Chemical admixture, Compressive strength, Splitting tensile strength, Young's modulus.

1. INTRODUCTION

In the construction industry, the demand of ready mix concrete (RMC) is increasing rapidly day by day. The primary reasons behind this are: convenience of using RMC in high rise structures, shortage of space at construction site, saving of time related to the preparation of concrete on site and better quality of RMC. In cities like Dhaka, the time required to travel from RMC plant to construction site is very high, because of severe traffic congestions, especially during weekdays. To keep concrete workable for such a long time period is very challenging. Moreover, high ambient temperature in summer makes the situation worse, since high temperature adversely affects the workability of fresh concrete (Burg, 1996; Mehta and Monteiro, 2006). Therefore, high workability has become one of the most desired and essential properties of RMC in Dhaka city. A conventional practice to improve workability of concrete is to add water in the concrete mix. But with the increase of water to cement ratio (W/C), the compressive strength of concrete reduces significantly (Schulze, 1999; Dhir et al, 2004; Wassermann et al, 2009). So to overcome this problem, in recent years, RMC manufacturers in Dhaka city have started using chemical admixture as a fourth ingredient in concrete apart from cement, aggregates and water. Chemical admixtures allow RMC to achieve high workability without compromising its quality at hardened state (Devi and John, 2014).

In light of the above discussion, it is necessary to conduct a comparative analysis among the fresh and hardened properties of concretes made with different chemical admixtures in order to identify the best chemical admixture. Therefore, this study plans to investigate the effects of chemical admixtures available in the local market on workability of fresh concrete. The effects of chemical admixtures on compressive strength, splitting tensile strength and Young's modulus of concrete are also aimed to be evaluated.

2. EXPERIMENTAL METHOD

In this study six different types of chemical admixture were used. Among the admixtures, WR is water reducer and SP1, SP2, SP3, SP4, SP5 are superplasticizers. All the types of chemical admixture comply with ASTM C494. The chemical and physical properties of the chemical admixtures, and their dosage ranges as recommended by the manufacturers are mentioned in **Table 1**. Crushed stones were used as coarse aggregates. Natural river sand was used as fine aggregate. **Figure 1** shows the gradations of coarse and fine aggregates. Both the gradations of coarse and fine aggregates satisfy ASTM C33 requirements. The physical properties of coarse and fine aggregates are summarized in **Table 2** and **Table 3** respectively. CEM Type II/B-M cement (as per BDS EN 197–1:2000) was used as binding material. Tap water was used for mixing and curing of concrete.

Mixture proportions of the cases investigated in this study are summarized in **Table 4**. In this study, the maximum recommended dosages of the chemical admixtures were used to prepare the concrete mixtures. In all concrete mixtures, sand to total aggregate volume ratio (s/a), water to cement ratio (W/C) and cement content were kept constant to 0.40, 0.40 and 340 kg/m^3 respectively.

Slump tests of fresh concrete mixtures were done at 15 minutes intervals according to ASTM C143. 100 mm by 200 mm cylindrical concrete specimens were also prepared and tested for compressive strength, splitting tensile strength and Young's modulus according to ASTM C39, ASTM C496 and ASTM C469 respectively.

Table 1. Properties and recommended dosage ranges of chemical admixtures

Chemical admixture	Composition	Appearance	Specific gravity at 25 °C	Recommended dosage range (ml/100 kg of cement)
WR	Lignosulfonate based	Dark brown liquid	1.17	200 – 400
SP1	Sulfonated naphthalene polymer based	Dark brown liquid	1.24	700 – 1800
SP2	Polycarboxylic ether based	Light brown liquid	1.05	400 – 1200
SP3	Second generation polycarboxylic ether polymer based	Light brown liquid	1.10	500 – 1200
SP4	Synthetic polymer based	Dark brown liquid	1.22	500 – 1500
SP5	Organic polymer based	Dark brown liquid	1.19	600 – 1140

Table 2. Properties of coarse aggregate

Aggregate type	Specific gravity	Absorption capacity (%)	Abrasion (%)	SSD unit weight (kg/m^3)	Fineness modulus
Crushed stone	2.56	2.39	38.30	1549	6.95

Table 3. Properties of fine aggregate

Aggregate type	Specific gravity	Absorption capacity (%)	SSD unit weight (kg/m^3)	Fineness modulus
River sand	2.45	3.30	1520	2.52

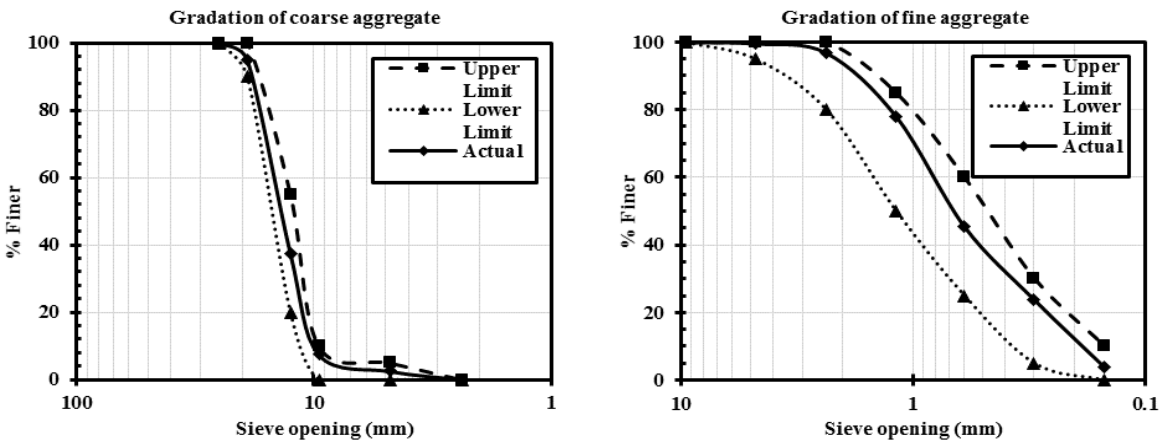

Figure 1. Gradation of coarse and fine aggregates

Table 4. Mixture proportion of concrete

Case ID	Chemical admixture	Dosage (ml/100 kg of cement)	W/C	s/a	Unit content (kg/m^3)			
					Cement	Sand	Aggregate	Water
C1	WR	400	0.40	0.40	340	743	1114	136
C2	SP1	1800			340	738	1107	136
C3	SP2	1200			340	740	1110	136
C4	SP3	1200			340	740	1110	136
C5	SP4	1500			340	739	1109	136
C6	SP5	1140			340	741	1111	136

3. RESULTS AND DISCUSSION

3.1. Effects of Chemical Admixtures on Fresh Properties of Concrete

Slump test results of fresh concrete mixtures prepared with different types of chemical admixtures are presented in **Figure 2**. It can be seen that concrete mixture prepared with sulfonated naphthalene polymer based admixture SP1 resulted maximum initial slump, it also remained workable for longest duration. The second best result was obtained for second generation polycarboxylic ether based admixture SP3. Concrete mixture made with synthetic polymer based SP4 resulted higher initial slump compared to concrete mixture with polycarboxylic ether based admixture SP2. However, concrete with SP2 remained workable for longer duration compared to concrete with SP4. Amongst the superplasticizers, organic polymer based SP5 imparted lowest workability to fresh concrete. Again, lignosulfonate based water reducer WR resulted poor workability compared to the superplasticizers.

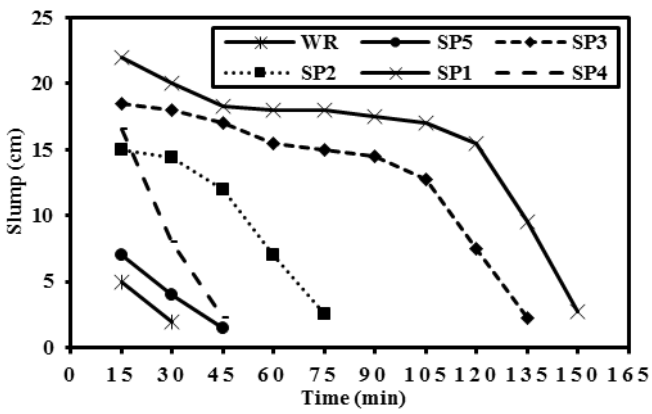

Figure 2. Slump test results of concrete mixtures made with different admixtures

3.2. Effects of Chemical Admixtures on Hardened Properties of Concrete

28 days compressive strengths of concretes made with different chemical admixtures are presented in **Figure 3**. It can be seen that concretes made with chemical admixtures exhibited better compressive strengths compared to the concrete made without admixture. The results therefore reapprove the findings of Mohammed and Hamada (2003), Papayianni et al (2005), Alsadey (2012), and Devi and John (2014) as opposed to the findings of Jerath and Yamane (1987) who concluded that the addition of superplasticizer in concrete mixture causes reduction in compressive strength. Papayianni et al (2005), and Devi and John (2014) attributed the reason of strength increase of superplasticized concrete to the improved workability of concrete in its fresh state which eventually leads to the formation of denser and less porous structure.

Figure 3 shows, second generation polycarboxylic ether based superplasticizer SP3 imparted the highest 28 days compressive strength to concrete. The second highest compressive strength was resulted by concrete with sulfonated naphthalene polymer based SP1. However, the compressive strengths of concrete with SP1 and concrete with SP3 were very close. Among the superplasticizers, organic polymer based SP5 imparted lowest compressive strength to concrete. Performances of the superplasticizers were better than that of the lignosulfonate based water reducer WR. The compressive strengths of concrete cylinders made with chemical admixtures were within the range of normal strength concrete specified by JSCE Guideline for Concrete (2007).

28 days splitting tensile strengths of concretes made with different chemical admixtures are shown in **Figure 4**. Concretes made with chemical admixtures resulted better splitting tensile strengths compared to the concrete made without chemical admixture. The results confirm the conclusions drawn by Shah et al (2014). Like compressive strength, the splitting tensile strength resulted by concrete with WR was less compared to the concretes made with superplasticizers. Concrete with second generation polycarboxylic ether based superplasticizer SP3 exhibited the highest splitting tensile strength. The second highest splitting tensile strength was resulted by concrete with sulfonated naphthalene polymer based SP1.

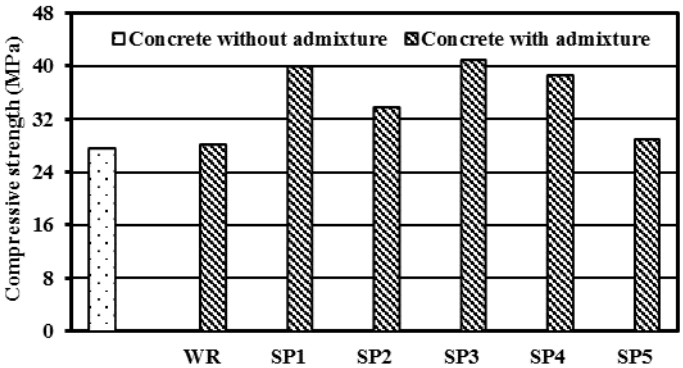

Figure 3. 28 days compressive strengths of concretes made with different admixtures

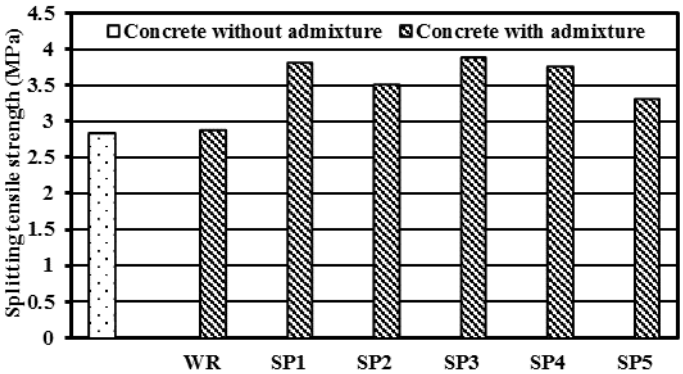

Figure 4. 28 days splitting tensile strengths of concretes made with different admixtures

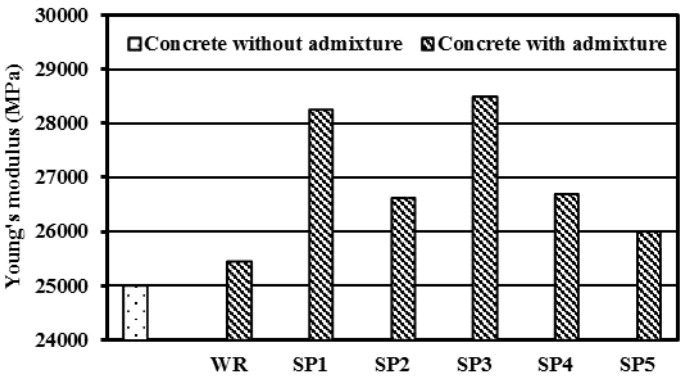

Figure 5. 28 days Young's moduli of concretes made with different admixtures

Young's moduli of concretes made with different chemical admixtures are shown in **Figure 5**. The results shown in **Figure 5** are similar to those presented in **Figure 3** and **Figure 4**. In this case also, concrete with second generation polycarboxylic ether based superplasticizer SP3 exhibited better performance compared to others. Concrete with lignosulfonate based water reducer WR resulted lower Young's modulus in comparison to the concretes made with superplasticizers.

Based on the results presented in **Figure 2**, **Figure 3**, **Figure 4** and **Figure 5**, it can be summarized that concrete with sulfonated naphthalene polymer based SP1 exhibited best performance in the fresh state of concrete. On the other hand, concrete with second generation polycarboxylic ether based SP3 showed best performance in the hardened state of concrete. However, in hardened state, the differences between the test results of concretes with SP1 and SP3 were insignificant.

4. CONCLUSIONS

Based on the results obtained from this experimental investigation, the following conclusions can be drawn:

- Sulfonated naphthalene polymer based superplasticizer shows best performance in improving workability of fresh concrete in comparison to other chemical admixtures. Sulfonated naphthalene polymer based superplasticizer also helps concrete to remain workable for longer time period in comparison to other admixtures. Second generation polycarboxylic ether based superplasticizer can be categorized as the second best chemical admixture in improving workability of fresh concrete.
- Concrete made with second generation polycarboxylic ether based superplasticizer exhibits higher compressive strength, splitting tensile strength and Young's modulus compared to concretes prepared with other admixtures. Concrete made with sulfonated naphthalene polymer based superplasticizer exhibits the second best performance in hardened state of concrete.
- Superplasticizers show better performance in improving fresh and hardened behaviors of concrete compared to lignosulfonate based water reducer.
- Concrete with chemical admixture results higher compressive strength, splitting tensile strength and Young's modulus compared to concrete without admixture, when the dosage of admixture is within the range recommended by manufacturer.

ACKNOWLEDGMENTS

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Utilization of Induction Furnace Slag in Concrete as Coarse Aggregate

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Abstract

An experimental investigation was carried out to explore the suitability of utilizing induction furnace slag as coarse aggregate in concrete. Cylindrical concrete specimens (100 mm by 200 mm) were made varying the W/C ratio (0.45 and 0.50), cement content (340 kg/m^3), and sand to aggregate volume ratio (0.44 and 0.48). The concrete specimens were tested for compressive strength and splitting tensile strength at the age of 28 days. For comparison, similar investigation was also carried out on burnt clay aggregate commonly used in Bangladesh. Experimental results show that slag aggregates absorb less water compared to the burnt clay aggregates. Compared to burnt clay aggregate, concrete made with induction furnace slag aggregate gives more workability, compressive strength and splitting tensile strength. For obtaining the maximum compressive strength and splitting tensile strength, the optimum amount of replacement of burnt clay aggregate by induction furnace slag aggregate is found at 50%.

Keywords: Aggregate; Concrete; Compressive strength; Induction furnace slag.

1. INTRODUCTION

Utilization of sustainable construction material and technology can be considered as an essential tool to ensure sustainable environment. Due to growing environmental awareness, as well as stricter regulations of managing industrial by-products, the world is greatly concerned finding ways to utilize the by-products as secondary raw materials in other industrial branches. In Bangladesh, most commonly used coarse aggregate is burnt clay aggregate (produced by crushing clay burnt blocks, locally known as brick). Due to rapid urbanization in the country, the demand of aggregate is increasing every year. In 2012, 8.6 billion bricks were produced (to use as coarse aggregate in concrete, to construct walls etc.). It was also estimated that the demand of bricks is increasing every year by 5.3% (Hossain, 2012). But brick industries are associated with a lot of negative environmental impacts. Therefore, it is necessary to find out possible alternative resources that can be used as coarse aggregate in construction works. An extensive study on recycling of demolished brick aggregate concrete as coarse aggregate was carried out for the sustainable use of construction materials in Bangladesh (Mohammed et al, 2015). Also, it is necessary to find further alternative building materials such as steel slag, which is a by-product of steel industry. The utilization of steel slag in concrete as coarse aggregate will help achieve sustainable use of construction materials and will also reduce the emission of greenhouse gases during the production of burnt clay aggregate. Raza et al (2014) conducted a study by replacing natural aggregate with iron slag aggregate at different replacement ratios, such as 0%, 10%, 20%, 30%, 40% and 50%. The results were compared with the results

obtained for conventional concrete. It was concluded that utilization of iron slag in concrete will enhance the compressive strength of concrete. Hiraskar and Patil (2013) also investigated the possibility of utilization of blast furnace slag as aggregates in concrete. It was found that similar level of strength of concrete can be obtained by utilizing steel slag as concrete made with natural aggregate. It is clearly understood that utilization of blast furnace slag in concrete as coarse aggregates has no negative effects on the short term properties of hardened concrete. However, it is still necessary to understand the properties of fresh and hardened concrete, if induction furnace slag aggregate is used to replace conventional brick aggregate. With this background, this study has been planned.

2. EXPERIMENTAL METHOD

Slag aggregate sample was collected from a crushing plant of induction furnace slag of a local steel manufacturing company. The slag and brick aggregates were tested for grading, unit weight, abrasion, specific gravity, absorption capacity and abrasion as per ASTM standards. The maximum size of aggregate was 20 mm and the grading of aggregates was controlled as per ASTM C33. Natural river sand was used as fine aggregate. The physical properties of coarse and fine aggregates are summarized in the **Table 1**. The air content in concrete was assumed to be 2% as no air entraining admixture was used. The mixture proportions are summarized in **Table 2**. High-range water reducing admixture (4 ml per kg of cement) was used for W/C = 0.45.

Table 1. Physical properties of coarse and fine aggregates

Type of aggregate	Specific gravity	Absorption capacity (%)	SSD unit weight (kg/m^3)	Abrasion (%)	FM
BC (Brick Chips)	2.14	19	1211	38.8	Controlled as per ASTM- C33
IFS (Induction Furnace Slag)	2.65	2.62	1550	35.2	
Fine Aggregate	2.59	3.30	1520	-	

Table 2. Mixture proportion of concrete

Replacement Ratios (%)		Cement content (kg/ m^3)	s/a	W/C	Case ID	Unit content (kg/m^3)				
						Cement	Fine Aggregate	Coarse Aggregate		Water
BC	IFS							Brick Chips	IF Slag	
0	100	340	0.44	0.45	0%BC+100%IFS-0.45	340	809	0	1053	153
				0.50	0%BC+100%IFS-0.50	340	790	0	1028	170
25	75			0.45	75%BC+25%IFS-0.45	340	809	213	790	153
				0.50	75%BC+25%IFS-0.50	340	790	208	771	170
50	50			0.45	50%BC+50%IFS-0.45	340	809	425	527	153
				0.50	50%BC+50%IFS-0.50	340	790	415	514	170
75	25			0.45	25%BC+75%IFS-0.45	340	809	638	263	153
				0.50	25%BC+75%IFS-0.50	340	790	623	257	170
0	100			0.45	0%BC+100%IFS-0.45	340	809	851	0	153
				0.50	0%BC+100%IFS-0.50	340	790	831	0	170
BC- Brick Chips, IFS – Induction Furnace Slag, W/C = water to cement ratio										

Cylindrical concrete specimens (100 mm by 200 mm) were made with different W/C ratios (0.45 and 0.50), sand to aggregate volume ratio = 0.44, and cement content = 340 kg/m^3. Brick aggregates (BC) were replaced with IFS by 0% (i.e., no replacement), 25%, 50%, 75% and 100% (i.e., full replacement). The grading curves of coarse and fine aggregates satisfy the requirement of ASTM C33 as shown in **Figure 1**. CEM Type II/A-M cement (as per BDS EN 197–1:2000) was used. Tap water was used for mixing and curing of concrete. After mixing concrete, slump was measured and then concrete specimens were made as per ASTM C31M–03. The specimens were cured under water till the time of testing. The specimens were tested at 7, 28, 60 and 90 days for compressive strength and tensile strength as per ASTM C39 and C496 respectively.

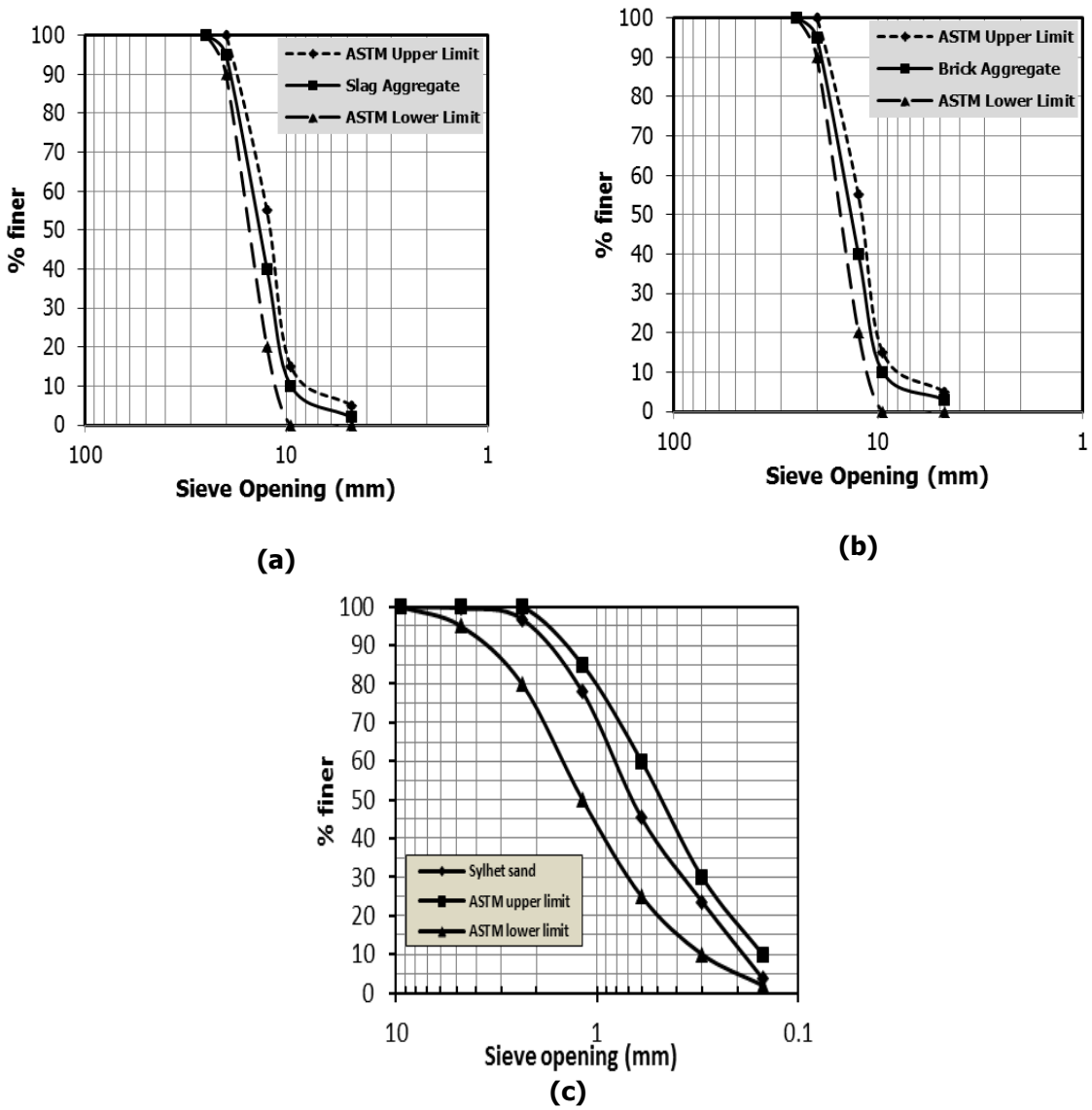

Figure 1. Grading curve of aggregates (a, b – coarse aggregates; c – fine aggregate)

3. RESULTS AND DISCUSSION

3.1 Workability of Concrete

The workability of concrete (measured by slump test) made with different replacement ratios of induction furnace slag is shown in **Figure 2**. It was observed that the workability of concrete increased with the increase of replacement of brick aggregate by induction furnace slag. Similar results were also obtained by Yuksel and Genc (2007). The reason of increase in slump with the increase of replacement ratio of induction furnace slag can be attributed to the lower absorption capacity of IFS aggregate compared to the BC. Another reason may be related to the shape of the IFS aggregate, IFS aggregates have more blunt edges compared to BC. It was also observed that, with the increase of s/a ratio (from 0.44 to 0.48), workability of fresh concrete reduced. It is due to the increase of amount of fine aggregate (sand) in concrete. With the increase of sand content, the total surface area of aggregate would also increase and therefore relatively more lubricating material would be required to increase slump.

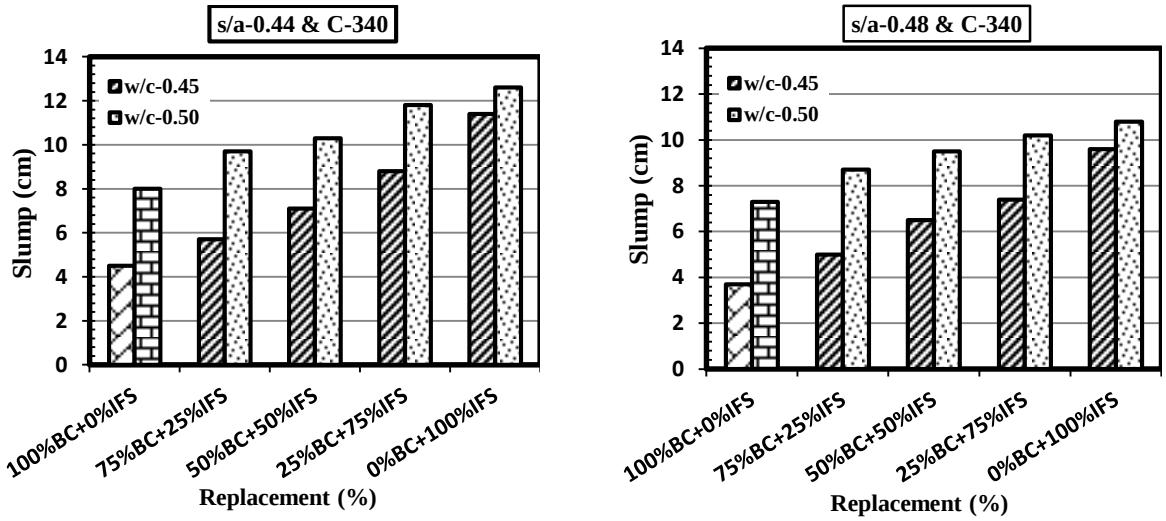

Figure 2. Workability of concrete

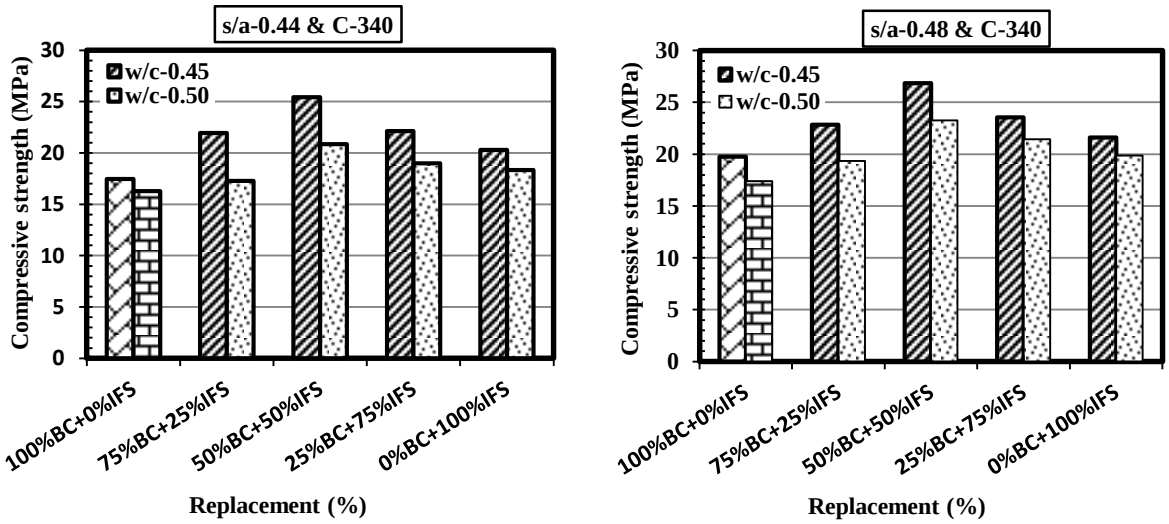

Figure 3. 28 days compressive strength of concrete

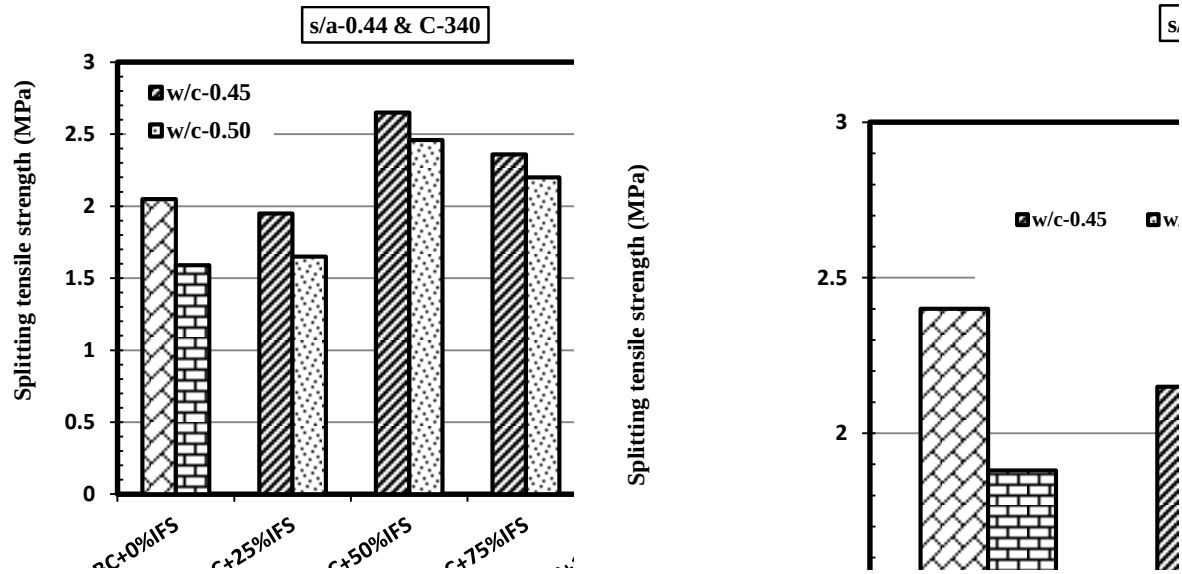

Figure 4. 28 days splitting tensile strength of concrete

3.2 Compressive Strength

The compressive strength of cylindrical concrete specimens made with different replacement ratios (0%, 25%, 50%, 75% and 100%) of brick aggregate by IFS aggregate is shown in **Figure 3**. It can be seen from **Figure 3** that compressive strength of concrete increased up to 50% replacement of BC by IFS aggregate. Beyond 50%, the compressive strength of concrete reduced but did not fall below the strength level obtained for the case of no replacement (100%BC+0%IFS). Similar results were also observed by Nadeem and Pofale (2012) and Murthi et al (2015). The increase in compressive strength can be related to the rough surface of IFS aggregate compared to the BC that would help to form a relatively stronger Interfacial Transition Zone (ITZ) around the IFS aggregate. It is also expected that at 50% replacement level, BC and IFS aggregate would produce a more compact system (amount of void in the mixture of aggregate will be the least) compared to the other replacement levels. As a result, maximum level of compressive strength of concrete was found at 50% replacement level. Irrespective of replacement level, concrete made with low W/C exhibited higher compressive strength compared to concrete made with high W/C. Using the mixture proportion as summarized in **Table 2**, it is possible to make concrete of 26 MPa and 22 MPa for W/C = 0.45 and 0.50 respectively by replacing 50% of BC with IFS aggregate.

It was found that with the increase of s/a ratio (from 0.44 to 0.48), the compressive strength of concrete increased. The reason may be attributed to the overall decrease in the volume of voids (in the mixture of fine and coarse aggregates) with the increase of the amount of fine aggregate. Similar results were also observed by Yang et al (2010), Mohammed and Rahman (2016), and Mohammed and Mahmood (2016). However, this study needs to be continued with further increase of sand to aggregate volume ratio (such as 0.50, 0.55 etc.).

3.3 Tensile Strength

The results of 28 days splitting tensile strength are presented in **Figure 4**. Same as compressive strength of concrete, the split tensile strength increased up to 50% replacement and then decreased with further increase of the amount of IFS aggregate. Again, the reason can be related to the improved interfacial transition zone (ITZ) around IFS aggregate. Nadeem and Pofale (2012) also observed similar results. They mentioned that excellent rugosity of slag aggregate ensures strong bonding and adhesion between aggregates and cement paste, and thereby increases the strength of concrete. It was also observed that an increase in s/a ratio resulted an increase in tensile strength which followed the trend of compressive strength as described earlier.

4. CONCLUSIONS

Based on the results obtained from this experimental work, the following conclusions are drawn:

- The absorption capacity of brick aggregate is much higher than the absorption capacity of induction furnace slag,
- The workability of concrete increases with the increase of replacement ratio of brick aggregate by induction furnace slag aggregate, and
- The optimum replacement ratio of brick aggregate by induction furnace slag aggregate for obtaining maximum compressive strength and tensile strength of concrete is found at 50%.

5. ACKNOWLEDGMENT

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Changes in Australian Rainfall Runoff and Its Implication for Estimating Design Rainfall

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Abstract

Recently, Geoscience Australia has released updated national guidelines for the estimation of design floods in Australia, commonly known as Australian Rainfall Runoff (ARR). The methodologies and guidelines proposed in ARR are crucial for an accurate estimation of flood risk and for the design of safe and sustainable infrastructures in Australia. The newly proposed ARR (ARR2016) has adopted new methods and data compared to old ARR (ARR1987), which resulted differences in the estimation of design rainfall across Australia. For example, in ARR2016 additional 30 years' rainfall data and 2300 extra rainfall stations have been included compared to ARR1987. Rainfall frequency analysis has been conducted by Generalised Extreme Value (GEV) distribution compared to Log-person Type III (LPIII), previously adopted in ARR1987. Bayesian Generalised Least Square Regression (BGLSR) is adopted in ARR2016 to predict daily rainfall from sub-daily rainfall statistics, which was previously done by Principal Component Analysis (PCA). In this paper, we review the guidelines for both ARR2016 and ARR1987 for design rainfall estimation and evaluate changes in design rainfall for selected stations in 8 major cities in Australia including Adelaide, Brisbane, Canberra, Darwin, Hobart, Melbourne, Sydney and Perth. Results presented in the paper will help Engineers and Managers of local governments to understand and implement new regulations proposed in ARR2016 for the estimation of design rainfall.

Keywords: *ARR1987, ARR2016, rainfall frequency, design flood.*

1. INTRODUCTION

Estimation of design rainfall is crucial to rainfall runoff modelling. Design rainfall, commonly known as intensity-frequency-duration (IFD) curves are frequently used by design engineers and scientists as an input to a wide range of design flood model, environmental studies and to design water infrastructure including bridges, culverts, stormwater drains, flood embankments and constructed wetlands. In Australia, the guidelines and methodologies for accurate estimation of design rainfall is provided by Australian Rainfall Runoff (ARR). The first edition of ARR was published in 1958, the second in 1977; however, the third version of ARR in 1987 (ARR1987) incorporated a complete revision of materials previously covered with a broader range of topics relevant to flood estimation (IEAust 2001). ARR1987 has been accepted and widely used by the relevant practitioners.

In ARR1987, IFD was developed using daily rainfall data from approximately 7500 stations with more than 30 years of record, and short duration (down to 6 minutes) data from approximately 600 rain gauges (pluviographs) with more than 6 years of record. The rainfall data was supplied by Australian Bureau of Meteorology (BOM), which operates and maintains such stations throughout Australia. With time, BOM has extended its network of meteorological stations and has improved the rainfall database significantly. Recently, the new ARR2016 is released, which is an updated version of ARR using daily rainfall data from 8074 stations and continuous (pluviographs) data from 2280 stations (BOM 2017); rainfall records of more than 30 years and 8 years are used, respectively for daily and continuous data. The inclusion of additional continuous rainfall data in ARR2016 has improved the accuracy and representativeness of the newly estimated IFDs, significantly. In addition, new probability distribution is fitted to the annual maximum series of rainfall data for at site and

regional frequency analyses, which is regarded more appropriate distribution across Australia (Green et al 2012). In this paper, we review the guidelines for both ARR1987 and ARR2016 for design rainfall estimation and evaluate changes in design rainfall for selected stations in 8 major cities in Australia including Adelaide, Brisbane, Canberra, Darwin, Hobart, Melbourne, Sydney and Perth.

2. SALIENT FEATURES OF ARR1987

ARR1987 was developed with the aim of providing spatially and temporally consistent IFD design rainfall information in a comprehensive, yet relatively simple way. Design rainfall can be estimated following one of the three procedures, namely (IEAust 2001):

- algebraic equation procedure;
- graphical procedure and
- computerised technique.

Algebraic equation and graphical procedures are provided in hard copies published by IEAust (1987). These two methods involve 8 steps to obtain complete IFD information for any location using input data from detailed *rainfall intensity maps*. The computerised method, termed as CDIRS (Computerised Design IFD Rainfall System) is available from the BOM website (BOM 2017). CDIRS provides a full set of IFD curves derived automatically for any given latitude and longitude. The system has a large data base covering Australia at a latitude and longitude resolution of 0.025 degrees for high rainfall gradient areas and 0.05 degrees for elsewhere. One of the advantages of using CDIRS is that the user does not need to read data from design rainfall maps; maps were digitised and gridded using a Laplacian smoothing spline technique, where grid point values were calculated iteratively.

In ARR1987, IFD curves consistent to topographical, spatial and temporal scales were obtained using *generalised technique*. The generalised technique is based on charts of rainfall intensity for six combinations of durations and Average Recurrence Interval (ARI). The combinations are 1 hour, 12 hours, 72 hours and ARIs of 2 years and 50 years; the combination commonly known as *master charts*. For any given location, the six specific intensities are obtained from the master charts along with a regionalised skewness, and interpolation and extrapolation methods are used to obtain a full set of IFD curves. There are thirteen standard durations have been selected using interpolation and extrapolation methods, namely, 5, 6, 10, 20, 30 minutes and 1, 2, 3, 6, 12, 24, 48 and 72 hours, and seven standard ARIs, namely, 1, 2, 5, 10, 20, 50 and 100 years.

Different probability distributions can be fitted to annual maximum rainfall series data including normal, log-normal, log-log normal, gamma, Gumbel, Pearson Type III, log-Pearson Type III and different extreme value distributions. However, log-Pearson Type III (LPIII) by method of moments was found to be more appropriate for most of the regions in Australia and availed in ARR1987 (IEAust 1987). Briefly, the random variable *x* has a log-Pearson Type III distribution if $y = log_a x$ has a Pearson Type III distribution. In general case, LPIII distribution is derived from the Pearson Type III distribution by change of variable on the logarithm of this variable (Bolgov and Korobkina 2013). Normally, the commonly used logarithm is the natural logarithm with base *e*. The density function of the LPIII can be written as:

$$f(x; a, b, m) = \frac{|a|}{\Gamma(b)x}[a(\ln x - m)]^{b-1} \exp[-a(\ln x - m)] \quad (1)$$

Where, a, b, m are scale, shape and location parameters of the distribution, respectively and $\Gamma(b)$ is the gamma function. Relationships between parameters of distribution and moments of logarithms of the observed series can be written as:

$$a = \frac{2}{C_{S_{ln}}\sigma_{ln}},\ b = \frac{4}{C_{S_{ln}}^2},\ m = M_{ln} - \frac{2\sigma_{ln}}{C_{S_{ln}}} \quad (2)$$

Where, M_{ln}, σ_{ln}, $C_{S_{ln}}$ are mean, mean square deviation and coefficient of asymmetry of series of natural logarithms of the observed data, respectively.

For the determination of LPIII distribution parameters a, b, m, the methods of moments can be applied to the observed data, as expressed below:

$$b = \frac{\ln \beta_2 - 2\ln \beta_1}{2\ln(1-{}^{1}/_{a}) - \ln(1-{}^{2}/_{a})}, \qquad m = \ln\ \beta_1 + b\ln\left(1 - {}^{1}/_{a}\right),$$

$$\Phi(a) = \frac{2\ln(1-{}^{1}/_{a}) - \ln(1-{}^{2}/_{a})}{3\ln(1-{}^{1}/_{a}) - \ln(1-{}^{3}/_{a})} - \frac{\ln \mu_2 - 2\ln \mu_1}{\ln \mu_3 - 3\ln \mu_1} = 0 \qquad (3)$$

Where, β_1, β_2, β_3 are sample moments about the origin and μ_1, μ_2, μ_3 are moments about the mean.

The LPIII distribution is mainly applicable to the south coast, the south-east high topographic areas, and the north-west and south-west regions of Australia. For determining short duration design values (less than one hour), Principal Component Analysis (PCA) of pluviometers data was found to be the most appropriate method and hence was adopted in ARR1987. Details of PCA method can be found in Haque et al (2013).

3. SALIENT FEATURES OF ARR2016

In ARR2016, the commonly used design rainfall frequency term, ARI is omitted; instead, Annual Exceedance Probability (AEP) has been used. There is a misconception about ARI that ARI of 100 years (for example) means that the event will occur once every 100 years. Rather the reality is, for each and every year there is a 1% chance, more elaborately, a *1 in 100 chance* that the event will be equalled or exceeded once or more than one time. In contrast, the AEP is the probability of a particular rainfall amount for a specified duration being equalled or exceeded in any 1 year period. The use of AEP to describe the chance of a particular rainfall is preferred as it conveys the probability or chance that exists for each year (BOM 2017).

Design rainfalls in ARR2016 are proposed in three sets of frequencies (BOM 2015):

- Very frequent,
- Frequent and infrequent, and
- Rare.

The *very frequent* (or sub-annual) design rainfalls are proposed for the estimation of small flood events, water sensitive urban design and some stormwater design tasks such as gutters. This ranges from 2 Exceedances per Year (EY) to 12 EY. Table 1 shows a comparison between the new EY terminology and its equivalency to the old terminologies.

Table 1. Very frequent design rainfalls proposed in ARR2016 (Green et al 2014)

EY (exceedance per year)	AEP (Annual Exceedance Probability (%)	ARI (Average Recurrence Interval) (Months)
12 EY	99.99	1 month
6 EY	99.75	2 months
4 EY	98.17	3 months
3 EY	95.02	4 months
2 EY	86.47	6 months
1 EY	63.21	12 months

The *frequent and infrequent* design rainfalls cover the probabilities of 1 EY, 50% AEP (2 years ARI), 20% AEP (5 years ARI), 10% AEP (10 years ARI), 5% AEP (20 years ARI), 2% AEP (50 years ARI) and 1% AEP (100 years ARI). This range of design rainfall probabilities commensurate to that proposed as ARI in ARR1987. The *rare* design rainfalls have probabilities less than 1% AEP (between 1 in 100 and 1 in 2000). This category of design rainfalls is used to design large bridges and for the assessment of adequacy of spillway of existing dam and other important infrastructure.

The at-site frequency analysis of annual maximum rainfall data in ARR2016 is conducted by Generalised Extreme Value (GEV) distribution fitted by L-moments instead of LPIII as done in ARR1987. GEV distribution is a continuous probability distribution that combines Gumbel, Frechet and Weibull distributions (Millington et al 2011). Like LPIII distribution, GEV also uses 3 parameters namely, location, scale and shape. While location parameter describes the shift in each direction on the horizontal axis, the scale parameter describes how spread out the distribution is, and the shape parameter determines the shape of the distribution and governs the tail of the distribution. The CDF and PDF of this distribution can be written as below (Hosking and Wallis 1997):

$$F(x)exp\left[-\left(1-\frac{\kappa(x-\xi}{\alpha}\right)^{1/k}\right]$$

$$f(x)\alpha^{-1}exp[-1(1-\kappa)y-\exp(-y)]\text{, where } y-\kappa^{-1}\log\left[1-\frac{\kappa(x-\xi}{\alpha}\right], when\ \kappa\neq 0 \qquad (4)$$

Where ξ is the location parameter, α is the scale parameter and κ is the shape parameter.

As stated in Section 2, for the determination of daily to sub-daily rainfall frequency in ARR1987, PCA followed by regression was used; however, one drawback of this approach is its inability to account for variation in record lengths from site to site and inter-station correlation (Green et al 2012). In ARR2016, sub-daily rainfalls are derived using Bayesian Generalised Least Squares Regression (BGLSR) (Haddad et al 2011; Haddad and Rahman 2014). This approach is found suitable as it accounts for possible cross-validation and can justify sampling uncertainty (by separating the sampling and statistical modelling errors) and inter-site dependence (Green et al 2012).

Another difference between ARR1987 and ARR2016 is that the gridding system in ARR2016 implemented by software package ANUSPLIN (Hutchinson 2007 reported in Green et al 2012) as compared to manual drawing of the isohyets in ARR1987. GEV parameters have been gridded in ANUSPLIN, which provides more flexibility in the choice of extracting design rainfall (Green et al 2012).

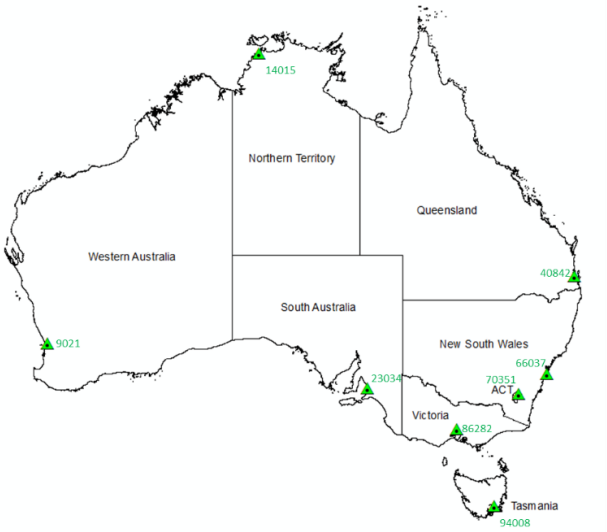
4. COMPARISON OF DESIGN RAINFALL ESTIMATION USING ARR1987 AND ARR2016

To see the impact of changes implemented in ARR2016 discussed above, rainfall gauging stations situated in airports of eight major cities across Australia were selected. Details and geographical locations of the stations are shown in Figure 1. It is assumed that selected stations are representative of geographical variations of a large country like Australia, to some extent. IFD curves were generated by computerised technique of ARR1987 and ARR2016 using online tool provided by BOM (BOM 2017). Three ARIs of 2 years, 50 years and 100 years, and corresponding AEPs of 50%, 2% and 1% were used. Three durations of 1hr, 12hr and 72hr were used for each combination resulting in a total of 9 combinations of IFD for each station. It should be noted that the selected combinations of durations and frequencies represent the six master charts as reported in ARR1987 (except the 100 year ARI or 1% AEP).

The relative differences in design rainfall between the new ARR2016 and the ARR1987 IFDs have been calculated by:

$$\%\ difference = \frac{(ARR2016\ IFD - ARR87\ IFD)*100}{ARR87\ IFD} \quad (5)$$

Percent changes in design rainfall using ARR2016 compared to ARR1987 for eight stations are shown in Figure 2 for 1hr, 12hr and 72hr durations.



Station number	Station Name	Coordinates
66037	SYDNEY AIRPORT AMO	-33.950, 151.170
86282	MELBOURNE AIRPORT	-37.670, 144.830
40842	BRISBANE AERO	-27.390, 153.130
9021	PERTH AIRPORT	-31.930, 115.980
23034	ADELAIDE AIRPORT	-34.950, 138.520
70351	CANBERRA AIRPORT	-35.310, 149.200
14015	DARWIN AIRPORT	-12.420, 130.890
94008	HOBART AIRPORT	-42.830, 147.500

Figure 1: Geographical distribution of stations used in the study

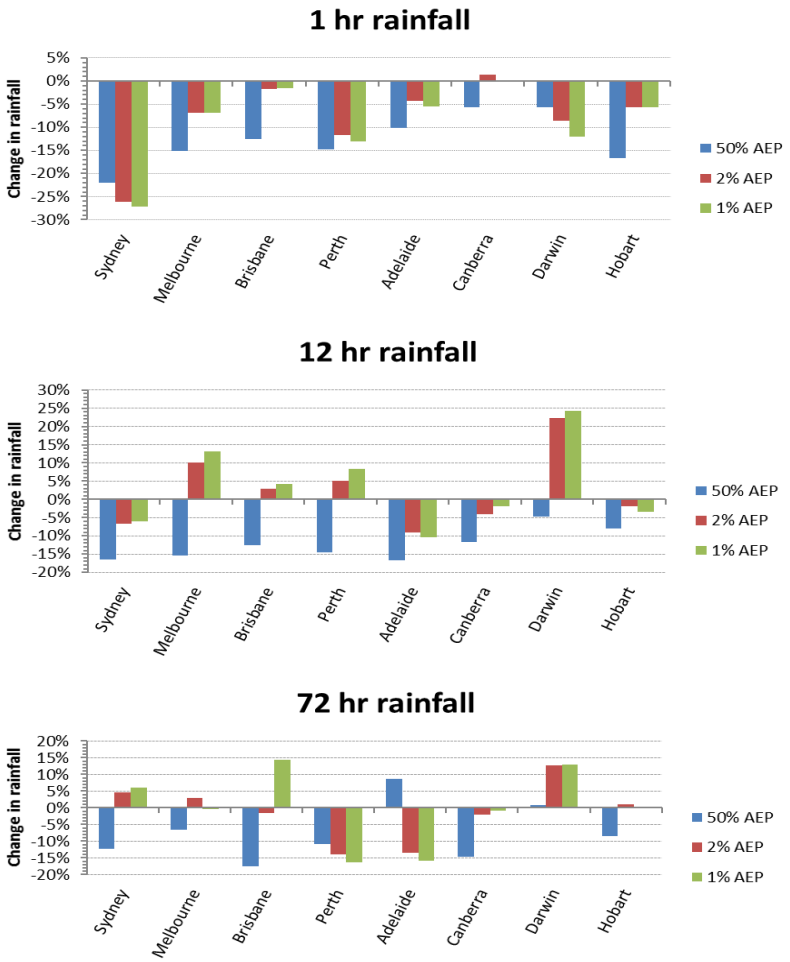


Figure 2: Percent changes in design rainfall using ARR2016 compared to ARR1987

As shown in Figure 2, for 1 hr duration, for most of the stations, percent changes in design rainfall are negative. This means the new method for the estimation of 1hr design rainfall gives smaller values compared to ARR1987. For 12hr duration of rainfall, stations in Melbourne, Brisbane, Perth and Darwin were found to have an increase in design rainfall values in the range of 3% to 22% for 2% AEP, and 4% to 24% for 1% AEP; for both these AEPs, Darwin had the highest increase (about 25%). For 12 hr duration, Sydney was found to have a decrease in design rainfall for 50%, 2% and 1% AEPs. For 72hr duration, Perth was found to have a decrease in design rainfall for 50%, 2% and 1% AEPs, Sydney had a decrease (by about 12%) for 50% AEP, but about 5% increase for AEPs of 2% and 1%, and Brisbane had the highest decrease (by 17%) for 50% AEP.

5. CONCLUSIONS

The paper presents a short review on design rainfall estimation methods in ARR1987 and ARR2016. Eight different locations are selected across Australia and design rainfall values are compared between ARR1987 and ARR2016 methods for three durations (1hr, 12hr and 72hr) and three AEPs (50%, 2% and 1%). It has been found that ARR2016 design rainfalls vary by about -25% to +25% compared with ARR1987 values. For 1hr duration, Sydney shows the highest change (by about by -25%), for 12hr duration, Darwin shows the highest change (by about + 25%), and for 72hr duration, Brisbane shows the highest change (by about -17%).

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Epidemiological Issues of Hazardous Medical Waste Management from Private Healthcare Facilities- Case Study from Dhaka City of Bangladesh

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Abstract

Medical waste constitutes a special category of hazardous waste because they contain potentially harmful infectious materials and it needs special care for its management. Most of the hospitals in Bangladesh are built up here and there without any environmental study and provision for its waste management. The team has conducted a research study on the location of selected private health care facilities in Dhaka City with their waste management provisions. Medical waste generation data for the selected healthcare facilities were collected from PRISM Bangladesh. Also, questionnaire survey was performed among the selected hospital's employees and residents living near the Matuail landfill and dumping site. This research has found out that most of these healthcare facilities lack proper provisions for hazardous medical waste management – such as inappropriate handling and disposal of medical waste. Such actions poses health risks to health workers who may be directly exposed and to people near health facilities, particularly children and scavengers who may become exposed to infectious wastes and a higher risk of diseases like hepatitis, HIV/AIDS and various types of skin diseases. The study thus focuses on how to use relevant chemicals, tools, techniques, materials and machineries etc. and what's their potential threats to the stakeholders with their epidemiological issues like mortality and other such sufferings. Also the study has generated hazardous medical waste generation rates on $kg^{-1}bed^{-1}day^{-1}$basis for the private healthcare sector of Dhaka city. Finally the study proposes some recommendations and sustainable management approach towards medical waste management in light of socio-economic condition of Bangladesh.

Keywords: Epidemiological issues, Infectious materials, Sustainable Management Approach, Hazardous waste generation rate.

1. INTRODUCTION

Unfortunately, in most developing countries, proper waste management system does not exist. Currently, for risk or hazardous waste management on small scale, some methods are in use such as on-site incineration, steam disinfection and autoclaving. The countries where the incineration practice is common are Brazil, Argentina, India, Pakistan, Bangladesh and Peru. Especially in developing and poorer countries, the performance of incinerators is very bad and mostly nonoperational. Normally, medical institutions focus on the installation of waste disposal technologies such as incineration but remain unable to enforce the waste management practices within the hospital. Globally, there are some techniques which are used in different countries such as Incineration, microwave disinfection, autoclave disinfection and chemical/mechanical disinfection. Other methods are also used to dispose of medical waste, e.g. burial, burning, dumping, removal of municipal bins and selling etc. (Akter et al, 2000).

Medical waste may contain highly toxic chemicals and can present a mechanism for transmission of diseases (Silva et al 2005). The growth of the medical sector around the world over the last decade (WHO, 2002) combined with an increase in the use of disposable medical products has contributed to the large amount of medical waste being generated (Silva et al, 2005).

Bangladesh is a developing country with a rapidly growing urban population, extensive health problems, low educational status and environmental pollution (Kabir et al, 2003). In Bangladesh, hospital waste is collected by sweepers, and then transported to the city's open waste dumping sites. In Government hospitals, there are no special techniques for waste handling, and due to lack of awareness, hospital staff normally treat all solid wastes carelessly. Municipal transport is used to collect and dispose of the hospital waste in any open dumping site alongside city garbage (Ahmed et al, 1997). Normally, hospital waste and domestic waste are mixed together on the roadside and then disposed of. Sometimes, to get rid of this waste, it is simply buried without complying with any rules and regulations. The bitter reality behind such actions is that although the necessary technologies and equipment are available to ensure proper hospital waste management within the country, the unawareness among staff and local people regarding effective disposal techniques and policies hinders their implementation. However, Bangladesh is at a crucial stage where there is an urgent need to create awareness about the hazardous impacts of waste on human health as well as the environment. Moreover, strict measures are needed to implement hospital waste management technologies.

2. DEFINING MEDICAL WASTE

There are different concepts of waste management. However, in this research the concepts proposed by WHO (2014) such as classification of medical waste, the nature of medical waste, and hospital waste management techniques and methods are described.

World Health Organization (WHO, 2014) defines medical waste as Waste produced by health care activities including a wide range of materials, from used syringes and needles to soiled dressings, diagnostic samples, body parts, pharmaceuticals, chemicals, blood, medical devices and radioactive materials. All types of wastes which are produced by hospitals, doctor's clinics or offices, medical and research departments are considered as medical wastes (Srishti et al, 1998).

World Health Organization (WHO, 2014) has classified medical wastes into eight different categories, which are as follows;

1) Pathological waste
2) Chemical waste
3) Sharps
4) Pharmaceutical waste
5) Pressurized containers
6) Radioactive waste
7) Infectious waste and potentially infectious waste
8) General waste

3. HOSPITAL WASTE MANAGEMENT SYSTEMS IN DIFFERENT DEVELOPING COUNTRIES

In developing countries, some processes e.g. on-site incineration, steam disinfection, microwave disinfection, autoclave disinfection, and mechanical/chemical disinfection are currently in use for managing harmful waste but on a very small scale. Normally, incineration practices are found in Argentina, Brazil, Peru, Pakistan, India and Bangladesh. In developing countries, hospital waste incinerators operate under sub-optimal conditions and mostly incinerators are non-functional due to different reasons. For example, hospital's administrations focus mostly on installing incinerators but they do not pay attention to its functioning and maintenance.

A study done by Subramani et al (2014) showed that in India, 420461 kg of biomedical waste is generated per day in which only 240682 kg of waste per day is treated. Asante et al (2014) also reported that, in Ghana, 6851 beds are available for patients and each bed is generating 1.2 kg of waste per day. Moreover, around 83% of the selected health care institution (total were 120 Health care institutions) in Ghana did not segregate their waste of which only 17% were segregated. A study done by Joshi et al (2013) indicated that in Nepal, around 1.7kg/person/day hospital waste was produced whereas 0.48kg/person/day health care risk waste was generated. This study also illustrated that in Nepal, mostly government and private hospitals do not systematically segregate the waste at the point of waste generation. Moreover, the guidelines of color coding and labeling of waste containers are not strictly followed by the hospitals. Normally, hospital waste is being collected in a big container then mixed with municipal waste, as a result the entire waste become hazardous and pollute the environment. Improper management of health care waste can badly affect the health of the hospital's staff, patients, waste workers and general public.

4. METHODOLOGY

At present, PRISM Bangladesh, a renowned NGO based in Dhaka city, has been operating at a large scale to collect hospital waste from healthcare facilities and to subsequently processing and treatment of these hazardous waste. Under their medical waste management program, currently 602 private healthcare facilities are registered. From these 602 HCF's, we have randomly selected 50 HCF's while maintaining the percentage of different types of HCF's. The overall categories of HCF's was 5. The duration of time for which the data was collected was April-May-June of 2015. We also attempted to perform case study analysis in 3 randomly selected hospitals. Personal visits were made to hospitals and waste disposal site to record the actual conditions.

5. CALCULATION OF WASTE GENERATION

For the purpose of estimation of hazardous waste generation from private HCF's, we modified the formula used by Patwari et al (2009). The final steps of the calculation are below:

$$\hat{y} = T_{hb} W_{hb} + T_{cb} W_{cb} + T_{dt} W_{dt} \quad (1)$$

- $\hat{y} =$ Total hazardous waste generated per day
- $T_{hb} =$ Total no. of hospital beds in Dhaka city (13650)
- $W_{hb} =$ Avg. hazardous waste per hospital bed per day in sampled hospitals (0.15 Kg bed^{-1} day^{-1})
- $T_{cb} =$ Total no. of clinic beds in Dhaka city (6102)
- $W_{cb} =$ Avg. hazardous waste per clinic bed per day in sampled hospitals (0.11 Kg bed^{-1} day^{-1})
- $T_{dt} =$ Total no. of diagnostic center test per day in Dhaka city (15100)
- $W_{dt} =$ Avg. waste per diagnostic test in sampled diagnostic centers per day (0.27 Kg test^{-1} day^{-1})
- $\hat{y} = (13650 * 0.15) + (6102 * 0.11) + (15100 * 0.27)$
  $= 6795.72\ kg\ day^{-1}$

Total solid waste generated in Dhaka city per day = *5000* metric tons
= *5000000 kgs*

So, hazardous healthcare waste generation percentage = *(6796 / 5000000) * 100*
= *0.14 %*

Total healthcare waste generated in Dhaka city per day = *42* metric tons
= *42000 kgs*

Hazardous healthcare waste percentage of the total healthcare waste = *(6796 / 42000) * 100*

=16.18 %

Our result shows that hazardous healthcare waste generation rates of private HCF's is 0.17 Kg bed^{-1} day^{-1}. This is lower than other research results. Because in Bangladesh, economically solvent people often visit private HCF's for minor health issues which results in lower waste generation. Also, our result for hazardous healthcare waste percentage of total healthcare waste $(16.18\ \%)$ is within WHO specified range$(10\% - 25\%)$.

Table 2: Comparison of healthcare waste generation rate of Bangladesh with other countries

Countries	Hazardous Health-Care Waste Generation Rate (Kg bed^{-1} day^{-1})
Bangladesh, Patwary et al (2009)	0.28
Jordan, Bdour et al (2007)	0.07
Norway, Bdour et al (2007)	0.68
UK, Bdour et al (2007)	0.57
Greece, Komilis et al (2011)	0.24
Brazil, Silva et al (2005)	0.57
Taiwan, Cheng et al (2009)	0.19
Iran, Mosaferi et al (2009)	0.30

6. MEDICAL WASTE TREATMENT AND DISPOSAL METHODS FOLLOWED IN DHAKA CITY

The collected information through interviews with the hospital staff and visual examination by the researcher demonstrated that in private healthcare facilities SWM (Solid Waste Management) system comprises of separate waste storage area at dedicated places within hospital vicinity. Sanitation staff including sweepers and waste collectors clean the hospital's individual area, collect the hospital waste and dispose of this waste at identical garbage heaps. PRISM Bangladesh collect this waste in covered vans and transport the waste to their waste disposal and treatment site in Gazipur. In these private HCF's, color coding scheme for the collection of plastic, paper, glass and other infectious waste was observed by the researcher.

For effective onsite segregation of medical waste, Kumar et al (2010) suggested a color coding scheme which is given below;

- Green: For organic waste
- Red: For risk waste with sharps
- Blue: For risk waste without sharps
- Black: For non-risk/General waste
- Yellow: For radioactive waste

6.1 Storage, Transportation and Final Disposal of Hospital Waste

In private healthcare facilities respondents told to the researcher that after collection of waste from each ward, for temporary storage we have separate container and trolley outside the hospital building, sweepers throw-off all of the hospital waste in "Blue color" container and "Yellow color PRISM

trolley". Respondents also told that, every early morning PRISM Bangladesh vans collect all of these wastes from these containers from the private hospitals, then transport this waste to PRISM Bangladesh's waste disposal and treatment site.

6.2 Burning of Hospital waste

In private hospitals researcher also observed that behind the hospital buildings, big holes were found. On asking from sweepers about the reality of these holes, they replied that, on the instructions of the hospital's administration, they filled these holes with infectious and hazardous waste which they got rid of by burning. On asking the reason of burning this hazardous waste, respondent replied that hospitals lack onsite incineration plant. Therefore, for hospital waste management there is no other option than burning to get rid of this infectious and hazardous waste.

6.3 Recycling of waste

Researcher came to know that empty glass/plastic bottles, containers and tins were mainly re-used by doctor's assistants and compounders without sterilizing them. However, recyclable products such as glucose bags, urine bags, tins, used syringes, paper, cardboard, plastic bottles and infusion tubes were collected and sorted by the sweepers and scavengers within hospitals premises and outside the hospital boundaries respectively. Sweepers and scavengers perform these duties without realizing the serious health issues such as Hepatitis B, C, HIV/AIDS and many other allergic issues which can be caused by handling and due to contact with these infectious/toxic wastes. On asking about the reason for the collected and sorted recyclable waste, respondents mostly replied that these recyclable products can be easily sold on for good prices at scrape yards.

7. CONCLUSIONS

From this study, the following conclusions can be drawn:

- Our result for hazardous healthcare waste percentage of total healthcare waste $(16.18\ \%)$ is within WHO specified range $(10\% - 25\%)$
- Per day, 6795 kg of hazardous waste generated from private healthcare facilities in Dhaka city which poses significant health risks to the people associated with waste disposal & treatment.
- Absence of government guideline and application of environmental law aided by the people's ignorance and reluctance to the conservation of environment along with economical insolvency has acted as the major catalyst behind the failure of waste management system of Dhaka city. Awareness and education on medical waste issues should be raised among the general people. The government has the responsibilities of formulating appropriate policy that needs to be followed by all the HCF's in Bangladesh.
- Participation on this aspect by NGOs like PRISM Bangladesh should be encouraged by the government. Also, through training and guidance supported by DGHS and NGOs, the application of guidelines and procedures associated with effective enactment of the law through DoE needs to be promoted.

8. IMPROVEMENTS RECOMMENDED FOR PRESENT HEALTHCARE WASTE MANAGEMENT FRAMEWORK IN BANGLADESH

- Training of personnel associated with healthcare waste collection, transportation & disposal.
- Formulation of national framework by government.
- Use of methods like 'Composting' & 'Vermiculture' for disposal of biological waste.

- Implementation of Energy from Waste (EfW) technology can generate electricity at low cost while providing zero discharge of waste.
- To increase mass awareness about the risk of exposure to healthcare waste**.**

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New Competency Framework for Fresh Engineering Graduates in Bangladesh

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Abstract

As a growing nation Bangladesh requires to work on restructuring the competency framework at institutional level for the upcoming human resources in technical sector. It is important more than ever because the changing business context and a rising entrepreneurial DNA that prevail in today's employment market- employability of fresh engineering graduates in Bangladesh is not depended on demonstrating technical mastery alone. Along with core applications of different technical branches engineers now-a-days are involved in supply chain, procurement, planning and other managerial roles. That is why the past competencies required to secure a job is proven to be inadequate considering the demands of employers in 21st century. The average lead time to secure a job has increased despite of the fact that many of these fresh graduates are coming out of country's top-most engineering schools and with excellent academic background. The war for talent along with rising complexity of specialist roles across different industries demands prospective job seekers to be capable in terms of both tangible and intangible competencies. In this context it is important that both academic institutions and industries work closely to redefine the required competency framework upon which prospective engineering graduates could be developed. This study aims to narrow the gap by providing a likely competency framework (the required knowledge, skill and attitude levels), which eventually would contribute in fulfilling the rising talent demand in the market.

Keywords: Competency, skills, Bangladesh, engineers, universities.

1. INTRODUCTION

The economy of Bangladesh is growing in an exponential rate along with its equally impressive level of industrialization. The Asian Development Outlook 2016 (ADO 2016) report published by Asian Development Bank confirms the fact that "growth has held up in developing Asia despite a difficult external environment". Bangladesh, being one of the 45 members of the Asian Development Bank is part of this select group of countries labeled as Developing Asia. In addition to the aforesaid positive economic indication, the war for attracting and retaining skilled industrial workforce is also at the rise. Bangladesh Bureau of Statistics (BBS) data shows 2% increase in industrial sector wages (i.e. wages given in production and construction related occupations) in the year 2015-2016 comparing to that of the year 2014-2015. All these evidences theoretically suggest a growing need of technical skills in Bangladesh's expanding economy. Therefore, the demand for engineering graduates should also be rising as more numbers of competent managers are required to lead larger number of blue color workers.

In reality, it has been observed that despite of the increased demand and growing need across different sectors, Recruitment Managers are finding it increasingly difficult to hire engineering graduates with the 'right' competences. A higher influx of engineering graduates still does not aid filling up vacant positions within the desired recruitment lead time. According to Esposito et al (2015), facilities and university must share common objectives in training undergraduate students. In addition, corporate grooming efforts to shape up fresh engineering intake through different development interventions (such as Campus to Corporate programs, customized trainings, and individual coaching) are also being

regularly incurred. In these above circumstances, undertaking an effort to review and understand the newly desired competencies by the employers is a demand of time, which this research aims to fulfill.

2. METHODOLOGY

In order to ensure triangulation of data in this qualitative research, three stratified sample sizes from a larger number of population (N=83) were selected based on the following criteria:

i. Professionals involved in hiring fresh engineering graduates (Sample group n1)
ii. Engineering graduates employed in different capacities but not involved in hiring decisions (Sample group n2)
iii. Academics involved in market research/business studies (Sample group n3).

As main purpose of the research is to understand the desired competencies that employers in Bangladesh seek for in fresh engineering graduates, it is logical to direct significant effort in reaching and interviewing a wider number of industry professionals representing different organizations. Accordingly, forty three personnel (under sample group n1) having different company designations namely Chief Executive Officers, Managing Directors, Recruitment Managers, HR Consultants, Departmental Managers and independent entrepreneurs from different sectors were approached through email out of which twenty responded positively. In-depth interview method was used for data collection purpose using both face to face and telephonic interviews.

In addition, survey was conducted using a predesigned questionnaire for sample groups n2 and n3. The questionnaire contained both open ended and closed questions. Under sample group n2, twenty probable respondents were approached out of which sixteen respondents provided their feedback and four respondents were discarded due to incomplete forms. Under sample group n3, another twenty probable respondents were approached and eighteen responded back by filling up the questionnaire correctly.

The overall response rate out of the population size 63 (N=n1+n2+n3) is thereby N/54 i.e. 86%, which is an excellent response rate for qualitative research such as this one.

3. DEVELOPING THE NEW COMPETENCY FRAMEWORK

Competency encompasses knowledge, skill and attitude on a given ability or subject. Being competent implies having job specific knowledge, appropriate skill set, and the right attitude. Phrasing a competency description is very important because through a single definition all three attributes of knowledge, skill and attitude must be covered. In competency definition the ability to transfer knowledge in to workable action is referred to as skill. A person can be knowledgeable but not necessarily skilled in applying that knowledge. Similarly, a person can be knowledgeable and skilled, but might not have the necessary willingness to put his/her knowledge into work. In order to be competent, one will have to have all three of the above mentioned competency elements present.

3.1. Competency levels

Different levels are observed while designing competency frameworks across industries (and organizations). The levels of a competency framework vary in terms of aptitude, means and gravity. Usually organizations tend to have 3 to 7 levels for each particular competency. For the purpose of adopting a general framework (which to be used and applied for development of fresh engineering graduates across multiple sectors), a five scale rating for each competency would be most ideal

solution. Starting from the beginning, these five levels would be Negative level (Level A), Awareness level (Level B), Competent level (Level C), Expert level (Level D) and Strategic level (Level E).

Negative level implies that an individual possesses zero knowledge, skill level and attitude to adopt a particular capability or perform a specific task/subject related to that competency. The general tagline often used to describe this level of competency is 'does not know, cannot do & does not want to do'. The slightly higher level from this previous one would be Awareness level. In this level it is assumed that the said individual has sufficient idea and knowledge on the given issue/subject, but does not know how to use this knowledge and solve problems by transferring the learning into real time environment (and nor has the willingness to do so). The general tag line often used to describe this competency level is 'Knows, cannot do & does not want to do'. In reality, many of the fresh engineering graduates would demonstrate specific competencies that would belong in either of the two aforementioned levels.

The minimum desired level by employers for any particular competency would be Competent level i.e. Level C. The tagline for this particular level would be 'Knows, can do and does perform'. The later element e.g. 'does perform' is the appropriate behavioral indicator that is most sought by many of the hiring organizations. The later competency levels (Expert and Strategic levels) are going beyond performing. In the Expert level one would not only possess job specific knowledge and the required skill-set to perform the desired task, but would also have the mentality to continually develop oneself and encourage others. The highest level of the competency framework would be Strategic level (Level E) in which the individual would fulfill all the previous traits as discussed above along with enforcement of others' performance and development. Although these two later levels are also often desired by many organizations, they are seldom available readily; and organizations need to invest focused time and resource to develop incumbents to these two levels.

Table 1. Suggested Competency levels

Competency Levels	Corresponding definition
Level A [Not competent]	Does not know, cannot do & does not want to do
Level B [Aware]	Knows, cannot do & does not want to do
Level C [Competent]	Knows, can do and does perform
Level D [Expert]	Knows, can do, does perform + encourages others to learn & perform
Level E [Strategic]	Knows, can do, does perform + <u>ensures</u> others also learn & perform

3.2. Elaborating the new competency framework

Based on the data received from multiple in-depth interviews and survey questionnaire as described in the methodology section, a number of core competencies were identified for fresh engineering graduates as listed below:

1. Problem Solving
2. Financial Accumulation
3. Emotional Intelligence
4. Managing Teams
5. Technical Mastery
6. Adapting the Industry
7. Resource Investigation

General definitions for each of the identified core competencies have also been derived from the interviews with sample group n1 along with combining expert views of the authors. It is also important that apart from identifying the core competencies and defining those, each of these particular set of competencies are also divided in to five competency levels as mentioned in section 4.1. These definitions along with the proposed competency levels are listed below:

4.2.1 Problem Solving

This particular competency deals with the notion of having the right skill and attitude for solving unforeseen problems and accomplishing difficult tasks by putting knowledge in to practice. The approach in problem solving needs to be proactive as well as reactive. Nowadays employers expect fresh engineering graduates to be skilled in identifying probable cause and eradicate problem elements (through innovations) even before downtime of a process/machinery takes place

Table 2. Problem Solving

Competency Level	Competency Element	Requirement(s)
Level A [Not competent]	Knowledge	Possesses no idea in regards to formal problem solving methods and analytical tools
	Skill	Unable to use different analytical tools or problem solving approaches both formally and informally.
	Attitude	Reluctant to use formal problem solving approach and often is the cause for delay in mitigating a given issue.
Level B [Aware]	Knowledge	Familiar with basic data collection tools (e.g. Flow Chart, Activity Diagram) and methods (such as Focused Group Discussion, Brain Storming Activity etc.)
	Skill	Does not demonstrate the ability to formally analyze a problem statement, identification of solution, or documenting and recording it.
	Attitude	Reluctant to use formal problem solving approach and often is the cause for delay in mitigating a given issue.
Level C [Competent]	Knowledge	Knows and understands the applicability of different analytical tools (such as Critical Path Analysis, RCA, SWOT) and associated monitoring tools (WBS, Gnatt Chart etc.)
	Skill	Always uses different analytical tools and approaches to initiate (and capture) innovations in solving an existing or future problem.
	Attitude	Possesses a profound sense of urgency with a can do attitude.
Level D [Expert]	Knowledge	Possesses formal training on problem solving and/or project management
	Skill	Proactively helps others to incorporate problem solving tools and techniques in one's day to day activity.
	Attitude	Actively encourages others to participate in solving a given problem, thus reducing lead time.
Level E [Strategic]	Knowledge	Ensure formal training or certification for other members of the team on problem solving or project management
	Skill	Ensures everyone in the team uses documented analytical tools and formally capitalizes team members' idea to solve problems.
	Attitude	Avoids reinventing the wheel and solves problem way ahead of time.

4.2.2 Financial Accumulation

Muradoglu & Harvey (2012) suggests that interdisciplinary research is becoming more widespread and it is likely that greater collaboration between finance and other disciplines will develop in the future.

Table 3. Financial Accumulation

Competency Level	Competency Element	Requirement(s)
Level A [Not competent]	Knowledge	Possesses no idea of the basic financial terminologies (such as Revenue, Net Profit, ROI, payback period etc.)
	Skill	Does not have the ability to use, interpret and analyze financial information related to the industry/organization
	Attitude	Deliberately avoids numerical values and target setting
Level B [Aware]	Knowledge	Possesses primary idea and application of common financial terminologies and metrics used in business
	Skill	Does not have the ability to use, interpret and analyze financial information related to the industry/organization
	Attitude	Talks about the importance of using numerical values and target setting but never uses numbers accordingly
Level C [Competent]	Knowledge	Completed a college/university level course in accounting or financial management
	Skill	Demonstrates ability by using, interpreting and analyzing different financial terms and metrics in own area of work
	Attitude	Makes habit of using numbers and quantifying business issues on a regular basis
Level D [Expert]	Knowledge	Inform and educate others on the usage of different financial terms and metrics for day to day business operation
	Skill	Demonstrates ability by using, interpreting and analyzing different financial terms and metrics in departmental level (forecasting, budgeting i.e. CAPEX/OPEX etc.)
	Attitude	Encourages others to use numbers and quantify business issues on a regular basis
Level E [Strategic]	Knowledge	Ensures mandatory training and certification on financial management or accounting across the team
	Skill	Demonstrates ability by using, interpreting and analyzing different financial terms and metrics used across the industry (such as industry growth, competitor analysis etc.)
	Attitude	Ensures others also use numerical values and quantify business issues to monitor and track one's own performance

4.2.3 Emotional Intelligence

Another important competency which has been highlighted by employers (sample group n1) and academics (sample group n2) alike during data collection phase is Emotional Intelligence (or EI in short). Being emotionally intelligent implies that a person would effectively control self-emotion and would use both emotion and cognition before deciding the immediate course of action. According to Cherniss (2010), emotional intelligence is positively associated with productivity.

Table 4. Emotional Intelligence

Competency Level	Competency Element	Requirement(s)
Level A [Not competent]	Knowledge	Does not realize the importance of understanding other's emotional state.
	Skill	Does not demonstrate the ability to control & filter own emotion in changing circumstances.
	Attitude	Reluctant in listening to others' views and acting stereotypically.
Level B [Aware]	Knowledge	Aware of the verbal and visual indicators to recognize special situations and other people's emotional needs.
	Skill	Sometimes fails to demonstrate the ability to control & filter own emotions in different circumstances.
	Attitude	Catches others' words but fails to reconnect to the same emotional resonance.
Level C [Competent]	Knowledge	Trained in observing and understanding surrounding environment and human emotions either through a formal certification process (such as NLP) or through self-development initiatives.
	Skill	Demonstrates the ability to control & filter emotions in different circumstances all the time.
	Attitude	Carefully listens to other people's views and interprets correct emotion without interrupting the flow.
Level D [Expert]	Knowledge	Facilitates other's capacity development in terms of emotional intelligence by conducting formal training and coaching sessions.
	Skill	Encourages others to control & filter emotions in changing circumstances from time to time.
	Attitude	Ensures emotional catharsis by helping to release tension through individual discussion.
Level E [Strategic]	Knowledge	Ensures Line Managers and others' involvement through a systematic process to develop EI capacity across the team.
	Skill	Ensures others in the team to control & filter emotions in different circumstances from time to time.
	Attitude	Develops a process to promote active listening by ensuring formal feedback sessions between Line Managers and employees.

4.2.4 Managing Teams

A significant part of this proposed competency also resolves around formation of team and leading it to the mastery of solving conflict situations as part of its own internal mechanism, thus making team leader's job easier through empowerment and delegation.

Table 5. Managing Teams

Competency Level	Competency Element	Requirement(s)
Level A [Not competent]	Knowledge	Has not been exposed to different team building tools and techniques along with no practical training on leadership
	Skill	Does not demonstrate the ability to handle team dynamism at all.

	Attitude	Negligent and oblivious of surrounding environment and team members' preferences
Level B [Aware]	Knowledge	Possesses some idea and personal experience in forming and handling team(s) but no formal training on teambuilding and leadership
	Skill	Fulfills the role as a team player rather than being the team leader
	Attitude	Tend to be negligent and oblivious of surrounding environment and team members' preferences
Level C [Competent]	Knowledge	Possesses a structured and well-designed formal training on handling team dynamisms and leadership
	Skill	Demonstrates effective handling of team dynamism across different levels of teams by defusing conflict situations
	Attitude	Fully aware and observant of surrounding environment and take mental note of others' behavior in different situations
Level D [Expert]	Knowledge	Shares experience and enables learning to fellow colleagues on building and leading teams
	Skill	Demonstrates proactive handling of team dynamism and eliminates any chance which could instigate a conflict situation
	Attitude	Encourages fellow team members to stay aware and observant of surrounding environment and to act as a single team
Level E [Strategic]	Knowledge	Ensures delivery of formal training/mentoring session to fellow team members on building & leading teams
	Skill	Builds a culture and practice where fellow team members are meant to resolve issues in a congenial way on their own
	Attitude	Ensures empowerment and ownership across the team

4.2.5 Technical Mastery

It is generally assumed that employers (in this case sample group n1) would be in the forefront to demand technical mastery as one of the core competencies for fresh engineering graduates. Interestingly, traits or need for this particular competency were widely observed while collecting and interpreting data from sample group n2 and n3! Technical Mastery implies that a candidate is knowledgeable in his/her own area of study and also has the ability to use the learning to accomplish given tasks. Continuous learning and excellence in capacity development is also another important part of this competency.

Table 6. Technical Mastery

Competency Level	Competency Element	Requirement(s)
Level A [Not competent]	Knowledge	No basic idea or understanding of own area of study/subject(s)
	Skill	Does not demonstrate the capability to put theories into practices
	Attitude	No willingness to learn or experiment.
Level B [Aware]	Knowledge	Can define the common terminologies, theories and equations but does not know the practical usability
	Skill	Does not demonstrate the capability to put theories into practices.
	Attitude	Tend to learn and experiment while under pressure or in a

		supervised environment.
Level C [Competent]	Knowledge	Knows own area of subjects well and its practicability or current usage across different industries
	Skill	Demonstrates the capability to put theories into practices
	Attitude	Continuously learning and often experimenting with different ideas.
Level D [Expert]	Knowledge	Teaches others. Pursues professional certifications relevant to own area of work/study and encourages others in doing so.
	Skill	Regularly encourages others to put theories into practices
	Attitude	Instigate others by asking thought provoking questions.
Level E [Strategic]	Knowledge	Formally identifies the development needs, arrange certification courses and administer assessments as part of other's development process
	Skill	Ensures fellow team members putting theories into practices.
	Attitude	Actively coaches and mentors fellow colleagues by embracing them as one team.

4.2.6 Adapting the Industry

One of the most crucial competency for any fresh engineering graduate as proposed in the new framework is Adapting the Industry.

Table 7. Adapting with Industry

Competency Level	Competency Element	Requirement(s)
Level A [Not competent]	Knowledge	Possesses no idea about recent industry trends and customer demands.
	Skill	Does not demonstrate the ability to be flexible and agile in particular way of working or thinking even if there is a need
	Attitude	Stereotyped and holds an obnoxious attitude towards job sector
Level B [Aware]	Knowledge	Aware about recent industry trends but fails to see the implication on own area of work or subject
	Skill	Takes significant amount of time or goes through difficulty to change existing way of working even if there is a need.
	Attitude	Demonstrates the tendency to be idealistic and often compares industries in terms of working condition or values
Level C [Competent]	Knowledge	Stays one step ahead from others in regards to industry update and knows how new trends would shape work.
	Skill	Adaptive and changes own way of working to meet changing customer requirements.
	Attitude	Receptive. Accepts reality in the way it is.
Level D [Expert]	Knowledge	Actively inform others about the trends and technology that are constantly taking place in the industry.
	Skill	Demonstrates agility and encourage others to embrace the same.
	Attitude	Acts as the change catalyst and welcomes diversity.
Level E	Knowledge	Develops a formal mechanism to facilitate knowledge

[Strategic]		sharing between different stakeholders in the industry.
	Skill	Ensure members in the team adapt to changing business demands without interruption productivity.
	Attitude	Focused, calm and appreciative to surrounding work conditions.

4.2.7 Resource Investigation

Resource Investigation encompasses effective communication, presentation and research capabilities, thus helping Management to reach to a balanced decision.

Table 8. Resource Investigation

Competency Level	Competency Element	Requirement(s)
Level A [Not competent]	Knowledge	Possesses no basic idea on research methodology.
	Skill	Does not demonstrate the ability to identify genuine information sources to gather data on a given topic/issue.
	Attitude	Remains introvert even when instigated.
Level B [Aware]	Knowledge	Familiar with certain terms and types of research but cannot distinguish which approach or methodology to use in what context.
	Skill	Compiles information only from a single source, which was directed and bestowed upon.
	Attitude	Occasionally engages in spontaneous and meaningful conversation.
Level C [Competent]	Knowledge	Knows about the difference and applicability of different research types and data collection methods.
	Skill	Collects information from multiple genuine sources through effective communication and investigative research(s) on the given topic/problem.
	Attitude	Inquisitive and proactively develops networks.
Level D [Expert]	Knowledge	Successfully completed a university/college level course on Research Methodology or Report writing.
	Skill	Demonstrates the importance of using multiple sources and compare information before presenting to Management.
	Attitude	Critical in evaluating information.
Level E [Strategic]	Knowledge	Ensures mandatory training/certification of other team members on research methodology or report writing.
	Skill	Develops a system of both way information flow and availability of data across the organization.
	Attitude	Extrovert and always gives attention to details.

4. CONCLUSION

The research results in development of a balanced competency framework consisting of seven separate competencies for fresh engineering graduates in Bangladesh, which coalesces both technical and business competences on a five point rating based framework. Adopting this new competency framework for fresh engineering graduates would enable universities and certification institutions to customize, design and develop course curriculum in alignment with real-time industry requirements. A growing need to modify the existing exam based assessment style of final year engineering graduates

have also been felt and suggestion is being made to shift from content based exams to competency based assessments across Bangladesh. Further research to validate the proposed competency framework with a higher number of population size is strongly recommended.

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Correlation Studies between Consolidation Properties and Some Index Properties for Dhaka-Chittagong Highway Soil

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Abstract

Collection of undisturbed soil samples is a very hard task either due to time constraint or heterogeneity of soil deposits or huge cost in collection process. So in most cases adequate and well-founded data on soil compressibility are not available. In the laboratory, 14 undisturbed soil samples from Dhaka-Chittagong expressway PPP design project were tested and analysed. The moisture contents of these samples were fully intact and collected from various depths. This paper suggests correlations between compression index with liquid limit, in situ water content, in situ void ratio and plasticity index. This paper also suggests correlation between swelling index and plasticity index. Based on consolidation pressure-void ratio equations, analyses were performed and results have been verified using the values of coefficient of determination (R^2). The correlated equations were compared with the equations already developed for various clays. Values of R^2 have been found 0.9805, 0.8997, 0.8864, 0.8619 and 0.8196 for the correlation between C_C-LL, C_C-W (%), C_C-PI, C_C-e_O and C_S-PI respectively.

Keywords: Consolidation, Correlation, Plasticity Index, Void Ratio, Water Content.

1. INTRODUCTION

It is obvious that Karl Terzaghi was the guiding spirit in the development of soil mechanics and geotechnical engineering throughout the world. The publication of Erdbaumechanik auf Bodenphysikalisher Grundlage (1925) by Terzaghi gave birth to a new era in the advancement of soil mechanics. It was Terzaghi who introduced the index tests. Atterberg played an important role by categorizing seven qualitative limits that determines how water content dominates cohesive soil behavior. Finally Casagrande standardize those indices according to engineering properties and they have been used universally essential for any site investigation. Less dependency has been put on the results of index test and evaluation of any soil for an engineer has become easy after the development of testing methods both in laboratory and in the field. According to Atterberg, the moisture content at the point of transition from semisolid to plastic state is the plastic limit and from plastic to liquid state is the liquid limit. These parameters are also known as Atterberg limits which establishes direct approach to make quantitative correlations between index parameters with other soil properties. It helps most importantly in the case of evaluating the quality of soil which will be used as engineering purposes. These procedures are used as a framework against various test results which can be judged for their consistency and reliability. But site investigation, sampling and testing soil are always preferred and best for the results.

Liquid limit is the assessment of the water content at which the soil begins to flow and the plastic limit test is the evaluation of the brittle/ductile transformation of the soil sample (Whyte, 1982; Haigh et al. 2013). The Atterberg limit provides values for plasticity index which is the difference between liquid limit and plastic limit of a soil. This can be empirically correlated against many soil properties and design. According to Casagrande, A-line which classifies soils into clays and silts based on a

correlation between soil type and a combination of liquid limit and plasticity index (Casagrande, 1947). In Bangladesh, the research works on this topic are not to be noted well since there is no established valid correlation between soil parameters has been developed. Based on experiment we suggest various correlations which enable to determine index properties. This will reduce experimental time and cost, most importantly predict the soil type and behavior of soils of Bangladesh.

2. MATERIALS AND METHODS

The soil samples are collected from Dhaka-Chittagong expressway PPP design project. Total 14 samples from various locations of the project were tested experimentally in the laboratory. Although collection of undisturbed soil samples is a very hard task either due to time constraint or heterogeneity of soil deposits or huge cost in collection process but experimented samples were undisturbed, collected from various depths and places in Dhaka-Chittagong highway. The natural moisture content was fully intact. Soil parameters, found from the laboratory tests, are shown in table 1.

In the laboratory, one dimensional consolidation test of soil has been conducted for all the collected soil samples. Applied loads are respectively 250 gm, 500 gm, 1 kg, 2 kg, 4 kg, 8 kg and 16 kg. For unloading, we removed 8 kg from the given loads. The values for compression index (C_C), swelling index (C_S) and pre consolidation pressure (P_C) have been found from the void ratio-pressure curve (e vs. log P). Standard test to determine liquid limit, plastic limit, specific gravity and moisture content has also been done in the laboratory. Soil samples were kept in the desiccator for further tests so that they could not get in touch with air by any chance.

Table 1. Experimental values of the soil parameters used in correlation

Test no.	Specific gravity, G_S	Moisture content, W (%)	In situ void ratio, e_o	Compression index, C_C	Swelling index, C_S	Liquid limit, LL	Plastic limit, PL	Plasticity index, PI
1	2.67	28.954	0.703	0.316	0.021	43.762	24.62	19.142
2	2.6	30.3	0.722	0.324	0.022	36.205	12.059	24.146
3	2.64	28.44	0.668	0.247	0.018	38.163	20.91	17.253
4	2.6	42.355	1.149	0.530	0.026	67.097	27.15	39.947
5	2.68	29.02	0.874	0.279	0.015	41.182	29.1	12.082
6	2.631	29.78	0.760	0.260	0.019	38.547	24.12	14.427
7	2.626	29.95	0.895	0.333	0.022	45.932	18.45	27.482
8	2.602	29.12	0.742	0.306	0.018	47.528	24.344	23.184
9	2.574	34.99	0.817	0.306	0.017	44.708	23.789	20.919
10	2.634	25.2	0.640	0.329	0.02	33.73	11.733	21.997
11	2.673	33.87	0.945	0.346	0.019	48.588	30.926	17.662
12	2.657	45.85	1.195	0.534	0.028	66.612	22.326	44.286
13	2.623	28.96	0.733	0.279	0.017	41.594	22.902	18.692
14	2.654	34.28	0.878	0.242	0.018	37.576	25.19	12.386

3. RESULTS AND DISCUSSION

3.1. Correlation between compression index (C_C) and liquid limit (LL)

In Figure 1, compression index (C_C) vs. liquid limit (LL) graph has been plotted. The value for R^2 has been found 0.9805. It presents that there is strong correlation between C_C and LL. Comparing our equation with Skempton (1944), shows very close relationship. The correlated equation is given below:

$$C_C = 0.01(LL - 13.61) \quad (1)$$

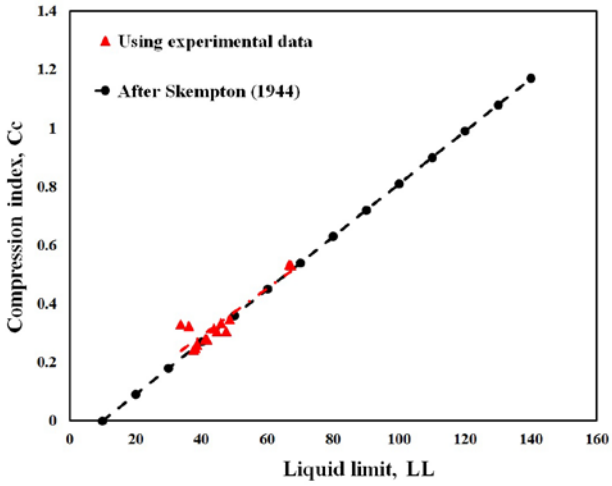

Figure 1. Correlation between C_C and LL

3.2. Correlation between compression index (C_C) and water content (W, %)

In Figure 2, compression index (C_C) vs. water content (W, %) graph has been plotted. The value for R^2 has been found 0.8997, which lies between acceptable ranges and hence it presents a strong correlation. The equation we have got is close to the equation derived for Chicago clays. The correlated equation is given below:

$$C_C = 0.0158W - 0.179 \quad (2)$$

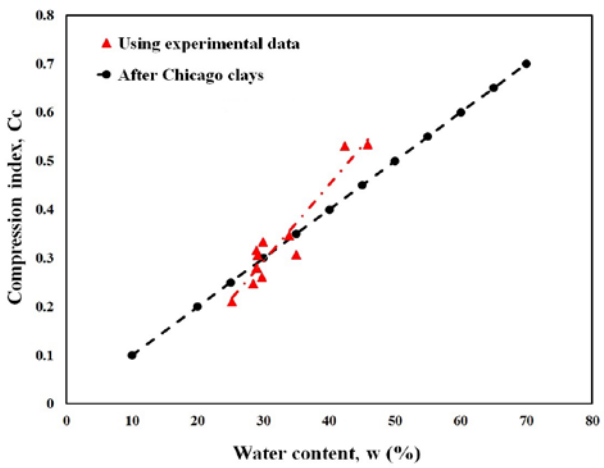

Figure 2. Correlation between C_C and W (%)

3.3. Correlation between compression index (C_C) and void ratio (e_o)

In Figure 3, compression index (C_C) vs. in situ void ratio (e_o) graph has been plotted. The R^2 value has been found 0.8619 which shows moderate relation between the two variables. The correlated equation was compared with the equation given by Nishida (1956). Experimental values have been deviated from Nishida's equation line as he proposed his equation for pure clay soil where our equation is for clay mixed with silt and sand by some amount. The correlated equation is given below:

$$C_C = 0.5562e_o - 0.1453 \quad (3)$$

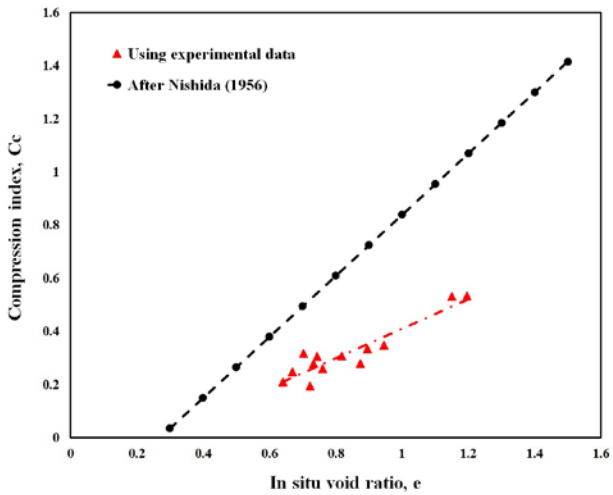

Figure 3. Correlation between C_C and e_o

3.4. Correlation between compression index (C_C) and plasticity index (PI)

In the figure 4, compression index (C_C) vs. plasticity index (PI) graph has been plotted. The value for R^2 has been found 0.8864 which indicates a strong relationship between the variables. The equation also seems to be closely matched with equation given by Kulhawy and Mayne (1990) where they took average specific gravity value 2.7. The correlated equation is given below:

$$C_C = 0.0091PI + 0.128 \quad (4)$$

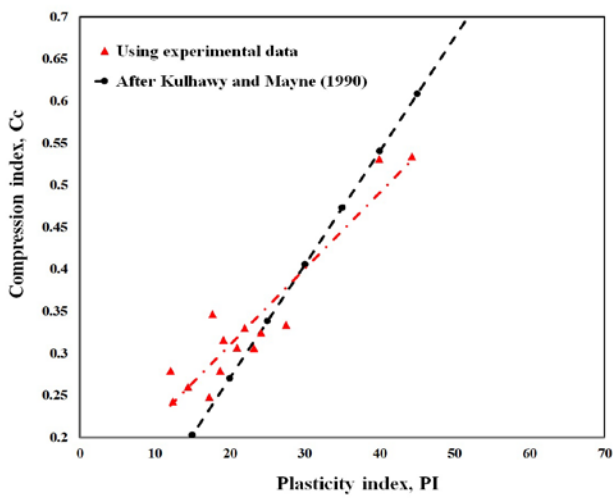

Figure 4. Correlation between C_C and PI

3.5. Correlation between swelling index (C_S) and plasticity index (PI)

In Figure 5, swelling index (C_S) vs. plasticity index (PI) graph has been plotted. The R^2 value has been found 0.8196, shows moderate relation between two variables. The equation derived from experimental values compared with the equation given by Kulhawy and Mayne (1990), shows some deviation as it was given for modified cam clay model but our results were from real field data where soil may be mixed with some impurities. The correlated equation is given below:

$$C_S = 0.0003PI + 0.0125 \quad (5)$$

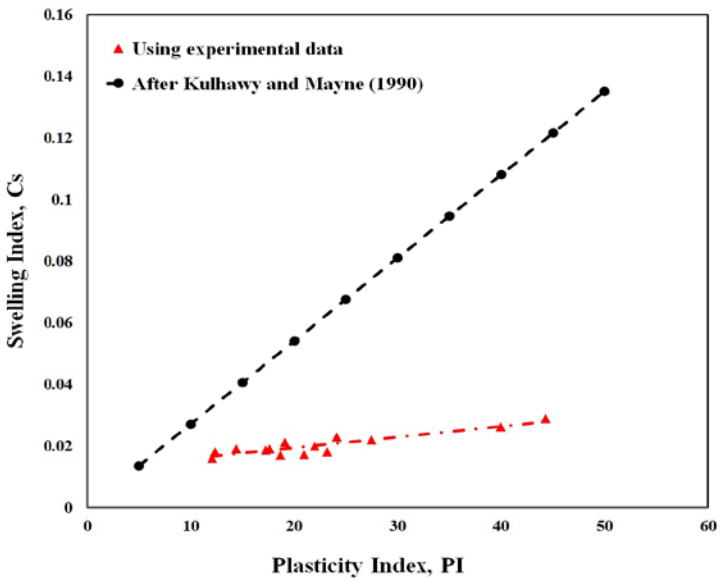

Figure 5. Correlation between C_S and PI

Summary of the results are shown in table 2.

Table 2. Correlated equations with their respective R^2 value

Sl no.	Correlated Equation	R^2	Compared Equation
1.	Cc = 0.01(LL-13.61)	0.9805	Skempton(1994): Cc = 0.009(LL-10)
2.	Cc = 0.0158W-0.179	0.8997	Chicago clay: Cc = 0.01W+3E-16
3.	Cc = 0.5562e_0-0.1453	0.8619	Nishida(1956): Cc = 1.15e_0-0.3105
4.	Cc = 0.009(PI+0.128)	0.8864	Kulhawy and Mayne(1990): Cc = 0.0135PI-2E-16
5.	C_S = 0.0003PI + 0.0125	0.8196	Kulhawy and Mayne(1990): C_S = 0.0027PI – 4E-17

4. CONCLUSION

From this research, the following conclusions can be drawn:

- The compression index (Cc) has been found to be exclusive functions of liquid limit, water content and void ratio, the equation being identical with that derived by Skempton, Nishida and for Chicago clays respectively.

- Equations derived in terms of liquid limit, water content and void ratio formed a basis for prediction of compression index (Cc) without carrying out any consolidation test.
- These findings can be used for predicting parameters without conducting either consolidation or Atterberg limit tests.
- This study shows a good correlation between plasticity index and compression index and plasticity index and swelling index.
- These equations can be used to determine the soil parameters of Bangladesh.

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Experimental Study of Cold-formed Steel Section for Wall Panel

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Abstract

The use of Cold-formed Steel (CFS) has rapidly increased in recent times in building construction as CFS wall frame panels due to their simple forming procedure and easy to assemble Most common CFS sections used in wall panel are C-channel and Z-sections. The behaviour of CFS is different from other steel known as hot-rolled steel. The structural behaviour of CFS wall frame panels subjected to flexural loading are normally characterised by many buckling modes which have not yet been completely understood. Buckling modes under flexural loading are generally either eliminated or delayed to increase the ultimate bending capacity of the members. So, it is very important to understand the behaviour of CFS under bending loading. This research studies the behaviour of cold-formed steel under flexural loading and an extensive four-point bending test (FPBT) are considered. This research mainly investigates the fundamental behaviour of CFS wall frame panels under elastic limits to further understand and improve these members. The research involved experimental study using three FPBT specimens. Two specimens are wall frames and one back-to-back steel stud. All specimens are tested using universal testing machine applying 75% of the design load of the studs. Test results demonstrate that back-to-back CFS studs can be used to overcome the buckling problem for light load bearing wall panels due to their higher rigidity. Gypsum plasterboard included in CFS wall panel also have significant influence on the failure modes, which is understood by testing two wall panels. Tested results are validated by comparing with finite element analysis results.

Keywords: Cold-formed steel (CFS), Wall panel, Gypsum plasterboard, and Four-point bending test.

1. INTRODUCTION

Nowadays, the use of composite Cold-formed Steel (CFS) gypsum wall systems in low to mid-rise buildings and houses is a widespread practice. The flexural strength of composite wall panels is greatly influenced by the strength of the studs. Amongst the numerous advantages over traditional timber wall framing, are their lightweight and simplicity in installation. Even though the concept of CFS in walls began in 1850's the application did not develop until 1940's (Hardwani and Patil, 2012, Yu, 1973). More recently, CFS are becoming increasingly more popular for mid-rise building when compared with timber wall frames, because of strength to weight ratio, consistency and greater span ability.

Typical composite cold-formed steel (CFS) wall panels consist of a stud, top and bottom tracks attached to gypsum plasterboard. Composite CFS wall panels are considered effective due to their advantages. According to (Gunalan, 2011), gypsum plasterboard can withstand lateral load. Therefore, gypsum plasterboard can be used to minimise the lateral buckling behaviours against steel wall panels. The understanding of buckling behaviour of CFS in wall panels is of high importance in regards to exposure to flexural loading. Additionally, CFS buckles before the yielding strength (Schafer, 2008).

Local, lateral and distortional buckling is critical for CFS members, due to having a high width to thickness ratio. Therefore, CFS buckle elastically under low compressive strength causing failures (Hancock, 2001). Research regarding the buckling behaviours of CFS is limited and not yet understood.

This paper investigates the composite behaviour of CFS wall panels under flexural loading. The behaviour of CFS wall panels with or without gypsum plasterboard is also investigated to understand the stud behaviour. The aim and objective of the research is to observe and analyse the buckling failure mode and to understand the composite behaviour of CFS wall panels under flexural loading.

2. EXPERIMENTAL STUDY

2.1.Test specimens

The CFS used for the experiment was taken from the same batch to ensure consistency in results. The studs used for the experimental study is $64 \times 25 \times 0.5$BMT CFS with a length of 1600 mm. In addition, the studs have 3 holes located in the middle of the web having a diameter of 25.4 mm and spacing of 150 mm to the edge and spacing of 600 mm between each hole. The holes in the studs also have a lip that helps it to withstand more stresses.

Clamps are used in the boundary condition to eliminate failure when load being applied. Clamps could also pose unnecessary buckling behaviours such as distortion buckling. Therefore, wooden blocks are needed inside the stud section where clamps are used to eliminate any buckling when load being applied.

Each wall frame consisted of 2 studs, 2 tracks, 4 wooden blocks, 4 clamps and 1 plasterboard or 2 depending on the wall frame. Each stud was connected with track using screw connection on the top flange and bottom flange, example of the connection is shown in Figure 1. Wooden block was also placed at each corner inside the stud section and just under the clamps, as shown in Figure 1. The four clamps were used to prevent unnecessary failures or buckling when placed on the universal testing machine. Plasterboard was connected to the stud's flange using screw connection with spacing of 150mm from the edge and 300mm between each screw connection, detail drawing is shown in.

Figure 1. CFS wall frame with gypsum plasterboard at the compression side

Figure 2. Back to back CFS stud

Back-to-back CFS stud consisted of two studs, four wooden blocks and two clamps. The two studs were connected back-to-back using two parallel screws at the web section with a spacing of 150 mm from the edge and 300 mm between each screw, as shown in Figure 2 . Therefore, ten screws were used for this specimen. The clamps for back-to-back CFS stud are different to wall frame specimens due to the specimen shape. The wooden block was placed inside the stud where the clamp is located. Clamps were used to hold two studs back-to-back, as shown in Figure 2 to prevent unnecessary failures or buckling at the elastic behaviour of the studs. Figure 2 shows the details of wall panel considered gypsum plasterboard in both side.

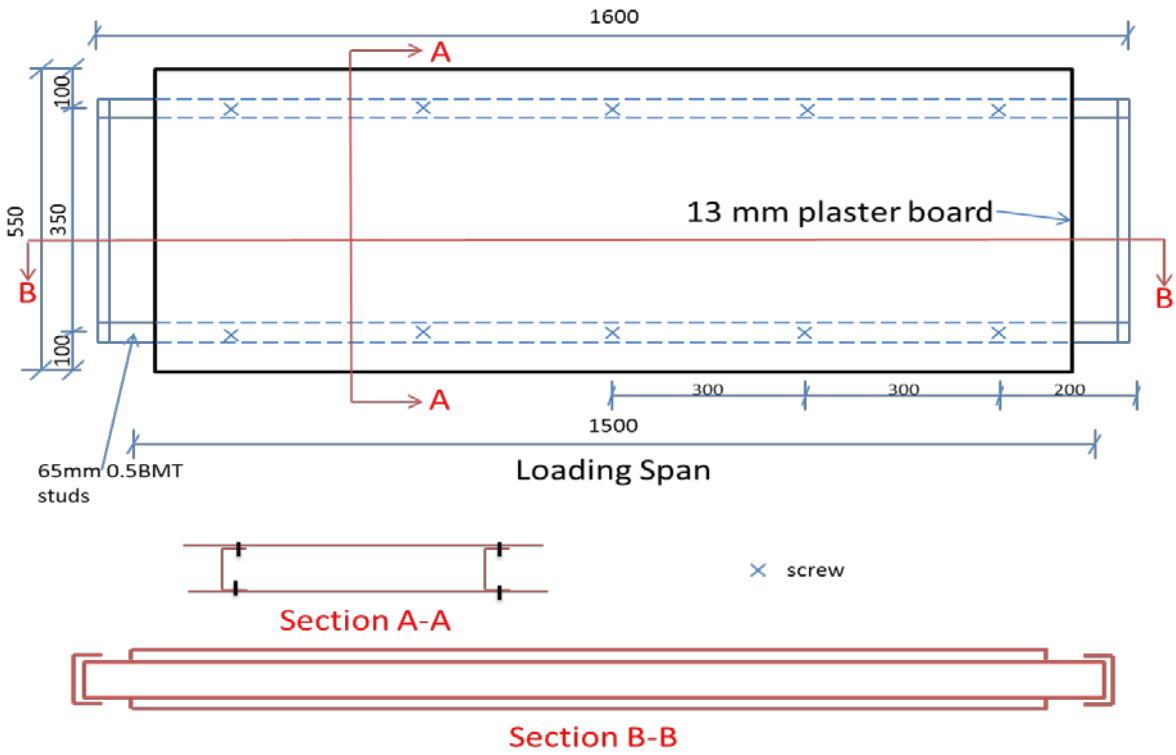

Figure 3. Details of wall panel with gypsum plasterboard in both side

2.2.Test setup and instrumentations

The FPBT method was conducted according to AS/NZS4600. The FPBT method involved applying two point loads on the top of the wall frame on each of the two stud's flanges. The FPBT was formed by having wall frame consisting of two studs connected to top and bottom tracks channel section using screw (connection. The frame placed horizontally on the universal testing machine, where the bottom flanges were placed on two pin supports and the top flanges had two roller loads applied at the top. The threes specimen of FPBT were conducted, including back to back CFS section, wall frame with gypsum plaster board at the bottom and CFS wall panel with double sided gypsum plasterboard.

2.2.1. Loading procedure

The load was applied at a gradual rate of 2.5*kN* which is approximately 75% of the design loading of the stud. Figure is side view of the specimens showing spacing of both the loads and boundary conditions. In order to effectively reduce errors in the results obtained, 10% of the loading was gradually applied in small increments to ensure no slip would occur and specimen would settle. This loading precaution was repeated twice before the section was subjected to the entire 2.5*kN* loading. However, some test specimens were incorrectly tested due to the lack of a 10% precaution load placed on the test specimen. Preliminary results showed that the presence of this precaution load yielded more accurate results. However, if the precautions load not considered the graph will not have straight linear line.

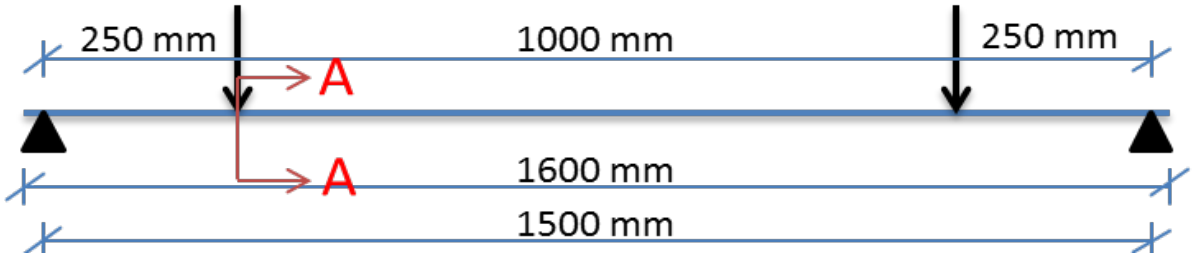

Figure 4. Stud length and loads spacing

2.3.Results and Discussion

Experimental results have elaborated the three specimen results for load versus mid-span, including discussion for each specimen readings.

2.3.1. Comparison of Test Results

The three specimens were tested under FPBT having maximum load 2.5kN. Figure 4 illustrate the linear reading of all three readings. The specimen's behaviour can be distinguished using the mid-span displacement. According to Figure 4, S5 reading obtained the least displacement due to both side plasterboard increasing its rigidity by having a displacement of 4.22 mm; S1 reading obtained 4.82 mm that is 1.65 mm less than S4 that had the maximum displacement of 6.47 mm. All 3 specimens are discussed below where some observation have been recorded for future research.

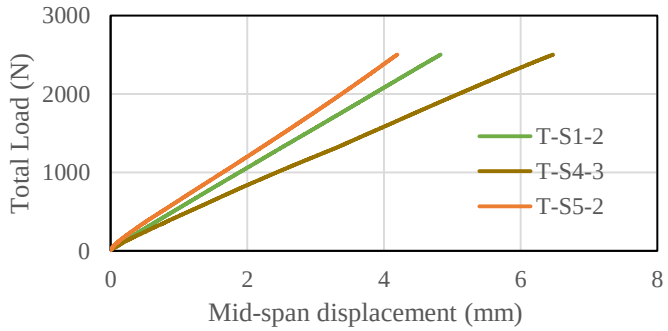

Figure 4. Comparison of the experimental specimens

3. VEREIFICATION OF EXPERIMENTAL AND NUMERICAL RESULTS

The results of the three finite element models are outlined in this section, including comparison with experimental results. The results data have been illustrated via force versus mid-span displacement graphs. Behaviour of each specimen has been discussed.

3.1. Comparison between Experimental and Numerical Results of Specimen S4

According to the reading in Figure 5, Figure 6, and Figure 7, the numerical results have achieved the elastic behaviour, no failure in any component was observed, since, the graphs achieved linear behaviour. According to Figure 5, Figure 6 and Figure 7 numerical results for S4, S5 and S1 achieved 6.35, 4.07 and 4.98 mm respectively. In comparison experimental results achieved 6.45, 4.20 and 4.89 mm respectively. Therefore, numerical analysis results are less than 10% away from the experimental results. In conclusion numerical analysis is validated. However, the reason for numerical results achieving small percentage in discrepancy, as shown in Table 1, is due to the contact between plasterboard and stud for S4 and S5 and both studs for specimen S1. In the experiment, 3 screw connection were used between gypsum plasterboard and each stud flange, while for numerical results full contact between plasterboard and studs were given, due to the limitations of ABAQUS.

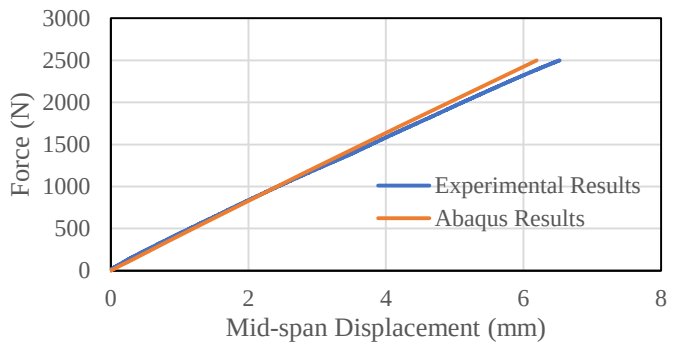

Figure 5. Comparison between test and numerical result of specimen S4

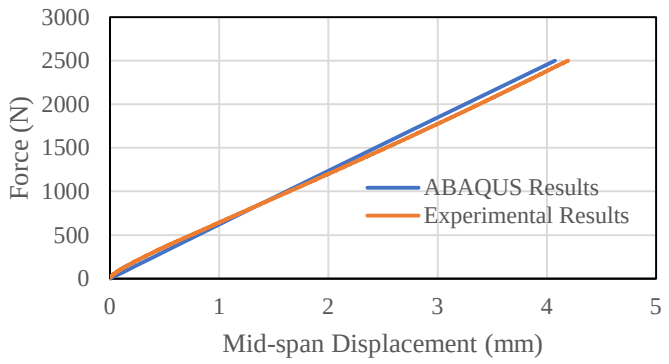

Figure 6. Comparison between test and numerical result of specimen S5

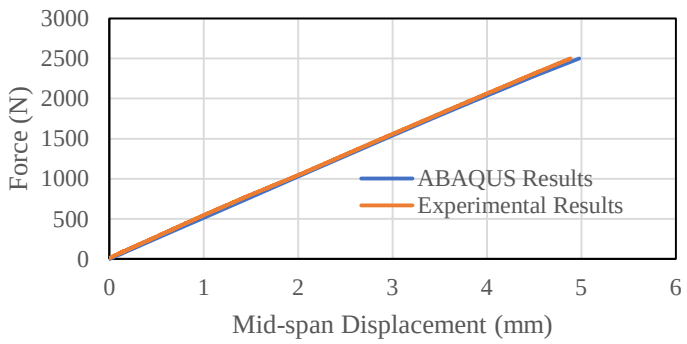

Figure 7. Comparison between test and numerical result of specimen S1

3.2.Verification of experimental study and numerical analysis summary

Comparative summary of the results obtained from experimental and ABAQUS is given in Table 1. Displacement reading was taken from the mid-span of both experimental and numerical results from Figure 5, Figure 6 and Figure 7.

Table 1 Difference between experimental and numerical results at the mid-span

Specimen ID	Experimental (mm)	ABAQUS (mm)	Difference (%)
S4	6.45	6.35	1.55
S5	4.20	4.07	3.09
S1	4.89	4.98	1.81

4. NUMERICAL ANALYSIS STRESS DISTRIBUTION

Contour plot was obtained from ABAQUS software to show the stress distribution of the steel studs, where the maximum stress for the contour plot was set as 512 MPa to show clear stress distribution, due to steel stud having a yield stress of 512 MPa. According to Figure , stress distribution of the stud is represented in a range of colour from the maximum stress (512 MPa) is in red to no stress (0 MPa) is in dark blue. In Figure stresses can be seen distributed to the stud where high stresses can be observed at the top flange and web-flange junction, where the top flange having higher stresses than the bottom base flange. Although maximum stress is achieved near loading and maximum stress located near boundary condition at the bottom flange lip. However, very low stresses can be observed at the middle of the web, except near holes at the sides medium to high stresses can be seen. When the load is applied on the loading plate, stresses are distributing from loading plate to the stud. Snowberger (2008), stated that the maximum stress occurs on a small area of the bending section (theoretically at the edge), this have been confirmed through the finite analysis. In this case, it was the bottom flange lip of the hole represented in light red with maximum stress of 400.0Mpa. Therefore, the steel studs did not reach its yield limit.

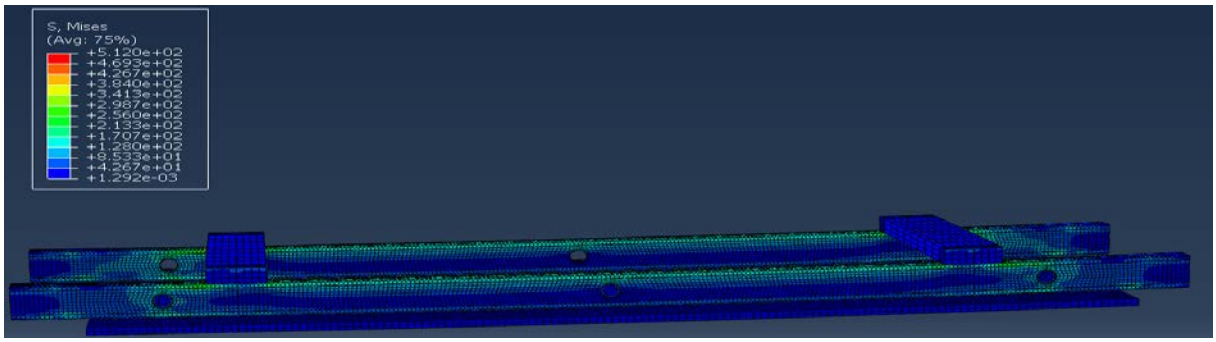

Figure 9. Finite analysis stress distribution contour plot at 2.5 kN loading (specimen S4)

4. CONCLUSIONS

The primary objective of this research was to investigate the composite behaviour of cold-formed steel section for wall panel under flexural loading. Four-point bending test was conducted as part of both the experimental study and numerical analysis to determine the behaviour of cold-formed steel under elastic limit. Numerical analysis has been conducted to validate the experimental results. It is seen from experimental and numerical results that when gypsum plasterboards are considered to the CFS wall panel, deflection of a CFS wall panel is decreased. Further research is required to understand the effects of different combinations of steel channel and gypsum plasterboard on the CFS wall panel.

ACKNOWLEDGMENTS

The technical support provided by Western Sydney University is gratefully acknowledged.

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Applicability of Kriging to Regional Flood Estimation Problem in Eastern Australia

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Abstract

Design flood estimation in ungauged catchments is a common problem in hydrology. Regional flood frequency analysis (RFFA) is widely used in design flood estimation at ungauged sites, which attempts to transfer flood characteristics from gauged catchments to ungauged ones. The most commonly adopted RFFA methods in Australia in the past included the Index Flood Method, Quantile Regression Technique and Probabilistic Rational Method; however, the new Australian Rainfall and Runoff (ARR) recommends a Parameter Regression Approach based on Log Pearson Type 3 distribution. This paper presents development of a new RFFA method in Australia based on ordinary kriging. It uses data from 558 gauged catchments from Victoria, New South Wales and Queensland States of Australia. These catchments are small to medium in size, with an upper limit of 1000 km^2. Based on a leave-one-out validation technique, it has been found that the relative error values in design flood estimates by kriging are in the range of 28 to 36%, which are smaller than Australian Rainfall and Runoff (ARR) recommended RFFE Model. However, kriging shows a relatively higher degree of bias than the RFFE Model. The findings of this study will be useful to enhance the RFFE Model in Australia in near future by applying kriging.

Keywords*:* Floods, ARR, Kriging, RFFE Model, RFFA

1. INTRODUCTION

The new edition of Australian Rainfall and Runoff (ARR) 2016 (Ball et al., 2016) include many recent advances regarding holistic planning, design and operation of flood management issues and involve many alternative approaches in design flood estimation. The development was necessitated to review and evaluate the available design procedures recommended in ARR 1987 (Pilgrim, 1987) as well as update the guidance to provide the best available methods and design data Australia wide under varying conditions. In many design flood problems, for example, culverts and small to medium sized bridges, only the peak flow of the flood hydrograph are the dominant variable of interest; whereas in some design applications such as flood storage design of retarding basins, an estimation of full hydrograph and also other flood characteristics are necessary (Pilgrim, 1987). In regions where adequate streamflow data are available, at-site Flood Frequency Analysis (FFA) can be applied (Stedinger et al., 1993; Kuczera, 1999); however where there is paucity of observed streamflow data, Regional Flood Frequency Analysis (RFFA) technique is recommended for design estimation (Flavell, 2012). Numerous RFFA techniques and procedures have been proposed around the world (Seibert, 1999; Hundecha & Bardossy, 2004; Young, 2006; Sivapalan *et* al., 2013).

The RFFA approach attempts to transfer flood frequency characteristics from a group of homogeneous gauged catchments to the ungauged catchment of interest. There are limited numbers of gauged

catchments throughout Australia over an area of about 7.5 million km^2, which limits the development of accurate RFFA techniques in Australia. The RFFA technique referred as 'RFFE Model 2015' is recommended for estimation of design peak discharges at ungauged catchments in Australia in the recent version of ARR (Ball et al., 2016). In the past, the most common RFFA methods in Australia included the 'Probabilistic Rational Method' recommended for general use in Victoria and eastern NSW (ARR 1987); 'Index Flood Method' relied on the assumption of regional homogeneity; and generalised least square based 'Quantile Regression Technique'. The later offered a powerful statistical method which is also popular in the US (Bates, 1994; Griffis & Stedinger, 2007; Hicks et al., 2009; Haddad & Rahman, 2012; Rahman et al., 2015). The performance of RFFA model is largely governed by the quantity and quality of the available data and the capability of the adopted statistical techniques to transfer information from gauged to ungauged sites within the region. It is reported that the relative accuracy of 'RFFE Model 2015' is likely to be within ±50%. Generally, the degree of uncertainty in RFFA technique is typically greater than at-site FFA (Rahman et al. 2012 and 2015).

This paper presents development of a new RFFA method in Australia based on ordinary kriging (Delhomme, 1978; Villeneuve *et* al., 1979); which may be applied as a potential tool for regional analysis of hydrological variables like flood quantiles (Daviau et al., 2000; Grover et al., 2002; Eaton et al., 2002; Chokmani & Ouarda, 2004). Although the range of the kriging method was limited to the field of mining initially, it has been noticeably expanded over the past years in Hydrosciences especially in Europe (Delhomme, 1978; Chokmani & Ouarda, 2004). Generally based on the neighbourhood information, kriging' more specifically ordinary kriging offers the best possible unbiased and optimum prediction of the unknown values. This paper adopts ordinary kriging to estimate flood quantiles using 558 catchments located in eastern Australia. The performance of the new technique is evaluated by applying a leave-one-out (LOO) validation approach (Haddad et al., 2013).

1.1. Data Selection

This paper uses streamflow data from 558 catchments from the States of New South Wales (NSW), Victoria (VIC) and Queensland (QLD). These catchments were also adopted in the development of ARR RFFE Model 2015. An upper limit of catchment size of 1,000 km² was generally adopted. These data were obtained from ARR Revision Project 5 (Rahman et al. 2015).

The selected streams are unregulated since major regulation (e.g. a large dam on the stream) affects the rainfall-runoff relationship significantly by increasing storage effects. Streams with minor regulation, such as small farm dams and diversion weirs, are not excluded because this type of regulation is unlikely to have a significant effect on large floods. The data sets for the selected potential catchments were prepared following a stringent procedure as detailed in (Haddad et al., 2010), gaps in the annual maximum flood series were in-filled as far as could be justified; low floods were censored in FFA (Lamontagne et al., 2013), errors associated with extrapolation of rating curves were investigated and the presence of trends with the data were checked (Rahman et al., 2015; Ishak et al., 2013).

Table 1. Summary of selected catchments from eastern Australia

State	No. of stations	Streamflow record length (years)	Catchment size (km^2)
NSW & Australian Capital Territory	176	20 – 82	1 – 1036
Victoria	186	20 – 60	3 – 997
Queensland	196	20 – 102	7– 963
TOTAL	558	20 – 102	1– 1036

2. METHODS

The RFFA method based on ordinary kriging presented here can be applied as a potential tool for estimating flood quantiles at ungauged sites. Using this powerful geostatistical technique, a spatial correlation model can be adjusted to be used for the estimation of specific quantiles. Amongst the neighbouring catchments, the closest one receives the greater weightage in general, as it is more likely to be similar to the ungauged catchment where flood quantiles needs to be estimated. In essence, the kriging allows predicting design floods at the ungauged sites using flood quantiles from the neighbouring gauged sites (Ouarda et al., 2008).

The kriging considers the spatial structure in the data and the distribution of the parameters along a function named 'variogram'. The structure function 'variogram' typically exhibits the pattern, configuration and intensity of the variable's spatial autocorrelation (Ouarda et al., 2008). In this study, to overcome the scaling effect, flood quantiles are standardised by the catchment area. A logarithmic transformation of variables is applied to the selected flood quantiles (2, 5, 10, 20, 50 and 100 years return periods). A variogram is developed using R package to identify the nature of spatial correlation. The variance level, usually called the 'sill', is reached at a distance, known as the 'range'. Furthermore, a 'nugget' effect is added to the model to identify the level of uncertainty relating to the local estimation, sampling or even localization errors. Thereafter, a predicted (theoretical) model is fitted to the observed (experimental) variogram.

For the performance evaluation of the kriging technique, a cross validation called LOO is adopted where each of the selected gauged catchments is in turn considered as ungauged catchment and the predicted flood quantiles are compared with the observed flood quantiles.

A range of evaluation statistics are calculated to evaluate the performance of the ordinary kriging model: coefficient of determination (R^2) (Equation 1), mean bias (BIAS) (Equation 2), relative mean bias (RBIAS) (Equation 3), mean square error (MSE) (Equation 4), root mean square error (RMSE) (Equation 5), relative mean square error (RRMSE) (Equation 6), Nash-Sutcliffe efficiency (NSE) (Equation 7) and relative error (RE) (Equation 8).

$$R^2 = 1 - \frac{\sum(Q_p - Q_o)^2}{\sum(Q_o - \bar{Q}_o)^2} \quad (1)$$

$$BIAS = \frac{1}{n}\sum\left(Q_p - Q_o\right) \quad (2)$$

$$RBIAS(\%) = \frac{1}{n}\sum\left(100 * \left(\frac{Q_p - Q_o}{Q_o}\right)\right) \quad (3)$$

$$MSE = \frac{1}{n}\sum\left(Q_p - Q_o\right)^2 \quad (4)$$

$$RMSE = \sqrt{MSE} \quad (5)$$

$$RRMSE = \frac{\sqrt{\frac{1}{n}\sum(Q_p - Q_o)^2}}{\overline{Q_o}} \quad (6)$$

$$NSE = 1 - \frac{\sum(Q_o - Q_p)^2}{\sum(Q_o - \overline{Q_o})^2} \quad (7)$$

$$RE = 100 * \left(\frac{Q_p - Q_o}{Q_o}\right) \quad (8)$$

where,

Q_p = Predicted flood quantile;
Q_o = Observed flood quantile; and
$\bar{Q}_o$ = Average observed flood quantile.

3. RESULTS AND DISCUSSION

Figure 1 shows developed variogram models for the six selected flood quantiles. Table 2 summarises the results, which indicates a decreasing trend of 'sill' and 'range' with the decrease in return period of flood quantiles. The 'range' indicates the distance after which data are no longer significantly correlated. The 'sill' represents the total variance where the empirical variogram appears to level off. Besides, the 'nugget' presented in Table 2 increases with the increase of return periods from 10 to 100 years; however, the trend differs for 5 and 2 year return periods. This indicates that the level of uncertainty increases gradually from 10 year to 100 year return period.

Table 2. Characteristics of developed variogram models

Quantiles	Q_2	Q_5	Q_{10}	Q_{20}	Q_{50}	Q_{100}
range	1.42	5.29	5.90	6.22	6.70	7.05
nugget	0.31	0.27	0.26	0.28	0.34	0.41
psill	0.38	0.47	0.51	0.53	0.56	0.59
sill	0.69	0.74	0.77	0.81	0.90	1.00

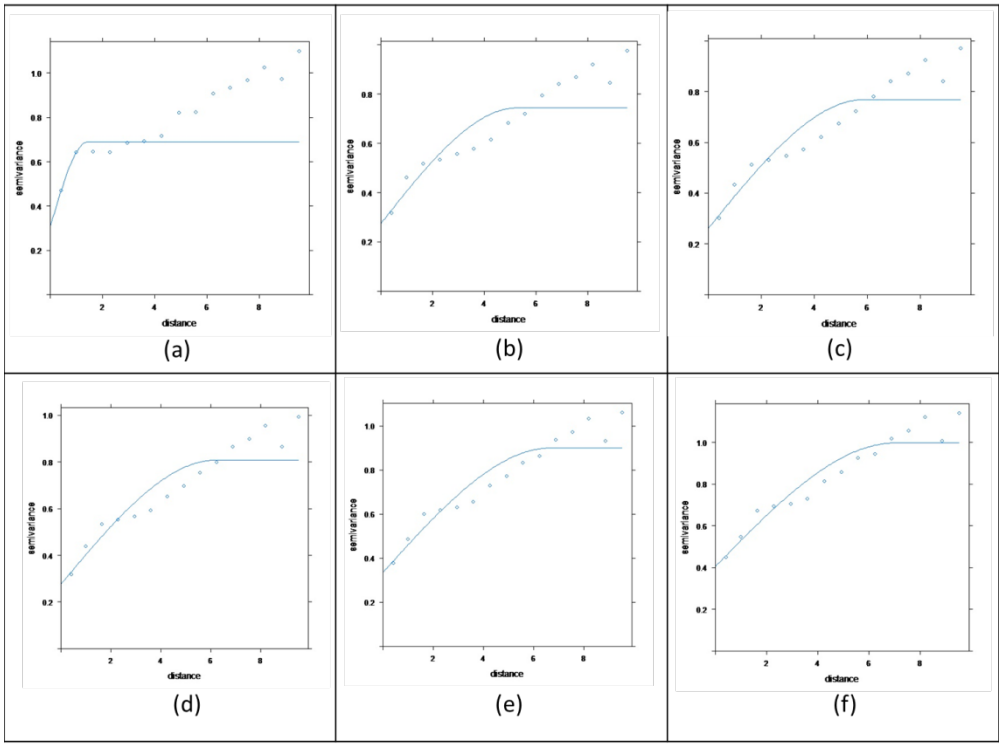

Figure 1. Variogram for the flood quantile models: (a) Q_2 (b) Q_5 (c) Q_{10} (d) Q_{20} (e) Q_{50} and (f) Q_{100}

The predicted (by ordinary kriging) (based on LOO) and observed flood quantiles for the 558 stations are plotted in Figure 2, and the histogram of relative error values for different flood quantiles are presented in Figure 3. These plots show that the predicted and observed flood quantiles match quite well. The performance of ordinary kriging is quite good (Table 3), with the NSE values in the range of 0.6-0.8. The NSE criterion compares the performance of kriging in relation to the observed mean value, where a negative value indicates that the estimation method is worse than using the mean value. On the other hand, the RRMSE value shows that, the least erroneous estimation is achieved for 20 year return period, which is followed by10 year return period, while the RRMSE value is found to be the highest for two year return period.

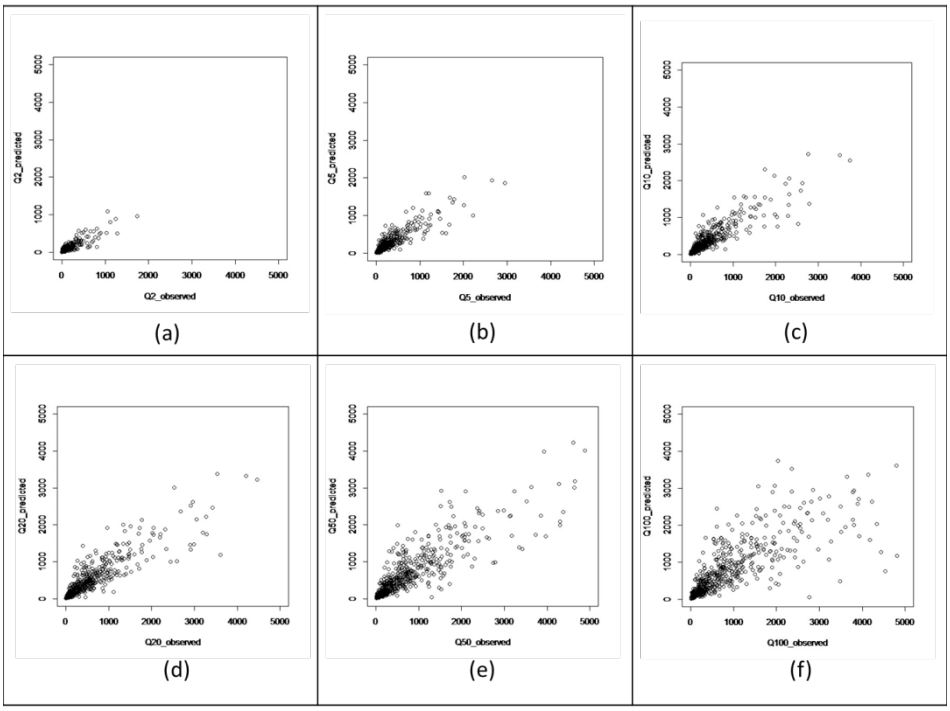

Figure 2. Predicted (by ordinary kriging) vs at-site flood quantiles (a) Q_2 (b) Q_5 (c) Q_{10} (d) Q_{20} (e) Q_{50} and (f) Q_{100}

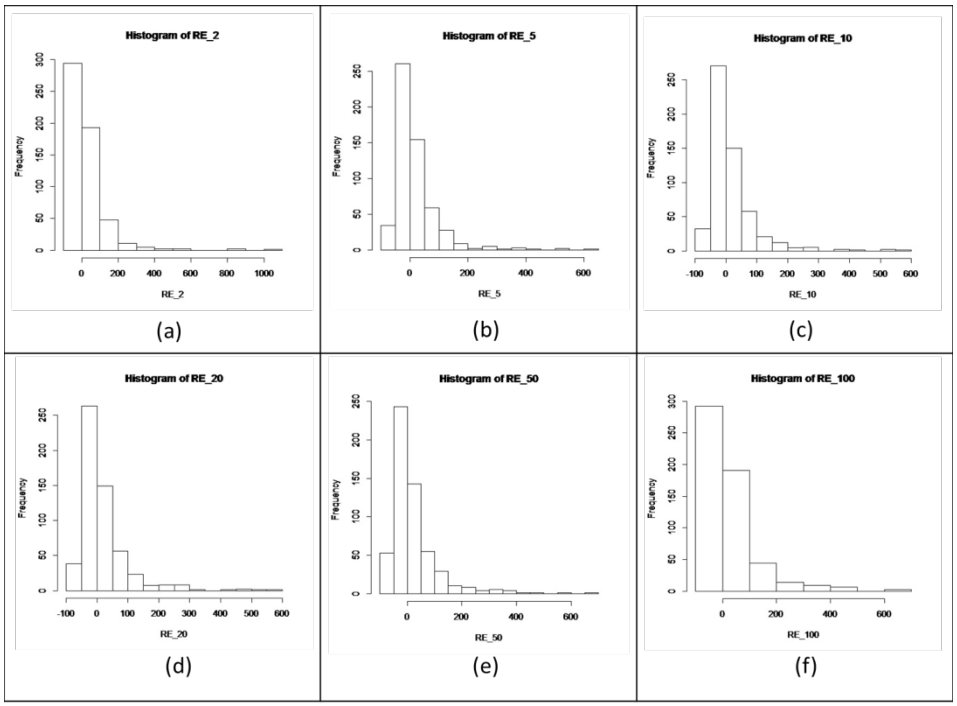

Figure 3. Histogram of relative error values for (a) Q_2 (b) Q_5 (c) Q_{10} (d) Q_{20} (e) Q_{50} and (f) Q_{100}

Table 3. Cross validation statistics from LOO for ordinary kriging

Quantiles	Q_2	Q_5	Q_{10}	Q_{20}	Q_{50}	Q_{100}
Mean_abs_RE	57.15	46.33	44.86	46.79	52.73	58.79

Median_abs_RE	34.20	30.55	28.75	28.41	31.86	35.37
mean (Qpred/Qobs)	1.23	1.16	1.16	1.17	1.20	1.24
median(Qpred/Qobs)	0.97	0.97	0.97	0.95	0.95	0.96
BIAS	-24.60	-31.30	-42.29	-58.29	-95.11	-145.46
RBIAS(%)	23.36	16.06	15.51	16.72	20.38	24.43
MSE((m^3/sec)^2)	10030.23	28597.01	54919.76	100666.20	233149.6	494243.4
RMSE(m^3/sec)	100.15	169.11	234.35	317.28	482.86	703.02
R^2	0.71	0.78	0.79	0.77	0.72	0.63
RRMSE	0.85	0.63	0.58	0.58	0.62	0.72
NSE	0.71	0.78	0.79	0.77	0.72	0.63

The negative value for the BIAS as found in the present case (Table 3) shows that the kriging method generally underestimates the observed flood quantiles. Furthermore, the coefficient of determination (R^2) is found to be decreasing generally with the increasing return period as expected. The validation results shown in Table 3 slightly differs this tendency for Q_2 and Q_5 probably due to sampling errors. The median RE values are the smallest for 10 and 20 year return periods and highest for 100 year. Overall, the median RE values are in the range of 28 to 36%, which seems to be an excellent result for Australia.

It should be noted here that LOO is a more rigorous validation technique compared with the split-sample validation where the model is tested on a smaller number of catchments (e.g. 10% of the total catchments). Hence, the RE that is generated by LOO is expected to be higher than if split-sample validation were used. The medians of the absolute relative error values from the LOO validation of the ordinary kriging and ARR RFFE technique are compared in Table 4, which show the ordinary kriging outperforms the ARR RFFE Model. The main conclusion from this analysis is that the quantification of uncertainty in the quantile estimates by the kriging technique is reasonable for the vast majority of the test catchments.

Table 4. Median relative error (RE) from leave-one-out validation of the ordinary kriging and ARR RFFE technique

Quantiles	Q_2	Q_5	Q_{10}	Q_{20}	Q_{50}	Q_{100}
Kriging	34.20	30.55	28.75	28.41	31.86	35.37
RFFE Model 2015	51	49	52	53	57	59

4. CONCLUSION

The study examines development of a new RFFA method in Australia based on ordinary kriging, which may be applied as a potential tool for regional analysis of hydrological variables like flood quantiles at ungauged sites. A large number of catchments (558) located in eastern Australia (NSW, VIC and QLD) ranging from small to medium in size, with an upper limit of 1000 km^2 have been used in the study. The data for this study were obtained from ARR Project 5. For the performance evaluation of kriging technique and compare the result with traditional method, a cross validation approach called leave-one-out (LOO) has been applied. The result has shown that the median relative error values in design flood estimates by kriging are in the range of 28 to 36%, which are smaller than the recommended ARR RFFE Model 2015. However, kriging shows a relatively higher degree of bias, which can be rectified by an interpolated bias correction factor. Overall, the results show the promise

of the application of kriging in regional flood estimation in Australia. A detail study is being continued on kriging and future publications will present further development on this.

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Bearing Capacity Analysis of Piled Raft Foundation by Numerical Analysis Using Finite Element Method (FEM) for Dhaka-Chittagong Elevated Expressway

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Abstract

Bearing capacity is one of the most important characteristics of any kind of soil. For every construction work it is compulsory to calculate the bearing capacity of soil of study area for particular type of foundation. Bearing capacity is generally calculated by some conventional equations like Terzaghi's bearing capacity equation and Meyerhof's bearing capacity equation and for different types footings these equations vary. In this research extended sub-loading tij model for Finite Element Method (FEM) is used to calculate the bearing capacity of piled raft foundation. Elasto-plastic constitutive model parameter identification is an important task for proper modeling of any soil. In this research, subsoil characteristics of study locations are presented based on field and laboratory test results. Elasto-plastic constitutive model parameters of study locations soil has been determined for extended sub-loading tij model. In this study some soil parameters are determined from laboratory tests and by using these, simulation parameters like Compression index for FEM tij simulation (λ), Swelling index for FEM tij simulation (Ḱ), Critical stress ration (RCs) and Void ratio at 98KPa (N) are calculated. Using these parameters, bearing capacity of piled raft foundation has been estimated for 0.05% settlement of soil section. Considering the effect of settlement in 2D Finite Element analysis have been conducted. It is found that bearing capacity determined by the conventional methods match well with the results of the numerical simulations.

Keywords: Constitutive Model, Bearing Capacity, Settlement, Finite Element Method etc.

1. INTRODUCTION

Bearing capacity is estimated by limit analysis using upper bound and lower bound theory. But the limit state analysis cannot consider the effect of Over Consolidation Ratio (OCR), bonding effect of soil. Therefore, in estimation of bearing capacity such parameters should be considered. A now-a-days FE method is widely used in different fields of Geotechnical Engineering. So, such condition can also be applied for bearing capacity estimation. However, the accuracy of the FE analysis depends on the constitutive models of soils. Available constitutive models such as Camclay model (Roscoe and Burland, 1968), Drucker-Prager Model, Mohr-Coloumb Model cannot properly consider or explain soil behavior of different densities. However, in this paper extended sub-loading *tij* model (Nakai and Hinokio, 2004; Nakai et al., 2011) is used which can consider influence of intermediate principal stress on the deformation and strength of soils, dependence of the direction of plastic flow on the stress paths, influence of density and/or confining pressure and bonding effect on the deformation and strength of soils (Shahin et al,. 2004; Nakai et al., 2010; Nakai et al., 2011).

Pile foundation is a popular deep foundation type used to transfer superstructure load into subsoil and bearing layers. However, accurate prediction of piles' settlement is particularly difficult concerning complicated consolidation process and pile-soil interaction (Kazimierz, 2015). Piles are commonly used to transfer superstructure load into subsoil and a stiff bearing layer. As it was emphasized by

Lambe and Whitman (1969), a pile foundation, even in the case of single pile, is statically indeterminate to a very high degree.

The present study is limited to sub-soil properties parameters for constitutive modeling of the ground where the proposed Dhaka-Chittagong elevated Expressway will be constructed. The main objectives of the study are:

1. Determination of elasto-plastic constitutive model parameters for extended sub-loading *tij* model.
2. Determination of load bearing capacity of piled raft foundation.

2. METHODOLOGY

2.1. Study Area

For the soil investigation, we have selected several places in Narayanganj and Comilla districts. In Narayanganj, we have collected soil from Sonargaon (23°38′51″N to 90°35′52″E with an area of 171.02 km^2) and Bandar (23°37′N to 90°31.5′E with an area of 55.84 km^2) upazila. In Comilla, we have collected soil from Comilla SadarDakshin (23°22′N to 91°12′E with an area of 241.66 km^2) and Chouddagram (23°13′N to 91°19′E with an area of 268.48 km^2) upazila, (BBS 2011).

2.2. Material Collection

Soil samples are collected as boring sample using Shellby tubes. Hence, the samples were undisturbed. The length of each tube was 450 mm. we have collected samples from different depths of earth i.e. 2m, 5m, 10m, 20m and 30m below from the earth surface. These samples are then tested in laboratory by different experimental procedures.

2.3. Laboratory Experiments

Undisturbed samples were collected during field investigation from the selected study areas. Laboratory tests were conducted according to ASTM standards (ASTM, 1989). Index and strength properties were determined to evaluate the sub-soil condition of study area. Moisture content test, specific gravity test, atterberg limit test, consolidation test and unconfined compression test of soil were conducted at four selected sites.

3. INDEX PROPERTIES OF SOIL

Index properties of the clay layer have been presented in Table 1. Specific gravity of the clay layer varies in the range 2.68 to 2.71. Dry unit weight of the soil varies between 14.11 and 16.20 kN/m^3. Natural moisture content of the soil varies in the range 23.0 and 40.0. Liquid limit and plastic limit of the clay layer vary in the range 48 and 56, 21 and 28, respectively. Sand, silt and clay content of the layer are 3~10%, 49~70% and 27~41%, respectively.

Table 1. Index properties of soil

Soil Parameters	Narayanganj		Comilla	
	Sonargaon	Bandar	Sadar Dakshin	Chouddagram
Specific Gravity, G_s	2.70~2.71	2.67~2.68	2.68~2.69	2.66~2.68
Dry unit weight, γ_d (KN/m^3)	15.79~16.20	14.65~14.96	15.07~15.86	14.11~15.02
Natural moisture content (%)	23.0~23.2	24.5~26.7	23.7~26.0	26.0~40.0
Liquid Limit, LL (%)	49~50	52~53	48~50	45~56
Plastic Limit, PL (%)	18~22	24~25	21~22	25~28
Unconfined compressive strength, q_u (KPa)	110~139	112~119	89~134	62~64

4. PARAMETER IDENTIFICATION FOR CONSTITUTIVE MODELING

4.1. Some Important Features of Sub-loading t_{ij} Model

An elastoplastic constitutive model for soils, called the extended subloading t_{ij}-model (Nakai, 2011), is used in the finite element analysis. This model, despite the use of a small number of material parameters, can describe properly the following typical features of soil behaviors (Nakai and Hinokio, 2004 & Nakai, 2011):

(i) Influence of intermediate principal stress on the deformation and strength of geomaterials.
(ii) Dependence of the direction of plastic flow on the stress paths.
(iii) Influence of density and/or confining pressure on the deformation and strength of geomaterials.
(iv) The behavior of structured soils such as naturally deposited soils.

A brief description of the above mentioned features of this model can be made as follows:

Influence of intermediate principal stress is considered by defining yield function f with modified stress t_{ij} (i.e.,defining the yield function with the stress invariants(t_N and t_S) instead of (p and q). The yield function is written as a function of the mean stress t_N and stress ratio $X \equiv t_S/t_N$ based on t_{ij} by Equation 1.

$$f = \ln\frac{t_N}{t_{N0}} + \varsigma(X) - \left(\ln\frac{t_{N1e}}{t_{N0}} - \ln\frac{t_{N1e}}{t_{N1}}\right) = 0 \tag{1}$$

Here, t_{N1} determines the size of the yield surface (the value of t_N at X=0), t_{N0} is the value of t_N at reference state and t_{N1e} is the mean stress t_N equivalent to the present plastic volumetric strain which is related to the plastic volumetric strain ε_v^p as

$$\varepsilon_v^p = \frac{\lambda - \kappa}{1 + e_0}\ln\left(\frac{t_{N1e}}{t_{N1}}\right) \tag{2}$$

The symbols λ and κ denote compression index and swelling index, respectively, and e_0 is the void ratio at reference state. In this research, the expression for $\varsigma(X)$ is assumed as,

$$\varsigma(X) = \frac{1}{\beta}\left(\frac{X}{M^*}\right)^{\beta} \tag{3}$$

The value of M* in Equation 4. is expressed as follows using principal stress ratio $X_{CS} \equiv (t_S/t_N)_{CS}$ and plastic strain increment ratio $Y_{CS} \equiv (d\varepsilon_{SMP}{}^{*p}/d\gamma_{SMP}{}^{*p})_{CS}$ at critical state:

$$M^* = \left(X_{CS} + X_{CS}{}^{\beta-1}Y_{CS}\right)^{1/\beta} \tag{4}$$

and these ratios X_{CS} and Y_{CS} are represented by the principal stress ratio at critical state in triaxial compression R_{CS}.

In elastoplastic theory, total strain increment consists of elastic and plastic strain increments as,

$$d\varepsilon_{ij} = d\varepsilon_{ij}^e + d\varepsilon_{ij}^p \tag{5}$$

Here, plastic strain increment is divided into component $d\varepsilon_{ij}^{p(AF)}$, which satisfies associate flow rule in the space of modified stress t_{ij}, and isotropic compression component $d\varepsilon_{ij}^{p(IC)}$as given in Equation 6.

$$d\varepsilon_{ij}^p = d\varepsilon_{ij}^{p(AF)} + d\varepsilon_{ij}^{p(IC)} \tag{6}$$

The components of strain increment are expressed as,

$$d\varepsilon_{ij}^{p(AF)} = \Lambda\frac{\partial f}{\partial t_{ij}} \quad and \quad d\varepsilon_{ij}^{p(IC)} = K\langle dt_N\rangle\frac{\delta_{ij}}{3} \tag{7}$$

Here, Λ is the proportionality constant, δ_{ij} is Kronecker's delta and <> denotes Macauley bracket. Dividing plastic strain increment into two components as in Equations 6 and 7 for the same yield function, this model can take into consideration feature (ii), i.e., the dependence of the direction of

plastic flow on the stress paths, (Mohammad et al., 2013). Adding the term $G(\rho)$ in the denominator of the proportionality constant Λ of normal consolidated condition, influence of density is considered. The proportionality constant Λ is expressed as,

$$\Lambda = \frac{\frac{\partial f}{\partial \sigma_{ij}} d\sigma_{ij}}{\frac{1+e_0}{\lambda-\kappa}\left(\frac{\partial f}{\partial t_{kk}} + \frac{G(\rho)}{t_N} + \frac{Q(\omega)}{t_N}\right)} = \frac{df_\sigma}{h^p} \tag{8}$$

$$K = \frac{1}{\frac{1+e_0}{\lambda-\kappa}\left(1+\frac{G(\rho)}{a_{kk}}\right)} \cdot \frac{1}{t_{N1}} \tag{9}$$

In feature (iv), the stress-strain behavior of structured soil can be described by considering not only the effect of density described above but also the effect of bonding. Two state variables ρ related to density and ω representing the bonding effect are used to consider feature (iv). The following relationships for $G(\rho)$ and $Q(\omega)$ are adopted in the model:

$$G(\rho) = sign(\rho) a \rho^2 \quad and \; Q(\omega) = b\omega \tag{10}$$

Where a and b are material parameters.

The parameters of subloading t_{ij} model are fundamentally the same as those of the Cam clay model (Roscoe and Burland, 1968), except for the parameter a, which is responsible for the influence of the density and the confining pressure. Parameter β controls the shape of the yield surface. The performance of the constitutive model has already been checked in numerical simulations (Shahin et al. 2004; Shahin et al., 2011; Nakai et al., 2010).

4.2. Layers of Soil Section with Piled Raft Foundation

Figure 1. is a section of piled raft foundation with different layers of soil.

4.3. Mesh of Soil Section

Figure 2. is the mesh with dimension of the same section which has been done for simulation work.

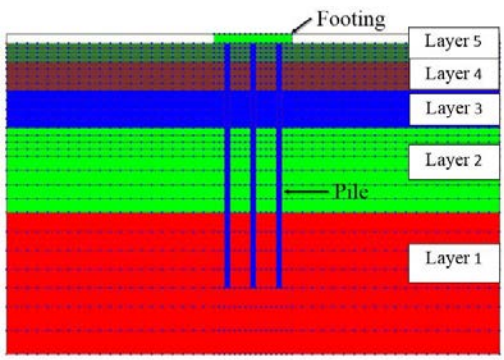

Figure 1. Layers of piled raft foundation soil section

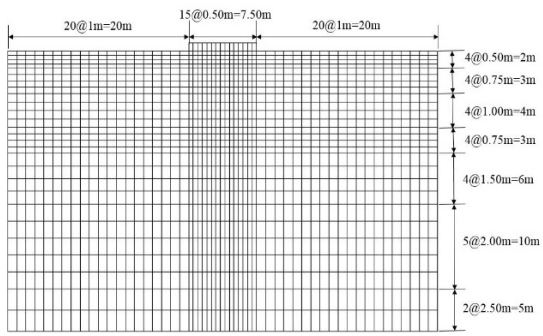

Figure 2. Finite Element Mesh for piled raft foundation

4.4. Determination of Soil Parameters

For getting parameters of the constitutive model (Table 2), consolidation tests for study locations have been carried out in laboratory. Figure 3 shows the relations between void ratios and mean effective stress in logarithmic scale. From these curves, compression index λ, swelling index $\acute{K}$ and void ratio at

98kPa, N are obtained for both soils by using Equations 11, 12 and 13. Using these values and fitting the computed curve parameter *a* (density parameter) of sub-loading t_{ij} model is obtained.

$$\lambda = 0.434 \times Cc \quad (11)$$

$$\acute{K} = 0.434 \times Cs \quad (12)$$

$$N = \text{Void ratio at 98 KPa} \quad (13)$$

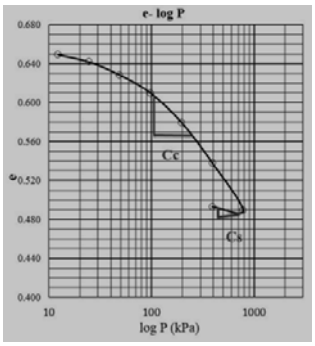

Figure 3. Calculation of C_c and C_s from e vs. logP curve

Table 2. Simulation parameters

Soil Layers	Depth (m)	λ	Ḱ	β	RCs	N	e0	aAF	aIC
1	0.00-2.00	0.07	0.0045	2	3.2	1.1	0.8	30	30
2	2.01-5.00	0.1038	0.00829	1.6	3.98	0.865	0.879	800	800
3	5.01-9.00	0.1018	0.00803	1.6	4	0.868	0.88	850	850
4	9.01-18.00	0.0819	0.00983	1.6	4	0.778	0.789	800	800
5	18.01-33.00	0.0879	0.00894	1.6	4	0.602	0.62	800	800

5. RESULTS AND DISCUSSIONS

5.1. Initial Stress Distribution of the Ground

Figure 4. shows the initial distribution of stress without piled raft foundation. Here, the stress in the deepest layer is the highest.

5.2. Stress Distribution of the Ground with Structure Load

Figure 5. shows the initial distribution of stress with piled raft foundation. Here, it is shown how the piles are distributing the loads in the soil layer.

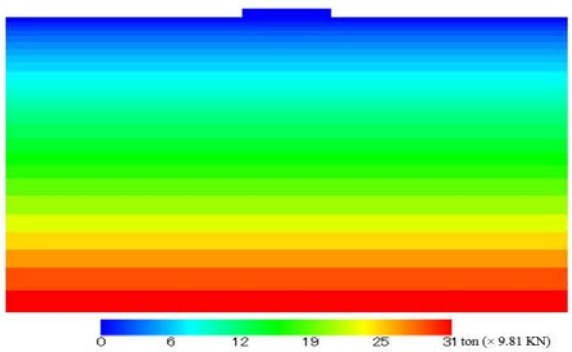

Figure 4. Stress distribution without piled raft foundation

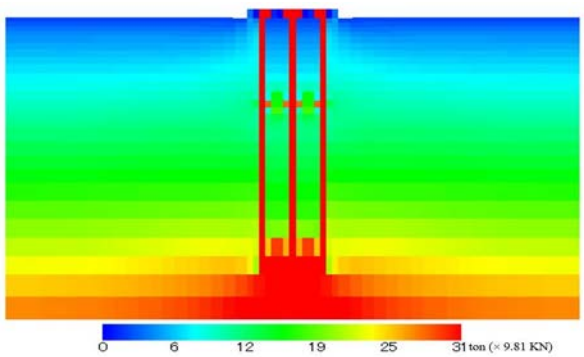

Figure 5. Stress distribution with piled raft foundation

5.3. Load-Displacement Relation

This the final result of our study through simulation. Figure 6. shows the load bearing capacity of soil. For 0.05% settlement the soil can take 880 ton load.

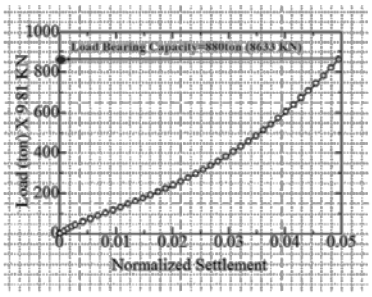

Figure 6. Load vs. Settlement curve

6. CONCLUSIONS

This paper determines soil parameters for Dhaka-Chittagong Elevated Expressway. This involved study area selection, soil sample collection, laboratory experiments, numerical analysis, calculation of simulation parameters and simulation by using Finite Element Method (FEM). It is concluded that load bearing capacity for 0.05% vertical settlement of soil is 8633 KN.

ACKNOWLEDGMENT

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Parametric Investigation of Cold-formed Steel Section for Wall Panel

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Abstract

Cold-form Steel Section has been known for its high strength due to high strength to weight ratio. Cold form steel (CFS) sections are used for load bearing primary members in substantial structures like high storey building and bridges, and non-bearing structural elements like composite roofing, slab decks and wall panels. Limited experimental study on CFS sections for wall panel has been conducted, which is not sufficient to understand its behaviour. As experimental investigation conducted on CFS sections for wall panel is expensive and time consuming, the development of FE models, well validated against experimental data, would be form a useful alternative to overcome any experimental limitations. In this paper, parametric study using finite element (FE) analysis has been conducted to investigate the behaviour of the CFS sections for wall panel under flexural loading. The FE model is validated by comparing the FE model results with test results. The extensive parametric study is conducted to investigate the effects of different combinations of steel sections and gypsum plasterboard, that can possibly influence the elastic behaviour of the cold form steel sections. The parametric results show that the presence of gypsum plasterboard has significant influence on the elastic behaviour of the CFS sections used in wall panel.

Keywords: Cold-Form Steel, Parametric Study, Local Flexural Buckling, Finite Element Analysis

1. INTRODUCTION

Cold-form Steel (CFS) sections and gypsum plasterboards are widely used to fabricate the wall panels. CFS sections with gypsum plasterboards provide a higher strength to weight ratio. The concept of wall panels is advantageous for the designer as it can act as load bearing infill walls around the perimeter of a structure. Gypsum is used on either side of the CFS as it effectively isolates sound and is fire resistant. These wall panels are not load bearing and are only used as partition walls. CFS sections are provided in the wall panels to support their own self-weight and reduce the overall weight of the structure. Hence, CFS sections in a wall panel not only helps a structure in reducing the total weight but may also acts as load bearing infill walls. In addition to the lateral loads, gravity loads are also resisted by the wall panels. Baran and Alica (2012) conducted several tests on different types of wall panels with diagonal struts and different sections and compared the results from various empirical

formulae. Gunalan et al. (2013) has conducted a research to determine the elastic behaviour and the analysis of CFS Wall system under high fire rating. The conclusion from the report was that the CFS Wall using composite panel system has higher structural and thermal performance than other load bearing walls with varying arrangements of gypsum plasterboard. However, the research did not look at fundamental behaviour of CFS section failure mode. Therefore, the research herein is going to study these failure modes.

CFS sections are also used for non-bearing structural elements like composite roofing, slab decks and wall panels. Lee and Miller (2000) conducted a research on a composite wall panel with two C-sections with gypsum plasterboards on either side. The assumption taken in their research was that the axial load acts on the centroid of the cross-section. The flexural as well as the combined effect of torsional and flexural buckling loads are calculated using the differential equation of equilibrium and an energy method. To study the behaviour of the local and distortional failure of the standard CFS sections, Yu and Schafer (2007) conducted a series of experimental tests on C-sections and Z-sections. They examined the influence of moment gradient on distortional buckling of CFS beams. Maduliat et al. (2015) conducted a research to study the failure behaviour of 42 CFS sections under pure bending. The conclusion of their research was that when the width to depth ratio, which is extremely important in the design of CFS, of a section is less, torsional buckling failure takes place. However, when the ratio is high, the failure is more inclined towards the distortional buckling failure. In addition, it has been observed that the specifications in many international standards are un-conservative.

In recently, Fairly (2016) conducted the experimental study to investigate the behaviour of CFS wall panel under bending. Fairly (2016) mainly investigated the fundamental behaviour of CFS wall frame panels under elastic limits to further understand and improve these members. The research involved experimental study using three four-point bending test (FPBT) specimens. Two specimens are wall frames (CFS section with gypsum plasterboard) and one back-to-back steel without gypsum plasterboard. All specimens are tested using universal testing machine applying 75% of the design load of the studs. Test results demonstrate that back-to-back CFS studs can be used to overcome the buckling problem for light load bearing wall panels due to their higher rigidity. Gypsum plasterboard included in CFS wall panel also have significant influence on the failure modes, which is understood by testing limited two specimens of wall panels.

It can be concluded from the previous studies that research on CFS sections for wall panel under bending is still very limited, and further research is required. The goal of this research is to further investigate the behaviour of CFS section with gypsum plasterboard as conducted in Fairly (2016). An extensive parametric study will be conducted to investigate the effect of various factors that can possibly influence the elastic behaviour of the cold form steel sections. The three validated models are further modified with different combinations of steel sections and gypsum plasterboard and seven more models are formed and the effect of gypsum plasterboard on the CFS section is observed.

2. FINITE EELEMENT MODELLING AND VALIDATION

2.1 General

To develop the FE model of CFS wall panel, ABAQUS software package was considered throughout the analysis. All components of a CFS wall panel with or with gypsum plasterboard are simulated by using 8-noded brick elements (C3D8R), with three translational degrees of freedom at each node. The surface to surface interaction is defined for the contacts between the plasterboard and CFS sections. Tie constraint is used to simulate the screw connectors. In ABAQUS, the normal interaction between the surfaces is defined as the "hard contact". The tangential behaviour of the surface to surface contact is defined using the Coulomb friction. Figure 1 shows the developed FE model of a CFS wall panel with both sides gypsum plasterboard.

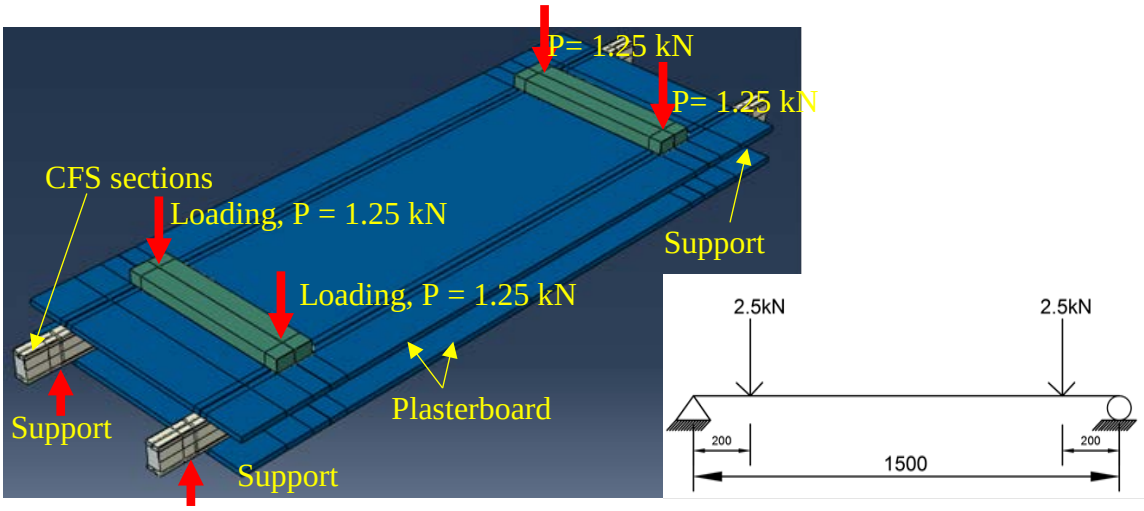


Figure 1. Load and boundary condition of a CFS wall panel with both sides plasterboard specimen1

2.2 Material Models

The elastic and inelastic properties of the steel section and the gypsum plasterboard are assigned . The wooden block material is assigned as of a high stiffness as it provides the support between the flanges and acts along with the boundary condition. The steel plate is assigned the same material property as the steel section as the only function of the steel plate is to disperse the load equally on the surface of the section. The elastic property of gypsum plasterboard and CFS Sections are shown in Table 1.

Table 1. Elastic properties of CFS section and Gypsum plasterboard

Characteristics Properties	CFS Section	Gypsum plasterboard
Modulus of elasticity	200 GPa	900 MPa
Poisson's ratio	0.3	0.2

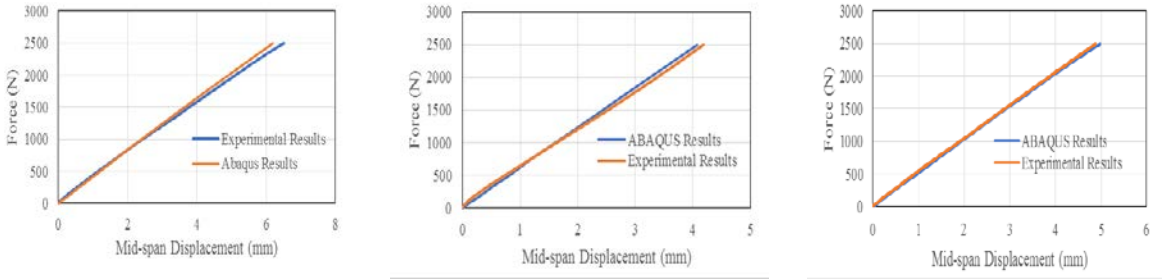
2.3 Boundary Conditions

Loading and boundary conditions are considered according the test conducted by Fairly (2016) for CFS wall panel under bending as shown in Figure 1. Loading has been provided on the steel plate which enables to convert the point load to the uniformly distributed load. The load applied is 5kN. The boundary conditions are taken as pinned support on both the edges making the frame symmetrical.

2.3 Validation of FE model

FE models are established to conduct the parametric analysis. Before conduct the parametric analysis, FE simulation results are compared with test data reported by Fairly (2016) to verify the FE model. Once validated, the parametric study might not be validated and it hence eliminates the time requirements and excessive resources used in the experimental studies. The accuracy of the models depends on how well the ABAQUS model stimulates the behaviour of the CFS section under the four-point bending test. The comparisons between FE and test results are shown in Figures 2 (a)-(c). The error percentage between the FE and test result of specimen 1 is less than 1% which is less than both other specimens. This may be because of the absence of gypsum plasterboard. Hence the increase in the error percentage inn specimen 2 and 3. Specimen 2 is uni-symmetrical with gypsum plasterboard only on the bottom. The gypsum plasterboard on the top may act as a compression flange of the composite section. Similarly, the gypsum plasterboard on the bottom may end as the tension flange, though the amount of stress on the compression would be more than the amount of stress on the tension due to gypsum plasterboard as gypsum plasterboard like concrete is good in compression than

in tension. In specimen 3, the neutral axis distance would vary less than in specimen 2 because of gypsum plasterboard present on either side of the CFS sections. Having considered these facts, it can be concluded that Figures 2 (a)-(c) show a good agreement between the FE models and the experimental studies.



(a) specimen 1 *(b) specimen 2* *(c) specimen 3*

Figure 2. Comparison between FE and test results

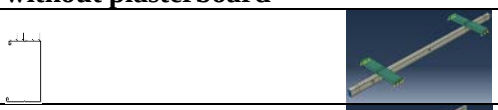
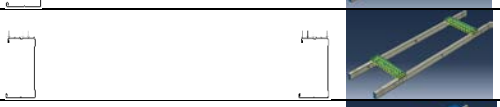
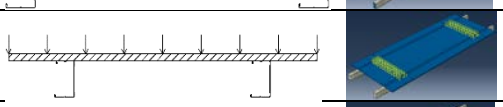
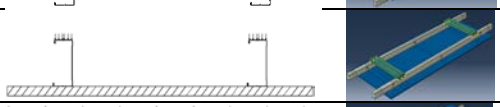
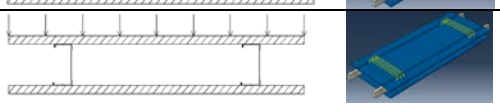
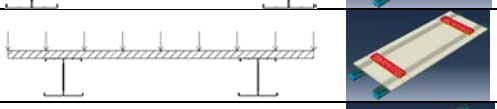
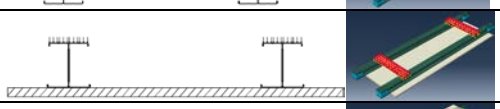
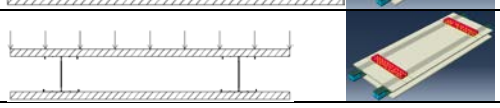
3. INVESTIGATION OF DIFFERENT PARAMETERS

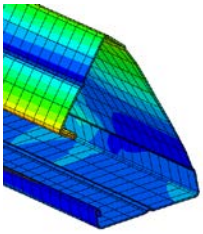
Based on the verified FE model, parametric study is conducted r to understand the behaviour of the composite combination of CFS sections know as studs and the gypsum plasterboard. The parametric study is conducted in order to expand the available structure performance data and to understand further the elastic behaviour of the frame and the steel studs. In the parametric study, the combinations of the gypsum plasterboard and the CFS section are changed to understand the effect of one on another. The load is applied on either of the part of the frame and the results are compared and discussed. The aim of the parametric study is to understand the different modes of buckling behaviour in the CFS sections with respect to gypsum plasterboard. This is done by observing the slenderness, structural behaviour and different failure modes. A total of 10 FE models was included. The ten FE models have been divided into two categories as can be observed from the nomenclature. Specimens A1 toA5 as shown in Table 2 are made using single stud with or without gypsum plasterboard. This has been done because the back to back channel studs results in a combined I-section. Channels are uni-symmetric but I-sections are bi-symmetric and thus have higher moment of inertia, and different behaviour patterns. Table 2 also shows another five specimens (specimens B1-B5) considered double studs (back to back channel) with or without gypsum plasterboard.

The deflection of these specimens subjected to the bending load of 2.5kN is summarised in Table 2. Deflection of each specimen is calculated at the mid-span of the frame. Deflection of CFS wall panel decrease when different combinations of gypsum plasterboard are used in back-to-back channel frame (specimens B2 to B5) and its values are lower compared to the single channel frames (specimens A2 to A5) with different combinations of gypsum plasterboard are used. The lowest deflection is observed when both sides plasterboards are considered.

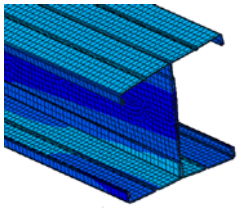
Buckling behaviour of the CFS sections with single side or back to back side channel is illustrated in Figure 3. As can be seen from the Figure 3(a) which shows the mode of failure at the centre of the stud section, local buckling of the compression zone of the flange and the web are observed for specimen A1. This local buckling is due to the absence y-axis symmetry of the channel. This behaviour is absent in the failure of specimen B1 as can be seen in Figure 3(b). The local buckling is minimised in the compression flange and the failure is eliminated is very negligible in the web. This is due to the connection of the webs into each other. The very minimum buckling of the web which is observed in the Figure 3(b) is due to the small gap between the centre of the two webs. The mirror studs provide a symmetry in the section, decreasing the stresses in the flange and hence increasing the stiffness, thus reducing the deflection. The deflection in specimen A1 is more than twice the deflection of the specimen B1. This is because the back-to-back section provides twice the moment of inertia.

Table 2. Parameters considered for CFS wall panels and their results

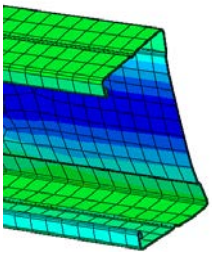
Sample No.	Stud Type	Gypsum plasterboard	Configuration of CFS sections with or without plasterboard	Deflections
A1	Single	Absent		11.293
A2	Frame	Absent		8.282
A3	Frame	Top		6.360
A4	Frame	Bottom		6.081
A5	Frame	Top & Bottom		4.325
B1	Single	Absent		4.780
B2	Frame	Absent		4.329
B3	Frame	Top		3.554
B4	Frame	Bottom		2.864
B5	Frame	Top & Bottom		1.699



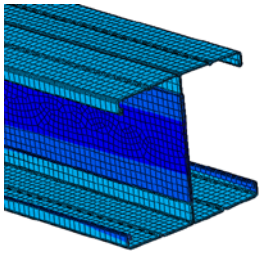
(a) Specimen A1



(b) Specimen B1



(c) Specimen A2



(d) Specimen B2

Figure 3. Buckling modes of specimens A1, A2, B1 and B2

However, comparing the specimen A1 and specimen A2, the failure behaviour of both specimens is almost same as shown in Figure 3(a) and (c). This is because the two studs are the same and the spacing between the studs does not affect the elastic behaviour of the frame. The combined frame provides twice the moment of inertia and reduced displacement. The same can be seen from the failure behaviour of back-to-back channel specimens B1 and B2, as shown in Figure 3 (b) and (d).

4. CONCLUSIONS

This research has described a detailed investigation into the behaviour of the composite action of CFS section and gypsum plasterboard in a wall panel subject to local buckling failure. The following conclusions can be drawn from the parametric investigations of CFS wall panels.

1) The local buckling of the compression zone of the flange and the web is observed on the single side channel of CFS sections when gypsum plasterboard is not considered.
2) For back-to-back side channel with plasterboards, the local buckling is minimised in the compression flange and the failure is eliminated is very negligible in the web.
3) When gypsum plasterboards are considered to the CFS wall panel, stiffness of a CFS wall panel is increased and thus reducing the deflection.

Thus, the buckling problem observed on CFS wall panel can be overcome by considering either gypsum plasterboard or back-to-back side channel.

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Queensland Flood in 2010-11: Will This Type of Flood Occur Soon?

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Abstract

During 2010-2011 Australia experienced one of the biggest flood events in Australia's history. Six major rain events affected large parts of the eastern states of Australia during this period. From December 2010 to January 2011, Queensland, Western Australia, Victoria and New South Wales experienced widespread flooding. There was extensive damage to both public and private properties, towns were evacuated and 37 lives were lost, 35 of those in Queensland. Three quarters of Queensland was declared a disaster zone, an area greater than France and Germany combined, and the total cost to the Australian economy has been estimated at more than $30 billion. The large scale of events, the number of lives lost and the scale of the damage incurred prompted numerous inquiries and review processes. The Queensland government convened a Commission of Inquiry to investigate the issues and consequences from the flood and to work towards learning lessons from the flooding to reduce the future vulnerability of the community to this type of disaster. To manage flood, minimise the risk, impact and damage due to flood, it is important to understand the cause of such flood and, frequency of occurrence. This paper presents the possible causes of such flood, possibility of its occurrence, insurance issues associated with flood, outcome of the 2010-11 floods commission of inquiry and the relation of global warming and climate change.

Keywords: Flood, flood insurance, cause of flood, chance of occurrence, impact of climate change.

1. INTRODUCTION

Flooding is the most common environmental hazard worldwide. It is part of the natural water cycle or a "Hydrologic Cycle". Floods occur when the amount of water flowing from a catchment exceeds the capacity of its drains, creeks and rivers. This process begins with rainfall, but is affected by many other factors. In Australia, high rainfall variability heavily influences flooding. Floods can also be caused by other factors including tsunamis, large tides, storm surges etc. In the last 35 years of the 20th century, seventy-seven (77) floods were recorded in Australia; eight major floods were recorded in the 19th century and six in the first decade of the 21st century (OQCS 2016). While Australia is described as the driest inhabited continent on Earth, right now it might seem hard to believe. Dangerous floods have occurred in every Australian state over the last 150 years. An Australian flood of 2010–11 principally affected three eastern states of Australia and was one of the worst in the country's history. Queensland (Figure 1), in the north, was hit hardest, but the widespread flooding of a scale not seen since the mid-1970s began in December 2010 spread southward to inundate portions of the neighboring states of New South Wales and Victoria by early 2011 (Murray 2015). This flood at the state of Queensland is known as Queensland 2010-11 floods. Flood affected much of central and southern Queensland. Three-quarters of the areas of Queensland were declared disaster zones, 35-people were killed, over 200,000 people were affected by it (Wikipedia 2016a). In Australia, floods are the most expensive type of natural disaster. Until recently, the costliest year for floods in Australia was 1974, when floods affecting New South Wales, Victoria and Queensland resulted in a total cost of $2.9 billion (OQCS 2016).

Figure 1. Location of Queensland and Brisbane (van den Honert and McAneney 2011)

2. CAUSE OF QUEENSLAND 2010-11 FLOODS

Australian flood records extend back as far as the 1840's, only a few years after European settlement of the area in 1824 (Figure 2) (van den Honert and McAneney 2011). The 2010/2011 floods occurred after a prolonged period of drought, in quick succession, compounded intermittently by three major storm events and cyclones.

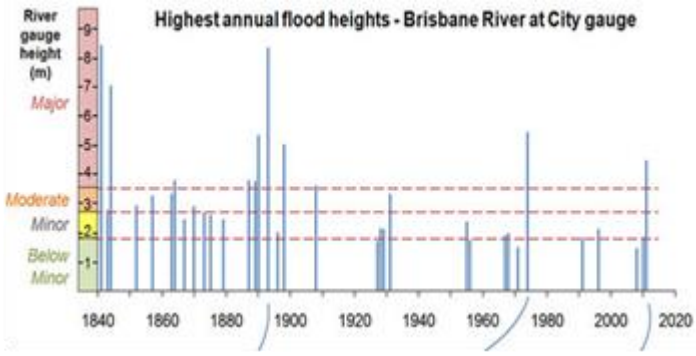

Figure 2. Highest annual flood heights at the Brisbane City gauge, 1840-2011 (van den Honert and McAneney 2011)

The 2010-11 Queensland floods were a series of floods in Australia which began in December 2010 and ended in January 2011. The record-breaking rainfall during the 2010–11 La Niña (BOM 2016e) (La Niña is an unusual weather pattern, which brings wet weather to eastern Australia often associated with extreme rainfall and widespread flooding) led to widespread flooding in many regions of Australia including severe flooding in southeast Queensland. The second half of 2010 and early 2011 was characterized by one of the four strongest La Niña events since 1900 (Figure 3). Figure 4 shows total rainfall across Australia for November 2010 to January 2011. Some gauge stations at north and west of Brisbane exceeded 1,200 mm rain. December 2010 was the wettest on record, with 107 places getting their highest rainfalls ever. Significant number of locations in Queensland experienced greater than 1 in 100-year rainfall. The state average rainfall level was 404.7mm compare to maximum 369mm in 1975. 2010 was the state's wettest spring since 1900 (Wikipedia 2016a) and the Australia's third wettest year (NASA 2011).

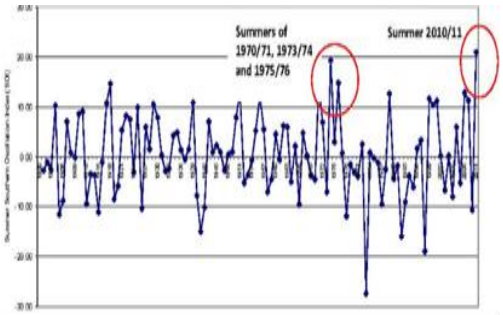

Figure 3. Average summer (October to March) Southern Oscillation Index (SOI), 1900/01 to 2010/11 (BOM 2011d)

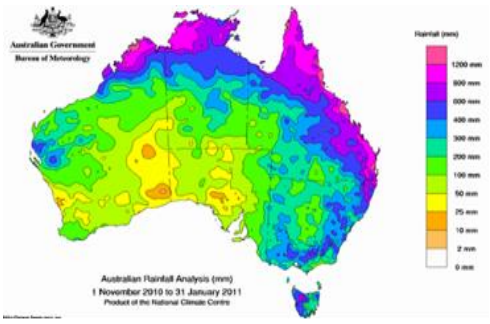

Figure 4. Total rainfall for the period November 2010–January 2011 (BOM 2011b)

Flooding started across parts of the state in early December 2010. These floods were caused by heavy rain from tropical cyclone "Tasha" that joined with a trough during a La Niña event. Australia. The 2010 La Niña was the strongest since 1973 (Wikipedia 2016a). Excessive rainfall caused rivers in southern Queensland reached record water levels including Brisbane and Bremer Rivers and caused flooding (BOM 2016f). Very intense localised rainfall caused severe flash flooding through the Toowoomba city (BOM 2011c). Heavy to very intense rainfall caused rapid creek rises and extreme flash flooding in the Lockyer Valley (BOM 2010a). Heavy rainfall caused very high water level at Wivenhoe and Somerset Dams that caused release of water from the dams resulting downstream flood. This flood was termed a “dam release flood” by hydrologists appointed by the Insurance Council of Australia. They suggest that a release of water from the Wivenhoe Dam was a principal cause of flooding at downstream of the dam (ICA Hydrology Panel 2011). Commission of Inquiry into the 2010–11 Queensland floods final report states that 59% of the downstream flooding was caused by water releases from the dam (Wikipedia 2016b).

3. NATURE, MAGNITUDE AND EXTENT OF QUEENSLAND 2010-11 FLOOD

The 2010-2011 floods across Queensland (the floods) were unprecedented in magnitude, scale and scope and in terms of extent, impact and severity, was amongst the most significant in Australia's recorded history. The January 2011 flood event can be categorized as a large to rare event by the Institution of Engineers Australia (Engineers Australia) national guidelines for the estimation of design flood characteristics (AR&R). The flood level classifications adopted by the Bureau of Metrology also define the Event as a major flood (SEQ Water 2011). Research published in Geophysical Research Letters has estimated that 2010-11 summer rainfall played a major role in average global sea levels dropping by seven millimeters (SCD 2015). Many places, including Condamine and Chinchilla were inundated by flood waters on multiple occasions (Hubert 2011). The most extreme flooding occurred in Central and South Queensland, particularly in the Fitzroy River and Condamine-Balonne River systems.

The extent and magnitude of the floods in Queensland were unprecedented in many places. As an illustration, this series of flood events yielded the filling of Lake Eyre for the third year in succession, a rare event. Further the floods contributed to the filling of the water reserves across the State, that is, both the surface reservoirs and the aquifers. Additionally, the river outflows into the Pacific Ocean constituted a series of freshwater turbid plumes that impacted the Great Barrier Reef. The floods in Queensland encompassed both large-scale floods and some flash flooding, including a deadly event in Toowoomba and the Lockyer Valley on 10 Jan. 2011. In 2011 (Hubert 2011), The Queensland flooding was Australia’s largest natural disaster in recent memory. With a ballpark estimate of US$ 15.9 billion in total damages and economic losses (with a public reconstruction cost of approximately US$7.2 billion), this is also one of the major international disasters of the last decade. The government and private sector have mobilized an estimated US$ 11.8 billion, representing 75 percent of the estimated damage and losses which is already above the 45 percent average of disaster coverage in

developed economies (The World Bank 2011). During these floods, approximately 3572 businesses were inundated. There were 5900 people evacuated from 3600 homes, approximately $4b in commercial losses across mining, agriculture and tourism sectors, and 19,000 km of roads were damaged. Three major ports were significantly affected. More than 28 per cent of the Queensland rail network was left twisted and displaced. An estimated 28,000 homes may need to be rebuilt while vast numbers of dwellings require extensive repairs (Harden Up 2016).

4. QUEENSLAND 2010-11 FLOOD INQUIRY

The 2010-11 wet seasons brought unprecedented rain and flooding to Queensland. The large scale of events, the number of lives lost and the scale of the damage incurred prompted numerous inquiries and review processes by different governments and organisations. On January 2011, the Queensland Floods Commission of Inquiry was established to examine the events leading to the floods, all aspects of the response and the subsequent aftermath, and to make recommendations about things that could be improved for the future (QFCI 2011).

The Commission's inquiries included considering over 700 written submissions, conducting 68 days of public hearings, taking evidence from 345 witnesses and convening community consultation sessions and meetings. The final report contains 177 recommendations directed at a broad range of matters related to the 2010/11 floods, including: floodplain management, planning and building issues, the performance of private insurers, the impact of floods on operational and abandoned mines, the emergency response to the floods and dam management (Queensland Government 2012). The Australian review of 2010-11 floods including Queensland Floods Commission of Inquiry reports varied greatly in their scope, but it virtually ignored the issue of climate change and its impact on flooding (Wenger et al. 2013).

5. QUEENSLAND 2010-11 FLOOD RELATED INSURANCE ISSUES

Following a natural disaster, the insurance industry plays a vital role in funding the rebuilding, repair or replacement of damaged homes, infrastructure and assets. While some insurers offer cover for riverine flood, many currently do not, something that many policy holders discovered only after the January 2011 floods (Johnston et al. 2011). Thus, whilst claims by insured victims of flash flooding in Toowoomba and the Lockyer Valley were generally settled by insurers, many residents along the Brisbane River had claims denied. Whilst the Insurance Council of Australia has put the proportion of denied claims at 15% (Trowbridge et al. 2011), the perceived lack of performance of private insurers in assisting the recovery from the flood has led to a community backlash. Many people were unaware that they were at risk of flood; others rejected flood insurance where offered because it was viewed as being prohibitively expensive (Johnston et al. 2011). The January 2011 floods have exposed the non-insurance or under-insurance of not just individual homeowners, they have also focused attention on the Queensland government's lack of insurance cover for its own infrastructure. At present self-insurance purchase by the State applies only to 25% of the reconstruction costs, with the Commonwealth covering the other 75% (van den Honert and McAneney 2011). Many insurers do not provide riverine flood insurance due to a lack of information on which to determine and price the risk. After Queensland 2010-11 floods, one of the biggest differences has been the rapid rollout of flood cover by insurers. In Queensland, 91.4 per cent of home and contents policies are now purchased with flood cover, compared with about 3 per cent in 2006 (FOS 2013). Nevertheless, a positive example of sensible land-use planning to arise out of the 2011 flooding occurred in the township of Grantham, where it is planned to relocate residential homes from the floodplain to higher ground outside the flood-zone (van den Honert and McAneney 2011).

6. WILL THIS TYPE OF FLOOD OCCUR SOON?

Understanding the chance of different sized floods occurring is important for managing flood risk. Floods occur in Queensland at irregular intervals. Six major floods occurred in Brisbane between 1885 and 1910, followed by more than 60 years without a major flood. Figure 2 also illustrates the sporadic nature of flooding, showing river levels at the Brisbane City gauging station since 1840 (OQCS 2016). Several factors are involved to predict whether flood event like Queensland 2010-11 floods will occur again or not. Firstly, we need to define an extreme event. Secondly, flow gauge record length is very short in Australia and this limits statistical analysis of the 'extremes'. Thirdly, climate is known to have varied at both decadal and centennial scales and this can also affect flood frequency predictions.

An extreme event (Based on IPCC definition, which is defined as equivalent to, or of greater than 90th quantile of the Australian Envelope Curve or AEC) can delineates the presently known upper limit of flood magnitude in Australia. The Queensland 2011 event lies below this upper limit in both the AEC and Global Envelope Curve (GEC) data sets suggesting that while large, events of higher magnitude can, and do, occur (The Big Flood 2016).

Brisbane Port-City gauge shows (Figure 2) extreme events in 1840s and 1890s are much larger than any flood events recorded since 1900. Most gauging stations in the region have very short record length (30-40 years). This is a limitation to use it to predict a rare event. Moreover, many gauging stations have not recorded an extreme event to guide the upper tail of the statistical distribution. Some gauging stations also do not have many extreme floods in their record. Based on 31 years of flood record at Lockyer Valley at the Spring Bluff gauge prior to the 2011 flood, the approximated 2011 flood peak is predicted to have an ARI of greater than 2000 years and with the inclusion of 2011 (32 years record), this reduced to 75 years and in 2016 with 36 years record the 2011 flood has a predicted recurrence interval of 90 years. This alone highlights the sensitivity of flood prediction to length of record of measured data. In addition, there exists decadal and centennial-scale climate variability. If short gauging records capture either a drought dominated or flood-dominated period, then these variations will bias predictions of flood frequency. Figure 7 shows that there are no clear trends in annual total Queensland rainfall over the 20th century. Rainfall varies from year to year and from decade to decade. La Niña is also associated with more heavy rainfall events. One of the key challenges is to extend the current gauging record length to better represent these climate fluctuations. One option is to use a Probabilistic Regional Envelope Curve (PREC) method (combine short record length gauging station with flood records from regions with similar characteristics) and then combined with traditional Flood Frequency analysis (FFA) on the combined longer record gauging data. Where available historical flood information (building mark, tree mark, newspaper) and history of past flood events stored within the floodplain sediments (named paleoflood record) can also be used. The inclusion of paleoflood records at-a-station can significantly decreases the uncertainty (90 % confidence Interval) for estimation of rare events in FFA (The Big Flood 2016). A review on coincident flooding in Queensland using joint probability and dependence methodologies done by the Queensland government for estimation of extreme flood did not specifically recommend applying this technique for estimation of extreme events (DoS 2012).

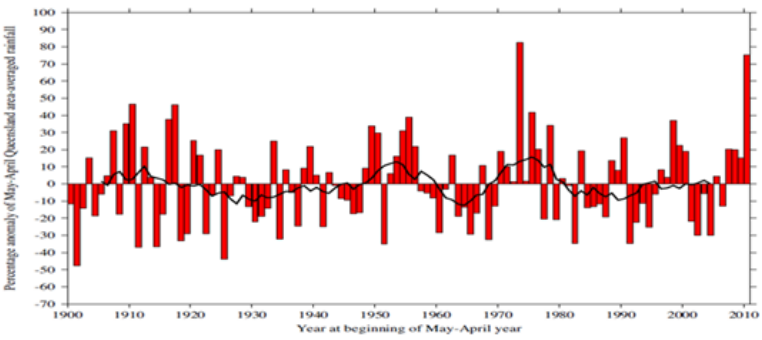

Figure 7: Queensland 100 years Rainfall (the red bars show the percentage deviation of each year's rainfall from the long term (1900-2010) average. The black line (11 year moving average) demonstrates decade-to-decade changes in rainfall) (DERM-QCCCE 2016)

7. CONCLUSIONS

Due to changing climate, the frequency and magnitude of floods in near future is expected to vary across Australia. As climate change will have notable impacts on the rainfall runoff process, thus hydrologic time series (e.g., flood data) can no longer be assumed to be stationary. A failure to take climate change into account can undermine the usefulness of the concept of return period, and can lead to underestimation / overestimation of design flood estimates (Wenger et al. 2013). Most climate models predict that the magnitude and frequency of storms and rainfall events will increase under a warmer 'greenhouse' climate (Nott et al. 1996). Because flood events are influenced by several factors, based on the current science it is difficult to confidently state that, overall, extreme flood events in Queensland will increase in intensity or frequency for climate change.

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Application of ANN in Regional Flood Estimation: A Case Study for New South Wales, Australia

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Abstract

Flood estimation in ungauged catchments is often needed in hydrology. Regional flood frequency estimation (RFFE) methods can be used for this purpose. The RFFE models in Australia are mainly based on linear models, such as Index Flood Method, Quantile Regression Technique, Parameter Regression Technique and Probabilistic Rational Method. The application of non-linear RFFE techniques such as Artificial Neural Network (ANN) is quite limited in Australia. In this paper, an ANN based RFFE model is presented for New South Wales (NSW) State in Australia. It uses data from 88 gauged catchments in NSW. A total of eight predictor variables are considered and five different model forms are tested. It has been found that when all the eight predictor variables are used, the ANN based RFFE model performs the best generally; however, the gain in model accuracy from using only three predictor variables is marginal. Furthermore, the performances of ANN based RFFE models vary across the six AEPs, and there no model that is the best with respect to all the evaluation statistics adopted here. The result shows that increasing the number of predictor variables does not necessarily enhance the performance of the ANN based RFFE models. The results demonstrate the potentials of ANN based RFFE models; however, further testing is needed using a larger data set before ANN based RFFE model can be recommended for practice in NSW.

Keywords: *Statistical Hydrology, RFFE, ANNs.*

1. INTRODUCTION

Flood is one of the worst natural disasters that cause significant damage to our society. To estimate design floods for ungauged catchments, regional flood frequency estimation (RFFE) methods are widely used in practice (Haddad et al., 2012). A RFFE technique attempts to transfer flood characteristics information from a homogeneous region to ungauged catchment location(s) of interest. Most of the RFFE techniques are based on linear modelling (e.g. Haddad et al., 2015; Micevski et al., 2015; Haddad and Rahman, 2012). The application of non-linear methods in RFFE such as artificial neural network (ANN) is quite few in numbers such as Aziz et al. (2014; 2015). To fill this current research gap, this study investigates the applicability of ANN based RFFE model for New South Wales (NSW) State in Australia.

The ANN is a data-based modeling technique, which was inspired by the working mechanism of human's natural neurons. ANN has been used frequently in the past few decades to solve complex mathematical problems. This was introduced by McCulloch and Pitts (1943). This technique has widely been adopted in medical and biomedical sciences (e.g. Baxt, 1990; Agatonovic-Kustrin and Beresford, 2000), and in different fields of engineering such as pattern recognition, forecasting, and data compression.

Lapedes and Farber (1987) first used ANN in modelling non-linear time series data. It has been found that ANN possesses better generalization capabilities than regression analysis and does not require the prescription of a mathematical functional form before the model building (Lek et al., 1996). ANN is

capable of discovering complex non-linear connection between observed and predicted data sets in many complex problems within science and engineering fields (Hsu et al., 1995).

The aim of this paper is to explore the applicability of ANN in RFFE in New South Wales (NSW) State of Australia. This, in particular, examines the selection of an appropriate set of predictor variables in ANN based RFFE modelling in NSW, Australia.

2. METHODOLOGY

This paper uses ANN to develop RFFE models. Unlike most of the other data processing methods, ANN finds the patterns and correlations in a sample data automatically without the use of a prescribed model form. Like human neural system, an ANN is made of plenty of single units which are called artificial neurons. Figure 1 shows a biological neuron which consists of three main parts: cell body (sama), dendrite and axon. Dendrites receive the information and transfer it to the cell body, then the signal moves through the sama and outputs through the axon, which are connected to other dendrites via synapses, if the signal received from the previous neuron reaches a certain threshold, the next neuron is activated and the signal is transferred between the neurons. Similarly, artificial neurons process the inputs (X) and compare them with a given threshold (θ) to estimate the proper output.

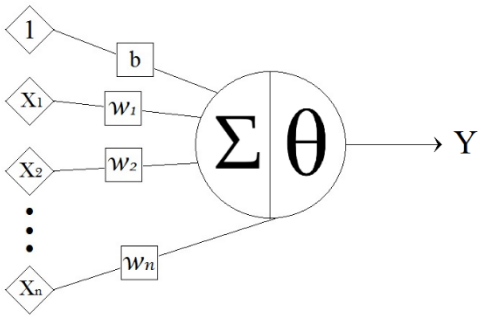

Figure 1 Schematic diagram of an artificial neuron

In Figure 1, X_i are the inputs and W_i are the specific weight matrix for each input, which determine the output by considering a given vector. It can be observed that there is a constant value of 1 among the inputs, which is introduced to the neuron by its unique weight b, known as bias. Bias allows the ANN to change the activation function. Consideration of bias for all networks is not essential, but it can improve the performance of a network significantly.

The mathematical relation of an artificial neuron can be shown as as below (Araghinejad, 2013):

$$I = W \times X + b \tag{1}$$

$$Y = \begin{cases} 1 \; if \; I \geq \theta \\ 0 \; if \; I \leq \theta \end{cases}$$

where X = inputs, W = weight matrix, b = bias, I = sum of all the weighted inputs, θ = threshold, and Y = output.

In this study, ANN is applied to develop prediction equation to estimate flood quantiles for six different annual exceedance probabilities (AEPs) i.e. 1 in 2, 1 in 5, 1 in 10, 1 in 20, 1 in 50 and 1 in 100 years. Model for each flood quantile is developed separately. Initially, for each of the selected gauged catchments, at-site flood quantiles of the 6 AEPs are estimated by using ARR FLIKE software

(Kuczera, 1999) by adopting LP3 distribution (with Bayesian method). The ANN based RFFE model prediction is compared with the at-site FLIKE estimates.

3. STUDY AREA AND THE DESCRIPTION OF DATA

A total of 88 small to medium sized gauging stations in NSW are selected for this study. The data preparation method of these catchments can be found in Rahman (2005), Haddad et al. (2010) and Rahman et al. (2009). The selected catchments are natural and free from major regulations. The catchments are smaller than 1000 km^2 with the median size of 260 km^2. The periods of annual maximum flood records range from 25 to 82 years. The following eight predictor variables are used in this study: (1) area of catchment (*AREA)* in km^2; (2) design rainfall intensity for 6-hour duration and 2-year return period (*I62*) in mm/h; (3) mean annual rainfall (MAR) (mm); (4) shape factor (*SF)*; (5) mean annual areal evapo-transpirationation (MAE) in mm; (6) stream density (*SDEN*) in km/km^2; (7) main stream slope of the central 75% of the main stream (S1085) and (8) fraction forest (FOREST). The data were obtained from Australian Rainfall and Runoff (ARR) Project 5 (Rahman et al., 2016).

The selected 88 stations were sub-divided into 70% (62 stations) for training, 15% (13 stations) for validation, and 15% (13 stations) for testing. All catchments in each of these groups were chosen randomly. Also, 5 different models based on different combinations of the 8 catchment characteristics were selected. A three-layer feed forward neural network was considered for the prediction with 2 hidden layers. The Levenberg-Marquardt algorithm was chosen for training purposes, and Hyperbolic Tangent sigmoid function was used as the activation function. The modelling was done using MATLAB software.

The following statistical measures were used to evaluate the performance of the ANN based RFFE models using the set of validation catchments (consisting of 13 stations, selected randomly):

Root mean squared normalised error:

$$\text{RMSNE} = \sqrt{\frac{1}{n}\sum\left(\frac{\hat{Q}_i - Q_i}{Q_i}\right)^2} \tag{2}$$

Relative root mean squared error:

$$\text{RRMSE} = \frac{\sqrt{\frac{1}{n}\sum\left(\frac{\hat{Q}_i - Q_i}{Q_i}\right)^2}}{\bar{Q}} \tag{3}$$

Coefficient of determination:

$$R^2 = 1 - \frac{\sum(\hat{Q}_i - Q_i)^2}{\sum(\hat{Q}_i - \bar{Q}_i)^2} \tag{4}$$

Mean bias:

$$BIAS = \frac{1}{n}\sum\left(Q_i - \hat{Q}_i\right)^2 \tag{5}$$

Relative mean bias:

$$rBIAS = 100\,\frac{1}{n}\,\frac{\sum(Q_i - \hat{Q}_i)}{Q_i} \tag{6}$$

Absolute median relationship:

$$absRE = \left|\frac{\hat{Q}_i - Q_i}{Q_i}\right| \times 100 \quad (7)$$

Ratio between predicted and observed quantiles:

$$r = \frac{\hat{Q}_i}{Q_i} \quad (8)$$

Here $\hat{Q}_i$ and Q_i are respectively the predicted (from ANN) and observed quantiles (from FLIKE), $\bar{Q}_i$ is the average of the observed quantiles for any given period, and *n* is the number of the considered catchments. These evaluation statistics are adopted from Blöschl et al. (2013).

4. RESULTS

Five different ANN-based RFFE models were compared: Model 1 (all the 8 predictors); Model 2 (AREA, I62); Model 3 (AREA, I62, SF); Model 4 (AREA, I62, SF, MAR, MAE, S1085, FOREST) and Model 5 (AREA, I62, SF, SDEN).

The current ARR RFFE model for NSW region (Rahman et al., 2016) consists of three predictors, AREA, I62 and SF (i.e. similar to Model 3 here). One important question here is whether addition of a greater numbers of predictors increases the model accuracy or not. The results from independent testing (using the validation data set) of the ANN based RFFE models using 7 evaluation statistics are summarised in Tables 1 to 5.

Close examination of results in Tables 1 to 5 show that Model 1 (based on all the 8 predictors) performs very well with respect to the median relative error (RE) for all the AEPs except 1 in 2. Overall, Model 3 performs the best with respect to the RE. The performances of models are different across the six AEPs, i.e., for AEPs of 1 in 2 and 1 in 20, Model 3 is the best, but for AEPs of 1 in 5, 1 in 10, 1 in 50 and 1 in 100, Model 1 is the best performer. In terms of BIAS, Model 4 is the poorest model. In terms of coefficient of determination (R^2), Model 1 is the best model, except for 1 in 2 AEP which shows a relatively smaller R^2.

The best two models (Model 1 and Model 3) are compared in Figure 2 with respect to RMSE, which shows that these two models perform quite similarly i.e. there is little differences in the RMSE values for the two models across the six AEPs.

Table 1. Summary of ANN-based RFFE model performance based on independent testing (Model 1)

Quantile	Median (ABS) RE(%)	Median Ratio	R^2	RMSE	RMSNE	RRMSE	rRMSE	BIAS	rBIAS
Q2	61.31	0.71	0.42	76.50	5.28	0.76	151.47	-20.08	-125.13
Q5	23.39	0.96	0.71	133.79	1.51	0.50	527.68	-5.89	-2.67
Q10	10.25	1.04	0.75	189.37	1.17	0.46	270.89	13.66	18.83
Q20	34.06	0.79	0.64	304.97	2.25	0.52	420.77	-102.98	-26.97
Q50	33.99	1.03	0.72	376.37	1.65	0.45	137.24	5.10	29.04
Q100	33.09	1.04	0.52	636.20	2.05	0.61	505.32	3.32	29.22

Table 2. Summary of ANN-based RFFE model performance based on independent testing (Model 2)

Quantile	Median (ABS) RE(%)	Median Ratio	R^2	RMSE	RMSNE	RRMSE	rRMSE	BIAS	rBIAS
Q2	36.13	1.02	0.71	53.65	0.89	0.54	89.44	2.26	-22.09
Q5	32.93	1.09	0.73	127.81	1.23	0.48	123.06	10.63	-40.19
Q10	34.19	1.06	0.68	212.53	1.79	0.51	319.50	3.44	-59.29
Q20	39.50	1.20	0.57	337.10	1.30	0.58	168.51	-16.63	-30.76
Q50	38.09	1.12	0.59	461.41	1.87	0.55	348.80	-2.96	-56.57
Q100	47.72	0.89	0.05	894.86	2.47	0.85	609.85	311.68	-54.28

Table 3. Summary of ANN-based RFFE model performance based on independent testing (Model 3)

Quantile	Median (ABS) RE(%)	Median Ratio	R^2	RMSE	RMSNE	RRMSE	rRMSE	BIAS	rBIAS
Q2	29.48	1.02	0.77	48.32	1.18	0.48	117.71	-0.91	-40.08
Q5	52.24	0.64	0.22	218.10	0.98	0.82	98.07	110.80	12.49
Q10	33.23	1.00	0.72	197.63	0.75	0.48	56.86	26.56	-7.02
Q20	33.43	0.98	0.68	287.39	1.30	0.49	168.22	21.09	-24.73
Q50	38.90	1.12	0.36	575.39	2.25	0.69	505.42	44.02	-68.93
Q100	38.32	0.93	0.37	726.09	3.50	0.69	1223.31	187.66	46.73

Table 4. Summary of ANN-based RFFE model performance based on independent testing (Model 4)

Quantile	Median (ABS) RE(%)	Median Ratio	R^2	RMSE	RMSNE	RRMSE	rRMSE	BIAS	rBIAS
Q2	38.28	1.23	0.72	52.62	3.29	0.53	329.24	-97.66	329.24
Q5	79.96	1.36	0.64	316.98	5.90	1.19	590.09	-12.04	590.09
Q10	29.84	1.09	0.62	231.52	3.25	0.56	1057.57	-59.87	1057.57
Q20	46.99	1.11	0.29	432.04	5.27	0.74	2777.88	-122.62	2777.88
Q50	30.69	1.07	0.55	483.04	2.07	0.58	429.41	-66.57	429.41
Q100	35.42	1.24	0.38	719.13	4.86	0.68	2366.70	-62.77	2366.70

Table 5. Summary of ANN-based RFFE model performance based on independent testing (Model 5)

Quantile	Median (ABS) RE(%)	Median Ratio	R^2	RMSE	RMSNE	RRMSE	rRMSE	BIAS	rBIAS
Q2	319.61	2.29	-9.77	329.04	27.88	3.29	2788.22	-174.27	53.26
Q5	84.89	0.99	0.04	242.55	7.67	0.91	767.27	-8.30	73.48
Q10	44.94	0.94	0.45	278.34	1.49	0.67	221.19	48.93	-15.29
Q20	38.44	0.98	-0.03	520.12	1.89	0.89	356.23	109.56	-20.44
Q50	46.69	0.76	0.39	559.18	4.84	0.67	2344.07	213.44	112.79
Q100	67.21	1.01	-0.21	1007.54	7.60	0.96	5769.06	164.90	-170.32

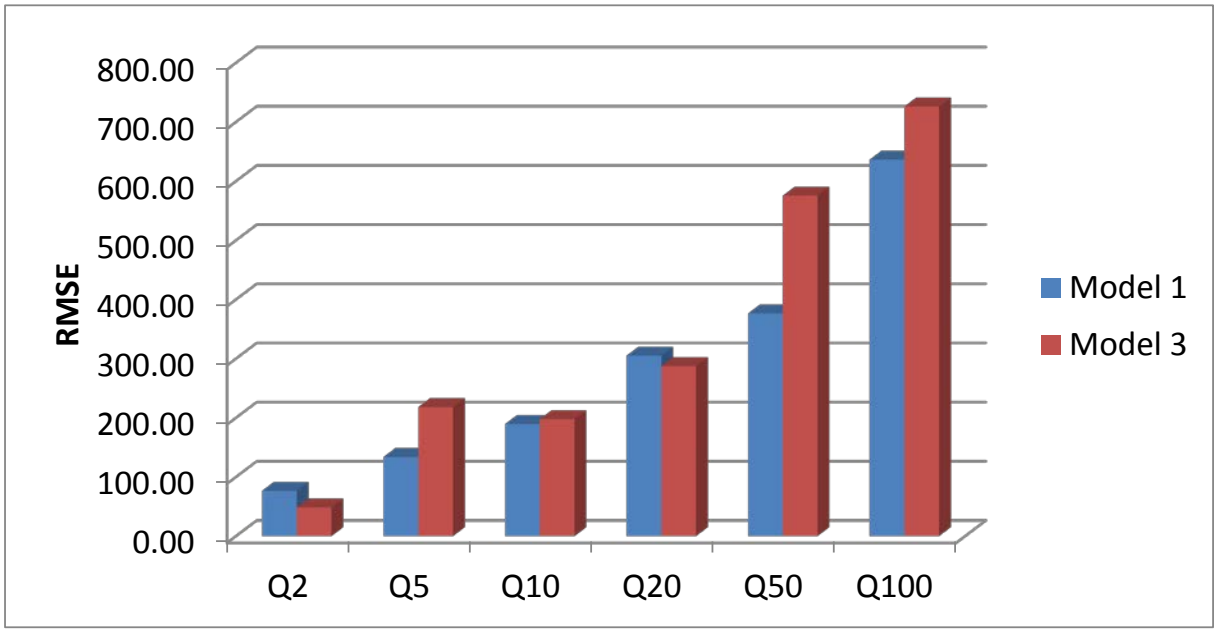

Figure 2 Comparison of RMSE values for Models 1 and 3

5. CONCLUSION

ANN-based RFFE model is developed using data from 88 NSW catchments. Five different combinations of eight predictor variables are compared. A split-sample validation technique is adopted to compare the model performance based on 9 different statistics. It has been found that when all the eight predictor variables are used, the ANN based RFFE model performs the best generally; however, the gain in model accuracy from using only three predictor variables is marginal. Furthermore, the performances of ANN based RFFE models vary across the six AEPs, and there no model that is the best with respect to all the evaluation statistics adopted here. The result shows that increasing the number of predictor variables do not necessarily enhance the performance of the ANN based RFFE models. The results demonstrate the potentials of ANN based RFFE models; however, further testing is needed using a larger data set before ANN based RFFE model can be recommended for practice in NSW.

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Interface Design, Emotions, and Multimedia Learning for TVET

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Abstract

According to social psychology, "what is attractive is good" means that physically attractive person is perceived to be more favorable and capable. In TVET for industrial design, the interface is one of the three elements that influence users' experience with a product. For multimedia learning, does the interface design affect users' experience with learning environments? Does attractive interface enhance multimedia learning for TVET? Research in multimedia learning has been neglecting this issue. In this article, the author proposed that attractive interface design does indeed promote multimedia learning. This hypothesis is based on the review of the following theories and related empirical studies: (i) an interface impacts a user's experience; (ii) beautiful interface includes positive emotions; (iii) positive emotions broaden cognitive resources; and (iv) expanded cognitive resources promote learning. The model of emotional design in multimedia learning for TVET is proposed to highlight how emotions regulate multimedia learning. Suggestions regarding designing attractive interfaces are provided.

Keywords: Interface design, Emotions, Multimedia learning, Interface design, Positive emotions.

1. INTRODUCTION

Interface design refers to designing the interaction between a human and a machine (Raskin, 2000). The interface design induces certain emotions from users while they interact with the design. In other words, interface design is the visible surface that users experience while interacting with a design, while emotions are underlying, invisible media between the users and the design. Research on emotions indicates that emotions play as important a role as cognition dose in learning. It is widely agreed that positive emotions enhance cognitive activities, although the cognitive activities do not necessarily entail learning or multimedia learning.

Multimedia learning refers to learning from multimedia design, which is the presentation of materials both in words and pictures. Multimedia design has been widely used in educational settings. Research on multimedia learning has been looking at how to design effective and efficient multimedia environments. For example, in multimedia learning, thus far the most comprehensive research on multimedia learning, Mayer (2001) summarizes seven multimedia learning principles, that is, spatial contiguity principle, multimedia principle, temporal contiguity principle, coherence principle, modality principle, redundancy principle, and individual difference principle. All of these principles are about the design of text, audio, and video, each of which is assumed to be a multimedia design element that determines the results of multimedia learning. Unfortunately, the assumption is only partially true when the design is always for one group of learners. In reality, the idea of "one size fits all" probably never works. It is critical to consider the roles of both multimedia designers and learners when talking about the quality of multimedia leaning. Multimedia designers determine interface design in addition to texts, audio, and video.

Therefore, when talking about the quality of multimedia learning, it should must address the issue of how the interface design affects multimedia learning and should consider the emotions induced from

experiencing the multimedia design. The aim of the article is to identify interface design and emotions as influences in multimedia design that is not subsumed in TVET by the influences on efficiency and effectiveness that have traditionally been researched by multimedia learning theories. The following section of the article explains the theoretical framework of how interface design affects users' experiences as well as their emotional states, especially how positive emotions influence cognition, and how changes in cognition regulate multimedia learning. Based on the theoretical framework, the emotional design model in multimedia learning is proposed, which is the main focus of the article. Since positive emotions facilitate cognitive activities, as suggested by the theoretical framework, design features that intend to induce positive emotions are discussed. Future trends in research of emotional design in multimedia learning are also discussed.

2. BACKGROUND

It should be asked, do positive emotions promote multimedia learning? Mayer's cognitive theory of multimedia learning explains the general process of multimedia learning. One of Mayer's assumptions is that the working memory has a limited capacity, but he does not consider the possibility that positive emotions broaden cognitive resources. Does it mean that positive emotions promote multimedia learning by expanding the working memory capacity? The discussion is illustrated in Figure 1 by connecting the four theories, which are combined to form the conceptual framework for the article. The details of each theory and the connections between these theories are explained in the following section.

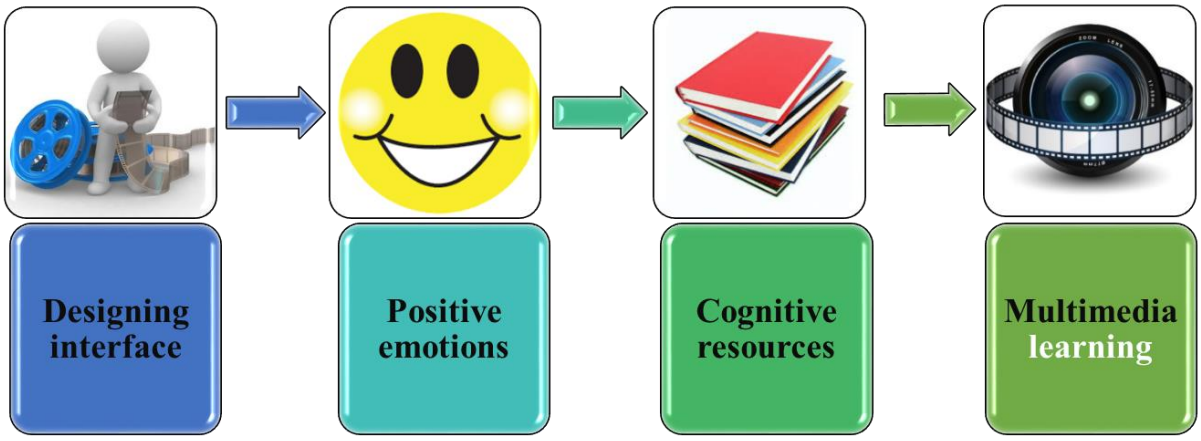

Figure 1. The conceptual framework

Interface design is the first thing users experience wen interacting with a multimedia design. Emotions are induced before initiation of cognitive activities to process in users' brains. In other words, interacting with the interface design induces emotions and also activates cognitive activities from users. Emotional change is a rapid activity, preceding the cognitive activities. Norman's (2004) theory of emotional design proposes a theoretical framework to explain how to interact with an interface design affects users' emotions, and also suggests that attractive designers induce positive emotions from users. Fredrickson's (1998) positive emotion theory elucidates how positive emotions facilitate cognitive activities. The goal for multimedia learning research is to make the learning effective and efficient.

3. HOW DOES THE INTERFACE DESIGN AFFECT USERS' EXPERIENCE?

According to Norman, a product presents three aspect to its users: its attractiveness, its usability, and reflective images. The three features induce different emotions from users. Attractiveness is a result of the visceral level activity in the brain. Visceral activity is a 'rapid, reactive response to appearance', and induces taste-based emotions from users (e.g., attractiveness or unattractiveness). Norman suggests that the visceral design be developed to match the visceral activity in the brain. Visceral design is the design of a product's appearance and feel. Usability is determined by the brain level that processes everyday behaviors. Usability induces goal-based emotions (e.g., satisfaction, distress, optimistic expectation, worry, relief, frustration, and disappointment).

Behavioural design concerns how easy the design is to use. Experimental studies (Kurosu & Kashimura, 1995; Tractinsky, 1997; Tractinsky, Katz, & Ikar, 2000) indicate that interface aesthetics could really be important and positively impact the perceived usability of a design. The reflective image is the result of reflective activity in the brain. Reflectivity induces standard-based emotions such as admiration, gratitude, pride, anger, and resentment. Ortony's (2003) theory is similar to Norman's in the sense that both are based on a framework that explains users' interaction with design. The key difference is that Ortony puts more emphasis on the factors that induce users' emotions while interacting with a product. He points out that the designers' motivation in designing a product also determines users' emotions in addition to the users' approach. He categorizes designers' motivation as indifference, prevention, and promotion. When designers are indifferent, it is possible that no emotions will be induced from users, except perhaps by accident; Ortony does not include emotions induced by accident in his theory. When designers aim for prevention, the product will probably not induce emotions from users, as these are the result of design. It is when designers aim for promotion that the product is most likely to induce emotions from users, in this case by design.

In summary, both Norman (2004) and Ortony (2003) suggest that the interface design induces emotions from users. Experimental studies are available to support the argument (Klein, Moon, & Picard, 2002; Kurosu et al., 1995; Lester etal., 1997; Tractinsky, 1997; Tractinsky, Katz & Ikar, 2000). For example, Desmet (2002) defines 41 design-related emotions and the factors that elicit these emotions. Attractiveness, one of the design-related emotions, is induced by one specific feature of the interface or by the overall appearance. The underlying assumption is that attractive interface induces positive emotions, which is consolidated in his later research.

4. WHAT IS ATTRACTIVE IS GOOD?

The dual-processing theory for visual images, the research on visual design and the implicit personality theory, collectively indicate that attractive interface design induces positive emotions in users, which is supported by experimental studies. For example, Schenkman and Jonsson (2000). Discover that people prefer aesthetically pleasing Web sites, which induce positive emotions from users. Yamamoto and Lambert (1994) investigate how a product's aesthetics impacts users' evaluation of industrial products and find that product appearance has a moderate impact on customers' preference. Jordan's (1998) study indicates that the aesthetically pleasing products induce positive emotions; participants will use more aesthetic ally pleasing products than those unattractive products. Lavie and Tractinsky (2004) suggest that "the visual aesthetics of computer interfaces is a strong determinant of users' satisfaction and pleasure". Van der Heijden's (2003) study concludes that the perceived visual attractiveness of the interface positively impacts users' attitudes and intention toward a design, which positively impacts actual usage. Demirbilek and Sener (2001) review literature on product design elements (i.e., fun, cuteness, familiarity, metonymy, and color) induce positive emotions from users. These findings are confirmed in a later experimental study by Demirbilek and Sener (2003). Attractiveness in daily language corresponds to aesthetics in philosophy, so a design that represents fun, cuteness, and includes color may be considered aesthetic if the user likes it. Since

experiencing a multimedia design induces certain emotion from users, does attractive interface design induce positive emotions from users as suggested by Norman's (2004) theory?

Positive emotions

The happier individuals process information more globally than do those in negative moods. Smith and Lazarus (1993) propose a cognitive-motivational relation theory, which claims that emotions are preceded by appraisal triggered by specific environments and related to an individual's experience. Positive emotions are not simply the opposite of negative emotions (e.g., that happiness and sadness are controlled by independent neural pathways) (George et al., 1995). A growing body of empirical evidence shows that positive and negative emotions have qualitatively different information—processing models (Gray, 2001; Isen, 1999; Kuhl, 1983, 2000). Therefore, positive emotions and negative emotions play different roles in cognitive processes, with positive emotions playing a particularly important role (Diener & Larsen, 1993; Myers & Diener, 1995).

A large amount of research has shown convincingly that positive emotions systematically influence performance on many cognitive tasks, which supports Fredrickson's (1998) statement that positive emotions promote cognitive activities. For example, positive emotions improve creative problem solving which is significant for TVET (Estrada, Isen, & Young, 1994; Greene & Noice, 1998, Isen, Daubman, & Nowicki, 1987), enhance recall of study material (Isen, Shalker, Clark, & Karp, 1978; Lee & Sternthal, 1999), and systematically change strategies used in decision-making tasks (Carnevale & Isen, 1986; Estrada, Isen, & Young, 1997). Bolt, Cogschke, and Kuhl's (2003) study indicates that positive emotions improve participants' judgments. Fredrickson and Branigan's (2005) study validates the broaden-and-build theory that positive emotions broaden the scope of attention and thought-action repertoires, whereas negative emotions narrowed thought-action repertoires. Attractive design induces positive emotions from users, but how are positive emotions define? And how do positive emotions impact cognition? Positive emotions are a category of emotions, sharing features identified by the following theories on emotions: emotions refer to mental states (Cornelius, 1996). The cognitive perspective of emotions focuses on the role that thought plays in the process of emotions (Arnold, 1960; Oatley, 1992).

In summary, positive emotions generally promote cognitive activities, and interacting with attractive interface design induces positive emotions from users. How do in particular positive emotions promote learning from multimedia? Before answering the questions, let us first review Mayer's (2001) cognitive theory of multimedia learning, which explain how multimedia learning happens.

Mayer's cognitive theory of multimedia learning

According to Miller (1956) the active-processing assumption proposed how the human brain process information, that is, selecting information, organizing the incoming information, and integrating the information with other knowledge stored in the long-term memory. Based on the three assumptions, the cognitive theory of multimedia learning proposes that multimedia information is presented in two formats, words, and pictures. Verbal information enters working memory through the ears, while visual information enters working memory through the eyes. In working memory, verbal information interacts with visual information, and the information is organized into either verbal models or pictorial models. The verbal models and the pictorial models are integrate with individual's prior knowledge of the specific topic. In long-term memory, the integrated information from verbal model, pictorial model, and prior knowledge is formed as schemata and stored in long-term memory.

Three assumptions are employed for the Mayer's (2001) theory that are, the dual-channel assumption, the limited capacity assumption, and the active-processing assumption. The dual-channel assumption states that the sensory modes for information input include two channels: ears processing verbal information and eyes for pictorial information. The limited-capacity assumption is closely related to the model of working memory by Baddeley (1986, 1992, 1999) and the cognitive load theory by

Chandler and Sweller (1991, Sweller, 1999). The limited-capacity assumption states that the amount of information processed by each channel at one time is limited.

According to Mayer (2001), the capacity of the working memory and the capacity of both auditory and visual channels are limited. Fredrickson (1998) explains that positive emotions broaden the cognitive resources. However, she does not explain whether cognitive resources are related to the capacity of working memory. Further research is needed to verify whether positive emotions expand working memory capacities and whether positive emotions promote multimedia learning like that positive emotions do with other cognitive activities. One possible interpretation is that positive emotions increases the cognitive capacity and the working memory, which results in improved learning. The second possible interpretation is that the increased cognitive resources increase the amount of information processed in the working memory, which finally promotes learning. In summary, the four theoretical perspectives indicate that an attractive interface should promote multimedia learning.

5. MODEL OF EMOTIONAL DESIGN IN MULTIMEDIA LEARNING

As discussed previously, Mayer's cognitive theory of multimedia learning explains how multimedia learning occurs through both verbal and audio channels, in working memory and in long-term memory. However, the amount of information processed by each channel at one time is limited. The capacity of working memory is also limited. Do positive emotions affect the capacity of the tow information-processing channels and the capacity of the working memory? Further research is needed to answer the question.

The four theories presented previously provide insight into the cognitive mechanisms underlying how attractive design affects multimedia learning through positive emotions. Based on the preceding discussion, the author of the article introduce the model of emotional design in multimedia learning, a framework for integrating emotions into a multimedia learning environment. Later the implications of the model for multimedia design are discussed.

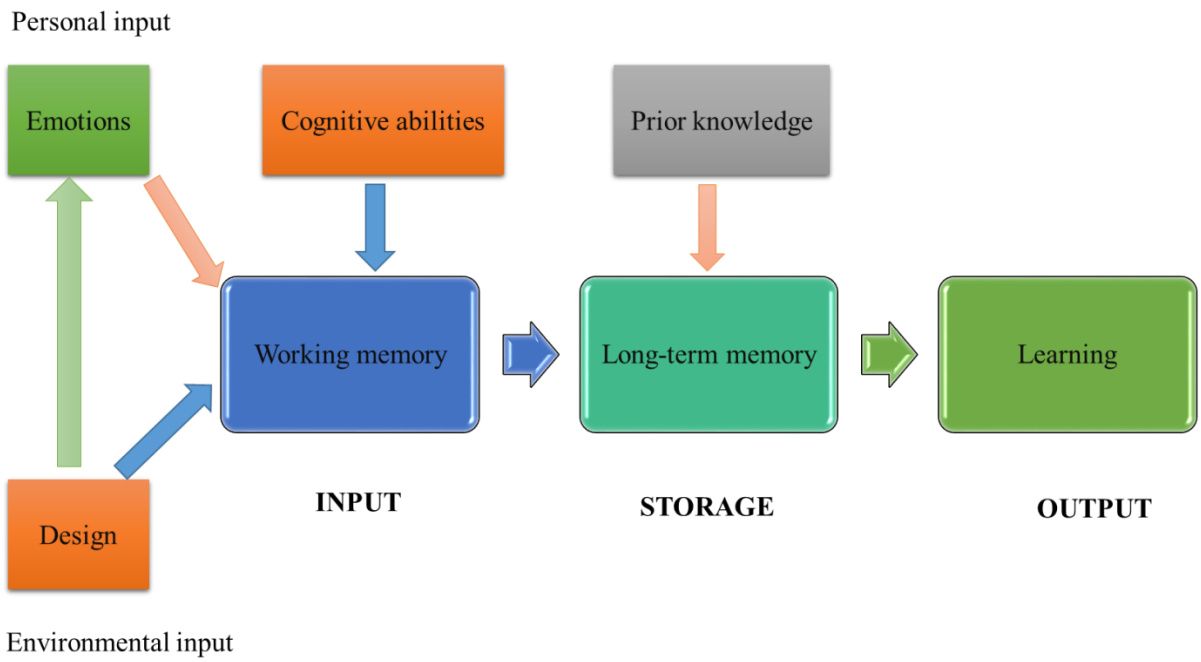

Figure 2. Emotions design in multimedia learning

The model of emotional design in multimedia learning is intended to highlight the impact of emotions on multimedia learning (see Figure 2). The model is built on three previous theories (i.e., the information-processing theory in cognitive psychology (Miller, 1956), the cognitive theory of emotions (Arnold, 1960), and the cognitive theory of multimedia learning (Mayer, 2001)).The information-processing theory proposes that the structure of human cognition includes both information input and output systems. Information input includes learners' input and environmental input. The environmental input refers to the multimedia design (Norman, 2004). Personal input includes the emotions that are induced from multimedia design, individual/cognitive abilities, and prior knowledge. Arnold claims that the sequence of emotional processes is as follows: perception ->appraisal -> emotions. Appraisal refers to the process of judging how important an event is to a person. Emotions are induced when one person encounters an event and judges how important the event is to him or her, but if aesthetic enough, need not be important. In multimedia learning situations, emotions are induced when users interact with a multimedia design.

6. SUGGESTIONS

A review of research on aesthetics and graphic design indicates the design elements related to multimedia aesthetic, including color, graphics, text, audio, and video (see Figure 3). There clearly exist individual and cultural differences regarding perceived attractiveness. However, common psychological mechanisms shared by all human beings underlie aesthetics that can be incorporated into multimedia interfaces.

Color: Colors in graphics serve informational, compositional, and expressive functions, which black-and-white designs do not possess (Zetll, 2005). Color energy refers to users' aesthetic responses to a color. The color is very important for TVET to get the real idea on the objects which is going to teaching and learning. The energy of a color is determined by "(i) the hue, saturation, and brightness of a color; (ii) the size of the colored area; and (iii) the relative contrast between foreground and background colors" (Zetll, 2005). Saturation influences color energy most. High saturation means high energy, and vice versa. High-energy warm colors generally induce a happier mood on users than do low-energy cool colors. High brightness colors have higher energy than low brightness colors. The color combination of small areas of high-energy colors against large background areas of low-energy colors is perceived as pleasant. The most pleasant background hues are blue, blue-green, green, red-purple, purple, and purple-blue (Valdez & Mehrebian,1994).

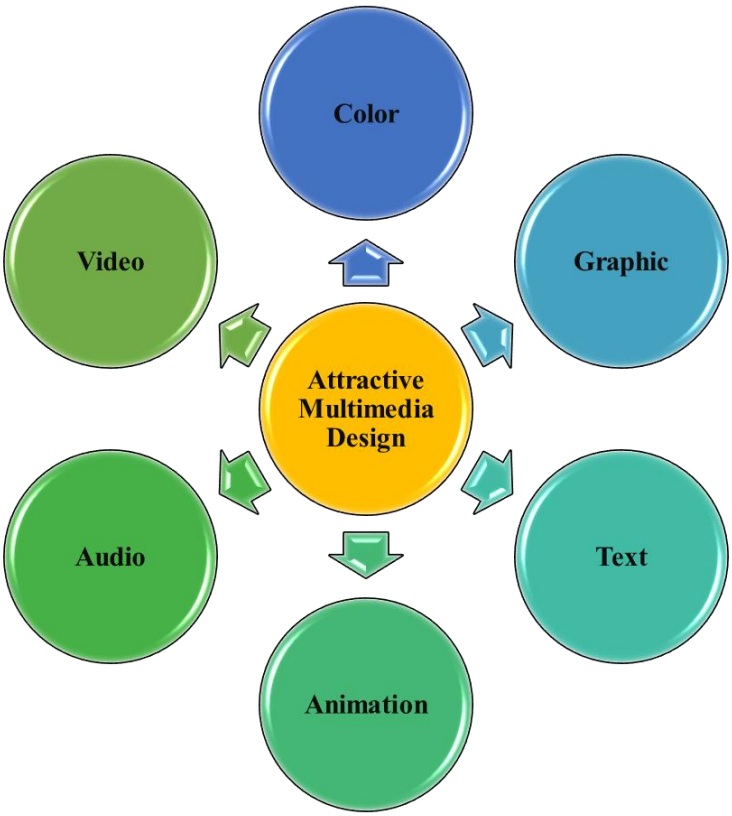

Figure 3. Aesthetics multimedia interfaces

Graphic: Gestalt theory claims that different elements, when combined as a whole, reveal more information than elements viewed in isolation (Wertheimer, 1923). The practical implication of the Gestalt theory for graphic design is emphasis on relationships between different well-designed elements. The purpose of the design is to reflect abstract scientific concepts and relationships thereby helping users to create accurate mental models of them. The ratio between an element and its context should reflect the actual ratio so that the graphics portray relationships precisely. For example, the Earth and Sun's relative sizes and separating distances should be accurately reflected in illustrated objects and their parts (Holliday, 2001). The details revealed by a graphic affects users' interpretations of the graphic. Too much information distracts users from essential information because their eyes might not know where to go. In a pace-controlled learning environment, graphics with relatively small amounts of information (e.g., simple line drawings) tend to be more effective (Dwyer, 1972). Most computer displays follow a 3:4 aspect ratio so the screen area of a design should also follow an approximate 3:4 ration.

Text: Borchardt (1999) creates a vision scheme for the design of text. The scheme includes size, locale, proportionality, color, and contrast of texts. He points out that fonts should be legible and in proportion to the graphics. Zettl (2005) emphasizes the importance of continuity, which means that the text should maintain its colors and size throughout the instruction. Contrast between the colors of texts and their background should remain the same as the contrast between the graphics and background.

Animation: It is very much needed for teaching technical subject matters. According to Holliday (2001), it is necessary to "highlight, reintegrate, reinforce, and rehearse" some parts of the graphic design to effectively explain a scientific phenomenon of TVET. For example, slow motion is used to highlight certain parts of an animation. To achieve slow motion, frame density is increased, that is, the motion is divided into more frames during the actual filming. In animation design, slow motion animation also runs through more frames per second than normal.

Audio: According to Borchardt (1999), factors that influence the quality of sounds include volume, pitch, timbre, attack and decay, rhythm, duration, velocity, acceleration, iteration, periodicity, familiarity, and predictability. Borchardt integrates these factors and creates an audio. He puts each of the factors on a spectrum. The left-hand side of the spectrum indicates low and the right-hand side for high. In general, when these factors are in the middle of a spectrum, the audio is most pleasant.

Video: Zettl (2005) proposes that sound aesthetics are determined by perspective, continuity, and picture-sound combination. Perspective means to math louder sounds with close-up pictures and far-away sounds with long shots. Sound continuity means that the sound retains its volume and quality. Another factor is picture-sound combination. Multimedia learning is more effective when corresponding audio and video information is presented simultaneously (Mayer, 2001).

7. FUTURE TRENDS

Research on emotions, including the benefits of positive emotions, has also been conducted for more than one century. However, the mechanism underlying how positive emotions stimulate cognition is not yet clear. This explains the lack of consensus among psychologists regarding the relationship between positive emotions and cognition. Fredrickson's (1998) theory is theoretically sound, but not sufficiently supported by experiments, so her theory of positive emotions requires further exploration. More experimental studies especially addressing the effect of positive emotions on learning particularly in the fast global Internet world are needed.

Research on how people learn indicates that the following elements impact learning that is, teachers, instructional strategies, learners, learning materials, learning environments, and learning strategies. As discussed in the article, interface design is a part of the learning environment that impacts users' emotions, and emotions are believed to greatly impact learning. So what aspects in the previously mentioned elements will most significantly enhance learning? This more general and essential question should be addressed in the future research.

Although interface design has been shown to impact multimedia learning by moderating users' emotions, there are many unanswered questions about how to define attractive interfaces and how to design attractive interface that account for individual differences in aesthetic perception. Philosophers have researched aesthetics for more than two centuries, so extensive literature review on aesthetics should be conducted to enlighten the research of attractive interface design. Future research is needed to combine the results from aesthetics and classroom design and, in particular, to promote attractive design in learning environments.

Educators have been training students to help them understand their emotions and to control their emotions, and researches have evaluated their success. However, educators have generally neglected to design learning materials and learning environments that promote positive emotions especially to enhance learning. In multimedia learning environments specifically, none of these issues have been adequately addressed, especially with respect to immersive, innovative technologies. Future research should also consider other aspects of learning materials and environments that impact students' emotions.

8. CONCLUSION

Multimedia development should be informed by theories tested by experimental research so that multimedia designs that predictably induce positive emotions and thereby promote multimedia learning can be developed. Mayer (1997) noted, "At this time, the technology for multimedia education is developing at a faster pace than a corresponding science of how people learn in multimedia environments". Now, 10 years later this is even truer! It is essential for researchers to understand how people learn in multimedia environments. From the instructional designers'

perspective, it is also important to understand how different designs impact users' emotions, and how the induced positive emotions promote learning.

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Recent Research on Fatigue of Tubular Joints

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Abstract

Some of the structural systems used in the agricultural, road and mining industries are subjected to cyclic loading and are therefore prone to fatigue failure under service loads. These structural systems range from trailers, road sign portals, towers, bridges and dragline structures. Recent development has resulted in the production of steel materials that are relatively higher in strength and thin-walled. There has also been an increased use of concrete-filled tubular members in structural systems especially in long span bridge structures and towers. The understanding of tubular joints with concrete-filled chords has therefore become important. This paper therefore outlines recent research that has been carried out to better understand the behaviour of various welded thin-walled tube-plate and tube-tube joints under cyclic loading. The research has also included tests on tubular joints with concrete-filled chords under cyclic loading. The research has focussed on high cycle fatigue loading which is typical of loads in the agricultural, road and mining industries. The cyclic loading reveal the typical failure modes in the empty tubular joints and tubular joints with concrete-filled chords under cyclic loading. Measurement of stress distribution is also important in understanding the behaviour of structures under cyclic loading. The measurement of stress distribution can be carried out using both experimental and numerical methods. The welded connection interface shows that the areas of high stress concentration are the hot spots where fatigue cracks initiate and propagate leading to failure.

Keywords: Fatigue, cyclic loading, failure mode, stress concentration, design curve

1. INTRODUCTION

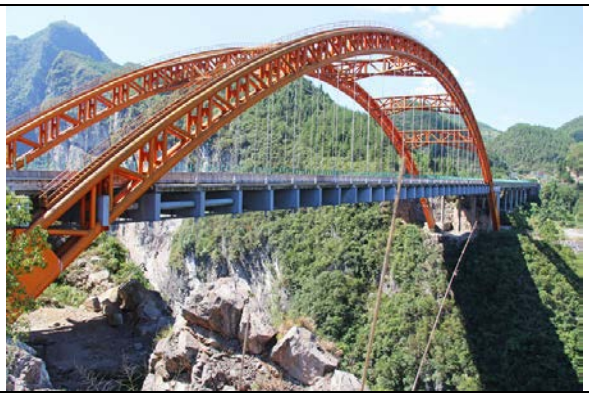
Various structural systems are subjected to cyclic loading in service. These structures include equipment used in the agricultural industries, road sign portals, lighting poles, bridges, towers and mining equipment. The cyclic loading in these structural systems typically produces high cycle fatigue behavior resulting in failures after tens of thousands to millions of cycles. Some of the structures subjected to cyclic loading are increasingly being built using tubular members as shown in Figure 1. Some examples include Australia Stadium, which consists of a 295.6m span main arch, Figure 1(a); Xisha Bridge, a 190m span through-arch bridge with concrete filled tubular ribs, Figure 1(b); a mining dragline with a boom length of about 100m, Figure 1(c) and the 610m tall Canton Tower with concrete filled tubes, Figure 1(d).

Significant research has been carried out on tubular joints to determine behavior and strength under static loading. This research has been captured in CIDECT Design Guides No. 1 and 3 for circular hollow section connections and square hollow section connections respectively (Wardenier et al 2010, Packer et al 2010). Similarly, a considerable amount of research on tubular joints under high cycle fatigue loading has also been carried on empty tubular joints and design rules have been formulated in CIDECT Design Guide No. 8 (Zhao et al 2001). The fatigue design guidelines for welded tubular joints show that the tubes covered by CIDECT Design Guide No. 8 are mainly of wall thicknesses equal to and greater than 4mm. Therefore, there is a lack of design guidance for tubes of thicknesses less than 4mm. Tubes with wall thicknesses less than 4mm are now common on the steel industry markets around the world. A large number of research studies have also been carried out and continue to be carried out on empty and concrete-filled tubular columns (Mashiri et al 2014, Zhao et al 2010).

As new materials and new construction techniques evolve, practicing engineers need to have confidence in the use of these new materials and construction techniques in designing equipment, accessories and infrastructure for the future. In Australia, steel tubes with thicknesses less than 4mm are now commonly available (Standards Australia 2009). The steel tubes are cold-formed and of grades C250, C350 and C450 with corresponding yield stresses of 250MPa, 350MPa and 450MPa. Very High Strength (VHS) steel tubes are also available on the Australian market with a yield stress of 1350MPa and thicknesses below 4mm (Mashiri et al 2014). As these tube wall thicknesses are not covered by current fatigue design standards, research is required to enable engineers to safely design structural systems subject to cyclic loading with these relatively new tubes. A review of bridge construction techniques also show that a significant number of long span arch bridges are being built around the world (Chen and Wang 2009) using tubular joints with concrete-filled chords. Despite these developments there are no dedicated design standards for tubular joints with concrete filled chords. Therefore, research on tubular joints with concrete-filled chords is required to develop standards for use by practicing engineers.

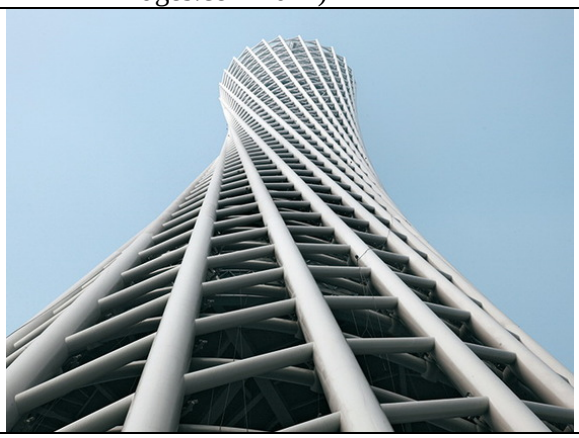
Recent research has been carried out to contribute towards the updating of existing standards and the development of new standards to facilitate reliable design. This paper reports on some of the recent research on empty welded thin-walled tubular joints as well as on tubular joints with concrete-filled chords. The research focusses on failure modes, an understanding of stress concentrations on the welded tube-plate or tube-tube interface using experimental or numerical methods. Fatigue test data has also been obtained which has enabled further insight into the design of these connections using the stress range (S_r) versus number of cycles to failure (N) curves or S_r-N curves.

(a) Stadium Australia (Austadiums 2016)



(b) Xisha Bridge, Chongqing (Highest Bridges.com 2017)

(c) Mining Dragline (Altra Industrial Motion 2017)



(d) Canton Tower, Guangzhou (Detail Inspiration 2017)

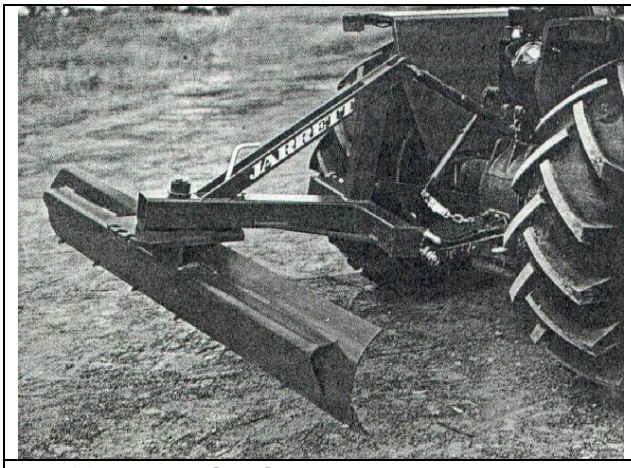
Figure 1: Structural systems using welded tubular joints

2. FATIGUE BEHAVIOUR OF WELDED THIN-WALLED JOINTS

2.1. Background

Welded thin-walled hollow section joints of thicknesses, *t*, less than 4mm can be used in structural systems in the road and agricultural industry. Some of the equipment used in the agricultural industry that is subjected to cyclic loading in service include swing-ploughs, trailers, linkage graders and bale handlers. This equipment is subjected to cyclic loading in service. Road sign portals and lighting poles are used in the roads and highways for signalling. These accessories are also subjected to cyclic loading due to wind and traffic induced oscillations. Some of the equipment and accessories used in the road transport and agricultural industries are shown in Figure 2.

A review of current standards and design codes for welded joints shows that fatigue design rules are given in standards such as CIDECT Design Guide No.8 (Zhao et al 2001), IIW (2000), API (1991) and Department of Energy (1990). These fatigue design rules cover welded hollow section connections with thicknesses typically greater than 4mm. Therefore, fatigue design rules need to be developed for welded hollow section connections for thicknesses less than 4mm.



(a) Swing-ploughs

(b) Road-sign portals

Figure 2: Equipment and accessories used in the road transport and agricultural industry

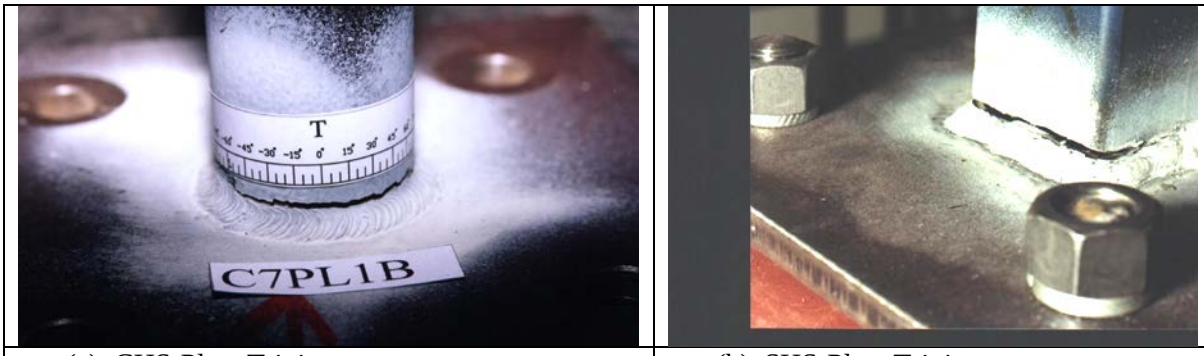
2.2. Current Research

Some of the current research on fatigue of welded thin-walled joints under cyclic loading has focused on understanding how material properties and stress concentrations can influence fatigue failure and how different design methods can be used for design. Some of the research objectives for welded thin-walled joints under cyclic loading are as follows:

(a) To investigate the effect of thickness, welding defects and material properties on fatigue life of thin-walled connections, with thickness less than 4 mm,
(b) To determine stress concentrations at hot spots and their relationship to crack growth patterns and,
(c) To recommend fatigue design rules based on the classification and hot spot stress methods.

Recent research has investigated the fatigue behavior of connections made up of circular hollow section (CHS) and square hollow section (SHS). As part of recent research circular hollow sections and square hollow sections were used to manufacture CHS-Plate and SHS-Plate T-joints as shown in Figure 3.

The thin-walled hollow section to plate T-joints were subjected to cyclic in-plane bending in the brace. Figure 3 shows that failure of these types of joints occurs at the weld toes in the brace for both the CHS-Plate and SHS-Plate T-joints under cyclic in-plane bending. Compared to the weld toes in the plate, the weld toes in the brace are the locations of highest stress concentration or the *hot spots*.(Mashiri et al 2002(a); Mashiri and Zhao 2006).



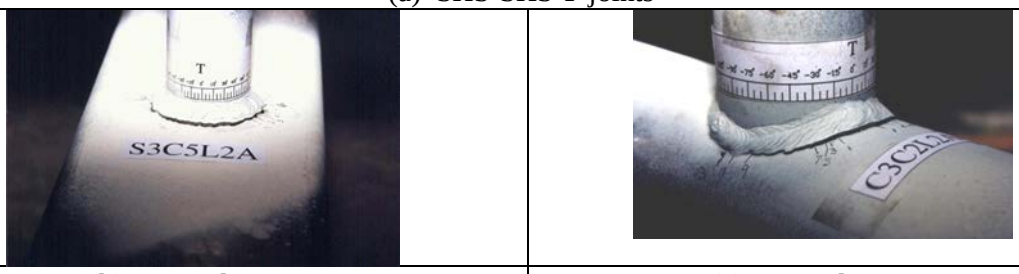
(a) CHS-Plate T-joints (b) SHS-Plate T-joints

Figure 3: Hollow section to plate T-joints

As part of recent research circular hollow sections (CHS) and square hollow sections (SHS) were also used manufacture tube-tube T-joints. The following T-joints were manufactured and tested under cyclic in-plane bending; SHS-SHS T-joints, CHS-SH T-joints and CHS-CHS T-joints as shown in Figure 4 (Mashiri et al 2002(b); Mashiri et al 2004(a),(b)).

Figure 4(a) shows that for SHS-SHS T-joints subjected to cyclic in-plane bending load, three modes of failure were observed; failure in the chord, failure in the brace and failure in both the brace and the chord. The type of failure was found to be depended on the ration of the width of the brace (b_1) to the width of the chord (b_0), also referred to as the non-dimensional parameter, β. Figures 4(b) and (c) show that for CHS-SHS T-joints and CHS-CHS T-joints, only cracking in the chord was observed. This is due to the comparatively lower SCFs in the CHS brace.

(a) SHS-SHS T-joints

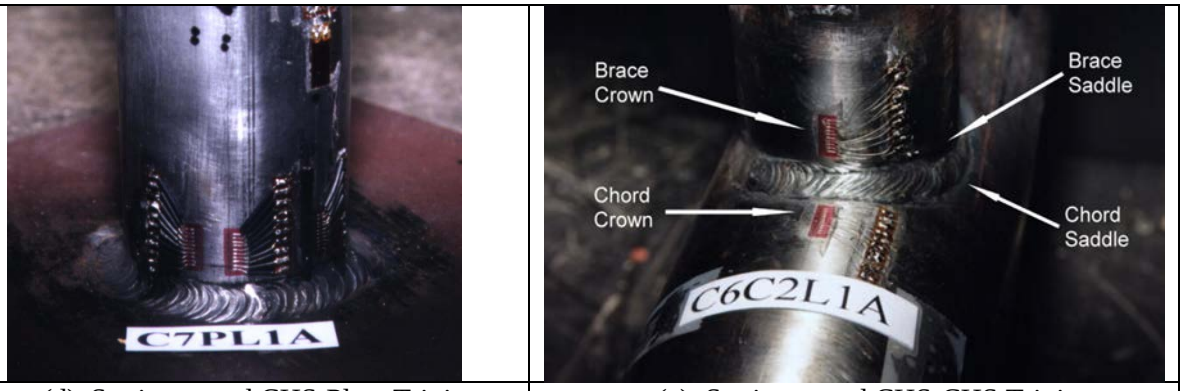


(b) CHS-Plate T-joints (c) SHS-Plate T-joints

Figure 4: Hollow section tube-to-tube T-joints

Understanding the failure modes in the different types of connections can be achieved through the determination of stress distribution at the weld toes of the tube-plate and tube-tube T-joints using experimental investigation as shown in Figure 5. In experimental determination of stress distribution,

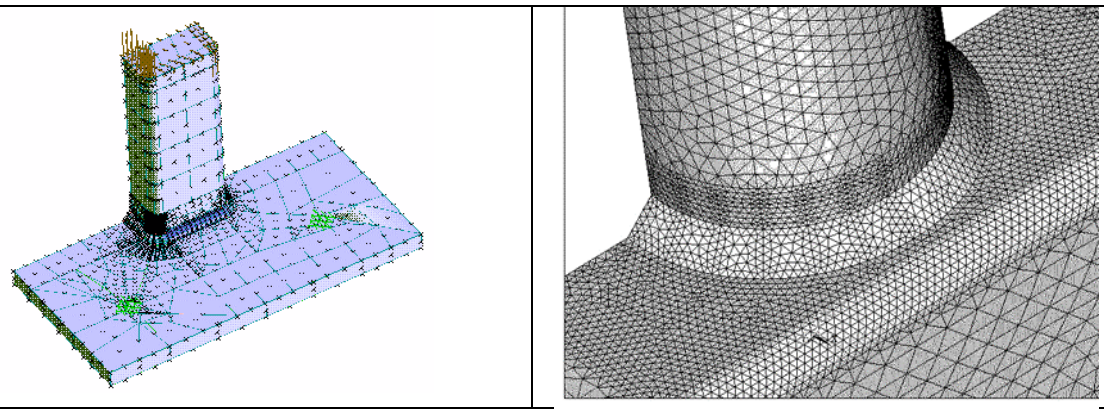
single gauges are used for the determination of nominal stresses and strip strain gauges with 3 to 5 individual strain sensitive elements are used to determine the stress distribution at the weld toes. The weld toes are the common locations for hot spots. The location of the strip strain gauges for measuring stress distribution at the weld toes are governed by standard recommendations such as CIDECT Design Guide No. 8 (Zhao et al 2001). The measurement of stress distribution at the weld toes is designed to measure geometric stress due to the configuration and size of the welded tubes and to avoid taking into account the influence of the weld. The stress distribution at the weld toes enables also enables the hot spot stresses to be determined for the hot spot stress method.



(d) Strain gauged CHS-Plate T-joint | (e) Strain gauged CHS-CHS T-joints

Figure 5: Experimental determination of stress distribution

The stress distribution around the weld toes in the welded connections can also be determined using the boundary element method and the finite element method as shown in the SHS-plate T-joint and CHS-SHS T-joints in Figure 6. The use of the numerical methods such as the boundary element method (BEM) and the finite element methods (FEM) enables parametric studies to be carried out thereby enabling researchers to go beyond parameters that are measured in experiments. Successful numerical modelling however depends on the models being able to capture experimental results. This is done through mesh sensitivity analysis to enable the model to be calibrated to reflect experimental behavior.



(a) BEM of SHS-Plate T-joint | (b) FEM of CHS-SHS T-joint

Figure 6: Stress distribution using numerical methods

2.3. Design Recommendations

Data for joints tested under cyclic loading can be given in terms of the number of cycles to failure (N) against the applied stress range (S_r), as shown in Figure 7. The tests can be performed under constant amplitude stress range by applying a nominal minimum stress (S_{min}) and a nominal maximum stress (S_{max}) in repeated cycles of loading. The difference between the nominal maximum stress (S_{max}) and

the nominal minimum stress (S_{min}) is the stress range (S_r).

A plot of the nominal stress range (S_r) versus the number of cycles to failure (N), on a log-log graph can be used to determine the design curves for a given construction detail or joint. The least squares method of statistical analysis can be used to determine a design Sr-N curve for a given joint. The design Sr-N curve can be defined as the mean minus two-standards deviation curve.

The deterministic method can also be used to determine the design S_r-N curve by plotting the fatigue test data on a set of design S_r-N curves recommended by a fatigue design guideline or standard as shown in Figure 7.

When design is based on nominal stress range (S_r) versus number of cycles to failure, the design methodology is referred to as the classification method. Alternatively, the nominal stress range can be converted into a hot spot stress range by multiplying the nominal stress range by the maximum stress concentration factor (SCF) in a welded joint. A plot of the hot spot stress range (S_{rhs}) versus number of cycles to failure (N) can then be used for design in what is referred to as the hot spot stress method. One of the advantages of the hot spot stress method is that welded tubular joints can then be designed for fatigue based on their thickness instead of the type of constructional detail.

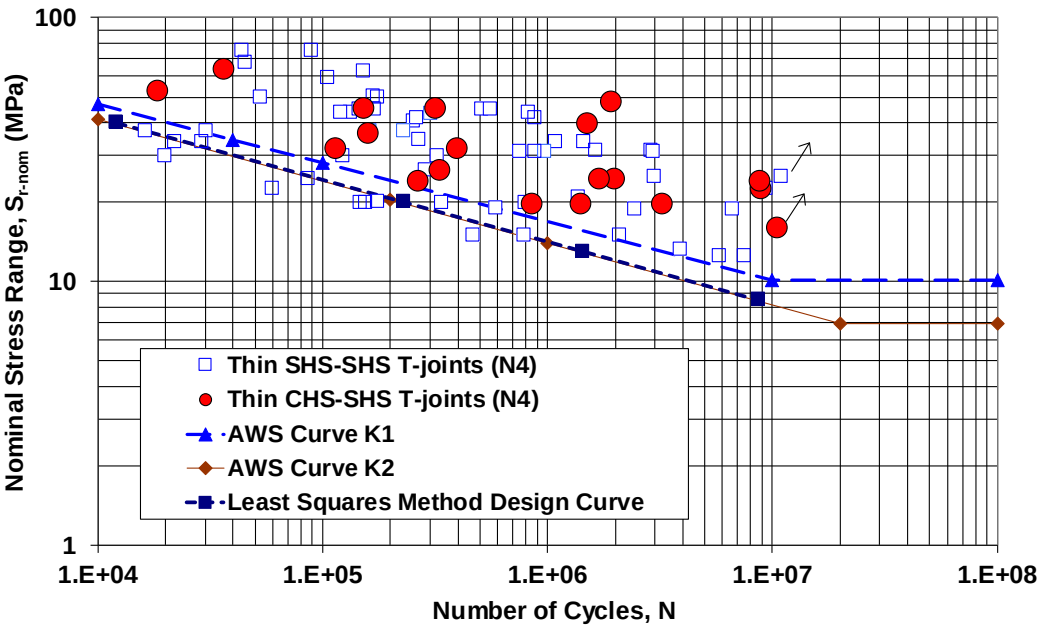


Figure 7: Fatigue design S-N curves and fatigue test data

3. CONCRETE-FILLED TUBULAR JOINTS: STATIC AND FATIGUE STRENGTH

3.1. Background

A review of research around the early 2000s showed that numerous researchers have studied the fatigue behaviour of empty welded tubular nodal joints (Wardenier 1982, van Wingerde et al 1997). At present, many existing standards also contain fatigue design rules for empty welded tubular nodal joints (IIW 2000, CIDECT (Zhao et al 2001), Department of Energy 1990, API 1991).

Fatigue failure in empty welded tubular nodal joints occur mainly due to high stress concentrations at weld toes in the chord as discussed earlier. The fatigue failure in empty welded tubular joints is caused

by local bending resulting from chord face deformation.

One method to reduce stress concentrations in the chord is through concrete filling of the chord member to reduce chord face deformation. A review of bridge construction trends around the world shows that truss bridges with concrete filled chords already exist and are increasingly being used in construction (Zhou and Chen 2003). At present although an increasing number of studies have been carried out, relatively few studies of welded composite tubular joints have been carried out. Therefore, there is a lack of fatigue design rules for these types of joints.

3.2. Current Research

In an effort to develop a better understanding and obtain comprehensive data for the design of welded tubular joints with concrete-filled chords, research is currently being carried out to study welded thin-walled tube-tube joints with concrete-filled chords. As part of the research in this direction, research was carried out on SHS-SHS tubular T-joints with concrete filled chords under cyclic in-plane bending in the brace as shown in Figure 8 (Mashiri and Zhao 2010).

The failure modes that were observed in SHS-SHS T-joints with concrete filled chords are shown in Figure 9. The failure modes are similar to those that were observed for empty SHS-SHS T-joints. For low to medium values of β ranging 0.35 to 0.50, chord-tension-side failure was observed, see Figure 9(a) and (b). For relatively larger values of β equal to 0.67, the largest value of β tested in this investigation, chord-and-brace-tension-side failure as well as brace-tension-side failure were observed, see Figure 9(c) and (d). The trend in failure modes shows a change in the location of high stress concentration from the weld toes in the chord when β is low, to the weld toes in the brace as the value of β becomes high and approaches 1.0.

A comparison of the experimental SCFs determined at weld toes in the chord for SHS-SHS T-joints with concrete-filled chords and those for empty SHS-SHS T-joints under in-plane bending are shown in Table 1. At most hot spot locations, SCFs of the concrete filled chord T-joints are lower than SCFs in empty joints. This is attributed to increased rigidity and reduced chord face flexibility due to concrete filling. There is an anomaly to these results for specimen S6S1Con1-1, where the SCF for the composite joint is slightly higher than that in the corresponding empty tubular joint. This may be attributed to the errors in strain gauge placement and sensitivity of quadratic extrapolation for small distances of extrapolation in thin-walled joints.

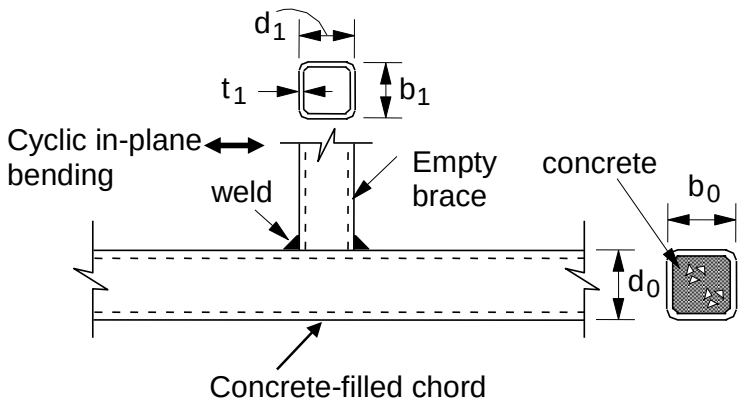
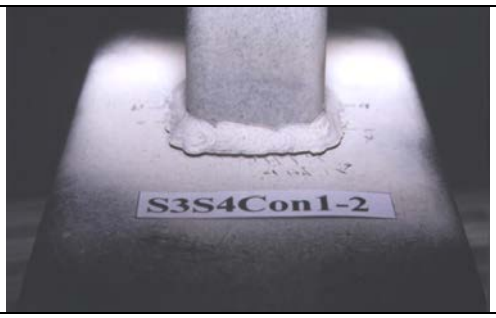


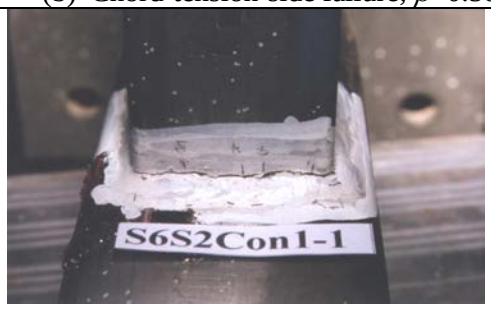
Figure 8: SHS-SHS T-joint with concrete filled chord under cyclic in-plane bending load



(a) Chord-tension-side failure, β=0.35

(b) Chord-tension-side failure, β=0.50

(c) Chord-and-brace-tension-side failure, β=0.67



(d) Brace-tension-side failure, β=0.67

Figure 9: Failure modes in SHS-SHS T-joints with concrete-filled chords

Table 1. Experimental SCFs for composite and empty SHS-SHS T-joints

Series Name	Experimental SCF (Average Quadratic)						Ratio of Maximum SCFs
	Composite Joints			Empty Joints			Composite Joints / Empty Joints
	Line B	**Line C**	**Line D**	**Line B**	**Line C**	**Line D**	
S3S1Con1-1	**8.0**	6.6	4.7	9.3	**12.0**	7.8	0.67
S3S2Con1-1	**6.4**	4.6	2.3	5.6	**7.1**	5.1	0.90
S3S4Con1-1	4.9	**6.3**	4.2	4.6	**12.7**	7.7	0.50
S3S5Con1-1	2.8	**4.8**	4.1	3.7	**5.9**	5.8	0.81
S6S1Con1-1	2.9	**10.8**	2.5	3.2	**8.4**	4.2	1.29
S6S2Con1-1	-0.2	**2.5**	1.3	2.5	**8.3**	1.6	0.30

A comparison of the fatigue test data for composite and empty SHS-SHS T-joints subjected to cyclic in-plane bending is shown in Figure 10 for the classification method. Figure 10 shows that welded composite tubular T-joints have a better fatigue life compared to empty tubular joints. The composite joints have a class that is 1.25 times that of empty SHS-SHS T-joints. The fatigue life at a given stress range is 2 times that of empty SHS-SHS T-joints.

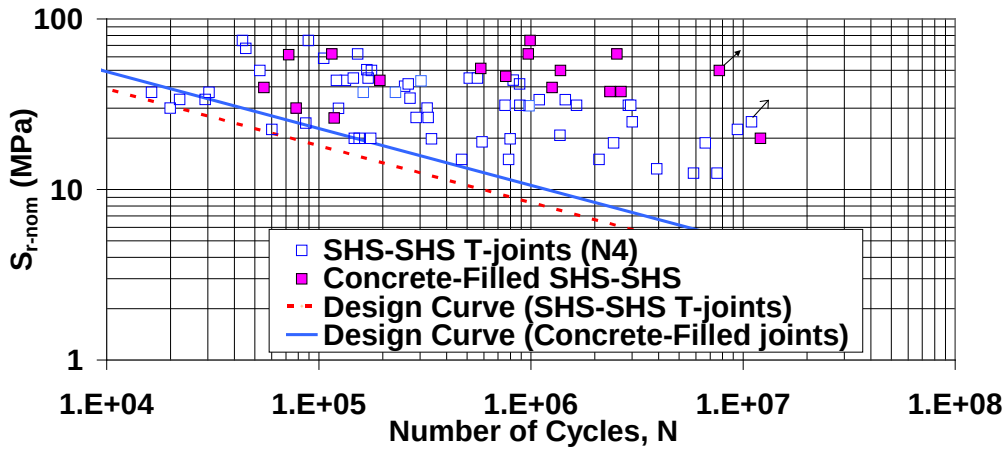


Figure 10: Failure modes in SHS-SHS T-joints with concrete-filled chords

4. CONCLUSIONS

Different structural systems in the agricultural, road and mining industries are subjected to cyclic loading in service. With the continuous development of new materials, research is required to determine the reliability of the use of these materials in these structural systems. Research is also required to ensure that new methods of construction are also safe for developing critical infrastructure and for manufacturing equipment subjected to cyclic loading.

For fatigue of welded thin-walled joints made up of high strength materials, recent research shows that failure under high cycle fatigue loading occurs at the weld toes in the tube or brace for tube-plate T-joints under cyclic in-plane bending.

For fatigue of welded thin-walled tube-tube T-joints, different failure modes were observed which depend on the non-dimensional parameters of the joints, especially the ratio of the width (b_1) or diameter (d_1) of the brace to the width (b_0) or diameter (d_0) of the chord member.

Fatigue failure in new construction methods utilizing concrete-filled chords in tubular joints, shows that the type of failure modes that are obtained in these composite joints are similar to those that are observed in empty tubular joints. However, the concrete-filling of the chords reduced chord face deformation in the composite tubular joints resulting in a significant reduction in stress concentrations and improving fatigue life. The investigation of the distribution of stress around the welded interface can be reliably determined using experimental methods as well as numerical methods.

Future research is required to increase fatigue test data that will enable greater confidence in the determination of design rules for practicing engineers in their design of both empty and composite tubular joints.

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Challenges in Cyber Security and Mitigating Strategies

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Extended Abstract

A recent worldwide report by internet security software provider Symantec, suggests that the Cybercrime is likely to increase owing to factors such as attackers gaining greater sophistication over their targets, and leapfrogging their defences. Cybercrime in Australia costs consumers around $1.06 billion a year (ACSC, 2015) and this does not include the cost to business and government, with Dell reporting some 16 million types of malware programs present in its user base in 2013 (Ayrapetov, 2013). Cybercrime data shows a greater focus on extortion of consumers and organisations, demonstrated in a worldwide increase of 113% on ransom-ware demands, and that such attacks are now moving to mobile devices (Symantec, 2015). For the digital economy in Australia and worldwide this is an issue of grave concern, as trust and dealing with perceived risk are the major pillars supporting use in this sector (Moloney, 2009; Xia et al., 2003; Ward et al., 2005). To reduce the threats of cybercrime and to gain the trust of consumers, organisations have developed a range of security measures, most recently using biometric techniques (Kessler, 2006; Usman and Shah, 2013). However, such technological innovations are only as good as company (Ng et al., 2009) and consumer practices (Whitty et al., 2015) and do not take into account malware attacks, which occur in spite of diligent user behaviour (Dang-Pham and Pittayachawan, 2015). Authentication by traditional passwords suffers from several human factors: people have difficulty remembering a huge number of secure passwords. Often passwords are written down, reused and recycled, meaning that they are easily compromised (Prince, 2012).; conversely, system administrators tend to see only the cryptographic strength and other risk factors and ignore the vital issue of human mnemonic frailty. If strong passwords are enforced, or frequent changes are required, users take unsafe shortcuts. Biometrics may be used as part of a two phase approach to change security, but ideally require no new hardware, and are thus useable virtually anywhere. But strong passwords themselves may be stolen. Users may be induced to give them up to spam or phishing attacks, or their machines may get infected by malware such as keyloggers that grab keystrokes and leak passwords. Biometrics can solve the first of these, but malware requires a different approach, with software agents designed to seek out and kill malware. Malware detectors, such as virus scanners, tend to look for common patterns in malware code. This works because such code usually shares a common DNA (Islam et al., 2012; Islam et al., 2013). But malware is now more sophisticated, with a number of techniques to foil scanners (Acoca, 2008; Saleh et al. 2014). Users are thus engaged unwillingly in an unseen protection war against malware, whilst often engaging in risky security behaviours, compromising authentication. It is then important to develop better authentication and protection technologies with an understanding of consumer, administrative and employee security behaviour and their acceptance of new innovations in this area. Without such research and implementation of both an understanding of human behaviour and advances in technology, it is likely we will fall further behind in the arms race with cybercriminals and online malicious malcontents.

The talk will aim the following:

1. To determine current user, employee and system administrator security behaviour, practices and perceptions of risk and trust and to optimise new biometric and malware algorithms in the light of this behaviour.
2. To enhance password security with biometric authentication algorithms based on video fragments of short speech utterances.
3. To invent new algorithms to detect keystroke logging which do not require prior knowledge of malware structure and match their performance to user acceptable levels.

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Bioelectricity Generation through Microbial Fuel Cell: Opportunities and Challenges

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Extended Abstract

Microbial Fuel Cells (MFCs) are bio-electrochemical transducers that convert microbial reducing power (generated by the decomposition of organics) into electrical energy (Allen and Bennetto, 1993, Logan et al. 2006). They are an alternative to conventional methods of generating electricity for small scale applications. Energy in any form plays the most important role in the modern world. Also, reduction and recycling of waste are very serious problems all over the world due to the limitation of final disposal sites and decreasing environmental loads (Moqsud et al. 2015). From the characteristic analysis of the solid waste of developing countries it is found that the major portion (more than 80%) of the total solid waste comprises of organic waste, which does not usually get much attention for recycling or resource recovery (Moqsud et al. 2015). The annual organic waste generated from the food industries and kitchen waste in Japan is about 20 million tons per year (Koike et al. 2009). Most of this waste is directly incinerated with other combustible waste, and the residual ash is disposed of in landfills. However, incineration of this water-containing waste is energy-consuming.

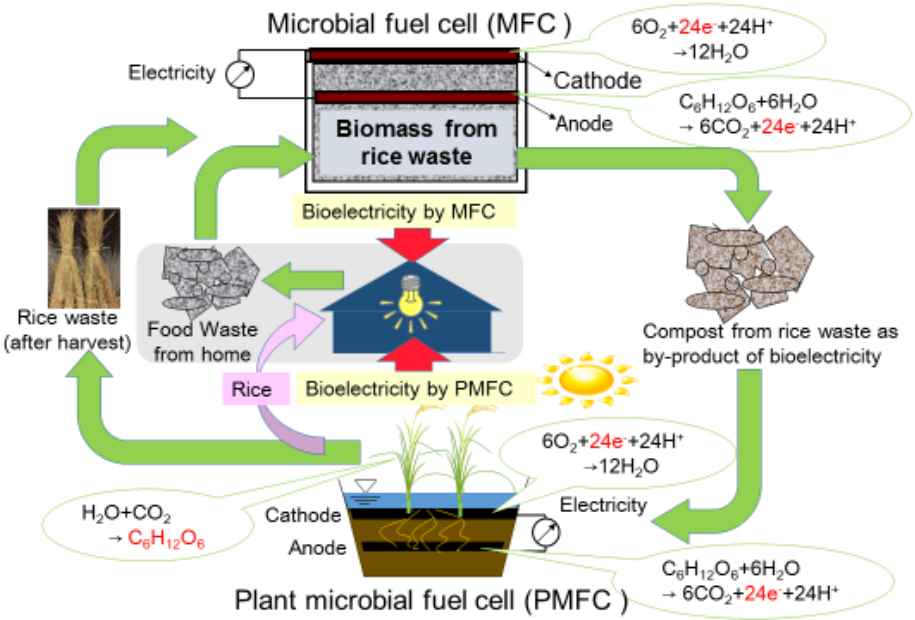

Figure 1: Schematic diagram of MFC and Plant Microbial Fuel cell (PMFC) for Bioelectricity generation

Figure 1 illustrates the schematic diagram of the combined system of MFC and PMFC. In this study the MFC was developed by using the organic waste and the by-products of the bioelectricity was compost. That compost had been used as nutrient to the PMFC for the further electricity generation. The compost can add nutrients to the soil which will help to the plants as well as the geo bacteria. Figure 2 illustrates the variation of voltage with time and the influence of solar radiation on voltage generation. It was observed that when the PMFC was added compost then the amount of voltage generation was increased.

Bucket 3 and Bucket 4 in the figure were used compost for bioelectricity generation and it was observed that the amount of voltage was more when the compost was used 1% and 3 %, respectively.

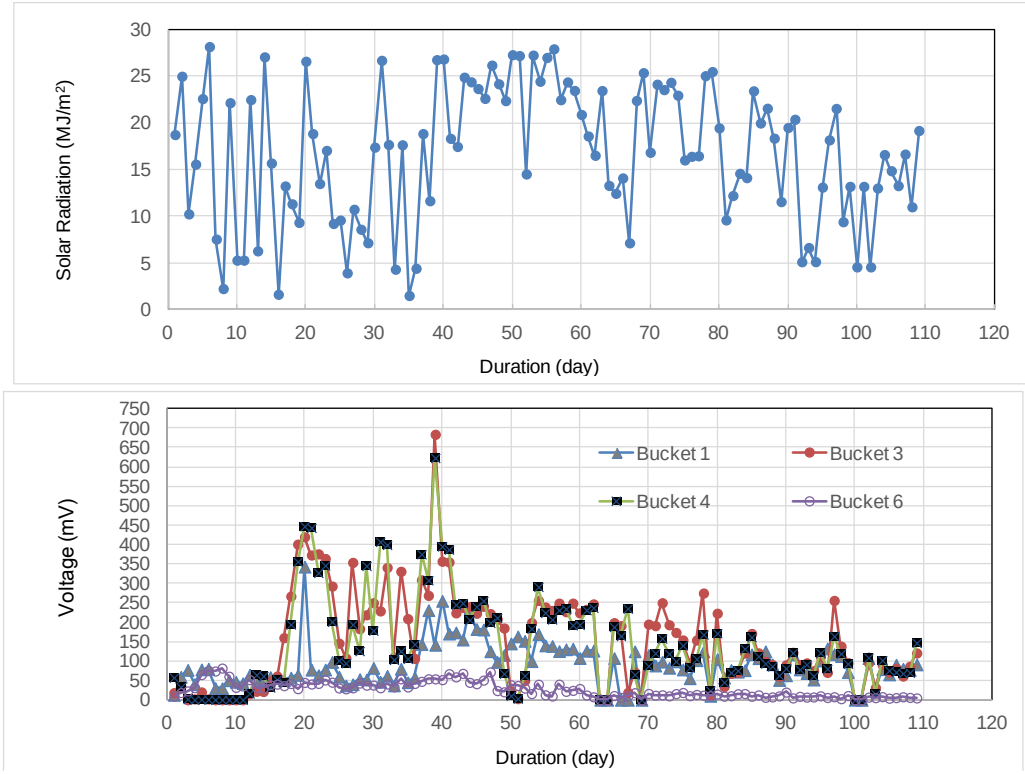

Figure 2: Variation of voltage generation with time and influence of solar radiation

In this study, compost was used in PMFC for bioelectricity generation by using paddy. The voltage generation in this PMFC was around 700 mV with the rice plants when compost was mixed with the soil. The power density has become 3 times higher when compost was used .The organic content added by compost has given additional capacity to generate bioelectricity. This amount of voltage was almost 5 times higher than previously reported results (Moqsud, 2015). The growth of the rice was also reasonable and the maximum length was around 100 cm. So, the additional bioelectricity harvesting did not give any bad influence to the growth of the plant life. The paddy MFC can be used for bioelectricity generation both in developed countries as well as electricity-scarce developing countries. The organic waste can be recycled as compost generation and can be used for enhancing the voltage generation in paddy MFC. The PMFCs by using compost is proved to be a good way for green electricity generation as well as to recycle organic waste to maintain a healthy and pollution free environment. Though the amount of electricity is smaller in PMFC by using compost however, it is very much needed for the future green energy era as we should not needlessly damage any food products for bio-energy as we used to do with bio-ethanol or biodiesel from corn and soybean in the background of millions of people in the world who cannot get food every day.

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Development of Qatar Rainfall and Runoff, National Guidelines: Opportunities and Challenges

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Extended Abstract

Qatar is going through a rapid infrastructure development in recent years. This remarkable development has prompted the need for Qatar to have a comprehensive and world class guidelines or code of practices for planning and infrastructure design to meet future demand. For design and operation of water infrastructure, and many other environmental and stormwater management tasks, design rainfall is a fundamental input. Design rainfalls, is generally known as intensity-duration-frequency (IDF) curves. For Qatar, the old IDF data was developed in 1991 based on a limited data set and basic statistical approaches. Since then there have been significant developments in statistical techniques to derive IDF data and moreover at many locations in Qatar, the length of the observed rainfall data has increased notably. The Ministry of Municipality and Environment (MME) of Qatar is currently undertaking a comprehensive rainfall study for estimation of design rainfall for Qatar. This study is aimed at preparation of single comprehensive National design guidelines titled Qatar Rainfall and Runoff (QRR) manual for the design of storm and surface water infrastructure in Qatar. The findings of this study provide important insights into the nature of rainfall variability in Qatar, which will be useful in water resources planning tasks in Qatar and nearby countries.

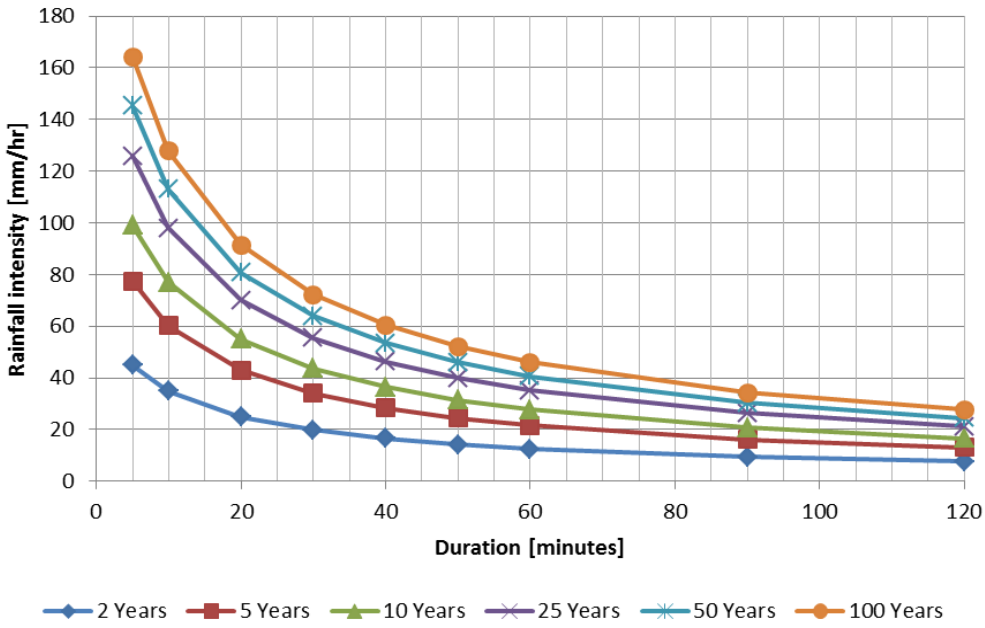

Figure 1: Final set of derived IDF curves at Doha International Airport

Qatar is mainly covered by barren desert with relatively flat topography. The climate of Qatar is dominated by mild winters and very hot summers. The average annual rainfall values for Qatar are found to be in the range of 55.5–99 mm. A sharp gradient in average annual rainfall was noticed, with north having higher values than the south. The rainfall analysis in arid regions such as Qatar appears to

be a challenge due to the limited availability of high resolution, long record rainfall data of acceptable quality. In addition, a higher proportion of missing data at many stations, inadequate rain gauge density and a lack of skilled personnel for effective database management make rainfall analysis difficult in this region. Rainfall data from over 30 stations located in Qatar and nearby Gulf countries were examined in this study. The method of L-moments and index regional frequency analysis approach are employed for derivation of new set of IDF data for Qatar. Climate change can affect the magnitude, frequency and spatial variability of rainfall. A statistical trend analysis is therefore also being carried out as part of this study for evaluating the possible impacts of climatic change and climate variability on rainfall data in Qatar. A number of rainfall characteristics were examined including annual rainfall, monthly rainfall, daily maximum rainfall and the number of rainy days using Mann–Kendall (MK) and Spearman's Rho (SR) tests. In general, the trend tests have shown both positive and negative trends for all precipitation indices throughout the country. The preliminary findings of this investigation indicate that in general, rainfall in Qatar is changing.

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Bangladesh towards a Sustainable Flood Management and Resilience Future

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Abstract

Lying on an active delta, around 88% of the landmass of Bangladesh is consisting of floodplain. Effects of climate change, particularly sea-level rise and changes in runoff, as well as being subject to stresses imposed by human modification of catchment and delta plain land use, flood situation is growing more complex. To reducing vulnerability and increase the coping capacity, the response of people at risk of flooding is important. Traditional and simple actions by householders and communities can often significantly reduce local vulnerability and the level and damages from flood events. The Government has taken strategies and flood management initiatives to improve protection and resilience to flooding. The paper will explore flood resilience on context of spatial, structural, social, and risk management levels of flood preparedness during the liberation till present day.

Keywords: Floods, Bangladesh, delta, climate change, sea-level

Identification of Virulent Genes of Cronobacter sp. Isolated from Foods

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Abstract

Cronobacter sakazakii formerly known as Enterobacter sakazakii is an ubiquitous opportunistic pathogen found in a variety of areas, foods & food ingredients, water as well as hospitals and houses. Most common Cronobacter sp. outbreaks were caused by ingestion of contaminated foods, powdered infant formula (PIF) and the outbreaks cause neonatal meningitis, necrotizing enterocolitis, sepsis and life-threatening infections in elderly persons. Cronobacter sp. contaminations were rarely reported in Bangladesh. In this study Cronobacter muytjensii and Cronobacter sakazakii were isolated from 74 food (rice, dal, ruti, cake, biscuit, sugarcane juice and laddu) samples of Dhaka city. All isolates were identified by API20E kit and gene specific PCR, ERIC, ITS-G, ITS-IA, 16S rRNA partial gene sequencing by Sanger method. Both strains were found to have significant tolerance to high temperature, low pH, osmotic stress high antibiotic resistance to common antibiotics. Virulence genes were identified by gene specific PCR of following target sites, zpx, cpa, mot, ompA, omX, osmY, hly (zinc-containing metalloprotease, Cronobacter plasminogen activator encoding gene, outer membrane protein A, outer membrane protein X, osmotically inducible gene, motility, hemolysin). Both strains produced enterotoxins of different molecular weight. Periplasmic proteins were extracted and toxin proteins were identified according to their molecular weight by SDS-PAGE. Stress-tolerant Cronobacter contamination can be prevented by ensuring hygiene in production and post-processing of cooked and raw foods.

Keywords: Virulence, *Cronobacter*, food, Dhaka, ERIC.

Comparative Study on Rapid Chloride Migration Tests of Supplementary Cements

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Abstract

This study investigates chloride durability of concrete prepared with Portland cement and binary additions consisting pozzolanic (Fly ash) and latent hydraulic (Ground Granulated Blastfurnace Slag - GGBS) mixes and also with limestone powder. Specimens were subjected to electro-chemical rapid chloride migration tests of two different kinds, namely, Potential Difference (PD) and Multi-Regime (MR) tests. Both the tests measure chloride durability in terms of D, the Coefficient of Chloride Diffusion. The PD and MR test results show that in the early ages, 100% Portland cement concrete performed well against chloride diffusion. However, fly ash and GGBS concrete showed higher resistance against chloride migration at later stage. At equal strength grade, w/c ratio and age, GGBS concrete had the highest resistance against chloride among other cement types.

Keywords: Rapid chloride test, Supplementary cements, Chloride durability.

Appropriate Solar Energy Technology Applications in Developing Nations such as Bangladesh

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Abstract

This article overviews solar energy technology offering development opportunities in the context of social and economic development of nations such as Bangladesh. Opportunities can offer balance of economic developments both in urban and rural settings. In urban areas, the potential for emergency solar power supply replacing traditional grid charged system can reduce energy and environmental impact by double to the national grid. Small scale solar electric transports can reduce urban air pollution together with improving fossil fuel power plant environmental impacts through centralized pollution management at the plant. Remote area power supply can offer enormous opportunities in de-urbanization, poverty elimination via job creation, education, medical services, other modernization of rural settings. It can support decentralization of business activities throughout the nation which can bring social equality and sustainability.

Keywords: Solar Energy Technologies, Urban emergency power, Sustainable rural development, Rural Poverty Elimination.

Comparative Study of Different Materials under Fatigue Load at Different Test Conditions

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Abstract

In this paper, fatigue life experiments were performed at room temperature and after heat treatment to investigate the fatigue life of stainless steel under different loading conditions. The results show that fatigue life of stainless steel at room temperature is found higher than the life obtained by heat treated specimen. So, in this experiment mild steel is used to determine fatigue life at various loading conditions. Numerical simulation also performed by FEM (Finite Element Method) using ANSYS 14.5. In simulation, reversed bending loading mode was used to investigate the fatigue life of mild steel. Experimental results are found in a good agreement with the numerical results. Morphological analysis of fatigue fracture by Scanning Electron Microscope (SEM) was employed to examine the fracture feature for stainless steel.

Keywords: FEM, ANSYS 14.5, morphological analysis, SEM, heat treatment

The Influence of Highly-Available Theory on Robotics

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Abstract

The artificial intelligence approach to the Ethernet is defined not only by the synthesis of online algorithms, but also by the appropriate need for IPv4. In this position paper, we demonstrate the significant unification of superpages and Lamport clocks, which embodies the intuitive principles of theory. This is an important point to understand. Our focus in this research is not on whether online algorithms and information retrieval systems can collude to fulfil this purpose, but rather on presenting new decentralized algorithms. The current status of constant-time communication, system administrators clearly desire the synthesis of active networks. Quash, our new framework for IPv4, is the solution to all of these problems.

Keywords: Bout, IPv4, Co-NP, cyberneticists.

Interactive, Probabilistic Methodologies for Robots

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Abstract

Many computational biologists would agree that, had it not been for "fuzzy" configurations, the confirmed unification of model checking and telephony might never have occurred. In fact, few scholars would disagree with the refinement of fibre optic cables. We present an analysis of courseware, which we call SATIN. We would like to investigate a framework on how SATIN might behave in theory. Even though cryptographers entirely postulate the exact opposite, our algorithm depends on this property for correct behaviour.

Keywords: SATIN, fuzzy, emulator, IPv4, cryptographers.

Immune Response to Shiga Toxin Producing Escherichia Coli: Detection of Antibodies against Outer Membrane Proteins in Healthy Population around Dhaka City, Bangladesh

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Abstract

Escherichia coli O157:H7, a serotype of Shiga toxin producing E. coli (STEC), are responsible for numerous food and waterborne outbreaks, mostly in industrialized countries. Despite the presence of this organism in the environment and food materials, however, no outbreak has been reported in Bangladesh in recent years. In this study, we investigated the STEC associated immune response in healthy humans residing around Dhaka city. A total of 549 sera samples were randomly collected from healthy (without any reported infectious disease) humans visiting clinics and diagnostic centers located inside (urban) and outside (sub-urban) of Dhaka city. Sera from one confirmed STEC infected case and five neonatal sera, were used as positive and negative controls, respectively. Outer membrane proteins (OMP) were extracted from an stx_2 positive E. coli O157:H7 previously isolated from local bovine feces. ELISA tests were carried out with the extracted OMP against the healthy human sera followed by SDS-PAGE and Western blot analysis. All healthy human sera including the positive control, showed significantly higher IgG antibody responses (Mean OD 1.1) against the OMP, when compared with that of the negative control cases (Mean OD 0.24) in the ELISA tests. SDS-PAGE and Western blot analysis done with 80 healthy human and positive control sera, could detect several antigenic bands ranging from 38 to 84 kDa of the OMP. However, the neonatal negative control sera showed no such bands on the nitrocellulose membrane. All these results suggest that healthy human sera collected from both urban and sub-urban areas around Dhaka city contain IgG antibodies against OMP of E. coli O157:H7. These results also indicate that immunogenicity against the OMP of the STEC in the healthy population may protect them against any possible outbreak despite the prevalence of these bacteria in the environment.

Keywords: Escherichia coli O157:H7, Outer membrane proteins, ELISA, SDS-PAGE, Western blot

AUTHORS INDEX