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Karachi Centre

ICEC
2017

Conference Proceedings

Striving Towards Resilient Built Environment

Edited By

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9TH INTERNATIONAL CIVIL ENGINEERING CONFRENCE




Jointly Organised by

The Institution of Engineers Pakistan
Karachi Centre

&

NED University of Engineering & Technology
Karachi

in collaboration with

Federation of Engineering Institutions of Islamic Countries (FEIIC)

For a Better Quality of Life
The Asian Civil Engineering Coordinating Council

FEISCA
Federation of Engineering Institutions of South & Central Asia (FEISCA)

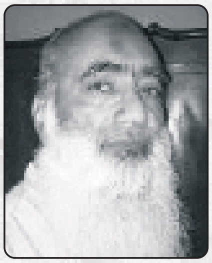
Balochsitan University of Information Technology, Engineering & Management Sciences

Council for Works and Housing Research
Government of Pakistan

Sir Syed University
of Engineering & Technology

"If Pakistan is to take its proper place among the progressive nations of the world, it will have to take up a good deal of leeway in the realm of scientific and technical education which is so necessary for the proper development of the country and the utilization of its resources. The establishment of institution like the Institute of Engineers will greatly stimulate technical research and help in disseminating available information. The Institute of Engineers will not only benefit the engineers themselves by improving their technical knowledge but also bring lasting benefits to public services which they are called upon to perform. I wish the Institute every success"

QUAID-E-AZAM'S message to the first inaugural meeting of the Institute of Engineers Pakistan on 20th June 1948.



Chairman

The Institution of Engineers Pakistan
Karachi Centre

I am indeed very pleased to see that with one day still in hand for completion of one year of the 8th International Civil Engineering Congress, ICEC-2016, held on 23-24 December 2016, you would once again be enjoying the vibrant technical sessions of the 9th International Civil Engineering Conference ICEC-2017 to be held in the same venue on 22nd & 23rd December-2017. This speaks volumes of the sincerity, commitment and zeal with which these thematic congresses/conferences are being under taken and which are becoming a regular feature of IEP Karachi Centre. Indeed the continuously increasing collaboration with NED University of Engineering and Technology, Karachi, is making these events not only an yearly focal point for international academics and researchers, but also developing faith and credibility amongst the international participants who now consider them as credible as any other such event across the globe.

Due to rapid urbanization in almost all the countries in general and in developing countries in particular and alongwith the recent advancement in Science and Technology and its widespread use calls for sustainable build environment. High investment in infrastructure development could only be beneficial if resilient build environment is being focused. While the main aim of the Conference remaining the same i.e. to bring academics and researchers of all areas of Civil Engineering together, however, due to the reasons cited above the theme of the conference have been chosen as "**Striving Towards Resilient Built Environment"** and includes, but not limited to the areas; Structural Engineering , Construction Engineering and Management, Earthquake Engineering , Transportation Engineering, Traffic Simulation and Modeling, Disaster Management, Coastal Engineering, Geo-Technical Engineering , Water Resources Engineering and Environmental Engineering .

The Institution of Engineers Pakistan (IEP) and NED University of Engineering & Technology (NEDUET) are jointly organizing the 9th International Civil Engineering Conference from 22nd & 23rd December, 2017 in collaboration with The Asian Civil Engineering Coordinating Council (AECC), Federation of Engineering Institutions of Islamic Countries (FEIIC) and Federation of Engineering Institutions of South and Central Asia, Sir Syed University of Engineering & Technology (SSUET) and Council for Works and Housing Research (CWHR).

I am sure that the 9th International Civil Engineering Conference shall be attended by large number of delegates including academics, researchers, professional engineers and engineering students from all over the country and abroad and will provide an excellent opportunity for the participants to benefit from the experience of one another and to find solutions to the current problems confronted across the globe. The knowledge transferred by this conference will indeed be helpful for the participants in increasing their professional ability and to find ways and means to tackle the national and international problems.

On behalf of the Institution of Engineers Pakistan, Karachi Centre and Advisory Board of ICEC-2017, I congratulate the Technical Advisory Committee and Management Committee for their untiring and excellent effort. Here I also wish to extend my gratitude for the cooperation extended by NED University of Engineering and Technology to make this event a success

Best wishes for the continuing success of the events organized by IEP in collaboration with NEDUET and good wishes for the successful completion of this Conference.

Engr. Prof. Dr. S. F. A. Rafeeqi, FIE(Pak)
Chairman
The Institution of Engineers Pakistan
Karachi Centre

Vice-Chancellor NEDUET

& Convener 9th ICEC-2017

It gives me an immense pleasure to welcome you to the 9th International Civil Engineering Conference (ICEC-2017) which is jointly organized by The Institution of Engineers Pakistan, Karachi Centre and NED University of Engineering & Technology. This conference provides platform for Researchers, Academia's, Engineers not only from Pakistan but also fromdifferent researchers around the world where they can share the on-going in the field of Civil Engineering as well as look for aspect for future collaboration.

The theme of the Conference i.e. Striving Towards Resilient Built Environment is of great significance to the society. In today's world resilience has been highlighted as the major concern equally for rural and urban settlements. The socio-economic growth and development requires that the societies are established in such a manner that the built environment is capable enough from strategic perspective to cater the challenges of the modern era. Such challenges are in actual providing opportunities to the nations, especially the developing nations, to strive towards resilience. This conference is a key effort in line with the aforementioned context.

I am proud to say that ICEC-2017 is one of that platform where participants can get benefits from each other experiences through knowledge sharing.

I wish all participants a successful congress in the beautiful city of Karachi. The Institution of Engineers Pakistan, Karachi Centre and NED University of Engineering & Technology would feel immense pleasure to welcome you in future congresses as well.

Dr. Sarosh H. Lodi
Vice Chancellor,
NED University of Engineering and Technology, Karachi
Convener, ICEC-2017

Chief Organizer, **9th ICEC-2017**
Member Executive Committee of
The Asian Civil Engineering Coordinating Council,
Federation of Engineering Institutions of Islamic Countries
& Federation of Engineering Institutions of South & Central Asia

As Chief Organizer of the 9th International Civil Engineering Conference jointly organized by The Institution of Engineers Pakistan and NED University of Engineering & Technology in collaboration with The Asian Civil Engineering Coordinating Council, (ACECC), Federation of Engineering Institutions of Islamic Countries (FEIIC), Federation of Engineering Institutions of South & Central Asia (FEISCA), Sir Syed University of Engineering & Technology (SSUET), Balochistan University of Information Technology, Engineering & Management Sciences (BUITEMS) and Council for Works and Housing Research(CWHR) on 22nd & 23rd December, 2017 at Karachi, it is indeed a proud privilege for me to pen a few words on this occasion. The Institution of Engineers Pakistan (IEP), the prime national Institution of Engineers of Pakistan having representation in World Federation of Engineering Organization (WEFO), Federation of Engineering Institutions of South & Central Asia (FEISCA), Federation of Engineering Institutions of Islamic Countries (FEIIC), Commonwealth Engineers Council (CEC) and other international organizations has always endeavored to disseminate the ever expanding knowledge in the various field of engineering to its members through arranging Seminars, Symposiums, Congresses, Workshops, Lectures, etc.

I would like to place on record my profound regards to the members of IEP Local Council, Karachi Centre, Faculty Members of the Civil & Architecture Department - NEDUET particularly Engr. Prof. Dr. S.F.A Rafeeqi, Chairman, IEP, Karachi Centre, Engr. Ayaz Mirza, Secretary, IEP, Karachi Centre, Engr. Prof. Dr. Sarosh H. Lodi, Vice-Chancellor, NEDUET & Convener of 9th–ICEC-2017, Prof. Dr. Asad-ur-Rehman Khan, Chairman, Civil Engineering Department, Prof. Dr. Abdul Jabbar Sangi and Engr. Dr. Farrukh Arif of Civil Engineering Department, NEDUET, Co-Convener of 9th-ICEC-2017.

Today, we are proud to welcome all the distinguished guests, learned speakers and delegates from all over Pakistan and abroad in this 9th ICEC-2017. I, take this opportunity to specially thank our distinguished guest speakers who have spared their valuable time and traveled a long distance to participate in this Conference.

The field of Engineering in the last two decades has achieved tremendous advancement specially, in the developed countries. It is the need of the hour to arrange such forums through which international exchange of technical knowledge took place and the engineers all over the globe may emerge as a unified team to meet the new challenges for sustainable development, economic use of fast depleting natural resources and develop new strategies for future needs specially, in the developing countries.

I am confident that the delegates attending this Conferece will be benefited by the presentation to be made by the experts from all over Pakistan and abroad, and will be able to improve their knowledge in the relevant fields of Civil Engineering.

My sincere gratitude are to Engr. Farhat Adil, Vice-President, (Civil) IEP, Engr. Dr. Izhar ul Haq, President, IEP, Engr. Mian Sultan Mahmood, Secretary General, IEP for extending their help in organizing this 9th –ICEC-2017. I am sure that this Conference will be a great success. I take this opportunity to pay my special thanks to Engr. Prof. Dr. Mir Shabbar Ali, Dean Faculty of Civil Engineering, NEDUET, Mr. Sikander Mannan, Mr. Shareef Khan Qadri, Mr. Saif-ud-Din and all other staff members of IEP Karachi Centre, for extending their full support for organizing 9th-ICEC-2017.

Engr. Sohail Bashir, FIE (Pak)
Chief Organizer, 9th-ICEC-2017
Vice-Chairman (Civil), IEP, Karachi Centre

Secretary
The Institution of Engineers Pakistan
Karachi Centre

As Secretary of the Institution of Engineers Pakistan, Karachi Centre, I take great pride in welcoming all the attendees of the 9th International Civil Engineering Conference on “Striving Towards Resilient Built Environment” scheduled for Friday 22nd & Saturday 23rd December, 2017 at Karachi, jointly organized by The Institution of Engineers Pakistan Karachi Centre and NED University of Engineering and Technology, Karachi in collaboration with The Asian Civil Engineering Coordinating Council (ACECC), Federation of Engineering Institutions of Islamic Countries(FEIIC) and Federation of Engineering Institutions of South & Central Asia (FEISCA), Sir Syed University of Engineering and Technology (SSUET) and Council for Works and Housing Research (CWHR).

Recent natural and human-induced emergencies have highlighted the vulnerability of the built environment. Although most emergency events are not entirely unexpected, and the effects can be mitigated. If a resilient and sustainable built environment is to be achieved, emergency management should be more proactive and receive greater input from the stakeholders responsible for the planning, design, construction and operation of the build environment, the need for emergency management to take a more systematic approach to hazard mitigation by integrating more with professions from the construction sector. In particular, design changes may have to be considered, critical infrastructures must be protected, planning policies should be reviewed, and resilient and sustainable agendas adopted by all stakeholders.

The Institution of Engineers Pakistan was founded in 1948 with the blessing of the father of the Nation, Quaid-e-Azam Muhammad Ali Jinnah with the vision of building bridges of cooperation between the different nations and cultures in Pakistan and abroad. With a clear mission set in front – qualitative and serious education for the young generations, IEP & NEDUET has put outstanding efforts and has organized Eight International Civil Engineering Conferences on various theme with fully dedicated all available capacities to reach a stage when its brand becomes renowned as reputational and prosperous center of education in the all regions of Pakistan and beyond. Besides our serious and comprehensive approach towards education, we broadened our horizons and addressed scientific research and social responsibility with humbleness. IEP & NEDUET are host to various scientific international conferences, international congresses, as well as cultural, social and other educational activities. We are open to cooperation with the public institutions and the business world, as we want to make sure that our Engineer gets endless opportunities to prosper in their careers once they graduate from any accredited Engineering University.

This tremendous achievement is the result of the restless work and sincere and proactive commitment of the Engineering Profession and Engineering Community, our Council Members, Members of the Organizing Committee and particularly Engr. Prof. Dr. S.F.A. Rafeeqi, Chairman, IEP Karachi Centre, Engr. Sohail Bashir, Chief organizer, Engr. Sarosh Hashmat Lodi, Vice-Chancellor, NEDUET, Engr. Dr. Farrukh Arif, Engr. Abdul Jabbar Sangi, the academic and administrative staff, and in particular, our volunteer students who are the focus of our mindset. Each and every one has put a stone on the three golden letters – Build Better World, and fortified the castle of knowledge and wisdom which we call our second home! We thank you all one by one!

As we celebrate this 9th International Civil Engineering Conference at IEP, we bear the highest responsibility to carry on even better and even further, since in our philosophy, there is no limit for quality and development. Under the guidance of Almighty Allah and the people who replace dreams with real work, we will engage ourselves in reaching new peaks ahead and do everything it takes to make IEP & NEDUET a worldwide education brand.

I sincerely hope that this conference will deliberate and discuss all the different facets of this exiting topic and come up with recommendations that will lead to a better, healthier, merrier world.

I am confident that this Conference will be a great success.

Engr. Ayaz Mirza, FIE (Pak)
Secretary
The Institution of Engineers Pakistan
Karachi Centre

Co-Convener

9th ICEC-2017

It is my pleasure to be the Co-Convener of the 9th International Civil Engineering Conference (9th ICEC-2017) organized by The Institution of Engineers Pakistan Karachi Centre and NED University of Engineering & Technology in collaboration with The Asian Civil Engineering Coordinating Council, Federation of Engineering Institutions of Islamic Countries and Federation of Engineering Institutions of South & Central Asia on Friday 22nd & Saturday 23rd December, 2017 in Karachi.

The theme of this year's conference: “Striving towards Resilient Built Environment” has been chosen to focus on the need and efforts required for sustainable, resilient built environment for developing countries, which are faced with immense challenges. The major challenges faced by developing countries are lack of sustainable infrastructure to meet rapid population growth and urbanization. These problems can only be solved by adopting innovative technological approach towards the development process, where civil engineers can play a major role. Incorporating notions of resilience and sustainability into the development process will ensure a built environment that will adequately address the infrastructure needs of the country.

I would like to thank IEP and NED committee members, volunteers, and the authors for their valuable contribution towards the event. I hope that this conference also would lead to meaningful interactions between industry, academia and scientific community, which will enable further research and development.

Engr. Dr. Abdul Jabbar Sangi
Co-Convener, 9th ICEC-2017
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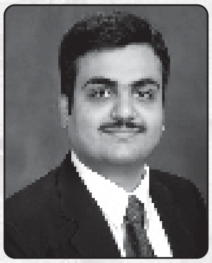

Co-Convener
9th ICEC-2017

I feel honor to welcome the learned authors, delegates and participants to the 9th International Civil Engineers Conference, 22nd and 23rd December, 2017. The event has been organized by The Institution of Engineers Pakistan, Karachi Centre and NED University of Engineering & Technology in collaboration with The Asian Civil Engineering Coordinating Council, Federation of Engineering Institutions of Islamic Counties and Federation of Engineering Institutions of South & Central Asia. Karachi Centre of the Institution of Engineers Pakistan has organized seven such conferences in the past and this is the seventh in the series. I wish all those attending the conference a happy and comfortable stay of two days during the conference.

Engineers have played significant role in the overall development of the Country. The Institution of Engineers Pakistan has helped the engineers in widening their engineering knowledge and techniques by holding various technical activities. Its services to the nation are exemplary. The Institution of Engineers Pakistan has also played vital role by deliberating over the pertinent issues and making appropriate recommendations to the government. The 9th ICEC-2017 is one of the feature events of these continuing development efforts of Institution of engineers Pakistan (IEP). The theme of the conference this year i.e. “Striving Towards Resilient Built Environment” is very pertinent to the future of the engineering and research community specifically that of Pakistan. The challenge to create resilient built environment is in actual the opportunity for the Civil Engineering Community to research and develop techniques and processes, in order to thrive towards the same.

I am sure the 9th International Civil Engineering Conference being attended by engineers from all over Pakistan and from around the world will provide an excellent opportunity to the participants to benefit from the experiences of one another and to find solutions to our current problems. The knowledge transferred by this Conference will be helpful for the participants, in increasing their professional ability and find ways and means to tackle the national and International problems.

The role played by the Institution of Engineers Pakistan is commendable and I wish every success for the Institution.

Dr. Farrukh Arif
Co-Convener, 9th ICEC-2017
Member IEP Local Council, Karachi Centre
NED University of Engineering and Technology, Karachi

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9th International Civil Engineering Conference (ICEC-2017)
"Striving Towards Resilient Built Environment"
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Factors affecting the Safe use of Tower Cranes at a High-Rise Building Project in Macau

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Abstract

Tower crane accident is one of the accident types which can occur on a construction site. These accidents may occur during crane erection, climbing, dismantling operations, maintenance, and testing of tower cranes. To minimize the occurrence of these accidents, it is necessary to adopt proper health and safety (H&S) procedures on construction sites. Factors affecting the safe use of tower cranes on a construction project were identified with literature review which were then analyzed with the help of a questionnaire survey. A high-rise hotel-casino building project completed in 2010 in Macau was used as a case study in this research. H&S policy requirements on this project were accomplished by main contractors who established specific aspects of safety program. The analysis of questionnaires and interview data, and the findings of case study for this project for different affecting factors provided recommendations about greater protection measures and safe workplace for the site workers. The results of this study emphasize towards improving the construction workers' awareness of workplace safety to prevent and avoid jobsite accidents.

Keywords

Safety, Tower cranes, High-Rise Buildings, Construction, Macau

1. Introduction

Tower cranes are commonly used for hoisting and transporting building materials on a construction site, however the related operation can be detrimental as the world crane accident statistics between 2000-2010 revealed that a total of over 1125 Tower Crane accidents happened throughout the world which resulted in over 780 deaths and many countless injuries without mentioning the accidents that have never reported (towercranesupport.com, 2010). In Hong Kong and Macau, several fatal accidents have happened during erection, climbing, dismantling or during operation of tower cranes. For example, on 10 July 2007, a tower crane at a Causeway Bay construction site in Hong Kong collapsed and caused multiple injuries and fatalities (China Daily, 2007). Tam and Fung (2011) have listed nine serious accidents involving tower cranes happened between 1998 and 2007 in Hong Kong. It is generally recognized that tower crane work on a construction site is potentially hazardous (Shepherd et al. 2000).

Damage and injury could be avoided if tower cranes are properly used during erecting, climbing, dismantling and operations. The Hong Kong and Macau government have set up regulations to warn and/or penalize the contractors guilty of tower cranes accidents (e.g. Legislative Council, 2017).

Macau, located approximately 60 kilometers south-west of Hong Kong in the southern part of China, is a major gaming market in the world. As a former Portuguese enclave, it was handed over to China in 1999 and is administrated as the Macau Special Administrative Region. Macau and Hong Kong use the same code of practice for safety regulations for the use of tower cranes. The regulation stipulates that in all construction sites using tower crane, it is mandatory to appoint supervising engineer, a registered professional engineer (RPE), for designing the method statement for all tower cranes and site safety supervisor, a registered safety officer (RSO), to perform safety risk assessment. On work site, foreman and competent workmen must have adequate qualification and good experience for safety checking and operation (e.g. Legislative Council, 2017).

With this background information on the tower crane safety on construction sites, the paper focuses on the identification and assessment of the factors belonging to various categories which affect the safety performance of tower crane. Related to the identified factors, the practices of H&S management system and regulations regarding the tower crane safety in Hong Kong and Macau will also be discussed. The findings of the paper will help to improve the safety related issues of tower cranes on a construction site.

2. Literature review

2.1 Safety Legislations and Regulations

In Hong Kong, Factories and Industrial Undertakings (Lifting Appliances and Lifting Gear) Regulations govern the occupational safety and health issues for tower cranes (Hong Kong Government, 1974). These regulations concern the safety in crane operation, systematic inspection, and testing. Besides, a code of practice by The Occupational Safety and Health Branch of Labour Department in Hong Kong is also available (Labour Department, 2011). The occupational safety and health rules are authorized by Macau Labour Department for projects in Macau. The laws are enforced to prevent illegal works and workers. If workers are without safety training, they are not allowed to enter the construction sites. General contractors who violate these laws will receive heavy penalties.

The Code of Practice on Safety Management in Hong Kong (Labour Department, 2002) contains 14 safety elements which employers must follow based on the following criteria. If a contractor employs 100 or more workers in a day working in a single construction site or the value of the construction works exceeds $100 million, it is required to adopt at least 10 elements of the safety management system. If a contractor employs 50 or more but less than 100 workers in a day working in a single construction site, it is required to adopt 8 elements of the safety management system. A contractor with 50 workers or less is exempted for the time being. The elements of the Code of Practice are safety policy, organizational structure, safety training, in-house safety rules, inspection programme, accident or incident investigation and emergence preparedness which are for general planning, evaluation, selection and control of sub-contractors, safety committees which are for inspection, testing and examinations and job-hazard analysis, safety and health awareness, accident control and hazard elimination and occupational health assurance programme which are for control and evaluation.

2.2 Factors affecting the Safe Use of Tower Cranes

Many factors affecting the tower crane safety can be identified. Beavers et al. (2006) attributed the negligence and misjudgment of participants in tower crane operations as the critical factor. The subcontracting practice is quite common in building construction which also includes tower crane operations. However, it places the safe use of tower cranes at risk as co-ordination, planning, allocation of safety responsibilities, housekeeping and communication among various parties become more challenging than with a single contractor (Tam and Fung, 1998). Tight schedules on construction project pose pressure on tower crane operators to complete the work quickly. Thereby, compromising the safety on the construction site (Leung and Tam, 1999). Shapira and Lyachin (2009) identified the safety training of tower cranes personnel as the important factor for the safe use of tower cranes. The information and training needed in different construction projects should be assessed according to the health and safety plan in the preliminary phase of all projects (Mohamed, 1999).

Maintenance level of the crane and lifting accessories was highlighted by Hinze et al. (1998) as the factor affecting the safe use of cranes. The inspection, examination and testing of tower cranes which can be performed by proper site-level safety management system requiring record-keeping documents and marking system for tower cranes, was discussed by Ng et al. (2005) as the factor to affect the safe use of tower cranes. Researchers have also pointed out the characteristics of superintendents also known as competent person a vital element ensuring the safe use of tower cranes and improving the safety climate on the construction site (Blismas et al. 2005). At the company level, the safety policy is a very important factor to address safety related issues affecting many other aspects including tower cranes (Ng at al. 2005). Operator experience and proficiency is the foremost ingredient in determining safety level of crane work and surroundings. However, age of the operator is generally not considered while gauging an operator’s proficiency (Neitzel et al. 2001). The issue of work stress on the operator has been found to affect the safe use of tower crane operations and is related with poor visibility of the operator (Holt, 2005). The safety promotion can be envisaged to affect safe use of tower cranes (Blismas et al. 2005). Safety related regulations perhaps play a vital role in the crane safety and it has been stressed by Hinze and Bren (1996) in relation with safety issues of cranes with powerline contacts.

In summary, the literature review has raised several substantial key points of safety performance for tower crane. These aspects are further analyzed in this paper.

3. Research Methodology

In this paper, the research methodology explored the underlying issues of descriptive research which includes surveys, case studies and interviews. Literature review findings were used to develop the survey questionnaire and to provide background material for evaluating the survey results. Questionnaire survey was conducted to collect data from a total of 100 workers and supervisors from building construction site mainly on the case study project. Typically, case studies and survey methods are frequently used to collect descriptive data. The case study focused on the legal, regulatory and safety policy related issues of the project. Further elaboration of the survey and case study results was obtained through an interview of an expert working on the case study project. Through the interview, the interviewee shared the experience of tower cranes health and safety issues. In the end, the results were discussed by correlating to previous findings by other researchers to corroborate the importance of the findings.

4. Results and Discussions

4.1 Questionnaire Survey

Most of the questionnaire forms were distributed on the construction site of case study to workers and supervisory staff at all levels. Majority of responders were quite experienced to answer questions in the survey form given to them as they had worked on these projects in the past. Since, it was a survey on a specific site, the response on almost all forms was received. The survey form included twelve questions related to the factors which may affect the safe use of tower cranes on the construction site. The responses were measured on 5-point Likert scale as shown in Table 1. To make objective outcomes of the responses, the responses were assigned with numbers from 5 to 1 corresponding to verbal scales from strongly agree to strongly disagree respectively. The weighted average of the responses was calculated based on these numerical scales and the frequency of each response. For example, the weighted average for factor 1 is calculated as [44(5) + 38(4) + 4(3) + 12(2) + 2(1)]/100 = 4.1

Table 1: Analysis of Factors affecting the Safe use of Tower Cranes

No.	Factor affecting safe use of tower crane	Strongly agree (5)	Agree (4)	Neutral (3)	Disagree (2)	Strongly disagree (1)	Weighted Average	Rank
1	Planned maintenance for tower cranes	44	38	4	12	2	4.1	5
2	Inspection, examination, and testing for tower cranes	60	30	9	1	0	4.49	2
3	Marking and documentation for tower cranes	31	13	32	20	4	3.47	8
4	Competent person supervision	40	8	5	22	25	3.16	10
5	Management and safe system for tower cranes	9	21	2	46	22	2.49	12
6	Age of the staff	40	29	10	20	1	3.87	7
7	Work stress	51	30	0	19	0	4.13	4
8	Safety promotion	45	25	14	7	9	3.9	6
9	Safety training on site	60	33	0	7	0	4.46	3
10	Work experience of the staff	73	19	1	7	0	4.58	1
11	Legislation and regulations	29	21	3	25	22	3.1	11
12	Housekeeping on work area	30	29	3	27	11	3.4	9

The results show that the responders, generally, agreed to most of the factors to affect the safe use of tower crane. The weighted average consolidates the responses in a single number. Higher the weighted average, higher is the importance of the factor for the safe use of tower crane works. The last column of Table 1 shows the rank of the factors based on weighted average. The simple average of all weighted average values is 3.76 which is a score between the factor ranked 8 and 9 out of 12 factors. This means that the distribution of weighted average is slightly tilted towards lower values.

According to the parameters used for the analysis, the five most important factors were (1) work experience of the staff; (2) inspection, examination, and testing of tower cranes; (3) Safety training on site; (4) Work stress; and (5) planned maintenance for tower cranes. The three least important factors were (12) Housekeeping on work area; (11) Legislation and regulation; and (10) Competent person's supervision. The rest of the factors were in the middle which most people considered important.

Most of the results in this study agreed to previous researches. For example, Shapira and Lyachin (2009) found that operator proficiency, site safety management, company safety management, maintenance management and operator characteristics were the factors ranked higher on the importance scale. Shapira and Simcha (2009) applied a more sophisticated Analytic Hierarchy Process (AHP) technique to rank these factors. This resulted into site-level safety management, operator proficiency, wind, superintendent character, crane maintenance management and company level safety management as the most important factors. While some factors matched in importance to the present study, some results diverged. For example, the role of superintendent/ competent person and the absence of wind factor from the present study. A study from Hong Kong with similar conditions as Macau (Tam and Fung, 2011) investigated four major factors to influence the safe use of tower cranes. These factors were (1) negligence or misjudgment of participants in tower crane operations; (2) inadequate training; (3) sub-contracting practices in tower crane operations; and (4) pressure from time constraints. To some extent all these factors are aligned with the present study. For example, safety training was rated higher in the present study. To investigate the practices of H&S management system and safety training approaches used regarding the tower crane safety on a major building project, the case study is presented as follows.

4.2 Case Study

4.2.1 Project brief

A high-rise hotel-casino building project completed in 2010 in Macau was used as a case study in this research. Two major construction companies from Hong Kong operated in a joint-venture as main contractor for a multinational client to perform the design and construction of this project. The project covered a total floor area of 106,000 m^2 of developed floor space. The floor space comprised 60-meter deep foundations and perimeter diaphragm wall, 4-level car-park basement, 41-level-curved tower with 410 luxury suites and 5-level podium for spa, villas, restaurants, retail promenade and additional VIP gaming space. Two tower cranes were used in this project. These cranes provided a maximum lifting capacity of 20 tons per crane. The cranes were mainly used to lift materials and concrete pours for the construction of the hotel tower.

4.2.2 Practice of inspection, examination, and testing for tower cranes on project

In the pre-erection examination, the owner of a tower crane engaged an RPE to examine all critical components of the crane in the depot before it is delivered to a construction site for erection. Moreover, visual inspection of all components of a tower crane and verification of genuine parts and components as specified by the manufacturer were recommended to be included in the examination. Besides, reference was made to the manufacturer's specification as well as the maintenance logbook provided by the owner of the tower crane. Furthermore, non-destructive tests to check the structural defects and mechanical strength of critical parts, testing of electrical components and connection of all switches and safety devices and thorough examination of all accessories such as telescopic cage, tie bolts, hydraulic system, climbing frame and ladders used for the operation were performed.

A Registered Structural Engineer (SE) checked the tower crane set up design and method statement by principal contractor or specialist contractor. Competent person, appointed by principal contractor carried out weekly inspection and completed the necessary forms. An RPE performed audit and monitored the process with the aid of the necessary forms.

The employer complied with the manufacturer's specifications and limitations applicable to the operation of all cranes and derricks. Where manufacturer's specifications were not available, the limitations assigned to the equipment were based on the determinations of a qualified engineer competent in this field and such determinations were appropriately documented and recorded. Attachments used with cranes were not allowed exceed the capacity, rating, or scope recommended by the manufacturer.

4.2.3 Safety training program on site affecting the safe use of tower cranes on project

Behavioral safety programme was adopted. It consisted of staff members' observing colleagues at work, providing feedback on safe and unsafe behavior and discussing improvements within themselves (Lingard and Rowlinson, 1997). A two-day 'observer' training course and one behavioral observation, per shift, at each location, were implemented as part of the initiative. Assessment was done through different categories which included personal protective equipment, housekeeping and plant and equipment safety. Observers included in team-building training improved their communication skills, problem-solving skills, and the ability to work together effectively. The implementing of the behavioral safety programme which included the training section contributed significantly to improve the safety performance of contractors.

4.2.4 Summary of the case study

According to the case study, inspection, examination, testing, and safety training were emphasized as important in affecting the safety performance of tower cranes. The case study thus supported the results of questionnaire survey.

4.3 Interview

To have a better insight into the health and safety management system and to strengthen the questionnaire results, an interview with the professional in the field was conducted. The interviewee was the construction manager on the case study project in Macau. The interviewee was asked similar questions as were in the main questionnaire survey so that more elaborated answers could be sought.

Interviewee pointed out that the most important points affecting the safe use of tower cranes were the high-risk activities, the training and the control of the sub-contractors. He explained that according to his experience, workers in Macau and Hong Kong always ignored the risks of the high-risk activities, as they were not aware of the consequences of the work process.

Moreover, they were influenced by the malpractice of the other workers, particularly the senior ones, who were not only teaching skills to the juniors, but also conveying their malpractices and wrong concepts in tower crane safety. Therefore, training is important to let them understand the hazards and the proper practice, to make them clear about the hazards of the works they performed and their consequences. Hence, it was crucially important to have good control of the sub-contractors. It was because if the sub-contractors did not participate in risk control, the workers could not perform work with proper practice and manner even though they understood the hazards throughout. The interview results are summarized in Table 2a Table 2b where the factors affecting the safe use of tower crane are listed in the decreasing order of importance as determined by the result of questionnaire survey.

Table 2a: Interview Summary - Factors Ranked 1-5

Rank	Factor affecting safe use of tower crane	Response	Comment	Survey Rating
1	Work experience of the staff	Strongly agree (5)	The work experience of crane operators is considered as an essential part of large-scale construction operations.	4.58
2	Inspection, examination, and testing for tower cranes	Strongly agree (5)	The person who conducts the inspection and examination for tower cranes must be qualified personnel with sound knowledge of crane safety usage and crane maintenance.	4.49
3	Safety training on site	Strongly agree (5)	Provision of adequate safety training to site workers and crane operators can reduce the injuries in crane operations. It is a good idea to refresh the workers' safety knowledge from time to time.	4.46
4	Work stress	Agree (4)	The stress from work can be a factor that affects the safe use of tower crane	4.13
5	Planned maintenance for tower cranes	Agree (4)	Ensure that all parts and components of the crane are maintained to the highest standards. Good maintenance keeps cranes in good operating condition i. e. to check for proper lubrication, fluid leaks, batteries, operating mechanisms etc.	4.1

Table 2b: Interview Summary - Factors Ranked 6-12

Rank	Factor affecting safe use of tower crane	Response	Comment	Survey Rating
6	Safety promotion	Agree (4)	Safety promotion can raise the awareness of safety. The site workers and crane operators would be more aware of the potential hazards that are embedded in crane operations	3.9
7	Age of the staff	Disagree (2)	The age of staff would not affect the safe use of tower crane	3.87
8	Marking and documentation for tower cranes	Neutral (3)	No comment given	3.47
9	Housekeeping on work area	Agree (4)	Crane can be operated smoothly with good housekeeping. Also, it provides better and easier surfaces for loading and lifting.	3.4
10	Competent person supervision	Agree (4)	The crane operator must hold a valid license and gain the required qualification from the related association.	3.16
11	Legislation and regulations	Agree (4)	The legislations imposed by the government can ensure the qualification of crane operators.	3.1
12	Management and safe system for tower cranes	Neutral (3)	No comment given	2.49

The opinions from the interviewee as given in the comments were very practical. The results of the interview were consistent with the results of the questionnaire survey for most of the factors notably for top 6 factors. Therefore, the results of all modes of research methods in this paper corroborate each other and the past researches. The notable disagreement is for age of the staff, competent person supervision and legislation and regulation. The disagreement in these three factors may be due to the different perspectives of responders. Some people consider age as an accumulator of experience while other consider ageing as barrier to attaining some types of work skills. The meaning of competent person may also have been misunderstood by the responders. It can have meaning of competent crane operator as well as competent crane supervising person. Similarly, the meaning of legislation and regulations for some may be only the rules which normally are only invoked when some problem occurs. They may not consider those rules for normal problem-free situations. More in-depth research in further exploring these factors is desired.

5. Conclusions

Following conclusions can be drawn based on the studies performed in this paper.

1. Work experience of the staff; inspection, examination, and testing of tower cranes; safety training on site; work stress of operator; planned maintenance of tower cranes; and safety promotion on site were the six most important factors affecting the safe use of tower cranes. The literature review, the case study, and the interview results support this main finding from the questionnaire survey.
2. Marking and documentation of tower cranes; management and safe system for tower cranes; and housekeeping were the factors found to be not much significant.
3. Staff age; legislation and regulations; and the competent person supervision could not be completely evaluated as significant factors as the results from different methods had some disagreement among their importance. The possible reasons for such disagreement have been discussed.

The results of this study may be useful for prioritizing the steps taken to ensure the general safety on a construction site and especially the safe use of tower cranes. This paper provides insights and valuable information for crane installers, operators, and field workers in various ways to minimize the risk of injuries while using tower cranes. Risk management is usually most effective at the early stages such as sufficient training and safety meeting as preventive measures. The list of factors identified and analyzed in this paper is by no means exhaustive. There may be other factors which must also be analyzed to manage the safety issues related to tower cranes.

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Factors Affecting Quality of Highway Projects of Pakistan

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Abstract
Successful projects are so called when the projects are completed within the standard quality, approved budget and on approved time. Unfortunately, like other developing countries construction sector of highway projects are also the substantial issue of quality of highway projects. The aim of this research is to recognize the factors which affect the quality of highway projects of Pakistan. 54 common factors of quality affecting in construction projects were identified by conducting a deep literature review. By conduction questionnaire survey among stakeholders of highway projects. Gathered data from questionnaires were analyzed by average index value the factors affecting the quality of highway projects were incompetency of contractor, poor quality of materials, improper supervision, corruptions, inexperienced staff in consultant and bidding at low cost were identified as major factors affecting the quality of highway projects of Pakistan. This study will help stakeholders to understand the problems of quality in highway projects of Pakistan.

Keywords
Quality, Highway projects, Factors, Pakistan.

1. Introduction

For successful construction projects, achieving the good standard quality is one of the vital objectives of all the stakeholders (Ofori, 2006). Quality can be defined specifications and standards approved and agreed by all stakeholders. It can also be described as the characteristics and features of any production process that bear on ability and capacity to satisfy the stated need (Aoieong, et al., 2002). However, Jha and Iyer (2005) described that it has been very difficult to achieve in practice in construction projects. Developing countries like Pakistan, poor productivity, and poor quality has been found as a major issue in the construction industry. Literature shows on contractor performance in the construction industry, there is need to evaluate critical factors which affect the quality performance in highway projects of Pakistan.

2. Performance Quality of Construction Projects

Construction industry like any other industries is facing different challenges like cost overrun, time overrun and poor quality which affect the overall performance and output of the industry. Identifying critical factors which affect the standard quality performance of construction projects before the initiation of projects will ensure customer satisfaction at the end of a project. Identifying the critical factors in highway projects will not resolve the problem of quality but extent help project team to reduce such factors. A very famous tool knows as Quality Performance (QP) aimed to give an information to find quality improvement standards for good productivity (Abdul, 2011). Performance of international projects is affected by many complex factors rather than local projects. These factors are due to grave external uncertainties like as political influence, economic issues, cultural and social risks (Kim and Park, 2008).

Atkinson et al (1997) described that owner of projects are not satisfied if completed project fails to meet the criteria like cost, standard quality, time frame and standard of delivery performance. According to Enshassi, et al.,(2009) that client, contractors, and consultant believe that a current number of construction projects running at the same time affects the quality performance of construction projects. From literature review for factors which affect quality performance of projects, Jha and Iyer (2005) found factors which were lack of commitment and management to quality improvement, lack of training staff for quality standards, management and leadership, and effective team work in stakeholders. It was also further identified that cost of material and equipment rarely affect the quality performance of projects.

Another study carried Jamaludin,et al., (2014) and identified fraudulent practices and kickbacks, poor planning, competitors in a project; coordination gap between contractor and design, financial control at site, wastage of materials at the site, lack of experience of the contractor in field and many changes in design. Recently another research carried by Mallawaarachchi and Senaratn (2015) recognized factors of poor quality were the shortage of technical staff, lack of resources to perform given a task, lack of commitment of employee, lack of education and lack of quality training.

3. Research Methodology

To achieve the main goal of this research a deep literature review was carried out and total 53 common factors which affect the quality of construction projects. New questionnaire was developed on basis of findings of factors from the literature review and distributed among 103 targeted respondents of highway projects from the client, consultant and contractors. The developed questionnaire was distributed among respondents from five provinces of Pakistan. The developed questions were designed by using Likert scale structured having four scale, that is, 1=Strongly Not agree (SNA); 2=Disagree (D); 3= Slightly agree (SA) and 4=Strongly agree (SA). Average index (AI) was used to calculate the weight of responses of each factor.

In this study average index method has been used to analyze data survey and explained as follows:

The Average Index Method is to analyze the data in the ordinal or rank

Average index method = $\left[\frac{\sum a_i x_i}{5\sum x_i}\right]$

4. Data Collection and Analysis

Total 78 questionnaires were distributed by the person, by using the post or by using google. From distributed 78 questionnaires, 67 received back from respondents. Out of 67 questionnaires, 63 questionnaires were valid for further analysis. Questionnaire distribution is shown in Table.

Table 1: Questionnaire Distribution

Parameter	Score
Distributed questionnaire	78
Received back questionnaire	67
Incomplete questionnaire	04
Valid questionnaire	63
Percentage of valid questionnaire	80

The factors which affecting performance on quality of highway projects in Pakistan are shown in table 2.

Table 2: Major Factor which affects Quality of Highway Projects

S.No	Factors	AI Value	Ranking
01	Poor quality of material	4.213	1
02	Design changes	4.186	2
03	Appointment of inexperienced subcontractors	4.132	3
04	Delays in making decision	4.032	4
05	Assurance of funding from client	4.003	5
06	Poor monitoring	3.921	6
07	Inexperienced consultant staff	3.847	7
08	Shortage of advance equipment's	3.811	8

After analysis of collected data, the factors whose average index value was more than 3.6 were designated as major factors (Sohu et al., 2017) which affect the quality of highway projects in Pakistan. The major factors were the poor quality of materials, design changes, an appointment of inexperienced subcontractors, delays in making the decision, assurance of funding from the client, poor monitoring, inexperienced consultant staff and the shortage of advanced equipment's were identified from expert's opinion after analysis.

4.1. Poor quality of material

Poor quality of the material was identified as a major factor which affects the quality of highway project in Pakistan with 4.213 value of the average index. Quality of material which is approved at the time of tender is not used in the construction site and poor quality of the material is mostly found at the site of construction which leads projects to low quality of highway projects.

4.2. Design changes

Design changes were also identified as a major factor which affects the quality of highway projects of Pakistan. Sudden changes in design can affect the quality because the many changes in one construction activity makes confused to staff which is available at the site. It was also verified by Oke et al.,(2017) that design changes causes and effects on quality of construction projects.

4.3. Appointment of Inexperienced Subcontractors

Appointment of the subcontractor is responsibility of main contractor but when inexperienced subcontractors are appointed by main contractor leads a project to the poor quality of construction at site. Inexperienced sub-contractor have not much knowledge and staff to complete projects as per given standard of quality.

4.4. Delay in Making Decision

Delay in making decision of different activities of projects causes poor and low quality of highway projects. If the decision of any activity is not taken on a particular time the required stuff and materials will be effected and that causes the poor quality of highway projects.

4.5. Assurance of Funding from Client

Contractor plays important role from initial stage to final stage of the project but when the contractor faced problem of funding from client and he is not sure that either he will be paid for good quality of material or equipment's. In such conditions, contractors try to complete the projects from available sources and which affects the quality of the highway projects.

4.6. Poor Monitoring

Many construction activities running at same in each project but for each activity adequate monitoring and controlling is very essential at site, but due to poor monitoring by client in construction site to check the standard quality of each project. Poor monitoring by client affects the overall quality standards of the project.

4.7. Inexperienced Consultant staff

Appointment of inexperienced staff by client leads projects in poor quality standards because appointed staff is not quite experienced to check quality of the construction activities at the site. It is very essential that to maintain the standard quality of the project experienced and competent staff should be hired by consultant.

4.8. Shortage of Advance Equipment's

Many construction activities can be done with standard with latest equipment's but due to shortage and unavailability of advanced equipment's at construction site causes poor quality of work at site. It is necessary that new and advanced type equipment must be hired or purchased by contractors for completion of project in standard quality.

5. Conclusion

The main objective of this study was to identify the main factors which affect the quality of highway projects of Pakistan. A deep literature review was carried out and 53 common factor of quality affects were identified. A questionnaire was designed and distributed among 78 respondents to identify the major factors which affect the quality of highway projects in Pakistan. After analysis of collected data poor quality of materials, design changes, appointment of inexperienced subcontractors, delays in making decision, assurance of funding from client, poor monitoring, inexperienced consultant staff and shortage of advance equipment's were identified as major factors. This study will help stakeholders to control these major factors which affect the quality of Pakistan highway projects.

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Attitudes and Perception of Construction Workforce on Waste Management: The Ghanaian Context

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Abstract

Construction waste has an impact on the performance of the industry and its sustainable goals. Majority of the causes essential material waste are directly or indirectly affected by the behaviour of the construction workforce. Waste occurs on site for some reasons, most of which can be prevented, mainly by changing the attitudes of the construction workforce. Therefore, exploring the attitudes and perceptions of the construction workforce is vital to enable prudent waste management strategies to be generated and implementation. Efficient compliance of this project management practice by a firm within the construction sector has yielded results such as minimization of waste which turns to maximize returns of the client, ensured fruitful communication among the project teams to avoid rework and accident amongst others. The study aimed at evaluating the attitudes and perceptions of the construction workforce involved during the pre- and post-contract stages to minimize waste. Design methodology approach for the study used the quantitative technique. This method proved suitable for this study, due to its consideration in attitudinal measurement based on opinions. In all, twenty (20) structured questionnaires were administered to a targeted population comprising of project managers/site managers, supervisors, labourers, and estimators via purposive sampling. The study revealed that positive perceptions and attitudes of the construction workforce towards minimizing waste would save lots of resources and also enable project targets accomplished. This study concludes that negative attitudes towards subordinates, attitudinal differences between different working groups, and a lack of training to enforce the importance of waste minimization practices have obstructed proper waste management practices in the industry.

Keywords: Attitudes and perceptions, Construction Industry, Employees, Ghana, Waste management.

1. Introduction

Construction output is rapid collective activities in most countries, that results in a corresponding increase in the utilization of natural resources. Holm (1998) argues that approximately 40 percent of the materials produced utilized in building and construction work. Furthermore, the construction industry consumes 25 percent of virgin wood, and 40 percent of the raw stone, gravel, and sand used each year globally. Ganesan (2000) states that materials account for the largest input into construction activities, in the range of 50-60 percent of the total cost. Also, a wide variety of materials used in the construction industry. Unfortunately, this large portion of materials is not utilized efficiently by the industry. Evidence shows that approximately 40 percent of the waste generated globally originates from the construction and demolition of buildings (Holm, 1998) and this forms a major portion of the solid waste discarded in landfills around the world. For instance, in the USA it is approximately 29 percent (Bossink and Brouwers, 1996) and in Australia 44 percent of landfills by mass (McDonald, 1996). Further, research indicates that 9 percent of the total purchased materials end up as waste (by weight) and 1-10 percent of every single material contributes to the solid waste stream of the site (Bossink and Brouwers, 1996). Many researchers have shown that there is a positive correlation between waste prevention and environmental sustainability (Federle, 1993; Lingard et al., 2001). Construction and demolition waste has become a burden to clients, as they have to bear the costs of waste eventually (Skoyles and Skoyles, 1987). The cost of waste blunts the competitive edge of contractors, making their survival more difficult in a competitive environment. Alwi et al. (2002) argue that construction waste can significantly affect the performance and productivity of an organization. Furthermore, the generation of waste is a loss of profits for the contractors due to extra overhead costs, delays and extra work in cleaning, lower productivity, etc. (Skoyles and Skoyles, 1987). Accordingly, this paper reports the findings of a survey carried out in Ghana to evaluate the attitudes and perceptions of construction workforce towards waste management practices.

2. Aim

The study aims to identify the attitudes and perception that influence the construction workforce during pre- and post-contract stages towards waste management practices in the Ghanaian construction industry.

3.Objectives

The following objectives formulated:

To identify the attitudes of contractors during the pre-contract stage (estimators) towards construction waste management practices on various issues and the attitudes of different levels of employees of contractor organizations (site managers, supervisors, skilled and unskilled labourers) regarding issues about construction waste management practices;

To compare and contrast the differential attitudes of employees at the pre-construction stage with employees at the post-construction stage; and

To identify the possible ways of developing waste management practices within the construction industry.

4. Literature review

4.1 Construction wastes

Even though there is widespread recognition across the world of the importance of moving towards sustainability, the construction industry is “notorious for producing huge amounts of construction and demolition waste” (Kwan et al., 2003). The Building Research Establishment (1982, cited in Skoyles and Skoyles, 1987) describes construction waste as the difference between the purchased materials and those used in a project. According to Hong Kong Polytechnic (1993), construction waste is the “by-product generated and removed from construction, renovation, and demolition workplace or sites of the building and civil engineering structures.” Further, construction waste defined as “building and site improvement materials and other solid waste resulting from construction, remodelling, renovation, or repair operations” (Harvard Green Campus Initiative, 2004).

4.2 Origins of construction waste

Construction waste stems from construction, refurbishment, and repair work, and can Emerge at any stage of a project from inception to completion. Generation of the stream of waste is influenced by various factors. Gavilan and Bernold (1994) classify the causes of waste into six categories: (1) design;(2) procurement;(3) double handling of materials;(4) operation;(5) residual-related; and (6)other.

The construction industry is labour-intensive. Thus, activities initiating from the inception to completion of a project are backed up by the human component. It can argue that a majority of the cause of waste is directly or indirectly affected by the attitudes and perceptions of the personnel involved in the construction industry. Accordingly, the human factor involved during the pre-contract stage has a significant influence on the prevention of waste. Ekanayake and Ofori (2000) identified “design change” during the construction project as the most significant cause of the generation of site waste. Awareness of waste generation factors and the attitudes of designers can help to minimize the generation of waste that originates from the “design” cause. For instance, proper identification of a client’s requirements, proper detailing of the documents, etc. can avoid most of the changes during the design stage, thus avoiding the rework which generates waste. Furthermore, the human factor involved during the post-contract stage can influence the minimization of waste in ordering materials according to the appropriate Quantity and quality, the use of proper storage facilities, proper handling of materials, etc. Formoso et al. (1999) argued that the lack of attention of site management to determining waste is a major barrier for the minimisation of waste. Teo and Loosemore (2003) highlighted the inadequate contribution of site managers to the development and implementation of waste management plans. Further, research has shown that the attitudes of construction labourers towards waste minimization activities are negative (Formoso et al., 1999; Alwi et al., 2002).

5. Attitudes of the construction workforce

5.1 Attitudes

Attitudes are an important concept in helping people to understand their social world. They help us to define how we perceive and think about others, as well as how we Behave towards them (Wayne State University, 2004). Judd et al. (1991; cited in Wayne State University, 2004) define attitudes as the “evaluation of various objects that stored in memory.” More simply, an attitude is a defined as a “psychological tendency to evaluate a particular object or situation in a favourable or unfavourable way, which causes someone to behave in a certain way towards it” (Ajzen, 1993; cited in Teo and Loosemore, 2003). Teo and Loosemore (2003), who emphasized the importance of attitudes to those who hold them, as they help people to categorize structure and prioritize the world around them. Thus, attitudes are important to managers as they determine people’s behaviour and provide an insight into their motivating values and beliefs. According to the tri-component model, an attitude includes affect (feeling), cognition (thought), and behaviour (Spooncer,1992).

(1) By changing the environment – Some people say that matters arranged so that people have to behave in a certain manner. Eventually, their attitudes will change in line with that way.. **(2) By changing attitudes** – In the second school of thought, it's stressed that if one could change people’s attitudes, their behaviour would change accordingly. For example, the importance of waste management practices can convey to employees. In practice, considering both points of view is significant, i.e. reuse of materials should be made a rule, as well as better knowledge, is given to employees regarding the importance of waste management practices.

5.2 Attitudes and perceptions of the construction workforce

Waste has been accepting as an inevitable by-product, with a strong belief that waste reduction activities will not be able to eliminate the generation of waste completely (Teo and Loosemore, 2001). These negative perceptions are the main barriers to effective waste management. As the construction industry is labour-intensive, the attitudes and perceptions of people influence its growth. This fact is unquestionably true for the generation and controlling of waste. The importance of attitudes in waste management was identified by Hussey and Skoyles as early as 1974 when they asserted that “it is a change in this attitude rather than a change in technique which is likely to have the most effective overall” (Hussey and Skoyles, 1974). Teo and Loosemore (2001) found that attitudes towards waste reduction have become one of the reasons behind the difficulties of the management of waste in the construction industry.

6. Methodology

A mixed-method technique was adapted. The technique proved suitable for this study, due to its combination consideration of both qualitative and quantitative information which provided attitudinal measurement based on opinions, views, and perceptions. Further, the method also provided means for contacting and receiving first-hand in-depth information from all the sets of respondents to via questionnaire. Project managers/site managers, supervisors, labourers and estimators within the construction industry were the targeted populations operating within the central capital city of Ghana. Face-to-face interpersonal structured questionnaires administered to these set of respondents selected through purposive sampling technique to establish primary data consisting their perception and views on the study. In all twenty (20) structured questionnaires were prepared for these set of respondents within the industry.

7. Results and discussion

Likert scales commonly used in attitudinal measurements (Ryerson University, 2004). Since this research focused on ascertaining the attitudes of the construction workforce, the questionnaires were prepared based on the Likert scale on a five-point scale ranging from, Strongly Agree, Agree, Neutral, Disagree, and Strongly disagree. Data is analyzed using the median and mode of the results where appropriate.

Data gathered through the questionnaires leads to the following findings. The study assessed the opinion of the respondents on the attitudes of contractors during the pre-contract stage (estimators) towards construction waste management practices on various issues and the attitudes of different levels of employees of contractor organisations (site managers, supervisors, skilled and unskilled labourers) regarding issues about construction waste management practices;

Table 1: The attitude and perception of construction workforce

Responses	Frequency	Percentage
Strongly disagree	1	5.0
Disagree	0	0.0
Neutral	2	10.0
Agree	11	55.0
Strongly Agree	6	30.0
Total	20	100.0

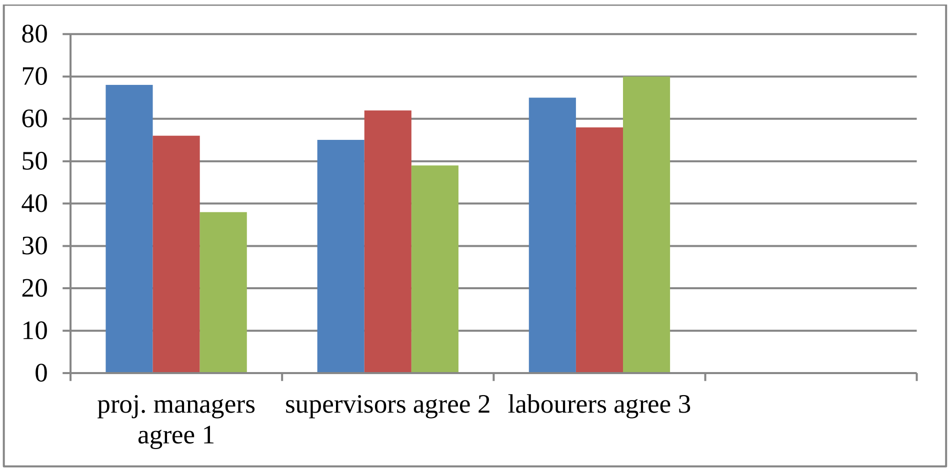
The respondents were asked to express their agreement or disagreement to the statement that the attitude of construction workforce contributes to construction waste generation in the industry. The result in Table 1, shows that 55% of the respondents agreed with the statement, 30% of them strongly agreed, 10% of them were neutral, and 5% strongly disagreed with the statement. This result indicated that in the construction industry workforce at one point in time have something to do with all kinds of materials which imply that attitudinal change towards construction material wastage.

8. Comparison of attitudes of construction estimators and contractors

A difference in the perception of estimators, site managers, and others was identified relating to the performance of organizations in the area of waste management. The majority of site managers (85 percent) believe that their organizations perform well in the area of waste management, while only 55 percent of estimators believe that their organization did well. However, only 65 percent of others believe that their organization has been adhering to waste management strategy to prevent material wastage. It indicates the differences in attitudes and perceptions of different groups of people within the same organization, one group having a negative perception and the other having a positive perception regarding the performance of waste management applications and the existence of an associated company policy. Therefore it can be argued that these contradictory perceptions negatively influence effective waste management practices at the organization level.

9. Developing Waste Management Practices within the Construction Industry

To establish waste management practices within all the levels of working groups; in an organization, proper recognition should give, and it should incorporate into all other operations on the site (Cole, 2000). The importance given by different personnel to waste management assessed as part of this survey. Although the overall attitude to the importance of waste management is highly positive in all the working groups, the attention paid to waste management in actual practice is not so apparent, as highlighted in Figure 1. Indicate the lack of time devoted to waste management practices in the real-life context. It can also identify that



- Waste management is as important as other functions of construction management
- Attention for waste management in the actual practices not sufficient
- Waste management is worthwhile irrespective of the cost gains.

Laborers gave the least attention to waste management practices. The reason behind this is suggested to be the time constraints of the construction industry and the lack of benefits gained by such practices. From the perspective of labourers, few personal benefits gained by adopting waste management practices. As construction work organized in a way to reward fast workers, and in most of the circumstances payments made on piece rate basis, tradespeople are more ready to use a new piece of material rather than spending little more time with cut pieces.

10. Conclusion

The study focused on the attitudes and perceptions of the construction workforce. Minimization of construction waste has emphasized regarding improving performance while achieving the sustainable goals of the industry. Since the construction industry is labour-intensive, the attitudes and perceptions of the workforce affect its growth, and minimization of waste is not an exception. Therefore, a change in the attitudes and perceptions of the construction workforce is vital to gain the maximum benefits from waste management practices. Thus, this research has focused on the Ghanaians construction workforce to evaluate and identify the influence of their attitudes and perceptions on waste management strategies. The research reported in this paper indicates the positive perceptions and attitudes of the construction workforce towards minimizing waste and conserving natural resources. However, the behaviour of the construction workforce in the actual scenario indicates a lack of effort in practising their positive attitudes and perceptions towards waste minimization. Alwi et al. (2002) argue that construction waste can significantly affect the performance and productivity of an organization. Moreover, the generation of waste is a loss of profits for the contractors due to extra overhead costs, delays, and extra work in cleaning, lower productivity, etc. The reasons behind this lack of practice of waste management applications were found to be other priorities during the pre- and post-construction stages, such as profit, time, cost, etc. Also, identify that for the firm, the necessary success factors are client consultation; planning, monitoring and control; and resource allocation compliant to enhance project success.

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Quantifying the Performance of Green Roof from Environmental Standpoint

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Abstract
Understanding the motivation driving green roof use in history expands our learning and understanding of its present uses as to resolve the issues by accomplishing resilient built environment. Green Roof, are additionally called a "rooftop garden" which is a vegetative layer grown on top of any building like commercial, residential, business and so forth to provide shade and expelling heat from the air through evapotranspiration which at last lessens temperatures of the rooftop surface and the encompassing air. These advantages incorporate; storm water mitigation, passive cooling of structures, expanded biodiversity, environmental advantages, sociological advantages, and money saving advantages. Life Cycle Assessment (LCA) is utilized to assess the advantages, from lessened energy consumption, coming about because of the expansion of a green rooftop. Building energy utilize is reproduced and a base up LCA is directed of a Civil Department building, NED University of Engineering and Technology, Karachi. The key property of a green rooftop is its low solar absorption, which causes lower surface temperature, eventually reducing heat flux through the roof ultimately sparing the cost.

Keywords
Life Cycle Assessment (LCA), urban heat island, cooling capacity, life cycle inventory, heat flux.

1. Introduction

A green roof is a rooftop with a vegetative cover which is one passive system that can be utilized to solve environmental issues in an urban setting. Green roofs are effective to mitigate issues related with storm water runoff, the urban heat island impact, air and water quality and untamed life environment. The reason of building proprietors to be reluctant in considering the utilization of a green roof is due to its increased initial expenses and vulnerabilities in the development, construction and upkeep maintenance of such roofs. In one of only a handful couple of concentrates that consider the life cycle of green roofs, it was discovered that the life cycle cost of extensive, i.e., shallow soil rooftops, are not as much as conventional rooftops but in comparison intensive roofs also known as deep soil roofs frameworks have a higher life cycle cost (LCC) than ordinary practice. In any case, these are not by any means the only central factors in picking a rooftop framework for building. For the duration of the life of a building, many environ-mental expenses will be brought about also. A watchful examination of the greater part of the environmental aspects of building and keeping up either a green or regular rooftop will be utilized to build up a life cycle assessment (LCA). LCA can be helpful to decide the ecological and cost benefits of green roofs.

The purpose for this paper is the environmental assessment about extensive green roofs throughout its life cycle (cradle will grave). To attain the objective, technique of life cycle Assessment (LCA) in a similar methodology of existing building is obtained to compare the difference between conventional and green rooftop system. The estimated service life is 90 years (Alves, November 2015).

1.1 Literature Review

There are 3 forms of Green roofs, extensive, intensive and semi-intensive. Extensive green roof supports between 50 and 150 mm of growing medium which is helpful for plant life. The weight of the green roof on the building structure is thus limit by the limiting size of plants used on the roof. For the most part foot traffic may be not permitted for extensive green roofs due to the shallow and delicate root framework of the vegetation. Normally this sort of green roofs is retro-fitted to existing buildings, minimizing the replacement of existing roof's structural supports. Intensive green roofs generally have 150 on 1200 mm of growing medium, which will help bigger plant life. Larger bushes and even trees can be planted on intensive green roofs. Foot traffic is however permitted for intensive green roofs and also adds value to the building structure by aesthetic means. However, this imposes an extensive dead and live weight load on the top of the building structure that obliges extra structural backing. Fig. 1 indicates to a typical cross-segment of a green roof. Same pattern of layers is generally adopted in both intensive and extensive green roofs (Lisa Kosareo, 12 June 2006).

Green Roofs are helpful to maintain the temperature as the surface temperature of green roofs can be cooler in comparison to air temperature in hot summer days where as the conventional roofs can be up to 90°F (50°C) hotter. Green roofs have been underway for over 100 years and now have become the key element in urban areas as these are valuable because of its cooling performances, efficiency and survival rates of plants. A standout amongst those key components for Green roofs is the vegetation layer on the top of building which incorporates certain variables to consider appropriate planting patterns for Green roof including dry season tolerant, sunlight based radiation tolerant and also cooling capacity from claiming plants (Lisa Kosareo, 12 June 2006).

1.2 Types of Green Roof

There are 3 major types of Green Roof systems as indicated:

1.2.1 Extensive green roofs

These Green roofs are light in weight which usually does not require much reinforcement and suitable for large areas and also for roofs with 0 - 30° slopes. These roofs are suitable for retrofit projects and look more natural having a thin soil layer and feature succulent plants like sedums that can survive in harsh conditions.

1.2.2 Semi-intensive green roofs

These roofs have greater diversity of plants and habitats along with good insulation properties and are energy efficient. They also have storm water retention capability with longer membrane life.

1.2.3. Intensive green roofs

These are more energy efficient with worthy storm water retention capability having a longer membrane life. These roofs are advantageous to simulate a wildlife garden on the roof that can attract visually. These roofs can often possibly use with more diverse functions i.e for recreation, growing food and as open space (Mohammed, 2006).

Figure 1: Three Types of Green Roofs

1.3. Benefits of green roof

Green roofs need higher beginning expenses over conventional roofing, green roofs have a different exhibit for possibility profits (Erica Oberndorfer, 2007) such as:

- Diminishing building cooling loads towards keeping abundance heat from entering buildings.
- Mitigating the urban heat island at suitable scales and density by providing a medium that uses additional heat to create water vapor.
- Reducing storm-water runoff by retaining precipitation.
- Sequestering carbon dioxide and pollutants in biomass.
- Refining aesthetic values or providing recreational benefits.
- Creating wildlife habitat; and Providing noise reduction in buildings

2. Life Cycle Assessment Methodology Framework

The LCA methodology adapted for this study is depicted through Figure 2. The detailed inputs/analysis for the particular case of green roof are provided in subsections.

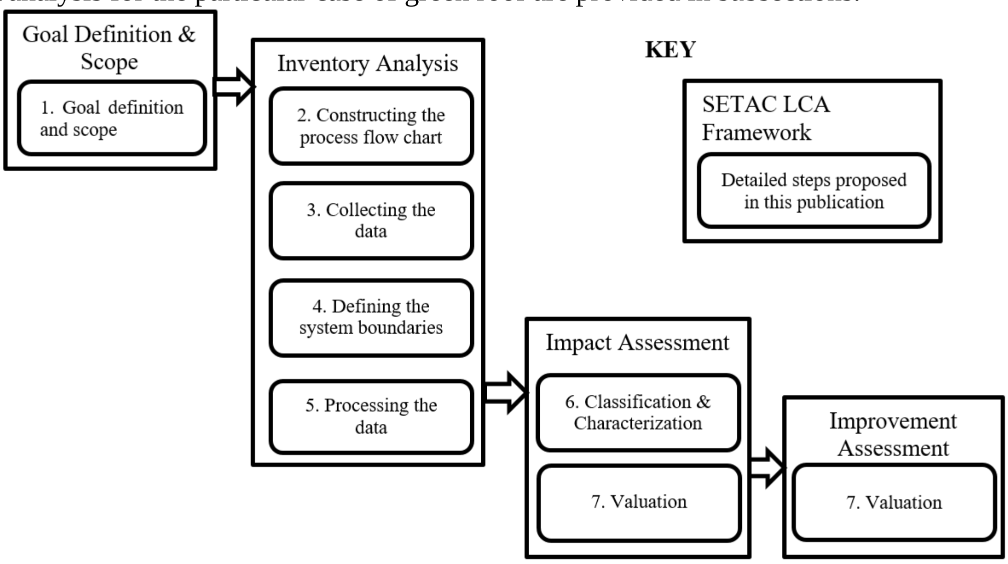

Figure 2: The Framework for Life Cycle Assessment

(Diagram adapted from: "Life Cycle Assessment: What It Is and How to Do It" Published by the United Nations Environmental Programme)

2.1 Goal Definition and Scope

The scope of this LCA is to compare the environmental aspects and potential impacts associated with constructing, maintaining, and disposing a green roof and determining the option with the lowest negative impact. In other words, the intention of this study is to find the environmentally preferable choice between an extensive green roof, and a conventional roof.

2.2 Inventory Analysis

Figure 3 provides process flowchart with reference to inventory analysis step for the green roofing system.

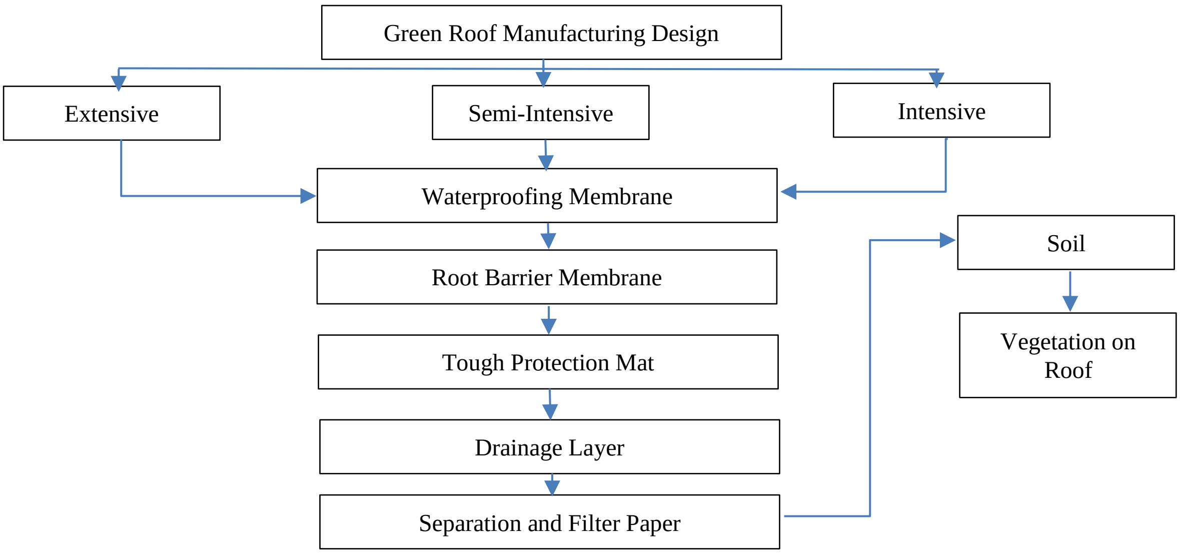

Figure 3: Process Flow chart

2.2.1 Collection of data

Table 1 provides the data collected for the case study roof, i.e. NED University New Block (N) Block class rooms' roof top.

Table 1: Collection of Data

Location	NED University (N Block Building- Room N8)
Green Roof material	Extensive
Area	15 ft by 40 ft = 600 sft
Temperature	37 Degree Celsius

2.2.2 System Boundaries

The system boundaries have then been defined for the LCA as shown in Figure 4.

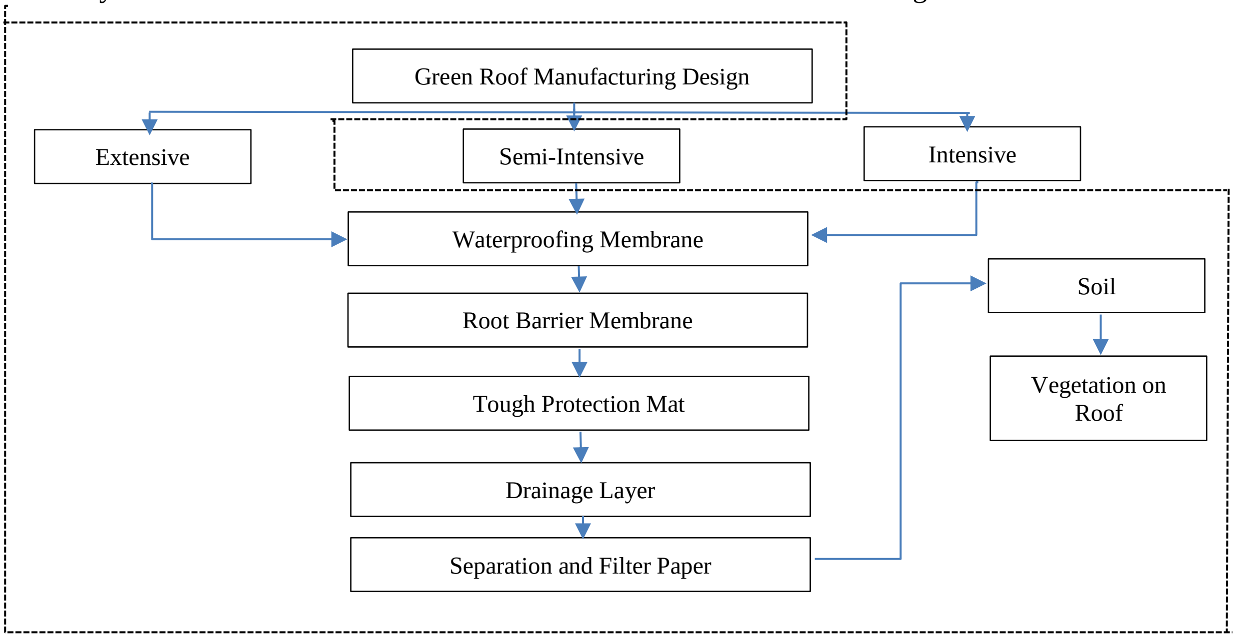

Figure 4: System Boundaries

2.2.3 Processing the data

Figure 5 provides a cross section of the possible green roof replacement for the case study roof, as per data collection, system boundaries and requirements. Moreover, Table 2 provides material requirement as per details of each layer shown in the cross section. Table 3 provides to the (assumed) Values of Resources and Externalities as per market and published literature.

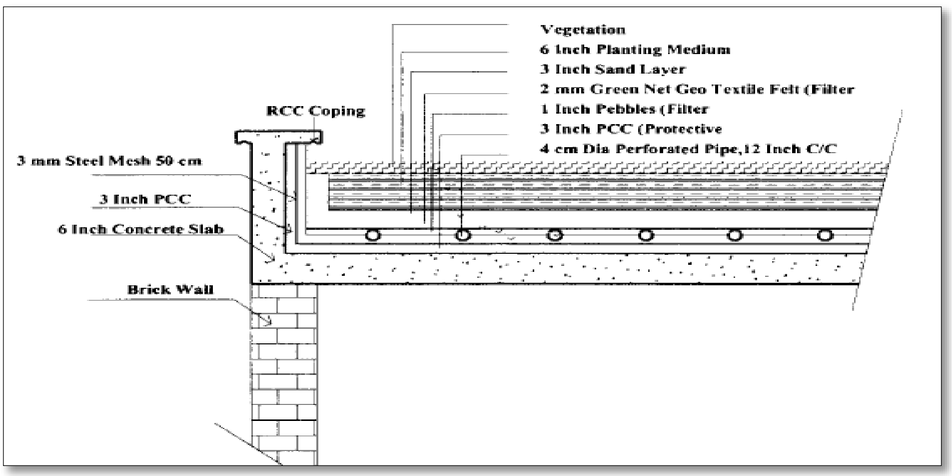

Figure 5: Section View for Green Roof Replacement

Table 2: Material Requirement- Green Roof Layers & Design

Protection Layer	It is used to protect structural roof slab from water leakage and other potential deteriorations caused by green system. For which, a layer of 3inch PCC (Plain Cement Concrete) of ratio 1:4:8 along with 3 mm steel mesh added on its top is installed between concrete roof slab and drainage system. **Additives (Waterproof material):** Feb-Proof **Iron Mesh:** Reported to protect the protection layer from contraction and expansion due to change in temperature **PCC:** Provide slope to roof of ratio 1:10
Drainage Layer	It is between filter layer and protection layer which allows water from rooftops to infiltrates into the drainage system. The purpose of PVC pipes is not just to drain but to retain water also in dry season. **PVC Pipes:** Top surface perforated 4 cm diameter **PCC Layer:** These pipes are half embedded in PCC layer (1 inch) and are placed on top of protective layer at 12 inch center to center.
Filter Layer	It is between planting medium and drain layer. It's a layer with 1 inch pebble bed with 2mm geo textile on its top. Purpose is to prevent planting medium eroded away by the infiltrating water and act as a root barrier.
Planting Medium	It contains 6 inch top layer of planting soil and 3 inch sand course underneath. Planting Soil: Mixture of loamy soil, 40 % silt and 10 % manures (locally used). Fertilizers: Little amount of phosphate and potassium
Vegetation	It transfers solar energy to the atmosphere, also act as insulation. Grass: Kori Grass, Korean Grass, Bent Grass, Bermuda Grass Ground Cover: Foliage, Yellow flower

Table 3: Values of Resources and Externalities (Assumed as per Market/literature)

	Greenhouse gases	**Electricity**	**Storm water**
Units	$/Metric Ton	$/kWh	$/kgal
Market Value	$21.47 (Rs. 2368.99)	$0.0982 (Rs. 10.83)	$2.27 (Rs. 250.47)
Reference	Capoor and Ambrosi (2008)	Energy Information Administration (2009)	Fisher et al. (2008)

2.3 Impact Assessment

Table 4 provides the characterization and classification for impact assessment of green roofing system, for the case study.

Table 4: Characterization and Classification

Classification & Characterization	
Storm water Quality	Acidification
Climate Change	Human toxicity
Urban Heat Island	Ecotoxicity
Global Warming	
Energy Depletion	

2.3.1. Valuation

The study is designed to be adopted in Karachi, Pakistan to see how efficient the Green roofing system in Pakistan is though there is no such vast use of Green Roofs in Pakistan. The Plan view (using google maps) for the case study roof, i.e. N-8 Room, N Block Building, Civil Depart, NED University, is shown in Figure 6. The challenge was to incorporate cost effective indigenous material while achieving a desirable degree of efficiency. For that purpose, a comprehensive investigation is required and a cost effective, durable green roof system can be installed for the civil department buildings. This work was developed for both the retrofit and new buildings. Pakistan previous record on dealing with environmental issues was unfortunate and climatic conditions harshly attack people like heat waves and unbearable winter as people cannot afford amenities like air conditioning or heating systems. The study is compare with conventional methods of roofing like 6 inch concrete roof slab superimposed with a bitumen layer. The conventional system does not help much in decreasing the heat entering the roof instead at the end of the day the ceiling temperatures exceeds the side out air temperature.. There are other methods of roofing also like Thermo pore system with top 2 layers replaced with polystyrene and washed gravels as insulation method while other is Concrete block system in which a layer of inverted box shaped concrete blocks is provided as a top layer.

Figure 6: Top View Plan for N-8 Room, N Block Building, Civil Depart, NED University

Table 5: Cash Flow table for LCA

<table>
<tr><th>Conventional Roof Systems</th><th colspan="3">New Green Roof System</th></tr>
<tr><td rowspan="8">Total cost of conventional RCC slab (6”) + Bitumen - Rs. 500/Sqft
(For 600 sqft= Rs. 300,000)</td><td>RCC Slab</td><td>Rs. 260/sqft</td><td>Rs. 156000</td></tr>
<tr><td>Waterproof (Geo membrane)</td><td>Rs. 30.00/sqft</td><td>Rs. 18000</td></tr>
<tr><td>PCC (Protective Layer + Drainage)</td><td>Rs. 80.00/sqft</td><td>Rs. 48000</td></tr>
<tr><td>PVC Pipe</td><td>Rs. 05.00/sqft</td><td>Rs. 3000</td></tr>
<tr><td>Pebbles</td><td>Rs. 80/sqft</td><td>Rs. 48000</td></tr>
<tr><td>Green geo felt</td><td>Rs. 25/sqft</td><td>Rs. 15000</td></tr>
<tr><td>Plantation Medium</td><td>Rs. 300 /sqft</td><td>Rs. 180000</td></tr>
<tr><td>Fertilizers</td><td>Rs. 3/sqft</td><td>Rs. 1800</td></tr>
<tr><td colspan="2">Total</td><td>Rs. 783/sqft</td><td>Rs. 469,800</td></tr>
</table>

All prices are as per local market survey.

The cost information of different roofing systems (see Table 5) shows that the cost for conventional roof systems found to be approx. Rs. 300,000/- for the area of 600 sqft; whereas the cost for green roofs as per surveys found to be Rs. 469,800 for the same area. The cost of Green Roofing System is 56% higher than the conventional indicating the capital cost is higher, however the long time benefits for green roofing are much higher than conventional, which will be discussed in proceeding sections.

2.3.1.1. Carbon Emission Evaluation

Table 6 provides information regarding the carbon footprint for conventional roof.

Table 6: Carbon Footprints for conventional roof

S. No.	Material	Density	Volume	Qty.	Avg. CO_2 Footprints	CO_2 Footprint
1	Concrete	150 lbs/cft	300 cft	1	26 lbs CO_2/cft	7800 lbs
2.	Steel	490 lbs/cft	0.945 lbs	30	1.6 lbs CO_2/lbs	45.36 lbs
					Total	7845.36 lbs

The CO_2 emission per sft for the tyical green roofing system is found to be approximately 23 lbs CO_2/sft, and thus for 600 sft roof it total carbon foot print becomes 6900 lbs (Blackhurst 2010).

2.3.1.2. Cash Flow Diagram

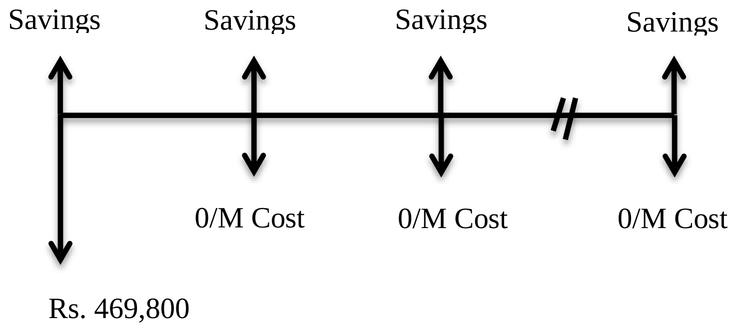

Figure 7: Cash Flow Diagram for Green Roof

Table 7: Reference table for Cash Flow Diagram

Savings = Benefits	
1) Air conditioning Cost (cooling loads)	4) Wildlife Habitat
2) Urban Heat Island	5) Sequestering Carbon dioxide and pollutants
3) Storm water runoff	6) Aesthetic
O/M= Operation and Maintenance Cost	

4. Benefit/Cost Analysis

The Figure 8 provides a comparative view of the benefits vs. Cost for the green roof based on the analysis presented in this paper. It can be seen easily that benefits are much more than the cost incurred to convert a simple roof into green roof for the purpose of sustainability. The quantification of benefits though, can require further investigation to have more tangible outlook of the same.

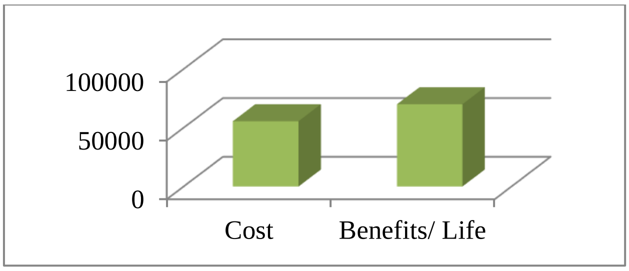

Figure 8: Cost V/s Benefits

5. Conclusion

The outcomes from the LCA model provide intuition in the environmental impact of diverse kinds of obtainable roofs. The energy benefits exemplified by the green roof possibilities make a notable influence in the life cycle assessment. The materials required to construct the roof are important when the energy needs are minimized, and when roof replacement cycles are short. Regardless of the requirement for additional resources initially, green roofs are the environmentally desirable choice when constructing a building due to the reduction in energy demand and the improved life of the roofing membrane. Conversely, it should be noted that this paper is limited to the building type, climate, and location of the study. Also, further research, case studies, and sensitivity analysis are desired to confirm or refute the conclusions made in this paper.

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Unmanned Aerial Systems "UAS" Bridge Inspection Case Study

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Abstract
The Gold Star Memorial Bridge carries Interstate 95 over the Thames River, Local Roads, and CV RR in Groton, Connecticut. The NB Bridge was originally constructed in the 1943 and reconstructed in 1975 and 2001. This 27 span bridge carries a five-lane roadway for a total length of 5,931 feet. An adjacent or "sister bridge" was built in 1972 to accommodate I-95 SB. The bridges are each comprised of 16 girder-floorbeam-stringer spans and 11 main deck truss spans. The main span over the channel is a hung deck truss span. The cantilevered-anchor spans of the main 3-span truss-unit have haunches lower chords and bearings required the use SPRAT repelling/climbing. AI proposed using an unmanned aerial system (UAS) given the constraints of these bridges. The scope of work include review and detail our implementation of the pilot project, and provide, among other things, details on our compliance with Federal Aviation Administration (FAA) regulatory requirements under the Section 333 Exemption process in order to legally use UAS technology for this project. On August 23, 2016 AI performed the UAS inspection of 4 key areas of both bridges with the intent to evaluate UAS inspection as a valid tool for bridge evaluation. The bridge owners (DOTs), particularly in the past 3 decades, have understood the value of on-time inspection, load rating, and maintaining/preserving the existing inventory of bridges (AASHTO Manual for Bridge Evaluation, 2011; AASHTO Manual for Load Rating; Bridge Load Rating Manual, v1.0, Connecticut Department of Transportation). AASHTO's "Manual for Bridge Evaluation" remains the fundamental guidebook for (1) INSPECTION, (2) EVALUATION, (3) MAINTENANCE and (4) PRESERVATION (AASHTO Manual for Bridge Evaluation, 2011; FHWA Bridge Inspector's Reference Manual FHWA-NHI-14-054; FHWA Recording and Coding Guide for the Structure Inventory of the Nation's Bridges, FHWA PD-96-001). This abstract will discuss these elements and how they are undertaken by New England/Northeast DOTs. Also, it will explore how new TECHNOLOGIES, such as UAS (drones) and real-time structural monitoring, can bring significant inspection process efficiencies.

Keywords
Inspection, Evaluation, Preservation, Maintenance, Technology

1. Introduction

An Unmanned Aerial Vehicle (UAV) is defined by the FAA as an aircraft flown with no pilot on board. UAVs are sometimes referred to as "drones" and the name can be used interchangeably. Unmanned Aerial System (UAS) is currently the term used by the FAA to describe these systems. The vehicle is controlled with the use of a remote control by a pilot from the ground and can carry a wide range of imaging and data collection technologies including still photos, video and infrared packages and other types of sensors. UAS are an emerging technology with many potential applications in the civil engineering field.

The primary goal of this Pilot Project was to provide relevant information to allow the Department of Transportation (Connecticut DOT Bridge Inspection Manual) to make an initial evaluation of the possible use and deployment of UAS as a part of the Bridge Inspection Program. It was designed to evaluate the potential benefits of using UAS as a tool to obtain remote photo, video and data.
To achieve this goal, the Project Team (Team) performed the following tasks:

1) Conducted an extensive review of current FAA rules, regulations and practices for the commercial use of UAS in areas open and closed to the general public.
2) Identified and researched emerging and related UAS and data collection technologies that could be employed to enhance the Bridge Inspection Program.
3) Using current FAA and public safety protocols, determine the appropriate staging areas, launch and access points, and general public safety and notification requirements.
4) Compare photographs taken during a hands-on inspection with photographs taken at the same bridge detail location with the UAS camera.

The Team investigated this technology on the Gold Star Memorial Bridge which carries I- 95 over the Thames River located between the Town of Groton and City of New London, Connecticut (See Figure 2). This bridge was chosen due to its difficult access, complex details, and extensive coordination required with stakeholders in the area, including the FAA and the U.S. Coast Guard. The integrated Team was comprised of staff from AI Engineers, Inc. and its UAS vendor Exponent Technology Services, Inc., with assistance from the UAV Systems Association. The Team also included members of the Department's Bridge Safety and Evaluation Unit.

The project tested advantages, disadvantages, and challenges dealing with the use of UAS to aid the Department in the bridge inspection process. The Team explored the legal and technical requirements of utilizing UAS for bridge inspections, and analyzed and evaluated current UAS technologies, including the means and methods necessary for their legal use

2. Background

One application that is routinely mentioned is the area of bridge inspection due to the logistical challenges to efficiently and effectively visually inspect a wide variety of structure types and components in challenging locations. This was the premise upon which this Pilot Project was based. The bridge selected for this Pilot Project was chosen based on its size, type and location, thus representing an optimal opportunity to study various predetermined variables as part of a single test project. This bridge is the *Gold Star Memorial Bridge* comprising sister bridges (Nos. 02514A and 03819) carrying I-95 over the Thames River located between New London and Groton, Connecticut. Our Pilot Project Team (Team) developed and implemented in conjunction with the Connecticut Department of Transportation (Connecticut DOT Bridge Inspection Manual), a detailed fieldwork plan to address safety, FAA rules, and inspection methods. The field test employed an image collection device capable of collecting digital photographs, video and infrared images. Various data was collected in the field and stored digitally enabling it to be managed using post processing techniques.

The bridge was selected based on the following factors: bridge type and size, location, FAA requirements and permits. This particular set of bridges also offered the Team with a unique opportunity to conduct test flights using an UAS for bridge inspection under the following conditions (See Figure 1):

1. The bridge carries a major Interstate Highway (I-95).
2. The bridge spans a major river (Thames River).
3. The bridge is located in a highly-dense populated area.
4. The bridge is located in close proximity to major railroad tracks.
5. The bridge crosses local roads and private property.
6. The bridge is located near an active seaport.

7. The bridge is located in proximity to and along the travel ingress and egress path of a U.S. Navy Nuclear Powered Submarine Base.
8. The bridge is located within 5 miles of an air-traffic controlled (ATC) airport.
9. UAS flight takeoffs and landings from a barge located in the river.

Figure 1: Location of the UAS Pilot Project

Figure 2: Gold Star Memorial Bridge, New London and Groton, Connecticut

On August 23, 2016 several UAS test flights were completed by the Team led by AI Engineers, Inc. with its UAS vendor Exponent Technology Services, Inc. These test flights were conducted under the general supervision of the Department's personnel. The UAS flights were able to obtain significant amounts of data on the targeted elements of the bridges. As part of its mission to develop, operate and maintain a safe and efficient transportation system, the Department monitors, inspects and surveys its highways, bridges and facilities inventory on an ongoing basis. The monitoring and inspection of infrastructure assets is done in many ways, such as with vehicles, traffic counters, bridge sensors, cameras and field or site inspections. There is a need to develop a method to be able to view certain structures and areas from an aerial view that is flexible, safe, cost effective, and have rapid deployment capabilities under emergency situations.

Previously, aerial imagery consisted of fly over photos limited to a specific time or with telescopic mounted cameras with limited viewing angles, which proved to be cost prohibitive and of little practical value. With the advancement of new technologies, it is now possible to examine structural elements that otherwise would be difficult to reach, and do it in a more economical and feasible manner with the level of data collection necessary to provide practical solutions and with the versatility needed to address various situations. This new technology is the emerging availability of aerial vehicles capable of delivering devices to capture high definition video, high definition photographs, low light visibility and thermal imagery at a lower cost than of alternative solutions. The recent advances in this technology warranted an investigation into its possible use as an asset in the inspection process. A UAS is an aircraft that is operated without a human being on board. It is controlled by a pilot operating a remote control from the ground or autonomously by computers in the vehicle. The UAS is said to be a flexible and a cost-effective approach to collecting real-time data from an aerial view. UAS have become more popular over the last couple of years in transportation planning, engineering operations, and several UAS options and designs have entered the market.

UAS are able to carry cameras with video capabilities, and their use in data collection can be expected to improve traffic and asset management operations. However, there are regulations and restrictions that control the use of UAS for commercial purposes in collection of data that affect or limit their use in public areas. The FAA Modernization and Reform Act of 2012 was introduced on February 11, 2011 and signed by the President of the United States on February 14, 2012. It included important provisions on the integration of UAS into the national airspace. Recently, newer applications are being studied, and several state governments have begun looking into the feasibility of using UAS for transportation and infrastructure inventory, inspection, emergency response and asset management. With the ability to transport data collection devices to a specific structure and hover on site for a period of time and take advantage of the high-resolution aerial photography and video creates a high potential for them to become an integrated system capable of collecting and verifying structural element data. Using UAS is now one of the most cost effective means for elevating a camera to record a video or photograph from an aerial view. Many UAS models can now legally carry a wide variety of data collection devices from ground level to heights up to 400 feet. Many of the data collection equipment have become simplified to the point where anyone with basic training can effectively operate them. The setup of this equipment takes very little room, leading to more available setup locations. Packages can be engineered for specific missions, such as, bridge and high pole inspections, area and perimeter awareness, and emergency incident responses. With the advent of these emerging technologies the failure rates of typical UAS has dramatically lessened over the past few years, making them safer and more affordable.

In order to evaluate the possible use of UAS, the Department authorized this Pilot Project and AI Engineers, Inc. to test and assess the feasibility of using UAS as a tool to assist in the bridge inspection process. The project involved utilization of a specific UAS based on its demonstrated ability to operate in a manner reasonably consistent with that needed to conduct a bridge inspection and deliver sufficient relevant data necessary to conduct a meaningful technical evaluation. In part, the project investigated the effectiveness of using UAS as a tool for providing additional data while reducing impediments to traffic flow. Although there are many different UAS models and technologies currently available in the marketplace the Team chose to utilize a specific UAS model, the Align M690L, based on its known technical capabilities and payload capacity for this Pilot Project.

A Section 333 Exemption and Certificate of Authorization (COA) was obtained by AI Engineer's sub-consultant, Exponent Technology Services, Inc., from the FAA for use of the Align M690L. Exponent provided the licensed pilot required by the FAA and they were instrumental in helping our team obtain all the required certifications and FAA approvals.

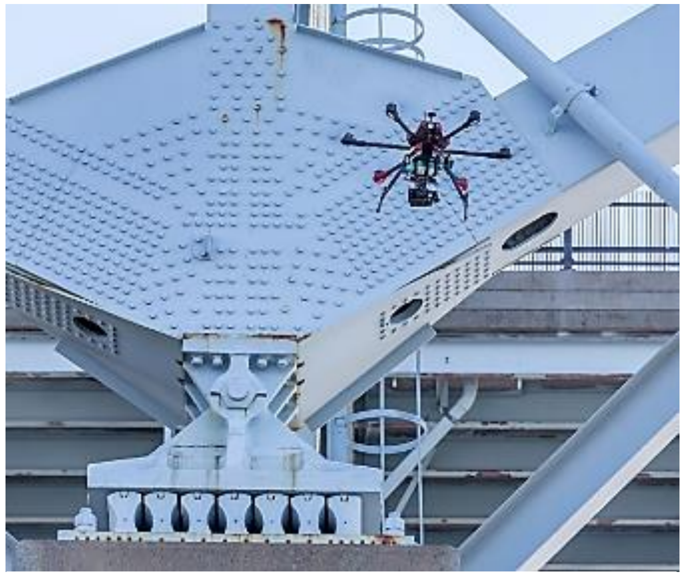

Figure 3: UAS during Gold Star Inspection

3. Project Conclusion

The Pilot Project identified that the use of UAS can be an effective tool to enhance the Bridge Inspection Program (See Figure 3). Although not a replacement for a hands-on inspection, it can be used for limited investigations for pre or post inspection information gathering. It also can be effective during emergency situations where a hands-on inspection cannot be performed immediately due to access limitations. This was the Department's first use of UAS and more field tests using different data collection technologies must be performed in order to make a more detailed evaluation. It has the potential to be very useful for certain inspection tasks such as high resolution data collection and mapping concrete cracking on large concrete superstructure and substructure elements.

The results from this pilot project were inconclusive for bridge inspection purposes due to the photographic resolution produced by the equipment used. Still images removed from the 1080P video did not produce high resolution detailing. The Department obtained valuable information on the FAA requirements and UAS equipment standards, specifications and limitations.
Other useful information obtained from the Pilot Project:

- The regulatory requirements of using UAS are in the early stages and are undergoing changes and refinements that must be monitored continuously.
- Utilizing a UAS for data collection and photos can be useful but the output must match the required usage of the data or photos.
- It is highly recommended to minimize the camera vibration during flight.
- Flight staging areas are minimal and manageable in various environments but can require boat or barge usage which adds costs.
- Flight set-up, launch and recovery, and flight times are manageable.

- UAS equipment (vehicle and data collection) should be required to meet minimum technical and quality standards.
- Without good airborne GPS data, sufficient ground points need to be provided around and inside the data collection block.
- UAS can access difficult to reach areas with minimal interference with traffic only under certain conditions.
- UAS can be a rapid response tool for emergency situations.
- Advanced camera options can include Infrared, GPR and 3D, although this was not employed during this Pilot Program except for low resolution infrared photos.
- UAS are adaptable to data collecting needs and can carry various types of data collection equipment.
- UAS flight time durations are currently limited.
- UAS have the potential to minimize public inconvenience for information gathering purposes.

Currently the NBIS does not address the use of UAS for bridge inspection.

4. Future Consideration

The Department will consider the following as it continues its evaluation of UAS as a tool to support the Bridge Inspection Program:

- Continue to follow activity in the State Legislature with regard to state agency use of UAS.
- Monitor the FAA web site for the latest developments on UAS testing and regulatory issues.
- Examine the UAS research conducted by educational institutions and private companies.
- Consult with other state DOTs that have completed investigations and continue to investigate UAS to learn more about how this technology is being developed and implemented.
- Monitor how multirotor UAS are being tested in other states in connection with bridge and other asset management programs.
- Research experience gained by other jurisdictions on how to do instrument calibration on aerial platforms for imaging purposes and data collection.
- Consider how other UAS applications (inventory documenting, photogrammetry, Hyperspectral sensing, vegetation and soil investigations, disaster response, sign and guardrail inspections, and roadside and roadway inspections) might be implemented by the Department.
- Attend UAS conferences to be appraised of the latest developments on UAS technology, applications, and regulatory environment.
- Apply matrices derived from previous research projects to define design specifications for a UAS that can fulfill their operational requirements.
- Talk with UAS commercial operators regarding defined UAS design specifications for asset inspection and inventory management.

Conduct additional tests as required to validate concepts and draft operational rules and regulations

5. Post Inspection Summary

A primary part of the scope for this UAS Pilot Project was to identify critical inspection points from hands-on field inspections on the bridge, and verify current conditions versus prior inspection conditions from photographs. The Team used digital photography and video footage of the bridge at preselected points and positioned the UAS during several flights to collect that data for subsequent collation and comparison using photo management software.

The takeoff and landing sites were identified prior to the field test, which helped to speed up the process of using the UAS to evaluate the several-targeted areas selected for this particular bridge inspection project. The UAS flights addressed and evaluated several factors including: an overall flight plan setting

forth the actual flight patterns and identifying the staging locations; setup areas; bridge element target locations; timing; flight durations; and data collection objectives. Each UAS flight followed a specific flight path based on predetermined criteria in accordance with mapped-out specific areas on the bridge structure that were targeted. Using the UAS, the Team was able to obtain photographs of the predetermined bridge components for comparison during a total of four UAS flights, with each flight lasting approximately 15 minutes. Vehicle traffic on the bridge was not disrupted and no traffic delays were reported during the flights. The flights confirmed that UAS could be an effective tool in delivering a data collection device (i.e., digital camera with video) to a predetermined bridge element and obtaining photographs of those elements in a shortened period with less cost and no traffic interference.
During the UAS flights, the Team was able to:

- Test and evaluate any limiting factors for setup locations, such as, area sizing, the need to avoid power lines, and pedestrian and vehicle traffic involvement.
- Use GPS to maintain an exact location for efficient data gathering, and test manual flight controls where the loss of GPS was experienced under the bridge structure.
- Note a list of concerns to account for in typical inspection project site locations and physical limitations.
- Evaluate aspects of the gimbal mount device utilized during the project and determined that it was an acceptable device for this project and other similar type operations.

Evaluate the UAS communication system and FAA flight area requirements

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Compressive strength evaluation by adding marble dust and wood saw dust as replacement of cement and fine aggregate

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Abstract
Marble stone dust and wood saw dust have been used in concrete mixes as partial replacement for fine and coarse aggregates separately in recent years. The increase or decrease in strength is due to the addition in the percentage of marble stone dust and wood saw dust. Studies have been conducted on determining the optimum marble stone dust and wood saw dust percentage to meet the desired strength of concrete in construction. In this study, marble stone dust (MSD) and wood saw dust (WSD) were used as partial replacement for cement and fine aggregate in concrete mix respectively. The test samples were prepared by replacing 0%, 2%, 4%, and 6% of cement and fines by weight of concrete with MSD and WSD. The combined effect of MSD and WSD on the workability and compressive strength of concrete was investigated. It was found that compressive strength gradually decreases with increase in the percentage of marble stone dust and wood saw dust.

Keywords
Comparative Assessment, Partial Replacement, Marble Stone Dust, Wood Saw Dust

1. Introduction

The cost of building construction is increasing regularly such as cement, granite, fine aggregates and coarse aggregate etc. Researchers are paying attention to other low cost materials including wastes and by products. Marble stone dust and wood saw dust are among them and have got interest as replacement. Due to the improper disposal, marble stone dust & wood saw dust pose great threat to environment and mankind. They release carbon-monoxide to atmosphere which depletes the ozone layer. There is a need to investigate the use of alternate locally available materials such as marble stone dust & wood saw dust in concrete production as a partial replacement of cement and fine aggregate. Additionally, the use of these wastes in concrete will contribute to the reduction in amount of carbon emission.

MSD and WSD have been used as a partial replacement of cement and fine aggregate in concrete separately. However, due to limited studies the effect of using MSD and WSD as partial replacement for cement and aggregate is yet not fully understood. In this study the MSD and WSD were used for partial replacement (0%, 2%, 4%, and 6%) percentages of cement and fines. Fresh and hard concrete properties were analyzed.

2. Review of literature

(Baboo Rai et al, 2011) partially replaced cement with MSD and reported that increase in the percentages of MSD the flow ability and compressive strengths of the concrete mix was increased. However, the MSD used as partial replacement for cement increases the compressive strength up to a limit and then decreases. The optimum limit of MSD replacement for cement beyond which the compressive strength of concrete cube decreases is 12.5% (Manju Pawar, 2014).

(Ankit Nileshchandra Patel et al, 2013) has reported that the compressive strength of the concrete increases with increase in the percentages of marble dust at all curing ages. (Kamel Al-Zboon et al, 2015) has reported that International Agency for Research on Cancer concluded that wood dust causes cancer of the nasal cavities, Para nasal sinuses, and nasopharynx wood dust is recycled of stone cutting slurry in concrete mixes. The compressive strength of sawdust concrete decreases with increase in amounts of sawdust. This was clearly exhibited by the 28 day strength.10% partial replacement of sand with sawdust provided suitable strength above the minimum compressive strength for lightweight concrete which is $17N/mm^2$.

(Olugbenga Joseph Oyedepo et al, 2014) the use of sawdust as partial replacement of sand between 0 to 25% will contributes to reduction in sawdust waste generated in the society without adversely affecting concrete strength.

(Tomas U. Ganiron Jr, 2014) Sawdust can be used as alternative substitute for fine aggregate in concrete production.

3. Methodology

Ordinary Portland cement, fine Aggregate, coarse Aggregate, marble stone dust, & wood saw dust were used as materials in this research work. In this study, the batching by weight method was adopted. Preliminary mixes of 1:2:4 (cement and MSD: fine sand and WSD: coarse aggregate) were investigated with water/cement ratio of 0.65. Water was also weighted and placed into the concrete mixer and mixed constantly until the batch was thoroughly mixed and then casted into $6 \times 6 \times 6\ in^3$ size moulds, as shown in **Error! Reference source not found.**. The experimental investigation was carried out in casting cubes normal concrete and the concrete made by the partial replacement of marble stone dust and wood stone dust by adding 0, 2, 4, and 6% percentages in addition. The concrete mix was thoroughly poured in the moulds of cubes. Every layer of cubes was tamped 25 times with tamping rod. The top surface was leveled with the help of a trowel and finished properly. After 24 hours cubes were de-moulded and cured at normal temperature for the time period of 7, 14 and 28 days.

4. Result

4.1 Slump

The workability was evaluated by conducting slump test in accordance with (ASTM International, 2003) as shown in Figure 1.

Figure 1: Slump Test

From the obtained results it was observed that there was decrease in the workability of the concrete with increase in the percentage of marble stone dust and wood saw dust, the loss in slump is shown in Figure 2.

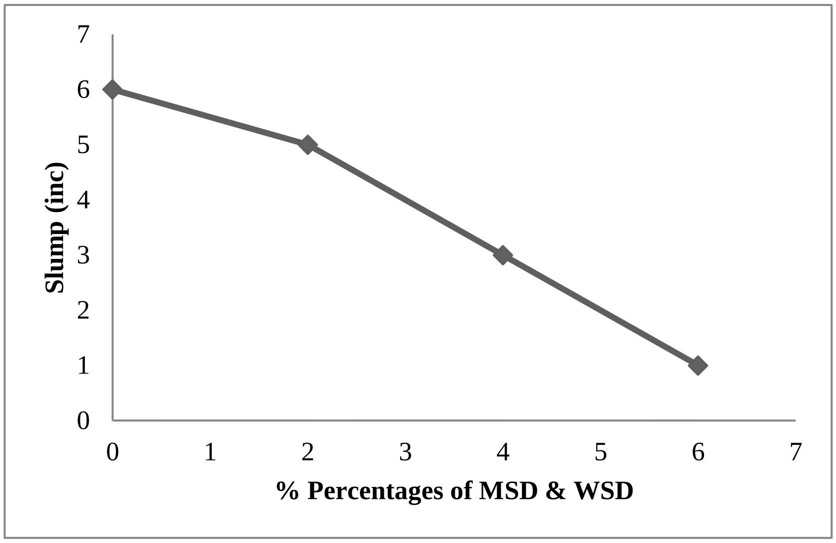

Figure 2: Slump with different % Percentages of MSD & WSD

4.2 Compressive Strength

The experimental work was done on the cubes for the compressive strength of concrete by replacement of cement and fine aggregate with marble stone dust and wood saw dust respectively. Compressive strength tests were conducted on compressive testing machine in accordance with (British Standard, 1983) to evaluate the strength development of concrete containing marble stone & wood saw dust as shown in Figure 3.

Figure 3: Compressive Strength Test

From the results it was observed that the average compressive strength of concrete cubes were decreasing with the addition of marble stone & wood saw dust it was due to the replacement of binder agent by marble stone & wood saw dust. The average strength of 6% marble stone & wood saw dust cubes cured for 28 days were decreased by 22.12% when compared to the normal concrete cubes cured for 28days as shown in Figure 4.

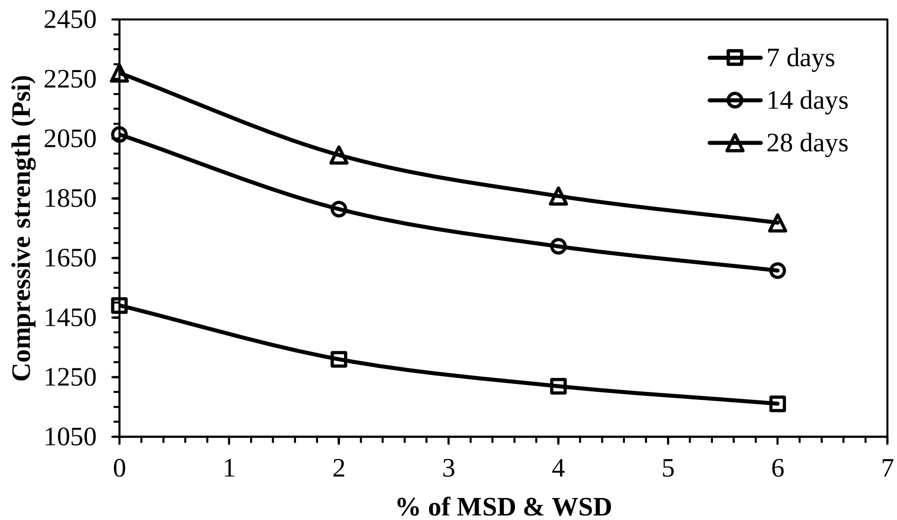

Figure 4: Average compressive strength

5. Conclusion

This research was concluded to find out compressive strength evaluation by adding marble dust and wood saw dust as replacement of cement and fine aggregate.

- There was a decrease in the workability of concrete, which could be due to the absorption of water by marble stone & wood saw dust in concrete.
- Also, the compressive strength of concrete at 0% replacement of cement and fine aggregate with marble stone & wood saw dust is greater than the compressive strength of concrete at 2%, 4% & 6%.

6. Recommendations

- Since the addition of marble stone & wood saw dust did not add strength to the concrete, it is recommended that these be added into the concrete to be used where moderate strength is required such as flooring etc.
- Based on the above conclusions further research is recommended to check the effect of marble stone & wood saw dust in concrete against thermal properties.

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Comparartive Analysis of Reinforced Concrete Structure With and Without Base Isolation

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Abstract
Base isolation is an effective retrofitting option which is currently being used throughout the world to mitigate the damages caused by earthquake. In base isolation, the structure vibrates and dissipates energy at the base of the structure as a result it reduces the major earthquake parameter which is base shear, drift and overturning moment and increases the natural time period of the structure significantly as compare to fixed base structure. The objective of current research is to check the performance of irregular midrise reinforced concrete building with and without base isolation. To this end, a case study building having (G + 12) story RC frame system has been analyzed considering with and without isolation interface at base level. A detailed computational model has been developed and process through linear static and linear dynamic analysis as per UBC-97 Section-16 (Division IV). Results are compared and presented in the form of engineering demand parameters such as story drift, story shear, overturning moment and base shear.

Keywords
Eathquake,Base isolation,EDP(Engineering Demand Parameter),Linear Static Analysis,Linear Dynamic Analysis.

1. Introduction

An earthquake occurs due to sudden release of energy that results in seismic waves. It is also referred to as quake, tremor or temblor. The seismic happening for an area is defined by frequency, type and size of earthquakes occurred for a specific time period (Wikipedia 2017). Earthquakes are measured using seismometers, namely moment magnitude and Richter magnitude scale. Modified Mercalli Scale measures the intensity of vibration. Earthquakes having magnitudes 3 or less are even difficult to detect while the ones having magnitude 7 or greater can cause severe damage over large areas, depending on their depths. (Wikipedia 2017).

They are the ones which can shake, damage, and can also destroy the structure. So we, as civil engineers, are only interested in surface waves and primarily Surface waves are of two types, Love and Rayleigh waves. (www.geo.mtu.edu/UPSeis/waves.html).

When earthquake hit the structure it induces acceleration in the structure, which produces extra forces resulting into displacements in buildings. Members, joints and whole structures might fracture, damage or even collapse because of these displacements. This can ultimately be dangerous up to the level of fatal. So to reduce its probability we need to design structure which can withstand the acceleration, forces and displacements induced by earthquake. For that purpose engineers have devised energy dissipating devices.

2. Types of Seismic Isolation

2.1 Energy Dissipating Devices

As the name suggests these devices are the one which dissipates energy from the buildings. Every device has its own mechanism to disperse excess energy and make the structure more flexible, hence earthquake resistant to a certain level. Some of the many devices are described here.

2.2 Tuned Mass Damper (TMD)

It is also known as harmonic absorber or active mass damper. This device uses the phenomena of pendulum. Placed at the top of the vertical structure it comes back to its original position when displaced by external lateral forces. While doing so it also brings the structure to its original position. It has a damping device which dissipates energy in the form of heat and a spring to oscillate. (www.theconstructor.org/structural-engg/tuned-mass-dampers/1198/)

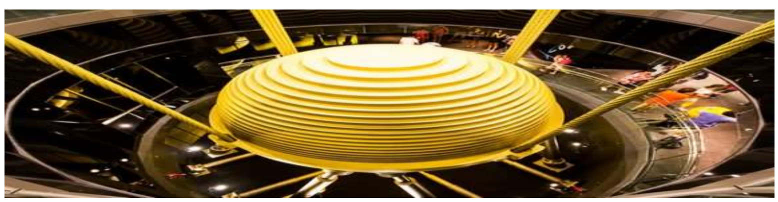

Figure 1: Tuned Mass Damper

2.3 Seismic Dampers

They are mechanical devices that deform during earthquake. They are usually placed diagonally in the structure. It dissipates energy so that the structure has to resist lesser amount of earthquake forces. Lesser amount of force will induce less deflection.

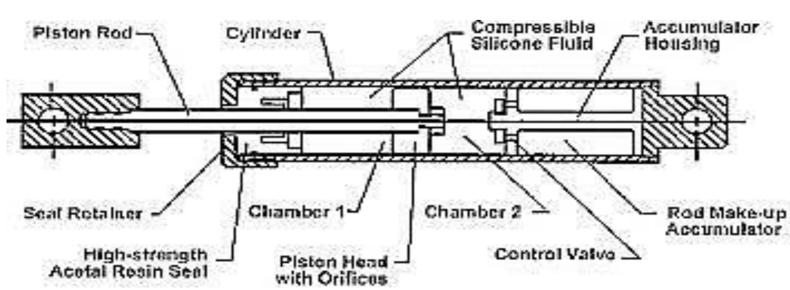

Figure 2: Seismic Damper

2.4 Base Isolators

Principally base isolator separates or decouples the structure from its foundation by providing a damping interface between them. By providing a base isolation device, which is essentially acting as damping device, between the structure and the ground the extent of earthquake force spread to the buildings can be minimized. Hence it will reduce forces in the superstructure which will reduce deflection. A simple base isolation system consists of two basic components which are isolation bearings and damper. The former protects the superstructure from collapse because of lateral movements based on earthquake forces, whereas the latter absorbs or dissipates the energy that base obtains during an earthquake. Materials used in Base Isolator are steel, rubber and may be lead which is essentially for its plastic property. Types of base isolator are elastomeric bearings, natural or synthetic rubber bearings and lead rubber bearings (Efiloglu 2013).

Figure 3: Base Isolator

2.5 Base Isolation of Building Structures

Earthquake engineering has devised many ways to tackle with the severe forces and its effects on the structure; base isolation is one of those concepts, which can be defined as separating or decoupling the structure from its foundation. That helps to reduce story shears, overturning moment, drifts and increase natural time period of the structure. (Efiloglu 2013)
Elastomeric bearing are further divided into two types:

a) Natural and Synthetic Rubber Bearings (NRB)

b) Lead Rubber Bearings (LRB)

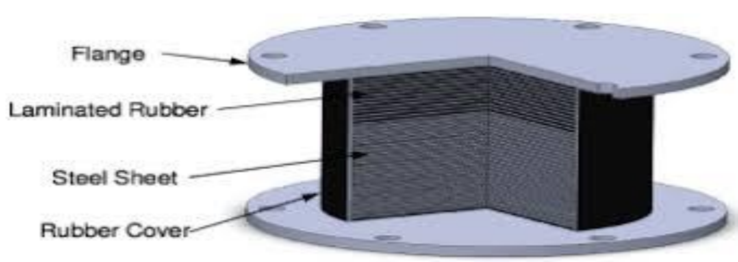

Figure 4: Rubber Isolator

2.5.1 Friction pendulum bearings

Two horizontal steel plates of friction pendulum bearings can slide over each other due to their shape and an additional articulated slider. The bearings are placed between building and its foundation. They are designed very stiff and strong to carry vertical load because of the weight of the building. (www.ideers.bris.ac.uk/resistant/isolating_pendulum.html)

Figure 5: Friction Pendulum Bearing

3. Case Study Structure

3.1 General Overview of Building

Following are the general details about the case study building of this report:

- The case study structure is located at Shaheed-e-Millat road, Karachi (zone 2B) surrounded by tall structures and 400 feet wide road.
- This structure consists of eleven residential floors and four parking floors. Each story is having a height of 12 feet.
- Soil beneath the structure is rocky sand.
- Total number of columns designed by structural engineer is forty-six having varying percentage of steel in cross sections of 15" x 36" to 18" x 36" throughout the height of structure.
- Shear wall of 8" thickness is also proposed by designer to control and mitigate seismic effects. It is located at center and corner of building.
- Slab thickness for parking slots is 6" while for rest of floors is 5".
- Beam cross sectional sizes are 12" x 24" to 12" x 35".
- Stair case is situated in between the shear walls.
- Material Properties:
  - Compressive strength of concrete for beams and slab is 4300 psi while for column and shear wall is 6000 psi.
  - Yield strength of steel is 60 ksi.

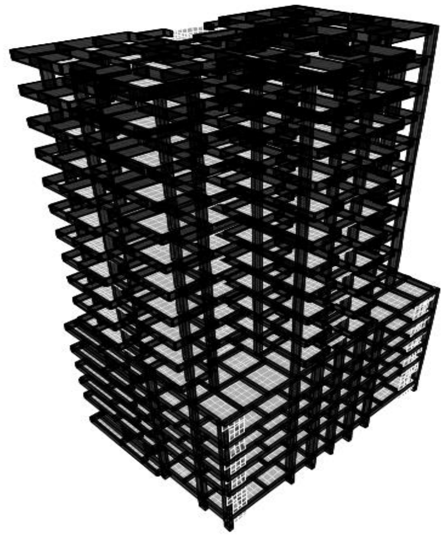

Figure 6: 3-dimensional view of case study model

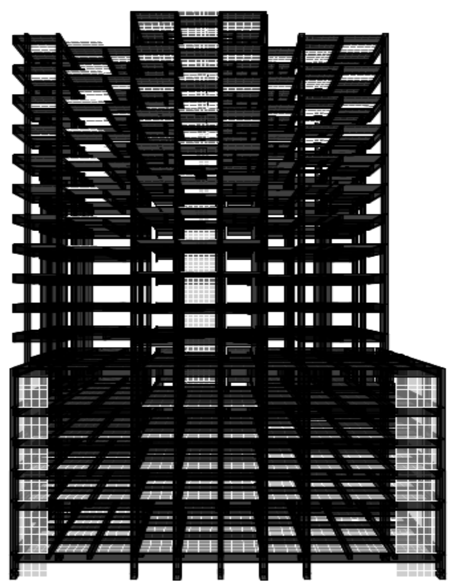

Figure 7: Elevation view of case study model

Following loads are assigned on structure:

- Live load of 100 psf for lobby, stairs and parking floors.
- Live load of 40 psf for residential floors.
- Dead load is calculated by self-weight multiplier of software except for stair case weight which is assigned manually.
- Masonry load on each beam is assigned except for roof beams, and is calculated by density of concrete, width and height of the masonry.
- Sunk slab load is assigned where sunk slabs were supposed to be provided, and are calculated by density of earth fill and depth.
- The isolator assigned on the given case study structure as per the loading is rubber isolator with damping in the range of range of 10-12%(UBC-97 Chap16 Div IV)

4. Results and Findings

Modal analysis was performed on the fixed base case study structure. ETABS software uses EIGEN value to calculate modal response of the structure. For prototype structure number of modes will be 48 but the practical modes are those which have the time period not lesser than 90% of the effective time period. Table 1 shows the practical modes and there corresponding time periods of the fixed base structure.

Table 1: Modal Analysis of Fixed Base System

Mode	1	2	3	4	5	6	7	8	9	10
Period (sec)	2.8	2.613	2.229	0.917	0.751	0.598	0.53	0.355	0.344	0.323

Table 2 shows practical modes and there corresponding time periods of the base isolated structure.

Table 2: Modal Analysis of Base Isolated System

Mode	1	2	3	4	5	6	7	8	9	10
Period (sec)	4.075	3.924	3.504	1.542	1.263	1.221	0.707	0.529	0.451	0.428

All the results and findings related to the comparison of prototype structure with and without base isolation. This includes the results of

- Seismic forces,
- Maximum story drifts, and
- Overturning moment

From linear static and linear dynamic analysis.
Conclusion extracted from the findings is discussed after every result.

4.1 Time Period Comparison of Fixed Base vs. Base Isolated Structure

The natural time period obtained from analysis performed on prototype structure on the software, ETABS for fixed and isolated base structure is as follows in Table 3.

Table 3: Time Period

TYPE OF BASE	FIXED	ISOLATED
TIME PERIOD (sec)	2.8	4.075

Time period obtained from base isolated structure is 1.45 times greater than that of fixed base which indicates that the isolators modeled are affecting the modal behavior of structure.

5. Conclusion

This research paper discussed about the implementation of base isolation in a structure to reduce alterations induced due to seismic forces. With the help of ETABS we implemented base isolation in a structure which was ground plus fifteen floors to get results. It can be concluded with the results that the following parameter of structure are reduced significantly and tables for seismic forces, interstory drift and overturning moment is not shown due to limitation of pages in research paper and the conclusion shown is below:

1. Time period of the structure increases by 46% for the seismic isolated structure.
2. Lateral forces at the base have been reduced by 59% for the seismic isolated structure and it reduces by 61% for approximate (UBC 1997) (appendix chapter 16 division IV).
3. Drift at the base has been reduced by 87% for the seismic isolated structure and it reduces 91% for approximate (UBC 1997) (appendix chapter 16 division IV).
4. Overturning moment has been reduced by 66% for the seismic isolated structure it reduces by 66% for approximate (UBC 1997) (appendix chapter 16 division IV).

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Comparative Dynamic Analysis and Design of Machine Foundations - A Real Time Project Scenario

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Abstract
Considering the importance of heavy dynamic machines with respect to their high cost and continuous usage, especially in the power sector, the analysis and design of such equipment's foundation require great attention. It is primarily because of the interaction of two important engineering aspects occurring simultaneously-the dynamic effects produced by the machine and the corresponding geotechnical response to it. A real time oil and gas compression / power generation facility required dynamic analysis of three similar machines-compressors package. Three types of analysis and design strategies were utilized and compared in detailed in this regard. Firstly, an in-house design sheet was developed for block type foundation. Secondly, a 3D finite element analysis software (STAAD Pro) was used. Finally, a more novel, relatively unfamiliar software (Clockwork), reasonably helpful for quick analysis of machines foundation was used. All types of strategies incorporated had certain advantages and limitations in their respective capacities. The various results thus obtained were synthesized in the real time project situation which involved the study of the response of these massive foundations and practical recommendations made. This accounted for a much optimized design solution, which was ultimately executed on site.

Keywords
Dynamic Analysis, Machine Foundation, Vibration, Resonance, Finite Element

1. Introduction

Machine foundations are an important and expensive part of any industrial setup. However their analysis and design is more complex as compared to a foundation that supports static loads only (Shamsher Prakash et al., 2006). In machine foundations, the structural designer has to specially consider, in addition to the static loads, the dynamic forces caused by the working of the machine. These dynamic forces are then transmitted to the foundation supporting the equipment. Therefore, the designer must be well acquainted with the method of load transmission from the machine as well as with the problems related to the interaction between the dynamic behaviour of the foundation and the soil underneath.

1.1 Analysis and Design Criteria

A machine foundation is needed to meet the following two conditions for satisfactory performance:

1.1.1 Static loads

- The applied loads should not exceed half the bearing capacity of the soil (ACI 351.3R-04). The static loads are due to the weight of the foundation, machine and accessories.
- It should be safe against shear failure
- There shall be no excessive settlement

The static analysis is a pre-requisite for the dynamic analysis and a strong correlation exists between the two types of analysis.

1.1.2 Dynamic loads

- The operating frequency of the machine shall not coincide with the natural frequency of the foundation-soil system at various modes (vertical, sliding, rocking, yawing etc.) to avoid resonance. Generally, the zone of resonance is defined by the code that is being followed or conversely by the manufacturer. Normally, the range is ± 20%, i.e. the two frequencies readings shall be at least 20 units apart.
- The amplitude of motion at operating frequency shall not exceed the limiting amplitude. The allowable amplitudes of the machinery are generally specified by its manufacturer that primarily is governed by the relative importance of the machine and the sensitivity of neighboring structures to vibration.
- It should be safe against all stability parameters due to seismic, wind, erection, operating and their combinations. These include the sliding and overturning checks (ACI 351.3R-04).

1.2 Data Requirements

The specific data required for dynamic analysis and design of different types of machine foundations vary, depending upon the type of machine it supports. The general requirements however, are as follows:

- Loading diagram showing the magnitude and positions of static and dynamic loads exerted by the machine on its foundation.
- Power of engine and the operating speed.
- Diagram showing the embedded parts/openings, grooves for foundation bolts, etc.
- The unbalanced primary and secondary forces and couples applied on the foundation through the machine.
- A comprehensive geotechnical investigation report consisting of all the soil characteristics related to its static and dynamic properties.

1.3 Parameters Influencing The Analysis And Design Process

There are various parameters that influence the design of a machine foundation:

- Center of gravity of the foundation,
- Moment of inertia of the foundation base,
- Mass moment of inertia,
- Effective stiffness of the base
- Damping.

The first three parameters mentioned above are referred as the "Geometrical properties of the machine foundation system”, while the remaining parameters are termed as the “Physical properties of the elastic base of the foundation”. The eccentricity of the center of gravity of a machine foundation with reference to the vertical axis passing through the center of elasticity of the base support induces coupling' of vibratory modes and this complicates the design procedure (Shamsher Prakash et al., 2006). It is, therefore, desirable in design practice to ensure that the eccentricities in the two horizontal directions (x and y) are within permissible limits. The moment of inertia of the base of the foundation and mass moment of inertia influence the dynamic calculations for the rocking (or twisting) mode of vibration. These two parameters are direction-dependent in the sense that their expressions differ with the chosen reference axis. The effective stiffness and damping presented by the base support depends on the type of the flexible base provided beneath the foundation-whether soil, springs, elastic-pads, etc.

2. Case Study Project

A comparative static and dynamic assessment of machine foundation is performed for a compression facility project to be utilized for power generation. It is located near Gujranwala in the Punjab province of Pakistan. The three dynamic machines, referred to as “compressor package” are being relocated from another site. There are certain design parameters that are either provided by the manufacturer or are furnished by the geotechnical investigation report. They have been summarized below in table 1.

Table 1. Input Parameters

Material Properties	
Structural Concrete Compressive Strength (MPa)	28
Blinding Concrete Compressive Strength (MPa)	14
Reinforcing Steel Characteristic Strength (MPa)	420
Geotechnical Parameters	
Bearing Capacity of Soil (KPa)	61
Unit Weight of Soil (KN/m^3)	19
Shear Modulus (KPa)	32732
Shear Wave Velocity (m/sec)	130
Soil Poisson’s Ratio	0.4
Machine Parameters	
Weight of Machine (KN)	744
Operating Frequency (Hz)	15
Time Period (sec)	0.067

The drawing, provided by the manufacturer is referred to as the “General Arrangement” drawing of the machine. It consists of the plan view, side views, elevations, isometric views, dead weights and the anchor bolts layout plan. Figure 1 below shows the isometric view of the compressor.

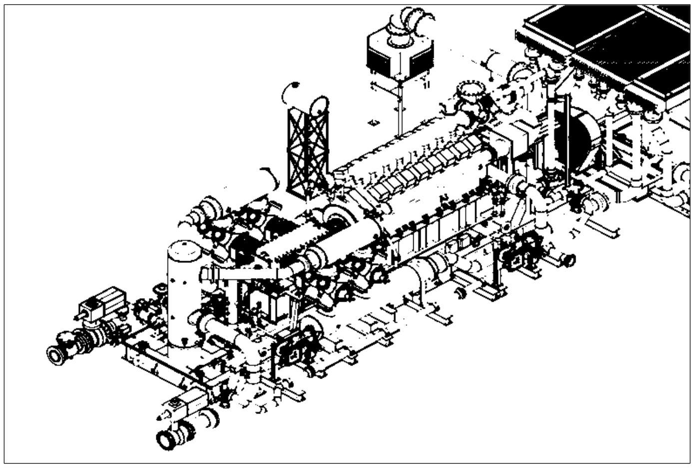


Figure 1. Isometric View of the Compressor

3. In-House Analysis and Design Sheet

An In-house sheet is formulated by the research participants for the static and dynamic analysis of machine foundations on Microsoft Excel software after a thorough literature review related to the matter. Firstly, static analysis is done in accordance with the ACI code and the size for the foundation is established. The static calculations are shown below in Figure 2.

Static Design of Compressor Foundation				
Data				
Weight of compressor	W	=	729.47	KN
Compressive Strength of concrete	fc'	=	28000.00	KN/m^2
Tensile Strength of steel	fy	=	415000.00	KN/m^2
Unit weight of soil	γs	=	19000.00	N/m^3
Unit weight of concrete	γc	=	24000.00	N/m^3
Unit weight of water	γw	=	9800.00	N/m^3
Allowable/Net soil bearing capacity	q_s	=	73.00	KN/m^2
Area of Footing				
Length of footing	L	=	13.50	m
Width of footing	B	=	7.50	m
Thickness of footing	h	=	1.20	m
Thickness of footing above ground	H1	=	0.80	m
Area of footing	A=LxB	=	101.25	m^2
Section modulus of footing in X direction	$Z_x=LxB^2/6$	=	126.56	m^3
Section modulus of footing in Z direction	$Z_y=BxL^2/6$	=	227.81	m^3
Weight of footing	Wf'=((L*B*h)*(γc)	=	2916.00	KN
FOS on Bearing Capacity of Static Calculation, q_{all} (FOS)		=	0.50	
	q_{max}= P/A (KN/m^2)	=	36.00	< 36.50
Ratio of Weight of Foundation to weight of Machine		=	4.00	> 3

Figure 2. Static Design Calculations

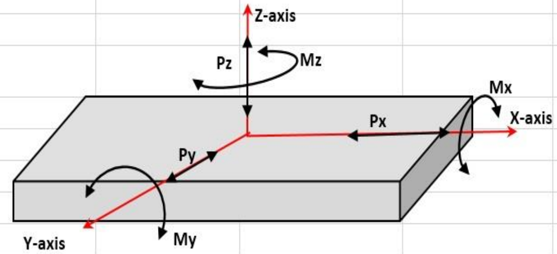
Geotechnical investigation report is carefully studied in collaboration with the geotechnical engineer for the proper understanding of soil characteristics as they are the dominant influences in the dynamic behaviour of the foundation. The report provides all the basic and relevant soil parameters essential for the calculation of spring constants.

The methodology followed for the calculation of the spring constants is based upon the elastic half space theory which requires the determination of shear modulus (G) and poisson's ratio (v) of soil preferably by an in situ dynamic test. The theory follows the undamped linear spring analogy of soil and requires the evaluation of certain soil parameters, listed below:

- Coefficient of elastic uniform compression (C_s)
- Coefficient of elastic uniform shear ($C_£$)
- Coefficient of elastic non-uniform compression ($C_Ø$)
- Coefficient of elastic non-uniform shear (C_λ).

These soil parameters are used for the evaluation of the spring stiffness of soil for calculation of natural frequencies of foundation at various modes of vibration. Utilizing the various analytical and empirical formulas developed as a result of in-depth literature review, the vibration analysis of foundation is performed on the sheet. Ultimately, a block type foundation size is finalized after employing trial and error solution method. Figure 3 below shows the calculation of vibration analysis.

DYNAMIC DESIGN OF COMPRESSOR FOUNDATION



Vibration modes of block type foundation

Data Input				
Operating Frequency Of Machine	fm	=	900	RPM
Weight of Compressor without Engine		=	49.88	tons
Weight of Compressor Engine		=	24.48	tons
Weight of compressor	Wm	=	74.36	tons
Weight of Foundation	Wf		414.72	tons
Horizontal Unbalanced Force in x-direction	PX	=	0.000	tons
Horizontal Unbalanced Force in y-direction	Py	=	0.103	tons
Vertical Unbalanced Force	Pz	=	0.045	tons
Unbalanced Moment about x-axis	Mx	=	0.000	tons.m
Unbalanced Moment about y-axis	My	=	2.081	tons.m
Unbalanced Twisting Moment	Mz	=	0.974	tons.m
Poisson ratio	v	=	0.4	
Shear Wave Velocity	Vs	=	130.0	m/sec
Shear Modulus	G	=	32731.9	KN/m2
Spring constant for vertical vibration	Fz	=	2.200	from graph
	Kz	=	127141.0	ton/m
Spring constant for horizontal vibration	Fx	=	0.950	from graph
	Kx	=	92235.0	ton/m
Spring constant for horizontal vibration	Fy	=	1.000	from graph
	Ky	=	97089.5	ton/m
A - Calculation for Natural Frequencies of Foundation for various Modes				
Natural frequencies for all six modes is calculated i.e.				
1 - Vertical translation (in Z-axis)		Uncoupled		
2 - Sliding and Rocking Motion in xz plane		Coupled		
3 - Sliding and Rocking Motion in yz plane		Coupled		
4 - yawing [or twisting moment about Z-axis]		Uncoupled		
1 - Vertical translation (in Z-axis)				
For the foundation Resting directly on soil the circular natural frequency for uncoupled vertical translation along the z-axis is given by :				
ωz = (Kz / m)^0.5			50.500	sec-1
Corresponding natural frequency of this mode, ωz / (2 x π)		=	9	Hz
Amplitude (in Z-axis)				
Vertical amplitude (az) = Pz / {m (ω2nz-ω2m)}		=	-0.000144	mm

Figure 3. Dynamic Analysis Input and Calculations

4. Finite Element Model on STAAD Pro

A 3D, finite element model (FEM) is developed of the machine foundation on STAAD PRO software utilizing the skid details provided by the vendor. The model is constituted of plate elements with uniform thickness. A detailed study of General Arrangement (GA) drawing is carried out to identify the location of the center of gravity (COG) in all directions (x, y & z) of the equipment. The static loads are applied at proposed locations as indicated by the manufacturer which includes the dead weight of the compressor including its engine, its other allied accessories and self-weight of the foundation. For dynamic analysis, all the unbalanced primary and secondary forces and couples are applied along with their proper directions at the COG. The vertical as well as horizontal forces and couples act in their respective planes, working about the center of the crankshaft. The forces and couples are generated by the unbalanced masses of equal size separated by a moment arm. In addition to the rotating masses, reciprocating masses also act in the horizontal direction. It is depicted below in Figure 4.

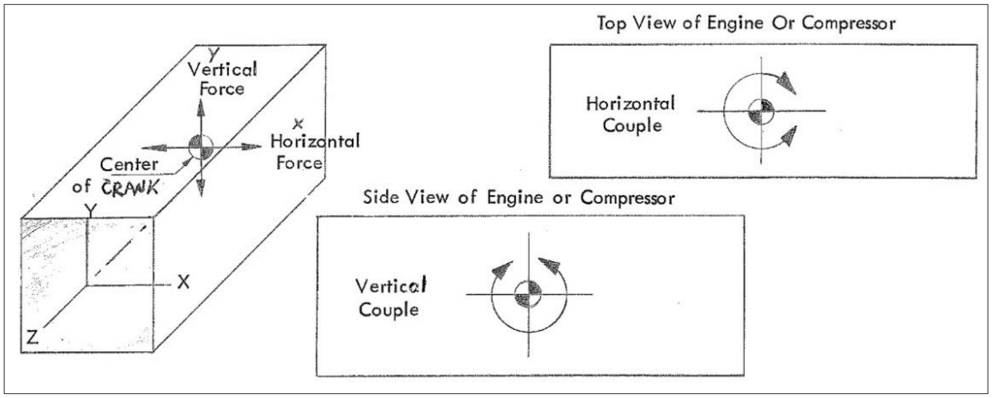


Figure 4. Maximum Unbalanced Forces and Couples Orientation

A time history function is defined for forces and moment independently with harmonic function selected. The frequency, amplitude and damping of the machine are provided as input for the performance of vibration analysis. The results show that the natural frequencies of the foundation in various modes and maximum relative displacements are within the allowable limits and hence, resonance will not occur. Figure 5 shows the 3-D FEM model of the foundation.

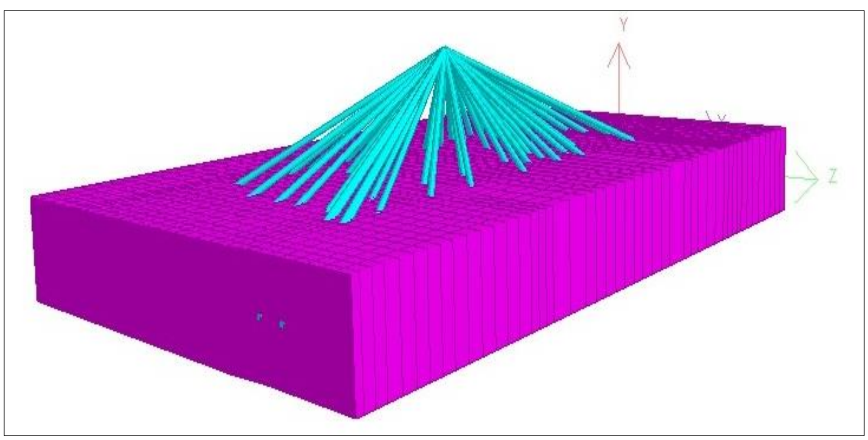


Figure 5. 3D Finite Element Model Rendered Image

5. Machine Foundation Model on Clockwork

One of the several methods available for analysis of vibration characteristics of machine foundations is the impedance function method and clockwork is the implementation of this method. Clockwork (CW) is an innovative 2D/3D CAD software for machine foundations which covers the design process, dynamic analysis and criteria for reciprocating machine vibration, rotation and impact. Clockwork allows to determine the six undamped natural frequencies of the foundation-machine system. The inputs for this software are very simple. It just requires the machine and geotechnical data for the whole vibratory process. Utilizing the skid details of the machine, a trial size is input along with the machine data as read from the vendor drawing. Then, the COG of the foundation is marked and machine load(s), unbalanced forces and moments are applied to it. The geotechnical data input involves the defining of soil layers under the influence of the foundation and then providing the soil parameters like shear modulus, unit weight and poisson's ratio.

Using this data, clockwork performed the vibratory analysis and displayed the frequencies of the foundation in six modes along with the allowable vertical vibration amplitudes. The researchers then checked if the response is acceptable. The foundation dimensions are changed and the response is recalculated until it comes within tolerable limits. Figure 6 below shows the graphical user interface (GUI) of this software along with the windows showing the input parameters entered into the software for analysis.

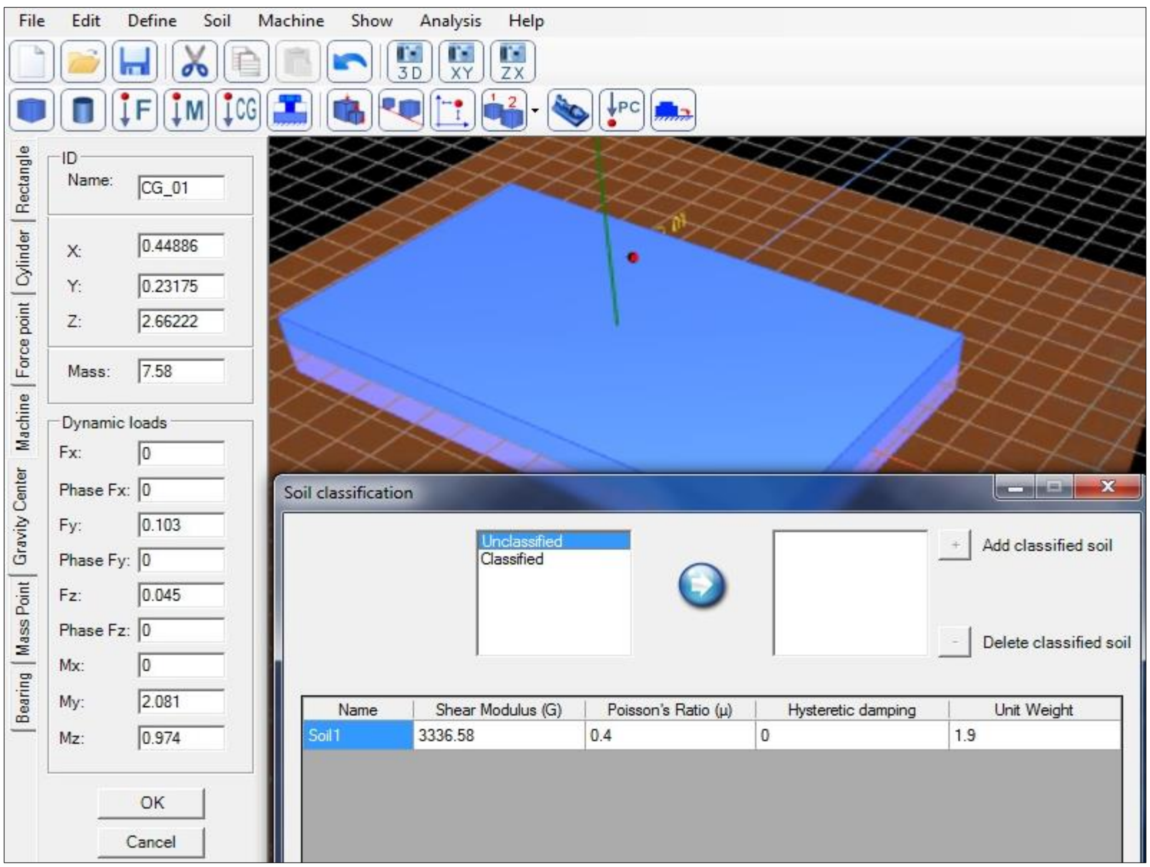


Figure 6. Graphical User Interface (GUI) of Clockwork

6. Comparison of Analysis Methods

The results obtained from the three types of methods employed indicate that at identical foundation sizes, apart, obviously, from the static design, they also yield similar dynamic outcomes to vibratory motions. Table 2 below shows the comparison of dynamic responses among the three methods with respect to the natural frequencies of foundation obtained in all possible modes.

Table 2. Natural Frequencies (Hz) of Machine Foundations by Different Methods

Vibration Modes	In-House Sheet	STAAD Pro	Clockwork
Horizontal & Rocking (Coupled) (Rx_1)	19	18	18.5
Horizontal & Rocking (Coupled) (Rx_2)	7	6.7	7.2
Horizontal & Rocking (Coupled) (Ry_1)	11	12	11.6
Horizontal & Rocking (Coupled) (Ry_2)	7	6.6	7.2
Vertical (V)	9	7.8	8.2
Torsional (T)	11	9	11.75

But, there are certain factors that need to be considered while executing a compression facility project to be utilized for power generation, as it is of high national importance. These include timely completion, optimized cost usage, quality control, serviceability and durability. Owing to the fulfillment of these requirements, it needs to be investigated that which is the best option to be utilized for this project.

The in-house sheet developed by the participants yield accurate results with very little computational effort. The trial and error method can easily be used for selection of a suitable size of foundation that conforms to the code/manufacturer requirements, when performing the vibration analysis. But there are several limitations/disadvantages of using this manual sheet. Firstly, it only accounts for the resonance check by calculating the frequency of the foundation at various modes and comparing it with the operating frequency of the machine. It lacks the ability to indicate the peak acceleration/displacement at various desired locations and also fails to identify the stress/vibration at any particular point. Secondly, it is restricted to only the design of a block type foundation and cannot be used for the design of box-type, wall-type or framed-type foundations. Also, it is limited to the design of a rectangular shape foundation only and cannot cope with an irregular shape, as may be the requirement of the manufacturer.

The model developed on the FEM based software STAAD PRO has numerous benefits. Apart from performing the vibration analysis and producing the overall results of natural frequencies to avoid resonance, it also has the capability to perform checks like rigidity at bearing locations, differential settlements, maximum displacements, and load distortions at specified locations. This is helpful in the comprehensive dynamic design of such foundations. But, the disadvantage of using this software is that it is tedious to model the whole foundation and also slight changes in the geometry of foundation requires a careful and time consuming adjustment in the model. Furthermore, every time while testing new dimensions for the foundation, the calculation for the spring constants needs to be revised.

The clockwork software is dedicated for the design process, dynamic analysis, standard and criteria of reciprocating machine vibration, rotation and impact. It requires no computational effort from the user and just uses the equipment and soil parameters data to perform the whole vibratory analysis. It increases the productivity of machine foundation design. Clockwork is accompanied by a library of soil properties labeled “Classified Soils” that contains parameters belonging to recognized standards used worldwide. It also takes into account the effect of embedded foundation, which is an important aspect in vibration analysis. Displacement, velocities and acceleration values at all pre-defined control points are available in results.

7. Optimized Design Solution

Since the compressors package is being relocated from another site, therefore it has already been designed along with its anchor bolts in accordance with the vendor data. But the present site location has inferior geotechnical parameters and hence, reduced bearing capacity. Therefore, neither the previous foundation design nor the anchor bolts lengths can be utilized here. The reduced bearing capacity limits the overall dimensions of the foundation and therefore the length of anchor bolts to be provided in order to pass the static analysis. Thus, in addition to the minimum reinforcement provided as per ACI code requirements, some extra reinforcement is required to compensate for the short length of anchor bolts. The easy but costly way to cope with this issue is to increase the overall reinforcement in the foundation (ACI 207.2R-95). But the researchers introduce a novel way for an optimized design. An anchor bolts design sheet is formulated in accordance with the ACI code (Appendix D). Utilizing this sheet, instead of increasing the overall percentage of reinforcement in the foundation, extra reinforcing bars are provided only in the influence of individual anchor bolts. Figure 7 below depicts the sheet developed by the research participants where the input parameters are shown. Figure 8 shows the results of the anchor bolt design as well as various failure modes of anchor bolts. This results in much cost reduction as it decreases procurement requirement of reinforcing bars.

Anchor bolt design based on Combined Tension and Shear								
Anchor Bolt Data								
			set N_u = 0 if it's compression					
Factored tension for design	$\mathbf{N_u}$	=	49.0	[kips]		=	218.0	[kN]
Factored shear	V_u	=	4.8	[kips]		=	21.4	[kN]
Concrete strength	f'_c	=	4.0	[ksi]		=	27.6	[MPa]
Anchor bolt material		=	A307					
Anchor tensile strength	f_{uta}	=	58.0	[ksi]		=	400	[MPa]
Anchor bolt diameter	$\mathbf{d_a}$	=	1.5	[in]		=	38.1	[mm]
Anchor bolt embedment depth	h_{ef}	=	36.0	[in]	900	[mm]		
Pedestal width	b_c	=	20.0	[in]	500	[mm]		
Pedestal depth	d_c	=	20.0	[in]	500	[mm]		
Bolt edge distance c_1	c_1	=	12.0	[in]			**OK**	
Bolt edge distance c_2	c_2	=	12.0	[in]			**OK**	
Bolt edge distance c_3	c_3	=	12.0	[in]			**OK**	
Bolt edge distance c_4	c_4	=	12.0	[in]			**OK**	
To be considered effective for resisting anchor tension, vertical reinforcing bars shall be located								
within $0.5h_{ef}$ from the outmost anchor's centerline.					$0.5h_{ef}$	=	**18.0**	[in]
No of ver. rebar that are effective for resisting anchor tension					n_v	=	8	
Ver. bar size No. dia of rebar		=	0.625	[in] dia	single bar area A_s	=	0.31	[in^2]
To be considered effective for resisting anchor shear, hor. reinft shall be located								
within min($0.5c_1$, $0.3c_2$) from the outmost anchor's centerline					min($0.5c_1$, $0.3c_2$)	=	3.60	[in]
No of tie leg that are effective to resist anchor shear					n_{leg}	=	4	no
No of tie layer that are effective to resist anchor shear					n_{lay}	=	1	no
Hor. tie bar size No.		=	0.625	[in] dia	single bar area A_s	=	0.31	[in^2]
For anchor reinft shear breakout strength calc								
Rebar yield strength - ver. bar	f_{y-v}	=	60	[ksi]	No of bolt carrying	n_t	=	1
Rebar yield strength - hor. bar	f_{y-h}	=	60	[ksi]	No of bolt carrying	n_s	=	1
For side-face blowout check use								
No of bolt along width edge	n_{bw}	=	1	No				
No of bolt along depth edge	n_{bd}	=	1	No				

Figure 7. Anchor Bolts Design Sheet Input Data

Results (ACI 318-08 - 12.2.1)					
Tension Failure Ratios			**Shear Failure Ratios**		
Anchor Rod Tensile Resistance	= 0.80	OK	Anchor Rod Shear Resistance	= 0.19	OK
Anchor Reinft Tensile Breakout Resistance	= 0.63	OK	Anchor Reinft Shear Breakout Resistance		
Anchor Pullout Resistance	= 0.94	OK	Strut Bearing Strength	= 0.03	OK
Side Blowout Resistance	= 0.29	OK	Tie Reinforcement	= 0.12	OK
Tension Shear Interaction		ratio	= 1.12		OK

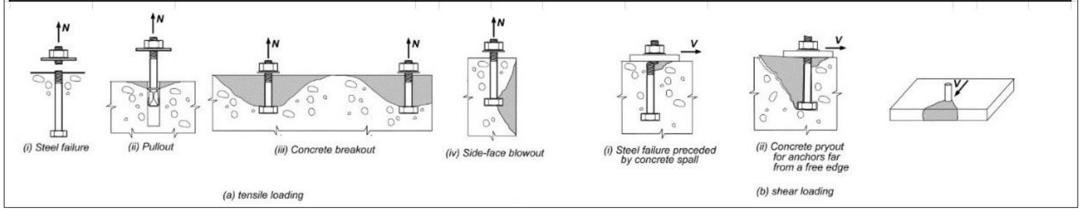


Figure 8. Anchor Bolts Design Sheet Results and Failure Modes

8. Conclusion

Three different methods for determination of dynamic response of machine foundation have been presented and discussed in detail. As a result of comprehensive and in-depth comparative analysis among these methods, following conclusions are drawn:

- All methods demonstrate similar results for vibratory analysis with respect to resonance calculations.
- Depending upon the equipment data provided by the vendor and project type and situation, different methods may be practicable, conditional to the circumstances.
- For quick analysis, clockwork and in-house sheet is feasible whereas for detailed analysis, FEM based software is effective as it has the potential to yield more results like acceleration, velocity, stress at any required point and location.
- However, for this project, the in-house sheet is found to be the best method for performance as it easily complies with the requirements of the equipment manufacturer as well as with the overall project situation.
- Additionally, the sheet formulated for the anchor bolt design sheet has resulted in a much optimized design.

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Traffic Study of Ulaanbaatar for Example on the Peace Avenue

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Abstract

The roads of any city are the complex structures contained in it in accordance with the general plans and urban planning with infrastructure and engineering facilities. Today's difficulties to solve the road problems in Mongolian capital city are related to its city planning. Ulaanbaatar city is located along the stretches of four mountains along the Tuul River from East to West and the major cause of the traffic today's roads as this city have only one avenue crossing it (Peace Avenue). Meanwhile, there are only three roads to reduce the traffic as the roads like from Sapporo to 3rd, 4th micro-districts, from Zuun Ail to KTMS, from Shar Khad to 10th micro-district, and from Bayanzurkh to Yaarmag's Bridge via Bogd Khaan Mountain's outer side. These roads have so little contributions to the city traffic. Other two roads can share the circulation of the main road but it is too limited and has low-pass capacities. As a result, it is necessary to study the Peace Avenue as the main cause of the traffic in Ulaanbaatar city. So therefore, I studied the throughout capabilities and capacities of the zone on the Peace Avenue between the Sapporo and Jukov's intersection junctions with regulated and unregulated traffic lights including the organization of their circulations.

Keywords
The capacity of the streets, throughput of the streets, passing capabilities and capacities of the intersections, traffic lights, and environmental problem of the street

1. Introduction

I studied the architectural structures and circulation features of the main street in the framework of the research work of the "Peace Avenue's traffic simulations" and made conclusions on the certain directions. The real data from the City Planning Department of the Capital, Traffic Management Department of the Capital city, Public Transport Department and Metropolitan Statistical Department were used in this study work. As result of the completion of the data after the mathematical processing I made partial engineering scenes of today's Peace Avenue as the longest street of the city with the most ecological dangers and as the road with the most accidents, injuries, damages and deaths.

2. The Traffic Structure and Capacity of the Peace Avenue

I made calculations on transferring the data to the light vehicles by using the traffic intensity where the circulation structures were put on every intersection. As example, I showed the traffic components of the Baruun Durvun Zam intersection including the results received from the calculations of the intersectional organizations in the Table 1.

Table 1: The Traffic Structure of the Baruun Durvun Zam Intersection and Its Transfer Values to Light Vehicles

The Baruun Durvun Zam intersection				
Route number	light vehicles	Bus	Truck	Transfer values to light vehicles
1	1166	50	40	1351
2	1092	4	34	1153
3	260	0	1	262
4	330	12	0	360
5	1047	101	4	1306
6	532	0	2	535
7	384	4	2	397
8	712	4	69	826
9	658	3	9	679
10	640	5	2	656
11	1012	106	12	1295
12	870	1	76	987

During the studies of the intersections and streets of the Peace Avenue, I found that the passing capability and capacity were too limited as the main street of the megacity as shown in Table 2. The new rules to use the first lane of the main road by the public transport and special-purpose vehicles in order to support the public transportation policy were a wrong decision which reduced its characteristics. Also, I made calculations on two different roads with and without these special-purpose lanes as shown in graph in the figure 1.

Table 2: The Capacity of the Peace Avenue

№	Section of Street	Without these special-purpose lanes		Without these special-purpose lanes	
		East to west	West to east	East to west	West to east
1	The Tsambagarav – Sapporo's intersection	327	269	269	327
2	Sapporo's intersection – 10th micro-district's intersection	659	542	542	659
3	10th micro-district's intersection – 25th pharmacy's intersection	254	219	159	194
4	25th pharmacy's intersection – Instituts of Transport's intersection	304	251	243	296
5	Instituts of Transport's intersection –Modnii hoyor's intersection	277	236	236	285
6	Modnii hoyor's intersection – Baruun Durvun Zam's intersection	136	122	134	148
7	Baruun Durvun Zam's intersection – Mergejiltnii hori's intersection	144	118	128	154
8	Mergejiltnii hori's intersection – Department store's intersection	320	263	263	320
9	Department store's intersection – Tsetseg tuv's intersection	224	184	184	224
10	Tsetseg tuv's intersection – Tuv shuudan's intersection	139	114	114	139
11	Tuv shuudan's intersection – Gadaad hergiin yam's intersection	164	135	191	220
12	Gadaad hergiin yam's intersection – Bagshiin deed's intersection	153	126	126	153
13	Bagshiin deed's intersection – Bukhiin urguu's intersection	467	384	384	467
14	Bukhiin urguu's intersection – Zuun Durvun Zam's intersection	295	262	286	319
15	Zuun Durvun Zam's intersection – The Jukov	412	356	339	429

The comparisons of the above mentioned indicators are shown in the figure 1.

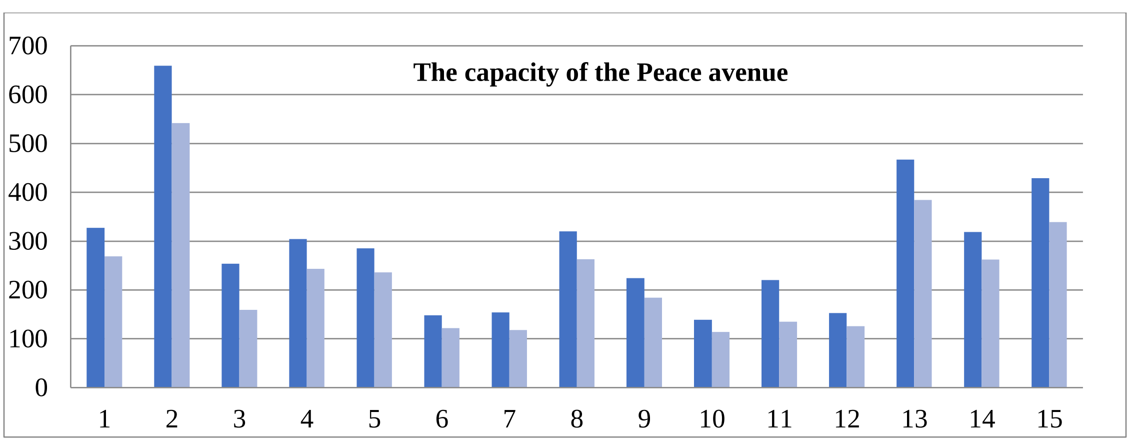


Figure 1. The Space of the Streets in the Peace Avenue (In Comparisons)

3. Congestion and Traffic Light on Intersection of Peace avenue

The cycle duration of traffic lights and their respective throughput are shown in table 3.

Table 3. Accounting Throughput of Traffic Control Intersection

№	Intersection	The cycle duration of the traffic lights, seconds	Accounting throughput , veh/hour
1	Sapporo	130	8446
2	10th micro-district	130	7865
3	25th pharmacy	130	7172
4	Institut of transport	130	6720
5	Modnii hoyor	130	6378
6	Baruun durvun zam	130	7495
7	Mergejiltnii hori	130	5686
8	Tsetseg tuv	130	6365
9	Tuv shuudan	130	6425
10	Gadaad hergiin yam	130	7486
11	Bagshiin deed	130	6000
12	Buhiin urguu	120	5800
13	Zuun durvun zam	120	6680

The passing ability of one lane in the intersection with regulated circulation can be determined by the following formula:

$$N_{зох} = \frac{3600 t_{н}}{T_{ц} \cdot q} \quad (1)$$

3600 is 1 second in 1-hour;

$T_{ц}$ – The cycle duration of the traffic lights, seconds;

$t_{н}$ – The green light duration of the traffic lights, seconds;

q – The distances between the vehicles (q=2.5-3 seconds);

The cycle of the traffic lights – the time from one green light to the next green light.

$$T_{ц} = t_{н} + t_{у} + t_{ш} \qquad (2)$$

(1) The passing capabilities of the traffic lights of the Peace Avenue calculated by the formula are shown in the Figure 2.

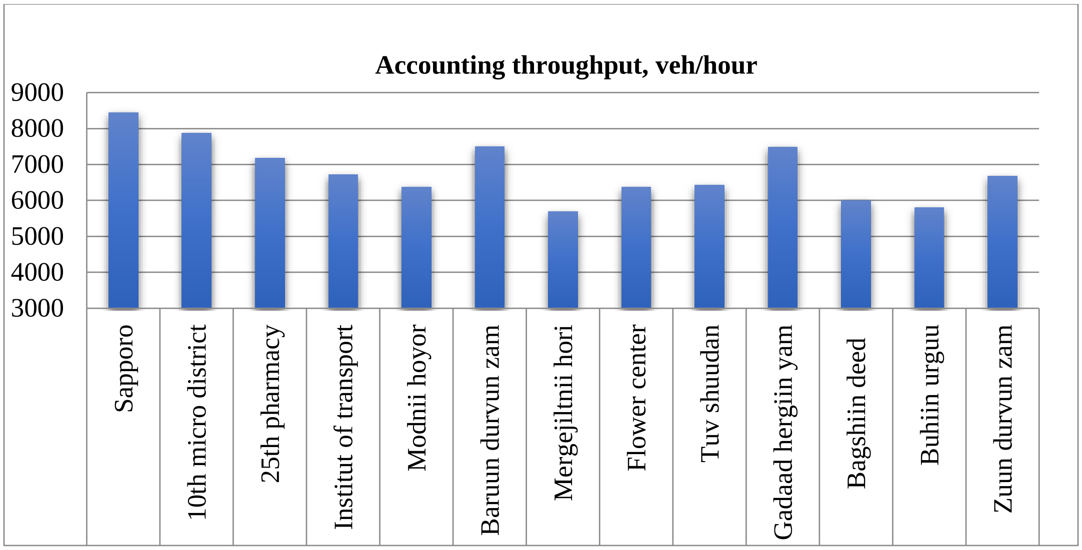


Figure 2. The Passing Capability of the Peace Avenue's Intersections

The passing capability of the intersection is calculated by the amount of the passing vehicles thru the unit time and the important indicator affecting to it are the organization of the circulations in the intersection and the load of the operational modes of the traffic lights (refer figure 3).

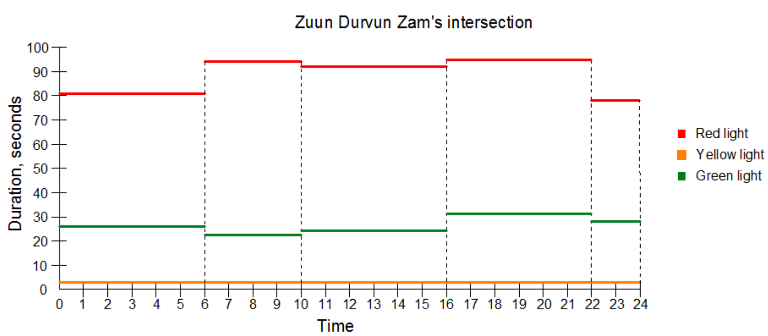


Figure 3. The Operational Modes of the Traffic Lights

The influence of the traffic lights to the intersection loads is calculated by using the statistical method programs in correlation of the regression equation.

$$M(Y/X = x) \int_{-\infty}^{+\infty} yf(y/x)dy \qquad (3)$$

This influence was used as the example in the calculation of the road between the Flower Center's and Central Post's intersections and is shown in table 4.

Table 4. Calculation of the Traffic Lights' Influence to the Traffic Intensity

Time	y	x1	x2	SUMMARY OUTPUT	
8.00-9.00	1375	100	245		
9.00-10.00	1728	100	245	Regression Statistics	
10.00-11.00	1553	120	245	Multiple R	0.594586
12.00-13.00	1650	120	245	R Square	0.353533
13.00-14.00	2060	120	245	Adjusted R Square	0.094946
14.00-15.00	1742	120	245	Standard Error	231.7095
17.00-18.00	1893	120	245	Observations	8
18.00-19.00	2081	120	245		

Where, x1 – the cycle of the traffic lights, x2 – the length of the street, y – the traffic power

The calculation of the influence of the traffic lights' cycles and length of the streets to the traffic intensity in the intersections was 0.59. The influence was reduced to 0.52 when the traffic lights' cycle was reduced by 30 seconds, as shown in table 5.

Table 5. Reworked Traffic Lights' Influence to the Traffic Intensity

Time	y	x1	x2	SUMMARY OUTPUT	
8.00-9.00	1375	70	245		
9.00-10.00	1728	70	245	Regression Statistics	
10.00-11.00	1553	90	245	Multiple R	0.528999
12.00-13.00	1650	90	245	R Square	0.27984
13.00-14.00	2060	90	245	Adjusted R Square	-0.00685
14.00-15.00	1742	90	245	Standard Error	223.2515
17.00-18.00	1893	90	245	Observations	8
18.00-19.00	2081	90	245		

As result, it was determined that the uses of the traffic lights' cycles with the preprogrammed time in the Ulaanbaatar city's intersections were playing the significant impacts on reducing the road traffic.

4. The Versions To Propose In Order To Improve The Passing Capabilities of Intersections

The proposed version is shown in figure 4. The passing capability of the T-shaped intersection located in the Peace Avenue in the southern corner of the Pedagogical University (in comparison) is shown in table 6 and figure 5.

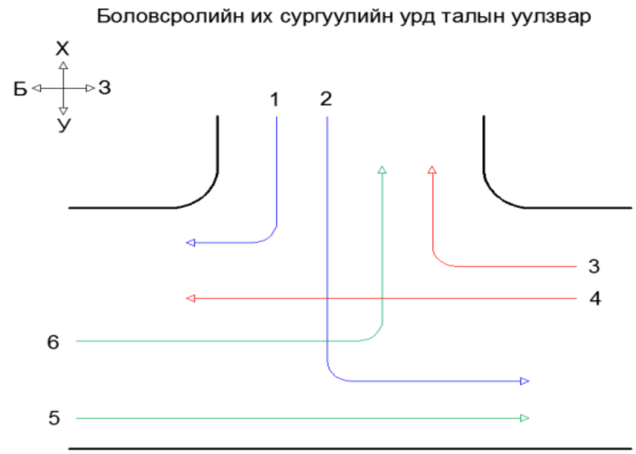


Figure 4. T-shaped Intersection (Bagshiin Deed)

Table 6. The passing capability of the T-shaped intersection located in the Peace Avenue in the Southern Corner of Pedagogical University (In Comparison)

Time	Now	Right when turning movement turned permanent green light/3/	1st row of the whole movement case turned on a permanent green light/5/	Turn right and the whole movement case turned on a permanent green light
00:00-06:00	6000	6393	6611	7004
06.00-10.00	6000	6305	6591	6895
10.00-17.00	6000	6393	6628	7020
17.00-20.00	6000	6394	6583	6977
20:00-00:00	6000	6406	6628	7034

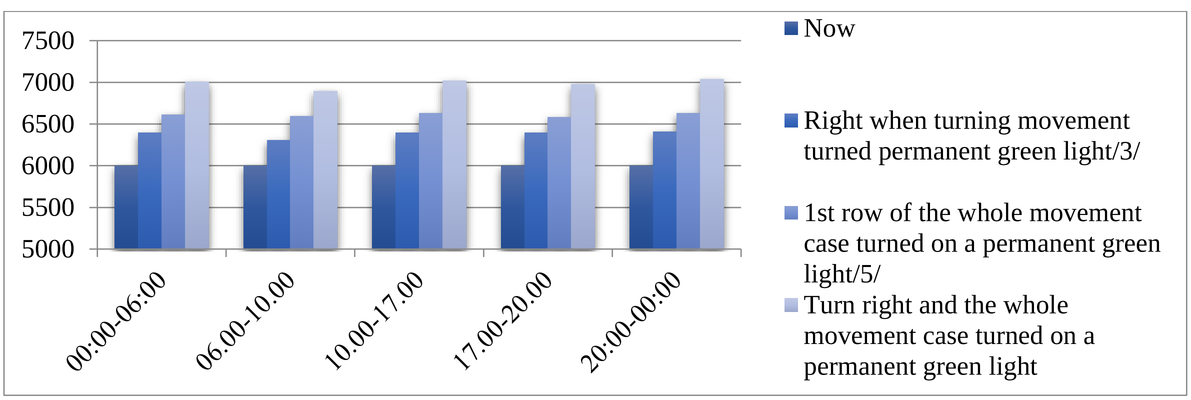
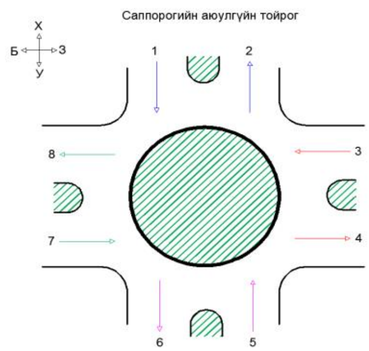


Figure 5. Thoughput of T-shaped Intersection (Bagshiin Deed)

Regarding to the above mentioned results from my calculations, making the first-lane going to the toward direction and making also the right turn with continuous direction in order to improve the passing capability of the T-shaped intersection it is possible to have the increase of the passing capability from 6.000 vehicles/hour to 7.000 vehicles/hour.

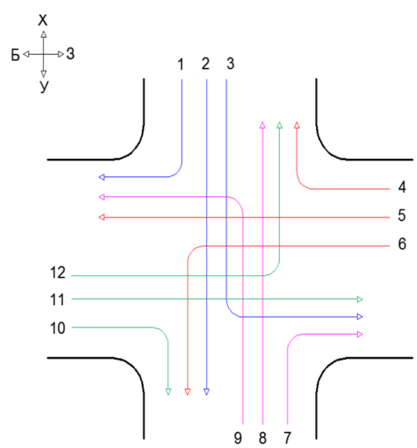
Now, let's compare by the passing capabilities of the circle and 4-road intersections using Sapporo's intersection as example. The traffic power censuses of 2011 and 2015 in the Sapporo's intersection were shown in the Tables 8 and 9.

Table 8. The Estimation of Traffic Power of the Sapporo's Intersection When It Was Roundabout



The Sapporo's intersection				
Route number	Vehicle	Bus	Truck	Transfer values to light vehicles
1	1557	28	188	1909
2	1329	36	147	1640
3	1543	96	223	2118
4	1880	80	69	2184
5	915	3	74	1034
6	1314	47	93	1571
7	1385	74	187	1851
8	1830	105	114	2264
Total				14571

Table 9. The Estimation of Traffic Power of the Sapporo's Intersection When It Was Intersection



Sapporo's intersection	
Route number	Transfer values to light vehicles
1	340
2	520
3	380
4	400
5	1600
6	700
7	400
8	720
9	510
10	400
11	1700
12	960
Total	8630

Regarding the above results, the roundabout has higher passing capability than the cross intersection four legs. Also, the circled intersection has more advantage from the cross intersection four legs as it has continuous and stable traffic.

5. Conclusions

The implementation of the special-lane for the public transport in the Peace Avenue of Ulaanbaatar city would have temporary effects but in the future it will have ineffective results as during the estimating calculation for the street capacities and passing capabilities during the presence and absence of the public transport lanes was determined to have bad results during the presented lanes.

The implementation of the traffic lights' cycles with the time sensitivity programs in the intersections during the normal traffic circulations plays significant impacts on reducing the heavy traffic on the roads of Ulaanbaatar city. Furthermore, it is possible to calculate depending from the seasons.

The number of the passing vehicles per one hour will become same when the passing capability of the intersections will be equalized. The differences in the passing capabilities of the intersections in the Peace Avenue are causing the formation of the heavy traffic.

It is clear that the intersectional passing capability will be reduced by about 15 percent if the right turning lane and the first lane of the T-shaped intersection will have continuous directions so using this method on the intersection with three roads will have same results as the use in the intersection with four roads and their passing capabilities will be increased too.

The roundabout has higher passing capability then the cross intersection with four legs as it has continuous passes without any obstacles, the transformation of the intersections located in the city outskirts including the supporting roads will reduce the traffic in Ulaanbaatar city.

The traffic problems of Ulaanbaatar city is closely related to its urban planning and architectural solutions. Although the city is stretched from East to the West, there is only one road connecting the two sides. No more roads are in the city to share its loads. Other roads are responsible for some parts of the loads only. Therefore, it is necessary to build more roads going from North to South then only from East to West of the city which will divide the traffic loads into different parts which also gives the conclusion as the construction of the highways from West to North would completely remove the traffic jams of Ulaanbaatar city.

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Calibration and Validation of Microsimulaiton Software for Intersection of Karachi

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Abstract
Traffic microsimulation has been one of the most widely used tools to solve the large scale urban congestion and planning problems. PTV VISSIM, a product of PTV group is amongst the most used traffic microsimulation software across the globe. The aim of the project is to form a set of calibrated traffic parameters values of the Weidmann 74, and 99 Car following models (models used by VISSIM) and parameters of lane changing behaviour for the signalized intersections of heterogeneous traffic of Karachi city. Two intersections are selected for analysis and modelling namely, i) Jauher Chowrangi and ii) Allah-Wali Chowrangi. The project consisted of conducting traffic count survey of one day to obtain traffic flow data, analysing data, and generation of a model for the signalized intersection under study in PTV VISSIM. A calibration exercise is conducted on data obtained from Jauher Chowarangi and validation of the set of obtained parameters is carried out on another intersection of the city i.e. Allah Wali Chowrangi. GEH statistics is used to develop relation between observed and simulated traffic flow. GEH statistic shows a strong correlation between the observed and simulated flow Based on the calibration and validation results of two chosen intersections, it was found that in Karachi, parameters for average standstill distance in the car following model, and lateral distances both standing at 0kmph and moving at 50kmph in the lane changing model are found to be the governing parameters. The importance of these parameters is more because the significant percentage of the total traffic consists of motorcycles.

Keywords
Micro simulation, Sensitivity analysis, Car following parameters, Heterogeneous traffic, Intersection.

1. Introduction

Karachi is the economic hub as well as the only metropolitan city of Pakistan. The associated seaport and vast opportunities offered by the city is resulting in an increase in the population of the city. Current population growth of the city is estimated at 4% annually. The busy congested city of 3527km^2 comprising of 23.1 million population in 2015, some estimated trips of 24.5 million every day (JICA, 2012). The city is connected by a complex road network. To keep up with the requirements of the growing city there has been a continuous but unplanned development in road infrastructure. Development of the road infrastructure is a costly affair and has not yielded the desired results.
In past few years, there has been a major awareness in the utilization of traffic engineering for the road infrastructure development of the city. This has encouraged the traffic engineers to use traffic engineering tools like microsimulation to model the road infrastructure and traffic control systems before actual

implementation to foresee its impacts. PTV VISSIM (PTV VISSIM, 2014), a micro simulator based on driving behavior is one of the most used traffic management and planning software. However, being a software based on driving behavior and patterns of Germany, it requires calibration of its parameters as per the local driving behavior. In developed countries such as Pakistan, India, Bangladesh, there is a problem exist, while using the micro-simulation software because of traffic heterogeneity and non-lane based traffic, as these models are mainly made for homogeneous and lane based traffic.

2. Literature Review

Siddharth and Ramadurai give a method for automatic calibration of VISSIM for a heterogeneous intersection of Chennai, India using C++. Similarly, Mathew and Radhakrishnan calibrated a micro-simulation software for heterogeneous traffic at a signalized intersection (Siddharth and Ramadurai, 2013). The traffic at that area is also Non-lane-based. In both the studies Genetic Algorithm (GA) is used to calibrate (minimize the difference between observed and simulated vehicles). Arkatkar et al., gives a methodology for micro-simulation for heterogeneous traffic on Indian (Delhi-Gurgaon) expressways (Arkatkar et al., 2014). Mehar et al., calibrated VISSIM for mixed traffic condition to find the highway capacity (Maher et al., 2014). Manfred et al., proposed a methodology for calibration of micro-simulation software based on mid-block section and an intersection. The calibration is done for heterogeneous traffic condition considering different vehicle class-wise driving. Also, they applied single and multi-criteria calibration using GA (Manfred et al., 2017). Another study outlines methodology for calibration of VISSIM in mixed traffic (Manjunatha, 2012). Above shown studies have calibrated micro simulation software for heterogeneous i.e. mixed traffic conditions mainly for India. However, there might exist some differences in the traffic due to differences in human behavior, vehicle composition, and vehicle types and sizes. Therefore, this study shows the calibration of a micro-simulation car following model and lane changing model for heterogeneous traffic of Karachi city.

This study comprises of calibration and validations of parameters of the car following models, Wiedemann74 and 99 and lane changing model used in PTV VISSIM for a signalized intersection of Karachi. Johar Chowrangi, a 4-legged signalized intersection, located in the heart of Gulistan e Johar area of Gulshan Town. The area surrounding the intersection comprises of residential cum commercial land use. It provides a link to the Karachi Airport and two of the most prominent universities of Karachi are situated in its vicinity. Development of many housing and commercial projects in the vicinity of the intersection make it significantly important. A traffic count was conducted to obtain accurate traffic data. This was followed by generation of a model in the software of the intersection. Based on the Sensitivity Analysis results, trial and testing a set of calibrated parameters for both the car following models were obtained. These calibrated parameters were then tested on another 4-legged known as Allah Wali Chowrangi located in the heart of Karachi City in the area adjoining Tariq Road, PECHS a busy commercial and residential area located in Jamshed Town, to validate the calibrated parameters.

3. Methodology

Research methodology for this study is shown in Figure . As discussed in previous section, literature review has been done to show the latest work done to calibrate the traffic micro-simulation software in different conditions. The second important step is to collect data. Mode-wise volumetric traffic data at 10-minute interval is extracted from video, recorded at Jauher Chowrangi on typical week day, Tuesday, 19 May 2015 for 16 hours i.e. 6:00 A.M – 10:00 P.M. The nature of traffic is heterogeneous therefore, traffic data is classified into different categories, which includes Motorcycles/Scooters, Rickshaws, Cars, Busses/Coaches and heavy vehicles.

The peak hour volume varied significantly from one direction of the intersection to the other, ranging from 2707 Vehicle Per Hour (vph) (from Pehelwan Goth side) to 5650 vph (from Johar Morr). From Johar morr, peak hour occurred between 6:00-7:00 PM, from university road, peak occurred between 5:50-6:50 PM with a volume of 4108 vehicles, it occurred between 5:20-6:20 PM from Pehelwan Goth and peak was observed between 7:30-8:30 AM from Rado bakery. Peak hour used for the model was 5:45 PM-6:45 PM. During this period 3 of the directions were close to their peak traffic flow volume. This is the peak traffic flow passing the intersection, all directions combined during the 16-hour traffic survey.

This variation is attributable to different traffic patterns on different routes, mainly due to school starting and off timings and office starting and ending time. The variation in the time at which the peak hour volume occurred from Rado bakery side i.e. morning time, is due to office and school starting time, because the traffic on this link is mainly coming from residential area and going towards commercial area i.e. offices.

The sixteen-hour volume to Johar Chowrangi from Johar more, University road, Pehlwan Goth road, and Rado Bakery is found to be 51155, 40229, 30056, and 38497 vehicles respectively. It can be seen that road from Johar more has the highest volume throughout the day. It is approximately 70% more than the volume from Pehlwan Goth road which is also serving airport. Car and bike shares approximately same percentage i.e. 40%. Buses and HGVs are 1% and 0.4% respectively. While, Qinqi and rickshaw has a share of 9% and 12%, respectively.

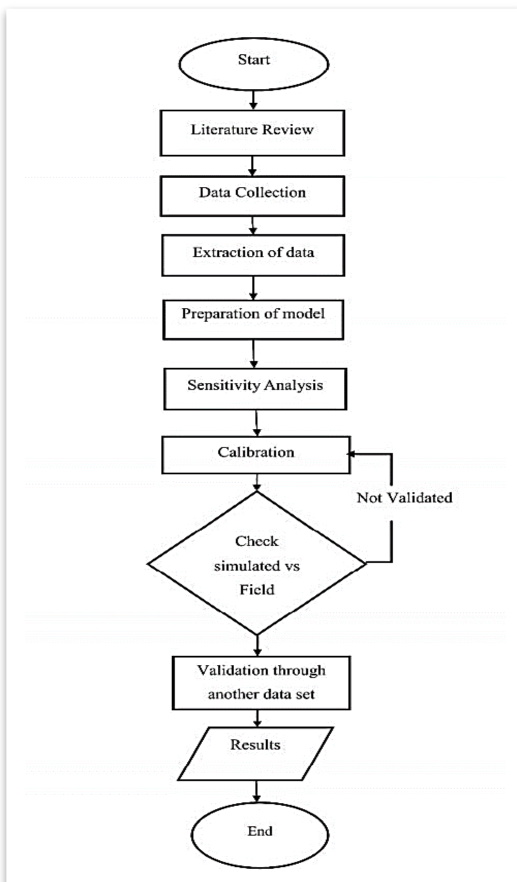


Figure 1: Procedure followed for calibration and validation of VISSIM

Car following model	No.	Parameter	Default Value	Input Range
Wiedemenn 74 parameters	1	Average standstill Distance	2.00	[1,3]
	2	Additive Part of Safety Distance	2.00	[1,3]
	3	Multiplicative Part of Safety Distance	3.00	[1.5,4.5]
	4	Collision time Gain	2.00s	[1,3]
	5	Minimum Longitudinal Speed	3.60km/h	[1.8,5.4]
Wiedemann 99 parameters	6	CC0: Standstill distance	1.50m	[0.75,2.25]
	7	CC1: Headway Time	0.90s	[0.45,1.35]
	8	CC2: ‘Following’ variation	4.00m	[2,6]
		CC3: Threshold for entering ‘following’	-8.00	[-12,-4]
	9	CC4: Negative ‘following’ threshold	-0.35	[-0.175,-0.525]
	10	CC5: Positive ‘following’ threshold	0.35	[0.175,0.525]
	11	CC6: Speed dependency of Oscillation	11.44	[5.72,17.16]
	12	CC7: Oscillation Acceleration	0.25m/s²	[0.125,0.375]
	13	CC8: Standstill Acceleration	3.50 m/s²	[1.75,5.25]
	14	CC9: Acceleration with 80 km/hr	1.50 m/s²	[0.75,2.25]
	15	Minimum Lateral distance standing at 0 km/h	1.00 m	[0.5,1.5]
	16	Minimum Lateral distance driving at 50 km/h	1.00m	[0.5,1.5]

Figure 1: Wiedemann 74 & 99 car following parameters, their default values and input range for sensitivity analysis.

The calibration of microsimulation traffic model is a key component to the success of the simulation modeling project. An effective calibration effort results in the confident future analysis of the study area. The proposed calibration methodology includes three major stages; 1) base model development, 2) calibration of the base model, and 3) model validation.

The first stage of the calibration process is the development of the base simulation model. The base model provides the input to the calibration stage. Careful considerations on the selection of the study area size, data collection requirements, and selection of time periods should be made during the base model development stage to ensure that the model will not encounter problems during the calibration stages. Potential calibration problems that may be avoided during the base model development stage include selecting the study area carefully which may not be very long or too short that it skips the important areas including bottlenecks and intersections.After a literature review, set of parameters of both road following

models i.e Wiedemann74 and 99 were observed on site and other input requirements of VISSIM such as signal timing and cycle length, lane dimensions were also collected.
The first step for preparation of model was to locate the intersection and lay out the geometry as on site. VISSIM is equipped with Open Street maps, which were used for location identification and laying out the geometry. As a signalized intersection, signal heads governed by a signal program was defined in the model. Signal timing for each movement was determined in the model as of obtained from the site. After the model was completed in terms of physical features, traffic data was added. A 4-legged traffic intersection can have four moments straight, left turn, right turn and U-turns. Traffic routing decision option in VISSIM was used to determine the share of each movement. Like in real life every intersection has conflict areas where two or more than 2 traffic paths overlap, so in order to make the model precise conflicts areas were defined.

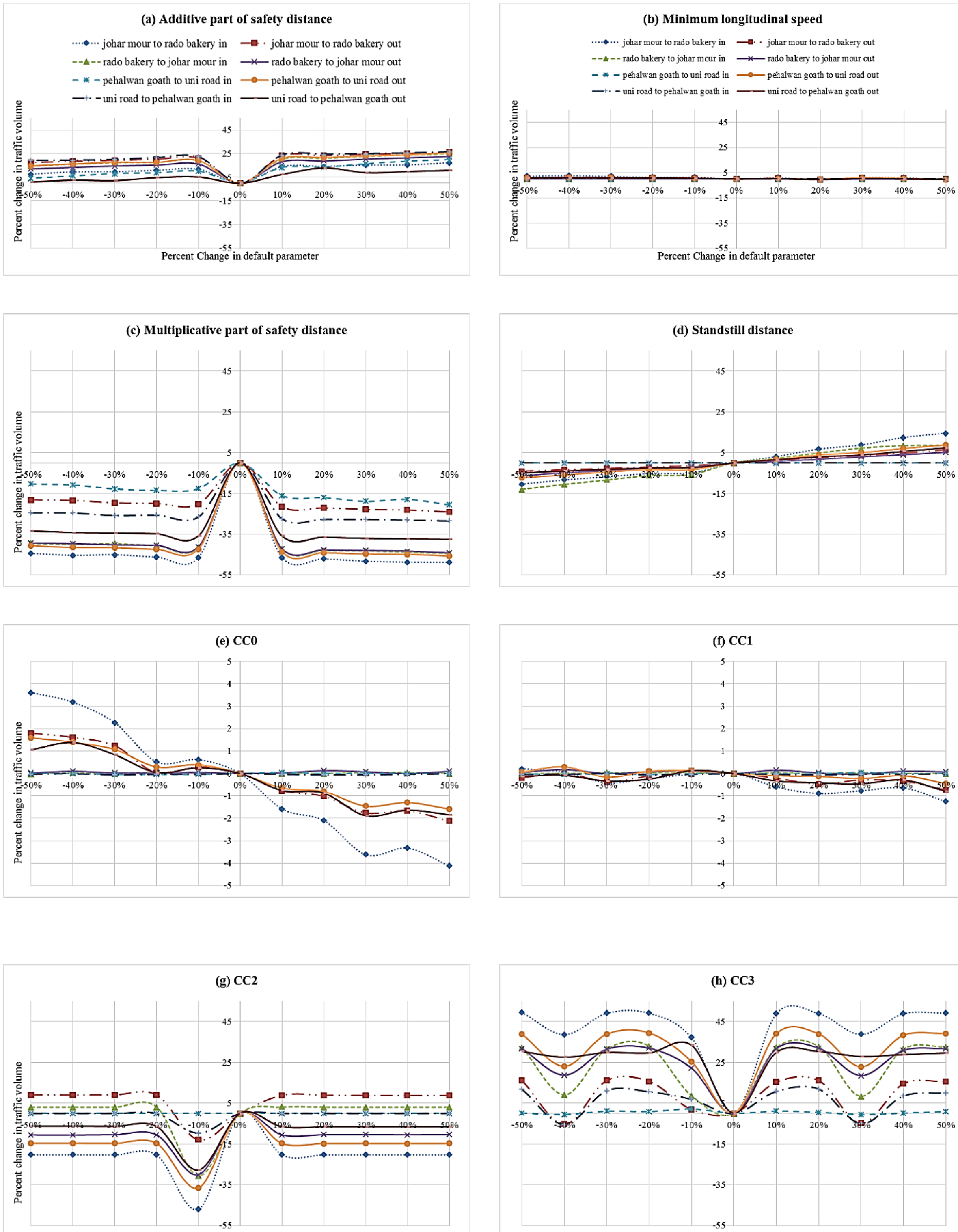
Sensitivity analysis is used to determine the parameters which have the most influence on the flow of traffic in the model. Sensitivity analysis prior to the calibration enables us to identify the parameters that govern our model and making the calibration process more accurate, time effective and result oriented. List of parameters with default values against sensitivity analysis values is shown in Figure 2. Few results of the sensitivity analysis are shown in graphs. Figure 3 Average standstill distance, Figure 4 Additive part of safety Distance, Figure 5 CC0, Figure 6 CC1, and Figure 7 CC2.
A graph for variation in volume simulated vs change in parameter for every parameter of both Wiedemann74 and 99 was plotted.

A maximum of 15% difference in the volume was observed for Average standstill distance, for Additive part of the safety distance the difference observed was 5%. Difference for CC0, CC1 and CC2 was found to be 5%. 3% and 4% respectively. Calibration of the model was conducted so that the obtained parameter values would generate results that should be near to the results obtained in real-world Conditions. Based on results observed from sensitivity analysis number of iterations or simulations were performed in combination of different sets of parameter values. Based on GEH Statistic tool (GEH statistics, 2016) and Percentage Difference method (Mathew and Radhakrishnan, 2010) the set of values of parameters which gave the closest results to that of the real-world were obtained. After successful calibration of the model, another model was developed for a different location of the city as discussed earlier and calibrated parameter values were used to perform simulations. Using the GEH Statistic Tool (GEH statistics, 2016) and Percentage Difference method (Mathew and Radhakrishnan, 2010) the results were compared to the one obtained from the real world data by classified counts conducted by a Transportation Engineering Firm. Based on the GEH Statistic Tool values (GEH statistics, 2016) and percentage difference method results (Mathew and Radhakrishnan, 2010), set of values of parameters proposed after the calibration was validated.

4. Result

Based on the sensitivity analysis performed for flow parameters in Wiedemann74, Average standstill distance and Additive part of the safety distance were found to be most sensitive parameters. For Wiedemann99, CC0 (standstill distance), CC1 (headway time) and CC2 (following variation), minimum lateral distance standing at 0 km/h and minimum lateral distance driving at 60km/h was found to be most sensitive. The graph obtained from sensitivity analysis of VISSIM is shown in
Figure **2**.
Figure **2**(a),(b),(d), (e), and (f) shows either direct or indirect relation between the values of parameters and change in volume. While, other graphs show a hap-hazard relation of volume change with dependent variable. A quick dip in the volume change near default value is seen in all these graphs i.e.

Figure **2** (c), (g), (h), (i), (j), (k), (l) and (m). This suggest that the car following parameters and lane changing parameters are very sensitive when the values used are between ±10% and needs further investigation which is out of scope of this study.



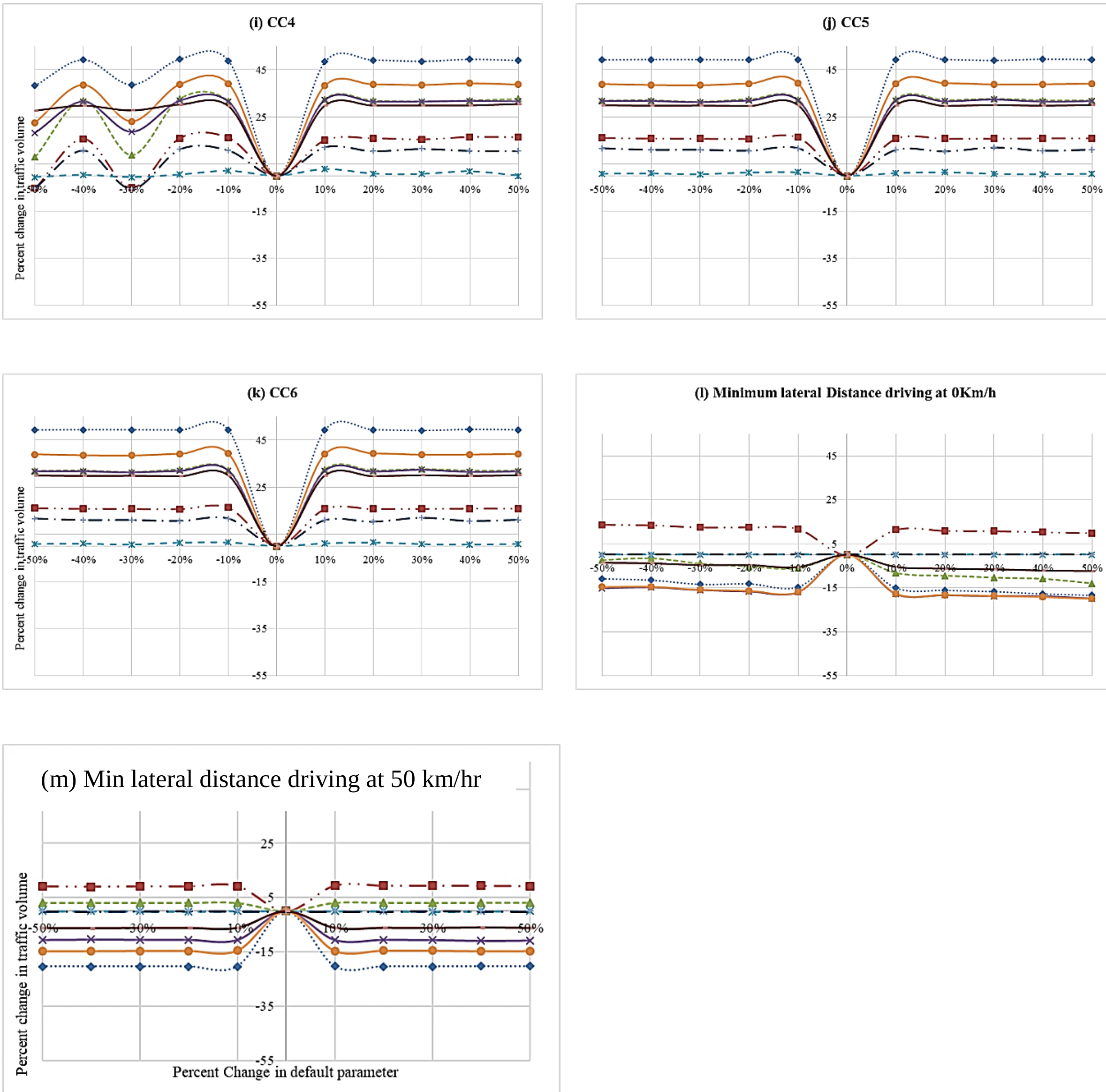


Figure 2 Graph of sensitivity analysis for (a) additive part of safety distance (b) Minimum longitudinal speed (c) Multiplicative part of safety distance (d) Standstill distance (e) CC0 (f) CC1 (g) CC2 (h)CC3 (i) CC4 (j) CC5 (k) CC6 (l) Minimum lateral Distance driving at 0 km/h (m) Min lateral Distance driving at 50 km/h

Calibrated values for parameters of Wiedemann74 and Wiedemann99 obtained after multiple simulation runs are shown in Figure 8 & 9 respectively. These sets yielded the minimum difference between the simulated and the observed results. GEH Statistical tool value less than 5 and percentage difference less than 15 (Mathew and Radhakrishnan, 2010) is considered as a very good calibrated model. Figure 10 & 11 is showing the simulation results for Wiedemann74 and Wiedemann99 respectively.

Parameter	Calibrated Value	Default Value
Average standstill Distance	0.70	2.00
Additive Part of Safety Distance	0.10	2.00
Multiplicative Part of Safety Distance	0.10	3.00
Minimum Longitudinal Speed	1.80 km/hr.	3.60 km/hr.

Figure 3: Calibrated and default values for Wiedemann74 parameters

Parameter	Calibrated Value	Default Value
CC0: Standstill distance	0.50 m	1.50m
CC1: Headway Time	0.72 s	0.90s
CC2: 'Following' variation	3.00 m	4.00m
Minimum Lateral distance standing at 0 km/h	0.50 m	1.00m
Minimum Lateral distance driving at 50 km/h	0.50 m	1.00m

Figure 4: Calibrated and default values for Wiedemann 99 parameters

Location	0 min to 15min	15min to 30min	30min to 45min	45min to 60min	Sum	Actual	Diffe-ence	%age diff.	GEH
Johar mour to Rado bakery in	1234	1138	1147	1138	4657	4822	165	3	2.39
Johar mour to Rado bakery out	1002	1003	960	1012	3977	4212	235	6	3.67
Rado bakery to Johar mour in	791	791	756	755	3093	3094	1	0	0.01
Rado bakery to Johar mour out	1041	1034	1023	997	4095	4126	31	1	0.48
Pehalwan goath to University road in	583	586	567	567	2303	2245	-58	-3	1.21
Pehalwan goath to University road out	756	737	768	772	3033	3195	162	5	2.90
University road to Pehalwan goath in	906	907	881	884	3578	3589	11	0	0.18
University road to Pehalwan goath out	603	570	579	589	2341	2556	215	8	4.34

Figure 5: Wiedemann74 simulation results

Location	0 min to 15 min	15min to 30min	30min to 45min	45min to 60min	Sum	Actual	Differ-ence	%age diff	GEH
Johar mour to Rado bakery in	1223	1124	1087	1052	4486	4822	336	7	4.92
Johar mour to Rado bakery out	982	982	949	983	3896	4212	316	8	4.96
Rado bakery to Johar mour in	792	791	756	755	3094	3094	0	0	0

Rado bakery to Johar mour out	1026	1048	1021	993	4088	4126	38	1	0.59
Pehalwan goath to University road in	583	586	567	567	2303	2245	-58	-3	1.21
Pehalwan goath to University road out	740	732	762	757	2991	3195	204	6	3.66
University road to Pehalwan goath in	906	907	880	885	3578	3589	11	0	0.18
University road to Pehalwan goath out	596	583	567	577	2323	2556	233	9	4.71

Figure 6: Wiedemann74 simulation results

After calibration, it is necessary to validate the model to assure that the model represents the field conditions. For another data set of the signalized intersection, Allah Wali Chowrangi was modeled with the calibrated values of Wiedemann74 and Wiedemann99. Results obtained are shown in Fig 12 & 13 for Wiedemann74 and Wiedemann99 respectively.

GEH Statistic value and Percentage Difference of the simulated and the original data is less than 5 (GEH statistics, 2016) and 15% (Mathew and Radhakrishnan, 2010) respectively, thus the validation results indicate that calibrated parameters were able to replicate field volumes. Therefore, GEH values for both calibration and validation is satisfactory.

Further to strengthen the Calibration, Level of Service was calculated for the original intersection in the VISSIM model and was compared to HCM delay calculation method (HCM, 2010). Delay observed were, 41.77s for Johar Mour to Rado Bakery, 38.86s for Rado Bakery to Johar Mour, 39.40s for University Road to Pehalwan Goath and 37.61s for Pehalwan Goath to University Road. The overall LOS for this intersection by HCM method was found out to be D (HCM, 2010). Average delay found from a calibrated model from VISSIM for Johar Mour to Rado Bakery, Rado Bakery to Johar Mour, University Road to Pehalwan Goath and Pehalwan Goath to Uni Road to be 40.22 sec, 39.36sec, 23.61sec and 21.09 sec respectively. These delays gave LOS D for Johar Mour to Rado Bakery, LOS D for Rado Bakery to Johar Mour, LOS C for University Road to Pehalwan Goath and LOS C for Pehalwan Goath to University Road and an overall LOS D.

Location	0min to 15min	15min to 30min	30min to 45min	45min to 60min	sum	actual	Differ-ence	%age diff	GEH
Nursey in	858	909	944	797	3508	3509	1	0	0.01
Nursery out	743	796	818	797	3154	3123	-31	-1	0.55
Numaish in	703	711	805	739	2958	2964	6	0	0.11
Numaish out	665	623	669	683	2640	2612	-28	-1	0.54
Tariq road in	224	278	296	266	1064	1020	-44	-4	1.36
Tariq road out	119	110	127	117	473	466	-7	-2	0.32
SMCHS in	175	192	193	187	747	748	1	0	0.03
SMCHS out	499	532	586	542	2159	2213	54	2	1.15

Figure 7: Wiedemann74 Calibration results

Location	0min to 15min	15min to 30min	30min to 45min	45min to 60min	Sum	actual	Differ-ence	%age diff	GEH
Nursey in	858	887	789	786	3320	3509	189	5	3.23
Nursery out	726	760	738	763	2987	3123	136	4	2.46
Numaish in	703	712	805	739	2959	2964	5	0	0.09
Numaish out	661	631	669	681	2642	2612	-30	-1	0.58
Tariq road in	300	305	311	303	1219	1220	1	0	0.02
Tariq road out	118	118	117	119	472	466	-6	-1	0.27
Sindhi muslim in	175	192	193	187	747	748	1	0	0.03
Sindhi muslim out	483	515	544	505	2047	2213	166	8	3.59

Figure 8: Wiedemann99 Calibration results

5. Conclusions

The calibration on an uptown signalized intersection and validation on another signalized intersection has given a set of calibrated parameters which can be further used to model any signalized intersection of Karachi city. Reduction of standstill distance, the lateral distance both at 0km/h and driving at 60km/h shows an aggressive driving behavior in Karachi City as compared to Germany. Comparison of LOS obtained from delay calculated by software and field calculation shows that the intersection has an overall LOS D (HCM, 2010). The delay observed is due to the traffic signal installed for traffic control.

6. Recommendations

This study was performed for an intersection in an uptown area of Karachi only and it is recommended that other types of intersections and roads should also be calibrated as well, including urban areas, city centers, freeways, and highways. It is suggested that before making any amendments in the physical, geometrical shape or changing any traffic control devices they should be modeled to get a realistic output. With the rapid growth in the area and congestion caused by the illegal traffic parking, LOS may drop further thus increasing delays so it's highly suggested to restrict lanes 300m in each direction as NO PARKING and provide parking area. Moreover, calibration of software for queue lengths and speed should also be done.

Also, the behavior of Weidmann Car following variables can be re-investigated in more detailed manner near default value region to get insights of sensitivity of car following parameters.

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Pavement Structural Evaluation of Peshawar Ring Road Using FWD Data

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Abstract
Nondestructive testing (NDT) of asphalt with use of a falling weight deflectometer (FWD) is a method that allows transportation engineers, municipalities and private engineering firms to gauge the characteristics of a roadway in situ. Specifically, depth and rigidity can be tested with the falling weight deflectometer. An accurate measurement of rigidity enables transportation engineers to understand how much displacement an asphalt system has relative to the load applied. Peshawar ring road from Charsadda road to Hayatabad is a 4-lane dual carriageway approximately 43Km in length (both directions) and is part of the Peshawar orbital constructed by Peshawar Development Authority (PDA). Its improvement and remodeling has been taken up the PDA and the project mainly entails improvement and widening of the existing carriageway to a six lane (3x3 lane) dual carriageway. Shortly after the winter rains in January 2012, hair line surface cracking at localized sections in the newly laid asphalt concrete base course layer was reported. Peshawar ring road over the previous year's especially after the year 2000 has evolved to be one of the busiest routes within the metropolitan. It not only caters for the local and suburban traffic but also provides a transit for heavy freight traffic bound for Afghanistan. Therefore, heavy axles have become a norm which could be witnessed at site. Themain objective of this research work was to evaluate the structural condition of the pavement layers both qualitatively and quantitatively. The assessment of the existing structural condition of the road sections included Non-Destructive Testing (NDT) using Falling Weight Deflectometer (FWD). Nondestructive testing was found to save approximately 80% the cost of destructive testing, not including the initial investment.The results indicate that there is a marked savings when using the falling weight deflectometer as a nondestructive testing method. Two significant factors that led to these savings were time spent in the field is decreased and labor needed to perform the testing in the field.

Keywords
Falling Weight Defelctometer, Non-Destructive Testing, Rehabilitation, NDT, PDA

1. Introduction

Peshawar ring road from Charsadda road to Hayatabad is a 4 lane dual carriageway approximately 43Km in length (both directions) and is part of the Peshawar orbital constructed by Peshawar Development Authority (PDA) nearly two (02) decades back. Recently, its improvement and remodeling has been taken up the PDA where approximately all the sections especially between the National Highway N5 and Hayatabad have been taken up for reconstruction. The project mainly entails improvement and widening of the existing carriageway to a six lane (3x3 lane) dual carriageway. To this date approximately l5 Km of length has been overlaid with asphalt base course and in the remaining sections other construction activities have been undertaken. Shortly after the winter rains in January 2012, hair line surface cracking at localized sections in the newly laid asphalt concrete base course layer was reported. Peshawar ring road over the previous years especially after the year 2000 has evolved to be one of the busiest routes within the metropolitan. It not only caters for the local and suburban traffic but also provides a transit for heavy freight traffic bound for Afghanistan. Therefore, heavy axles have become a norm which could be witnessed at site. As part of traffic operation plan during construction, the road was opened to two way traffic soon after the AC base layer was laid. The progression of distress was noted to get severe where intense cracking caused alligator form of cracking at localized sections causing the pavement to deteriorate at a much faster rate.

1.1 Objective of Research

The objectives of this paper are to evaluate the structural condition of the pavement layers both qualitatively and quantitatively. The assessment of the existing structural condition of the road sections included Non-Destructive Testing (NDT) using Falling Weight Deflectometer (FWD). The pavement structural evaluation comprised the following main elements, which were followed by detailed analysis on the site:

a) A structural evaluation by Non-Destructive Testing (NDT) using Khyber Pakhtunkhwa Highway Authority's (PKHA) Falling Weight Deflectometer (FWD);
b) Analysis/evaluation of " FWD deflection data and extrapolation of back-analyzed stiffness values for pavement layers;
c) Combined interpretation of observed distress at site and the measured deflection data by the FWD
d) Recommendation of suitable rehabilitation measures with regards to pavement structure.

1.2 Scope

The highway system serves as an important factor to a country's economic development and defense. Ring road is 43Km long carriage way stretching from Hayatabad at north to Charsadda road. It is the main circumferential road used for trade from Afghanistan to lower part of the country.

2. Literature Review

2.1 Falling Weight Deflectometer overview

Pavement surface deflection measurement plays an indispensable role in evaluating a flexible pavement structure. It can be used to monitor the pavement performance, calculate the pavement layer moduli and the subgrade resilient modulus, and identify potential problem areas in the pavement. Many nondestructive deflection testing equipment's are available for pavement engineers. These equipments can be divided into three categories: static deflections (Benkelman Beam), steady state deflections (Eagleson *et al.*,1982). Falling Weight Deflectometer (FWD) is a nondestructive test device widely used in pavement engineering. It plays an important role in evaluating the physical properties and performance of a pavement. The main components of a FWD system include: control system, hydraulic system, loading weight and plate, load cell and deflection (Eagleson *et al.*, 1981).

Figure 1: Dynatest Model 8000 Falling Weight Deflectometer (Dynatest) Sensors
(Eagleson *et al.*, 1981)

During the test, the FWD applies a load to the pavement surface by dropping a large weight onto a load plate positioned on the pavement surface. This load simulates the magnitude and duration of a moving wheel load. The pavement response (surface deflection) due to the load is then measured by a series of deflection sensors mounted at various distances from the loading point (one sensor is located directly over the loading point). Usually, the deflections are measured at 0 inch, 8 inches, 12 inches, 18 inches, 24 inches, 36 inches and 60 inches away from the center of the loading plate. The measured deflections at each sensor called deflection basin. Figure 2 shows a schematic of FWD load and deflection measurement.

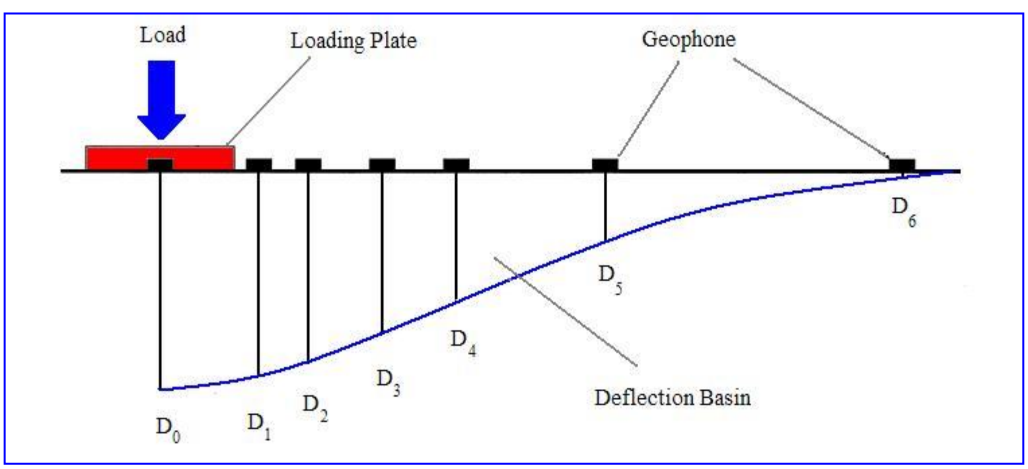

Figure 2. Schematic of FWD load and deflection measurement (Eagleson *et al.*, 1981).

The advantages of FWD test includes: it is accurately simulated the traffic load, it is quicker (can test up to 60 points in an hour) and can be operated by one person. The loading range of a FWD varies from 1,500 to 27,000 lbf (Dynatest, 2009).

2.2 MICHBACK Overview

MICHBACK is a computer program for the back-calculation of flexible and composite pavement layer properties using data obtained with a falling weight deflectometer (FWD). MICHBACK has many user-friendly features and advantages over other back-calculation programs.

- It is menu-driven
- has on-screen forms to facilitate data entry, with error trapping
- allows the use of English or SI units
- can accommodate up to 10 FWD sensors
- can read files created by the KUAB FWD
- displays a graphical preview of sensor data contained in FWD files
- allows preprocessing of FWD data
- detects and reports suspicious FWD drops
- can back calculate at any number of stations in a single run,
- can back calculate the "stiff layer" depth or one layer thickness in addition to the layer

3. Distress History

Almost 15 Km of the road has been rehabilitated and upgraded to 3 lanes as per construction strategies provided in the design document. The sections were completed up to asphalt base course. Asphalt wearing course has not been laid since it has been learnt that laying of surfacing shall be executed in one smooth operation alter preparation of other sections. Based on the information provided by the PDA site staff the first sighting of crack initiation in the newly laid asphalt base course was observed during the months of January and February 2011. Subsequent to the initiation of cracking distress the problem got widespread across the completed sections of the project. The pavement distress prevalent at site starts with the initiation of minor hairline cracks in the asphalt base course surface and grows in intensity until it is more visible on the surface. This is typical of cracking which once initiated propagates at a much faster rate. The cracks if not attended properly give Way to more ingress of water into the unbound pavement structure thus causing the pavement to fail under water action.

3.1 Back Calculations

Back-calculation is the process of computing pavement layer moduli and the subgrade resilient modulus based on pavement deflection basins generated by Falling Weight Deflectometer (Muench, et al., 2003). In order to conduct a back-calculation, the initial moduli of pavement layers should be first assumed, the values are usually estimated base on engineer’s experience or equations. After assuming the initial layer moduli, pavement surface deflections can be calculated using pavement response models. The calculated deflections are then compared to the measured values. By adjusting the pavement layer moduli, a good match (within some tolerable error) between the measured and theoretical deflections can be reached. The process of back-calculation is usually iterative. Many programs were developed for back-calculation such as Modulus 6.0,

Elmod 6.0, and Evercalc 5.0.
The figure 3 below presents a basic flowchart of back-calculation program. The main components in a back-calculation process include (Ceylan *et al.*, 2005)

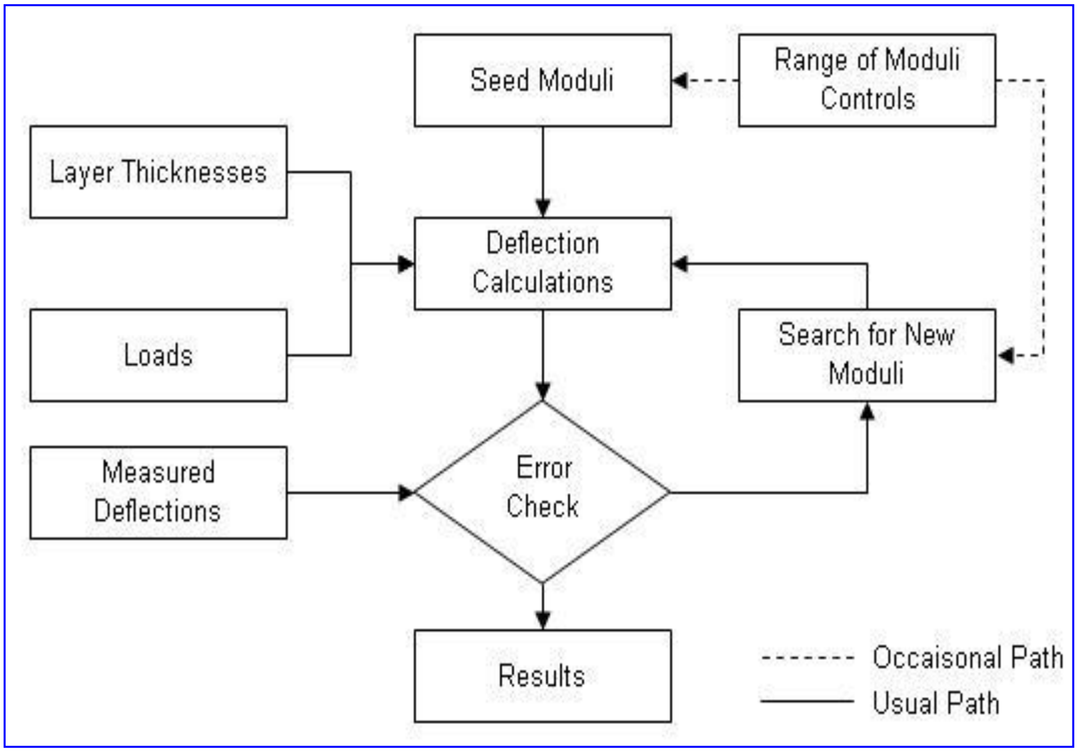

Figure 3: Back Calculation Flowchart (Ceylan *et al.*, 2005)

Layer thicknesses and loads: Thickness of each pavement layer and load levels applied on the pavement surface.
Measured deflections: Surface deflections measured during FWD tests.
Seed moduli: Initial modulus used to compute theoretical surface deflections.
Deflection calculation: Use pavement response models to calculate theoretical surface deflections.
Error check: Compare the calculated and measured deflections.
Search for new moduli: Iteratively search for the new moduli of pavement layers until the calculated and measured deflection are matched (within acceptable error).
Controls on the range of moduli: The back-calculation programs usually can define a range of modulus for each pavement layer to prevent unreasonable pavement layer moduli

4. Analysis & Results

Field Work pertaining to visual condition surveys, destructive testing of the pavement comprising pit logs, recovery of the soil and material at site, other non-destructive tests with regards to the permeability of the asphalt concrete surface has been done earlier. To evaluate the pavement structural condition of the existing pavement i.e recently rehabilitated up to the asphalt base course level, through non-destructive testing, Khyber Pakhtunkhwa Highway Authority's (PKHA) Falling Weight Deflectometer (FWD) was used at site on 18" July 2012. For this research work only the right carriageway from Hayatabad to N5 (GT Road) has been the focus of this analysis where rehabilitation of nearly 15 Km has been completed up to the asphalt base course level in intermittent sections.

4.1 Existing Pavement Layers Configuration

From the study of the materials it was observed that in total 48 number of test pits were excavated out of which 24 test pits were excavated for existing pavement while remaining 24 test pits were excavated in the proposed area for extension of carriageway. The underlying unbound material has been designated in top and bottom unbound material layers. The thickness of asphalt varies from 8-15 cm averaging to approximately 10 cm. The thickness of top unbound layer varies from 38-90 cm with thickness of bottom unbound layer in range from 50-82cm. The depths for taking the sample material also varies in each pit log. The layer coefficients (as used for pavement design) and corresponding pavement layer thickness are shown in Table 1:

Table 1: Design of Flexible Pavement

Layer	Layer's Coeff (/inch)	R-S – 1 (cm)	R-S-2 (cm)	R-S – 3 (cm)	R-S-4 (cm)
Asphalt Wearing course	0.40	5	5	5	5
Asphalt Base Course	0.40	10	10	10	Levelling+Patch work
Aggregate Base Course	0.135	22.5	15	0*	-
Granular Sub Base	0.126	20	30*	30*	-
Subgrade (30% CBR at 95% MDD)					

*** indicates existing pavement layers (material which already existed prior to this rehab)**

In total four construction strategies are applicable across the entire project in varying rehabilitation of the existing carriageway and widening sections are given as follows.

- **R-S-1:** Total rehabilitation.
- **R-S-2:** Base stabilization
- **R-S-3:** Replacement of existing asphalt concrete
- **R-S-4:** Asphalt concrete wearing course overlay with Patchwork.

It may be noted that the construction strategies with regards to the Chainage length have been slightly modified as per site condition during construction where instead of R-4, RS-3 has been applied for the Chainage length rehabilitated, **Table 2** below defines the application of strategies with regard to the sectional length under investigation:

Table 2: Application of Rehabilitation Strategies versus the Chainage Length

Carriageway /Direction	Section*	From (Km)	To (Km)	Rehabilitation Strategy Applied
Right / Towards N5	1	6+450	11+500	RS-1
Right / Towards N5	2	12+700	17+500	RS-3
Right /Towards N5	3	l8+700	23+000	RS-3

*** Denotation for referencing of the sectional lengths tested**

4.2 Falling Weight Deflectometer Testing (FWD)

The objective of Non-Destructive Testing (NDT) through use of Falling Weight Deflectometer (FWD) is to measure the pavements structural response to heavy dynamic loads, similar in magnitude and duration to those produced by moving wheel loads. The collected deflection data is then used to determine the pavement layer material properties and subgrade support. These values may be used to determine the structural capacity of the pavement. The equipment used for the deflection testing is KUAB Two Mass Falling Weight Deflectometer (KUAB 2m-FWD). The FWD testing was ‘carried out on l8th July 2012. Testing was performed on the centre lane of the rehabilitated pavement which is predominantly an existing lane and has been overlaid. The Section wise testing program is given in Table 3:

Table 3: FWD Testing Matrix

Carriageway/Direction	Section	Test spacing	From (Km)	To (Km)	Sec:Length (Km)
Right/ Towards N5	1	500	6+450	11+500	5.05
Right/ Towards N5	2	100	12+700	17+500	4.80
W.Right / Towards N5	3	100	18+700	23+000	4.30
TOTAL					**14.15 – 15 Km**

The Contact pressure applied by the FWD varies slightly from point to point which is due to the variability of pavement response. The FWD data was obtained by applying two different approximate loadings, **9000 lbs** and **15000 lbs**. Due to variations in pavement response, the Contact pressure applied by the FWD varies slightly from test to test. To compare test points for this survey, all the data was normalized to its respective load. The following deflection parameters, which indicate the performance of various layers of the pavement, were used for the interpretation purposes:

a) Central deflection (d_l): indicator of the overall pavement response

b) Deflection difference (d_1-d_2): indicator of the response from bituminous layer

c) Deflection difference (d_2-d_4): indicator of response from unbound granular Layer

d) Outer deflection (d_6): indicator of the subgrade response

A study of the profiles enables the relative condition of the pavement layers and subgrade to be assessed qualitatively. Generally, the higher deflections indicate weaker area, The peaks in the deflection profiles indicate poorer performance or distressed areas. A statistical analysis was performed using the values of the FWD deflection parameters. A summary of the 50 percentile (average) and 85 percentile (i.e. only 15% of the deflections are worse) deflection parameters are given in Tables 4 and 5 for representative sections and different loading conditions respectively.

Table 4: FWD Deflection Parameters (50th & 85th percentile values) - 9000 lbs

Sections	Percentiles	Deflection Parameters			
		d_1 (mils)	d_6 (mils)	d_1-d_2 (mils)	d_2-d_4 (mils)
Km 6+450 To Km 11+500	50th	6.8	0.9	2.3	2.5
	85th	7.2	1.1	2.5	2.8
Km 12+700 To Km 17+500	50th	13.4	1.3	4.6	5.2
	85th	16.5	1.9	5.4	6.8
Km 18+700 To Km 23+000	50th	13.0	1.3	4.4	5.0
	85th	16.1	1.6	5.3	6.6

Table 5: FWD Deflection Parameters (50th & 85th percentile values) - 15000 lbs

Sections	Percentiles	Deflection Parameters			
		d_1 (mils)	d_6 (mils)	d_1-d_2 (mils)	d_2-d_4 (mils)
Km 6+450 To Km 11+500	50th	10.7	1.7	3.2	4.0
	85th	11.3	2.1	3.6	4.3
Km 12+700 To Km 17+500	50th	20.9	2.4	6.3	8.2
	85th	25.5	3.5	7.3	9.9
Km 18+700 To Km 23+000	50th	19.2	2.3	5.7	6.9
	85th	23.7	3.0	6.8	9.4

4.3 Back Analysis of FWD Data

All the FWD test points were included in a detailed back-analysis procedure to determine effective stiffnesses of the pavement layers. The pavement was considered to be fully flexible. The pavement structure was modeled as three layered structure (bituminous + unbound granular layer + subgrade). The aggregate base course and sub-base material has been considered as a one single layer to extrapolate the effective stiffness modulus for the unbound material. The combination of both the unbound layer into one layer is attributed to the fact that the aggregate base and granular sub-base if modeled separately may not yield to the designer’s confidence to extrapolate realistic modulii values.

The results of the back-analysis are made in the form of approximate correlations between the effective stiffnesses obtained through the back-analysis and the appropriate deflection parameter, the material stiffnesses corresponding to the 50 and 85 percentile deflection parameters were abstracted and are shown in **Tables 6 & 7** for both loading conditions on representative sections respectively. Since the evaluation of the pavement structure of the respective sections was considered to be critical with regard to the existing; strength and subsequent recommendation, therefore, it was pertinent to analyze the deflection data on different back-calculation software programs to account for any variability among the strength values of the pavement layers. For this purpose four softwares can be used namely; ERIDA, MICBACK, RADAN 7 and MODULUS computer programs. The result obtained from these softwares for each pavement layer material is presented in **Tables 6 & 7**:

Table 6: Back-Analysed Effective Layer Stiffnesses (50th & 85th " Percentile Values) - 9000 lbs

Sections	Percentile	Asphalt Layer Modulus (ksi)			Granular Layer Modulus (ksi)			Sub-grade Modulus (ksi)		
		Erida	Mich	Mod	Erida	Mich	Mod	Erida	Mich	Mod
Km 6+450 To Km 11+500	85th	76	403	164	122	41	153	47	80	51
	50th	83	742	185	133	67	170	53	86	57
Km 12+700 To Km 17+500	85th	65	367	364	38	12	12	23	33	33
	50th	80	577	483	59	20	25	30	44	43
Km 18+700 To Km 23+000	85th	73	364	359	27	17	14	33	37	38
	50th	105	544	485	35	23	25	42	48	48

Table 7: Back-Analysed Effective Layer Stiffnesses (50th & 85th Percentile Values) - l5000lbs

Sections	Percentile	Asphalt Layer Modulus (ksl)			Granular Layer Modulus (ksl)			Subgrade Modulus (ksl)		
		Erida	Mich	Mod	Erida	Mich	Mod	Erida	Mich	Mod
Km 6+450 To Km 11+500	85th	75	472	177	129	34	134	39	71	45
	50th	87	550	195	156	39	157	46	73	52
Km 12+700 To Km 17+500	85th	75	367	484	27	12	13	29	33	31
	50th	103	574	622	37	20	26	36	44	41
Km 18+700 To Km 23+000	85th	69	364	473	29	17	15	34	37	35
	50th	92	544	637	43	23	28	42	48	48

Source: Peshawar Development Authority

The values obtained from each analysis for each pavement layer were checked and subsequently **RADAN 7** is used to check the respective pavement layer is reported in **Tables 8 and 9**:

Table 8: Back-Analysed Effective Layer Stiffnesses Resulting from RADAN 7 – 9000 lbs

Sections	Percentiles	Asphalt Layer Modulus (ksi)	Granular Layer Modulus (ksi)	Sub-grade Modulus (ksi)
Km 6+450 To Km 11+500	85th	241	405	60
	50th	337	123	65
Km 12+700 To Km 17+500	85th	265	21	29
	50th	380	34	39
Km 18+700 To Km 23+000	85th	265	19	36
	50th	378	28	46

Table 9: Back-Analysed Effective Layer Stiffnesses Resulting from RADAN 7 - 15000 lbs

Sections	Percentiles	Asphalt Layer Modulus (ksi)	Granular Layer Modulus (ksi)	Sub-grade Modulus (ksi)
Km 6+450 To Km 11+500	85th	242	99	52
	50th	277	117	57
Km 12+700 To Km 17+500	85th	309	17	31
	50th	434	28	40
Km 18+700 To Km 23+000	85th	302	20	36
	50th	424	32	46

4.4 Discussion of Results

The FWD testing as described above was carried out in right carriageway. Discussion on the extrapolated results for asphalt concrete, unbound granular material and subgrade for the representative sections is given below:

Section # 1 (Km 6+450 To Km 11+500)

This section is comparatively an intact section rehabilitated with RS-l strategy which is predominantly a full depth overlay comprising all the pavement layers. The condition survey carried out previously also show minor intensity cracking. It can be noted from FWD back analysed stiffness values from table 8 and 9 above that the average stiffness values corresponding to 50 and 85 percentile deflection parameter are in range of 337 ksi and 241 ksi indicating a reasonable strength of the asphalt layer with pavement temperature of 65^0C. this also ties up with the condition survey results where surface racking is of a lesser degree (if any). The unbound granular material layer shows 85 and 50 percentile modulus values to be in range of 123ksi to 105ksi indicating strong granular layer with higher thickness. The subgrade is generally in good condition with adjusted values of Mr being in range of 20ksi indicating a good foundation support.

Section # 2 (Km 12-700 To Km 17+500)
It can be seen from the deflection profiles and tables 4 & 5 above that the overall pavement deflections are high in localized stretches. This indicates relatively weaker pavement section. It can be seen from tables 8 and 9 (for both loads) that the average asphalt layer stiffness values corresponding to 85 and 50 percentile deflection parameter are in range of 265ksi and 380 ksi indicating that the material has weak stretch. This ties well with the condition survey observations where deep patching has been carried out to rectify the cracked locations.
The unbound granular material in the subject section represents much variation in terms of strength. The 50 percentile modulus values for both the loads are evaluated to be in range of 28 to 34ksi which is reasonably good however, the 85 percentile stiffness values are as low as l7ksi and 21ksi indicating relatively weak zones in the granular layer. The review of the deflection profiles shows that the unbound material in the subject section does have weak zones where the deflection are higher than the normal limits indicating higher moisture levels. It is also observed that this section has been rehabilitated with RS-3 strategy and therefore no substantial unbound material has been provided therefore the unbound base layers are weaker. The sub-grade is generally in fair condition with modulus values being in range of 10 to 13ksi providing a reasonably support. However, due to inadequate unbound material at the top, sub-grade support may not be exaggerated to heavy loading conditions which play a vital role in limiting deflections at the surface.
Section # 3 (Km 18+700 To Km 23+000)
It can be seen from the deflection profiles and tables 4 & 5 above that the overall pavement deflections are high in localized stretches. This again indicates relatively weaker pavement section. It can be seen from tables 8 and 9 (for both loads) that the average asphalt layer stiffness values corresponding to 85 and 50 percentile deflection parameter are in range of 265ksi and 424ksi indicating that the material has weak stretches. This ties well with the condition survey observations where deep patching has been carried out to rectify the cracked locations.
The unbound granular material in the subject section represents much variation at terms of strength. The 50 percentile modulus values for both the loads are evaluated to be in range of 28 to 32ksi which is reasonably good however, the 85 percentile stiffness values are as low as l9ksi and 20ksi indicating relatively zones in the granular layer. The review of the reflection profiles shows that the unbound material in the subject section does have weak zones where the deflection are higher than the normal limits indicating higher moisture levels. It is also observed that this section has been rehabilitated with RS-3 strategy and therefore no substantial unbound material has been provided therefore the unbound base layers are weaker. The sub-grade is generally in fair condition with modulus values being in range of 12 to 15ksi providing a reasonably support. However, clue to inadequate unbound material at the top, sub-grade support may not be exaggerated to heavy loading conditions which play a vital role in limiting deflections at the surface.

Table 10: Avg. Back-Analysed Effective Layer Stiffnesses Resulting from RADAN 7 - 9000lbs & 15000 lbs

Section	Percentiles	Subgrade Modulus(ksi) 9000lbs	Subgrade Modulus(ksi) 15000lbs
Km 6+450 To Km 11+500	85th	20	17
	50th	21	19
Km 12+700 To Km 17+500	85th	10	10
	50th	13	13
Km 18+700 To Km 23+000	85th	12	12
	50th	15	15

4.5 Direct Effective Structural Capacity Prediction Using FWD Data

Direct Effective Structural Capacity Prediction approach (AASHTO, 1993) is used for the purpose of evaluation of existing pavement from the deflection data. The main philosophy is that the combined stiffness influence of all the layers within the pavement system determines the overall structural capacity of the pavement. The approach relies on outer deflection values to estimate the sub-grade modulus and the measured maximum deflection (at the centre of the load) to predict the effective structural capacity of the existing layered pavement system. This approach used is independent of the layer thicknesses and provides the overall structural capacity rather than the structural stiffness of individual layers, Sub-grade modulus of the existing pavement structure can be predicted by the following relationship.

$$Mr = \frac{0.24P}{dr(r)}$$

Where:
M_R: back calculated sub-grade resilient modulus, psi
P = Applied load, lbs
d_r = deflection at a distance r from the center of the load, inches
r = distance from center of load, inches
Deflection used to backcalculate subgrade modulus must satisfy the following relationship.

$$R \leq 0.7a_e$$

$$a_e = \left[a^2 + \left(D \left(\frac{E_P}{M_R} \right)^{1\backslash 3} \right)^2 \right]$$

Where
a_e = radius of the stress bulb at the sub-grade-pavement interface, inches
a = NDT load plate radius, inches
D = total thickness of pavement layers above the sub-grade, inches
E_p = effective modulus of all pavement layers above the sub-grade, psi
Effective Structural Capacity (E_P) for the existing pavement layers can be determined by the following relationship.

$$d_0 = 1.5pa \left[\frac{1}{M_R \sqrt{1 + \left(\frac{D}{a} \sqrt[3]{\frac{E_P}{M_R}} \right)^2}} + \frac{\left[1 - \frac{1}{\sqrt{1 + \left(\frac{D}{a} \right)^2}} \right]}{E_P} \right]$$

Where

d_o = deflection measured at the center of the load plate (and adjusted to a standard temperature of 68° F), in
p = NDT load plate pressure, psi
a = NDT plate radius, in
D = total thickness of pavement layer above the sub-grade, inches
M_R = sub-grade resilient modulus, psi
E_p = effective modulus of all pavement layers above the sub-grade layer, psi
The only unknown in the above equation is the E_p. The d_0 corresponds to the deflection under the load. AASHTO recommends the computation of Effective Structural Number (SN_{eff}) from Effective Structural Capacity (E_p) by making use of the following equation.

$$SN_{eff} = 0.0045D\sqrt[3]{E_P}$$

Where:
D = total thickness of all pavement layers above the sub-grade, inches
E_P = effective modulus of pavement layers above the sub-grade, psi
SN_{eff} (50th percentile and 85th percentile) for different sections are shown in Tables 11 and 12:

Table 11: **Effective Structural Number (SN_{eff}) by AASHTO method - 9000 lbs 85th and 50th percentiles**

Sections	Effective Structural Number	
	85th	**50th**
Km 6+450 To Km 11+500	4.45	4.64
Km 12+700 To Km 17+500	3.28	3.53
Km 18+700 To Km 23+000	3.01	3.25

Table 12: Effective Structural Number (SN_{eff}) by AASHTO method - l5000 lbs 85th and 50th percentiles

Sections	(SN_{eff})	
	85th	**50th**
Km 6+450 To Km 11+500	4.64	4.75
Km 12+700 To Km 17+500	3.47	3.66
Km 18+700 To Km 23+000	3.20	3.46

It may be noted that originally the Structural Number Requirement over sub-grade was evaluated to **4.370** which can well be compared with the analysis given in Tables 11 & 12 above.

4.6 Required Structural' Capacity for Future Traffic, SN_f - A Review

The required structural capacity in terms of structural number is computed assuming it to be a new pavement to accommodate the future traffic. The required structural number was estimated based upon AASHTO specified approach. The Cumulative ESALs have been computed during the initial design stage by the design consultant for the subject sections over the design life of 10 years which comes to approximately in range of **22.3 million** ESAL's. Details of traffic calculations with bifurcation into sections can be found in design report and will not be repeated here. The pavement design carried out by the design consultants is based on AASHTO specified approach using higher values of layer coefficients of existing unbound material i.e. aggregate base and granular sub-base layers to satisfy the structural requirements for predicted traffic based on 10 years design life. It may be noted that following specified approach, experience of the designer while providing the pavement layer configuration is important since it is largely empirical. It may also be noted in instances such as the project design slight error in the construction methodology or quality control procedures is not permissible since the entire design is dependent on the protection of underlying pavement layers as far as the structural integrity is concerned. The design implemented at site following the construction strategies noted above defines series of rehabilitation options. The inadequacy of the provided pavement structural layers (except for full depth rehabilitation) can be argued to greater extents.

5. Conclusions

1. Structural assessment of pavement by FWD of Section # 1 (Km 6+450 To Km 11+500) shows that the section is structurally intact. Any localized cracking is result of porosity through asphalt base course layer.
2. Comparatively, Section # 2 (Km 12+700 To Km 17+500) & Section # 3 (Km 18+700 To Km 23+000) are structurally weaker with the minimal support from the unbound material. This structural weakness has further worsted problem due to deficiencies in the asphalt mix relating to permeability.
3. The sub-grade strength as quantified provides a reasonable support for the overlays in almost all the respective sections evaluated.

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Study of Traffic Decongestion in Phuentsholing Town

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Abstract

Phuenthosling city being the commercial hub of the country is facing tremendous problems related to traffic. Due to the ever increasing number of vehicles, the city is facing serious traffic problems leading to congestion. Traffic congestion is the condition that results when there are too many vehicles for the available road and therefore it becomes mandatory to have solutions which can bring down the problem. This paper aims to provide engineering solutions by studying the traffic network of the city, which in turn can aid in decongestion of traffic. Three alternative solutions are suggested in line to fulfilling the aims and objectives of the study. By studying the traffic scenerio in the city, the first alternative suggests reclaming some portion of land to increase the carriageway thereby increasing the number of outlets for the vehicles. Alternative two and three suggests diversion of route and possible location of flyover bridge for future diversion, respectively.

Keywords

Traffic congestion, Traffic decongestion, Land reclamation, Route diversion, Traffic volume

Introduction

Bhutan is a developing country and good traffic network plays a very important role for the economic development of the country. Good traffic network can be developed only when there is smooth flow of vehicles in the area. With reference to Road Safety and Transport Authority of Bhutan (RSTA), the number of vehicles has been increasing all over the country on yearly basis and has a huge impact on traffic congestion. Traffic congestion has high impact on road users as well as impacts on non-vehicular road space users.

Proper traffic management has to be done by providing alternatives for the solution of traffic congestion. Mainly traffic congestion occurs during peak times in the morning and evening i.e. when people are travelling to work and then from work (Kuensel, 2013). According to RSTA's vehicle statistics data, the number of vehicles as of March 31st, 2017 that was registered in Thimphu is 44621 and in Phuentsholing is 30693. Since the vehicles are increasing on yearly basis, it is important to introduce new route for diversion of traffic and also increase the road capacity to have proper traffic management.

Phuentsholing City is a commercial hub with narrow-roads accommodating only one vehicle in a lane. Lack of space makes it impossible for road widening with minimum outlets. Once the vehicle gets into the town, the same route has to be taken to exit (Pem, 2017).

The main objective of this study focuses on studying the traffic network in the Phuentsholing town area and determining and suggesting a possible diversion route. Designing the possible diversion routes and suggesting the future diversion structures.

Literature Review

Traffic congestion in Phuentsholing has always been a problem, as many of the residents say it is faster to walk than to drive (Pem, 2017). It becomes prominent to conduct traffic volume studies in the area to find out which particular route is being congested and needs solution. Traffic volume count is basically a count of vehicular traffic passing through a specific point/station which is usually expressed in terms of number of vehicles per time. Traffic counts during a Monday morning rush hour and a Friday evening rush hour may show exceptionally high volumes and are not normally used in analysis. Therefore, counts are usually conducted on Tuesday, Wednesday, or Thursday (Currin T. R, 2001). Also, in places where traffic congestion is seen as a dominant problem, the construction of flyover bridges has proven to be a good solution in reducing the congestion. For instance in Songkhla, Thailand, with the flyover bridge in place, it was found that about 35-40% of the traffic volume was diverted to the bridge and the time delay was considerably reduced to about 30% (Salatoom & Taneeranannon, 2015).

Methodology

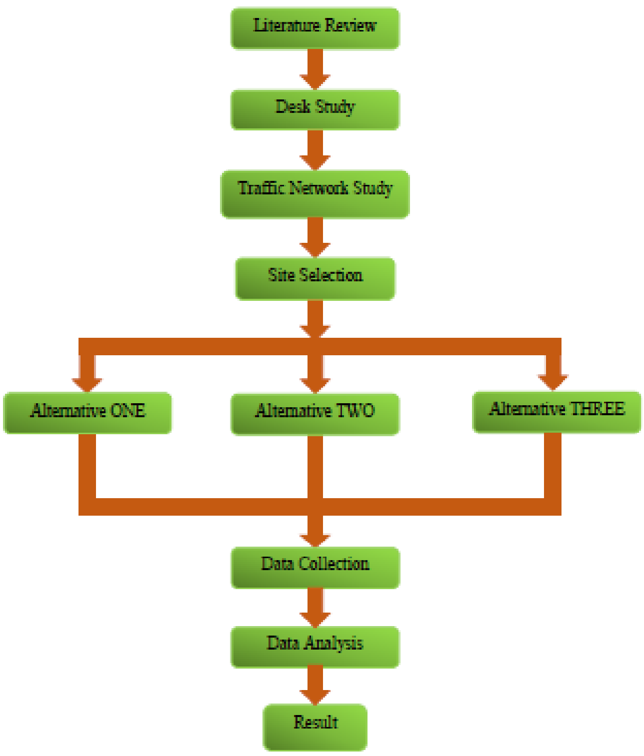

Figure 1: Methodology

Route Diversion Along Om Chu River (Alternative One)

The study of the traffic flow, vehicle route was studied for the completion of the project. The traffic volume count was conducted twice in a week to verify the past studies carried out for the town. The map of Phuentsholing town was used to select the most relevant site for the project and to come up with engineering solutions to traffic congestion that is currently prevailing in the town and for past decades. The site for land reclamation was also chosen from the map. Study and data collected showed that the routes were neither feasible for new construction nor widening within the town area. Therefore, route diversion was the most appropriate solution for the town.

This alternative route for diversion is suggested mainly for the vehicles entering into the town, to exit from the town via a route without involving back into the main town area. After route analysis; the route along Om-chu river was the most feasible diversion route with least hindrance to the vehicular movement and smooth exit from the town. Thus, reclaiming the land along Om-chu was suggested with engineering solutions.

The existing road along the Om-chu River starts from the downstream of the river till the current Taxi Parking Area. Currently, the road is not open to traffic as the road suffered much damage during the devastating flood back in year 2000. The existing length and width of the road is approximately about 760 m and 6 m respectively. Major damages have been caused by the river to the embankment and river protection structures during the past floods. Therefore, this particular road can be reused for traffic by widening the road by reclaiming some portion of the land available along the length of the river.

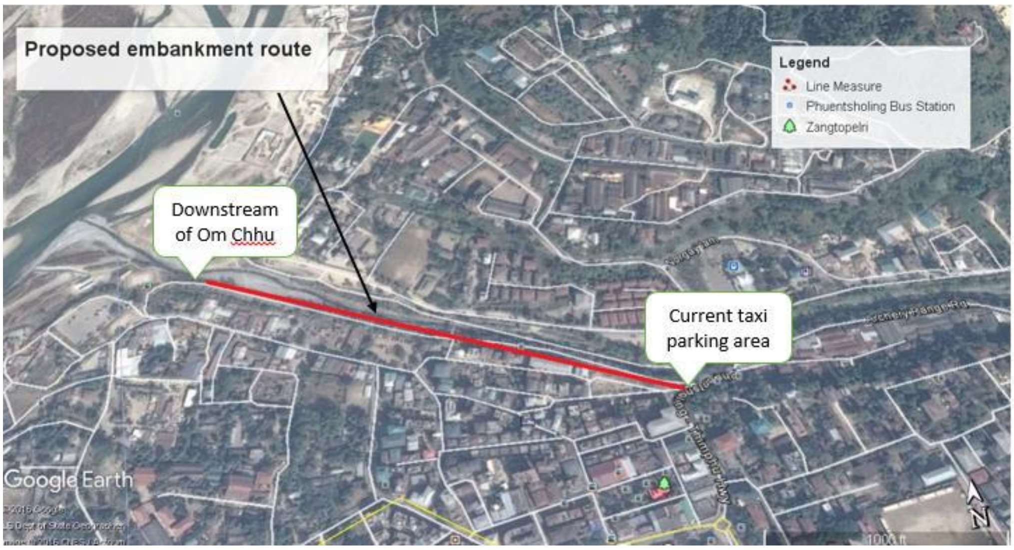

Figure 2: Location of proposed exit route

Om-chu diversion route design and details

This suggested route shall be a double lane, one way route of approximately 760 m long with 10 m of width. Cantilever retaining wall to withstand the lateral road as well as the loads from the moving vehicles (surcharge load) shall be designed with accordance to IS: 456. This retaining structure which will allow backfilling for the proposed route will also serve as a river training structure. Backfilling shall be done using the fill material available from the site (well graded sand) and shall be compacted with the help of vibratory roller to its desired thickness. And then flexible pavement is to be paved on top of it to allow the vehicles to run over it to exit from the town area.

Design data

Some of the data extracted for the study are:

i) Peak Discharge (Q)= 221.6 cumecs
ii) Highest flood lever (HFL) = 195 m above mean sea level i.e., 2.5 m above the bed level corresponding to year 2000
iii) Cohesion of soil (C) = 0 i.e., cohesionless soil

Table 1: Test and results

Test	Results	Remarks
Sieve analysis test	Uniformity coefficient Cu=11.4>6, Coefficient of curvature Cc = 1.17 (1 < Cc <3)	Well graded soil, hence can be used as a fill material for backfilling
Standard Penetration Test(IS 2131:1981)	Bearing capacity of soil= 320.7 KN/m^2	Good at resisting the pressure from the structures above

The cantilever retaining wall in place as shown in the figure below shall aid in providing an exit route for the vehicles. With this Om-chu diversion route in place, about 21% of congestion in town can be considerably reduced.

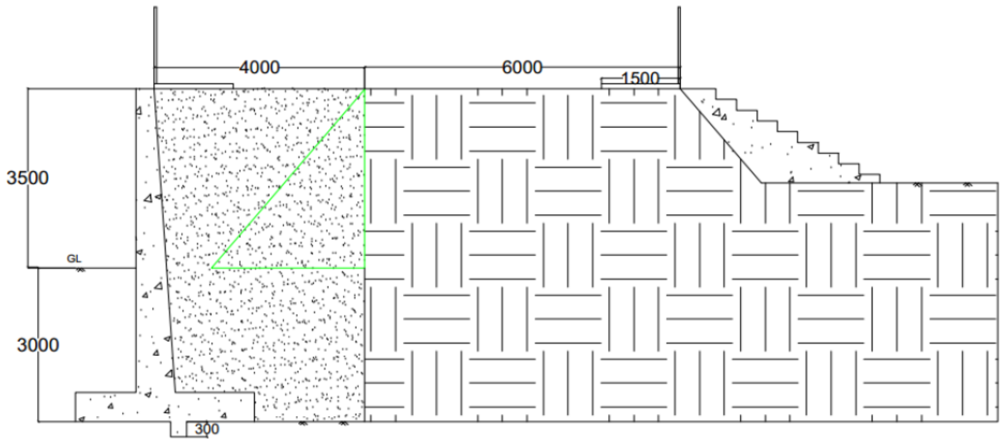

Figure 3: Cross sectional view of proposed reclaimed route along Om-chu river

Route Dversion At Gaki Lam (Alternative Two)

Alternative two suggests the widening of Gaki Lam after carrying out a study on the current exit routes adopted by the vehicles. The current routes adopted by the vehicles are as illustrated below.

Figure 4: Image showing the exit routes

Traffic volume count was conducted at the respective stations to find out the most common route adapted to exit from the town area. It was found that the exit route 2 was widely adopted by the vehicles. Study concluded the possibility of widening Gaki Lam (Exit route 1) to reduce the demand on exit route 2.

Figure 5: Image showing the allocated stations for volume count

The volume count at the respective stations were conducted by manually by observing the number of vehicles passing through the particular station and recording in the book. As per the literature review suggested, the volume count was conducted on all days excluding Monday and Friday. (Note: The average data of Tuesday, Wednesday and Thursday are recorded under Weekday and likewise, average data of Saturday and Sunday are recorded under Weekend.)

Table 2: Table showing the volume count at respective stations

<table>
<tr><td></td><td colspan="3">Morning (8-9 AM)</td><td colspan="3">Evening (5-6 PM)</td></tr>
<tr><td rowspan="6">Weekday</td><td rowspan="2">STN 1</td><td>Zhung Lam</td><td>Norkhil lam</td><td rowspan="2">STN 1</td><td>Zhung Lam</td><td>Norkhil lam</td></tr>
<tr><td>142</td><td>255</td><td>201</td><td>215</td></tr>
<tr><td rowspan="2">STN 2</td><td>Towards P.L</td><td>Zhung lam</td><td rowspan="2">STN 2</td><td>Towards P.L</td><td>Zhung lam</td></tr>
<tr><td>206</td><td>99</td><td>230</td><td>97</td></tr>
<tr><td rowspan="2">STN 3</td><td>Pelkhil lam</td><td>Gaki lam</td><td rowspan="2">STN 3</td><td>Pelkhil lam</td><td>Gaki lam</td></tr>
<tr><td>95</td><td>111</td><td>103</td><td>127</td></tr>
<tr><td rowspan="6">Weekend</td><td rowspan="2">STN 1</td><td>Zhung Lam</td><td>Norkhil lam</td><td rowspan="2">STN 1</td><td>Zhung Lam</td><td>Norkhil lam</td></tr>
<tr><td>122</td><td>125</td><td>187</td><td>175</td></tr>
<tr><td rowspan="2">STN 2</td><td>Towards P.L</td><td>Zhung lam</td><td rowspan="2">STN 2</td><td>Towards P.L</td><td>Zhung lam</td></tr>
<tr><td>201</td><td>89</td><td>289</td><td>110</td></tr>
<tr><td rowspan="2">STN 3</td><td>Pelkhil lam</td><td>Gaki lam</td><td rowspan="2">STN 3</td><td>Pelkhil lam</td><td>Gaki lam</td></tr>
<tr><td>97</td><td>104</td><td>187</td><td>102</td></tr>
</table>

Figure 6: Image showing the route that can be widened

The blue line showed in Figure 5 shows the route which can be widened for use. It has the capacity to expand it to two-lane road which can be used effectively as a diversion route at times of congestion.

Location of Possible Flyover Bridge For Future Diversion (Alternative Three)

This alternative provides the possible flyover bridge location for future diversion. This is necessary for controlling the future congestion in the traffic as it is evident from the number of vehicles that has been increasing as year passes by. Data acquired from the Road Safety and Transport Authority (RSTA), the percentage of light vehicles and heavy vehicles as per 2016 is found to be 56% and 44% respectively. This route shall be in compliance with the heavy vehicles originating from Jaigoan and headed towards the Industrial Estate in Pasakha.

First bridge could be located at the downstream of Om-Chu River and the other bridge location could be near the Phuentsholing South Treatment Plant (STP). As the core town has already permanent settlements and the vast space required for flyover bridge construction is available only in the vicinity of Om-Chu River, the provided suitable locations are as shown in Figure 6.

Figure 7: Location of the flyover bridges

Figure 8: Image showing the suggested new route for heavy vehicles

Decongestion can also be reduced with the new route following the heavy vehicles from Jaigoan to Pasakha from lower market to bridge one and to bridge two and then to curved bend to head towards Pasakha.

If this suggested route is used effectively by the heavy vehicles, around 15.87 % of traffic congestion in the town area can be reduced.

Conclusion And Final Recommendation

This paper studied the traffic network in the Phuentsholing town area and determined, designed and suggested possible route diversion. The most effective solution is to reclaim 10,000 m^2 of land along Om-chu river with proper retaining walls. It also has the possible locations for flyover bridges for future diversion and an alternate route for heavy vehicles. All three alternatives studies through this project can reduce the congestion in town area by 40%.

The recommendations made in through this study provide opportunity for looking into the feasibility of designing and planning possible flyover bridge and for urban planner for future planning and design of the town area.

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Optimizing Traffic Signal Control for a Selected Corridor in Karachi

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Abstract
The issue of traffic congestion is one of the prevailing problems for a metropolis like Karachi, which is experiencing continuous growth in population and vehicular traffic. Traffic congestion and delay at signalized intersections is a major component of overall travel time and delay. Most of the traffic signals in Karachi are pre-timed which are installed without any proper study. Congestion at traffic signals creates extra delays and contributes in signal violations and excessive emissions. Therefore, this study is intended to evaluate the performance of existing signal timing plans and to improve the traffic conditions at signalized intersection through optimization and synchronization of signals. For this study, a segment of Khayban-e-Ittehad is selected, starting from Korangi crossing towards Sea View. 12-hour traffic data on a typical working day is collected through video recording technique. Traffic flow rates and delays are extracted through video analysis. The flow rates were used in three different optimizing software, namely DISCO, SIGMIX and PTV VISTRO. The optimization results of signal timing obtain from the software, shows improvement in the existing delay. The study shows that optimum signal timings based on accurate data can improve delays and traffic conditions. This study can be replicated to other signalized intersection of the city as the prevailing conditions of this intersection is same as other intersections of the city.

Keywords
Signal optimization, Delay, PTV VISTRO, Synchronization, Signal Timings

1 Introduction

Traffic congestion and delays are one of the important and emerging problems of today's world. The traffic management systems mainly rely on traffic signals. Continuous efforts are made to improve delay at signalized intersections by using real-time measurements and adaptive traffic control systems. Adaptive Traffic Control Systems (ATCS) use real-time measurements from traffic sensors which include pneumatic tubes, inductive loops, video camera, tracking Bluetooth and Wi-Fi devices in passing vehicles etc.

A number of Urban Traffic Control Systems (UTCS) and ATCS have been developed and deployed in various cities around the globe. SCATS and SCOOT are among the most commonly used UTCS. Kumar and Dhinakaran (2012) published a study which discusses various problems related to delay estimation under mixed traffic conditions in a developing country (India) and proposes a method to improve the precision in estimating delay. Jain et al. (2012) proposed a local de-congestion protocol that coordinates traffic signal behavior within a small area and it can locally reduce traffic congestion by sustaining time variant traffic bursts. Bhuiyan et al. (2014) evaluated selected signalized intersections and concluded that the optimal signal timings can reduce control delay for all movements at the intersection, particularly for peak-hour for certain movements. Lal et al. (2016) proposed various remedial measures that focuses and targets on junction improvement, alternative operation plan and junction signalization. Patel et al. (2016) discussed a solution for the effective control of road traffic network having high heterogeneity and poor lane discipline.

In contrast to traffic signal operations in developed countries, all the traffic signals in Karachi are pre-timed signals with fixed timing plans. In most cases, signal timings are implemented without appropriate traffic studies. Inappropriately designed traffic signals create extra delays and results in signal violation, traffic accidents and excessive emissions. ATCS being used in developed countries have not been implemented for traffic management in Karachi, as the traffic of Karachi does not follow lane discipline and it is extremely heterogeneous. ATCS require real-time measurements from traffic sensors and at present no real-time measurement systems exist which can measure such heterogeneous and undisciplined traffic in real-time. In absence of such measurement system, signal timing plans based on accurate traffic data can also improve the performance of signalized intersection.

This study highlights the optimization of delay at traffic signals through signal optimization and synchronization by using selected signal optimization software. This study utilizes three different software, namely, Dynamic Intersection Signal Control Optimization (DISCO), PTV VISTRO and SIGMIX. DISCO and SIGMIX use Cell Transmission Model (CTM) for prediction of traffic flow while Genetic Algorithm (GA) is applied to determine optimal signal timings. VISTO uses microscopic traffic flow models for traffic flow propagation and GA for optimal signal timings. GA is one of the techniques based on artificial intelligence to optimize traffic signal network and it is widely used in several areas where automatic optimization is needed.

This research paper evaluates the performance of signalized intersections in Karachi and highlights the potential for improvement in performance of signalized intersection by optimizing and synchronization of the signals. For this purposes, a segment of road with three signalized intersection in close proximity was selected. Existing delay and traffic volumes were observed for a typical working day. The geometric and traffic data was used to estimate the optimal signal plan and offset values. The optimized delay and signal timings were compared with the existing values to evaluate the performance of existing signal and to determine the potential of improvement in performanc.

This paper consists of four sections. Section-2 describes the methodology of this study. Section 3 represents the simulation results and comparison of optimized delay with the existing one. Section 4 concludes the findings of this research and presents the way forward.

2 Methodology

Site Selection → Data Collection → Data Extraction → Optimzation → Performance Evaluation

2.1 Site Selection:

For the optimization and synchronization of pre-timed signals, a segment of urban arterial with three signals in close proximity was selected. Three consecutive signalized intersections of arterial in Karachi, named as, Khayaban-e-Ittehad were selected. Figure 1 shows the location of selected intersections on Google Map.

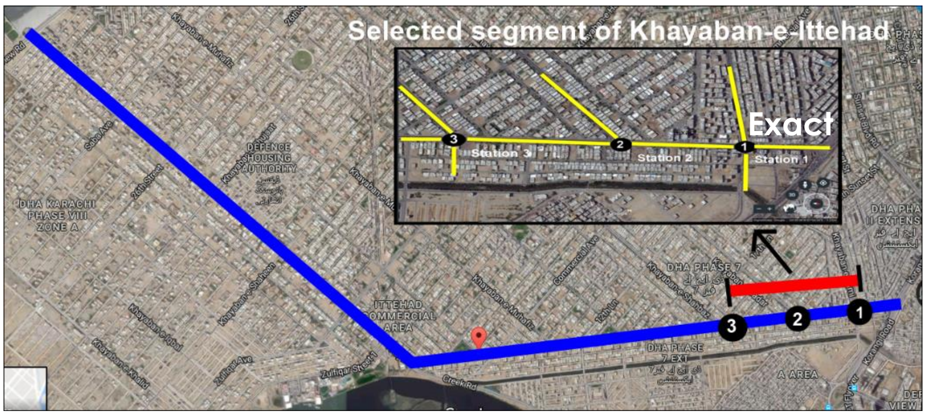

Figure 1: Google map of study area

2.2 Traffic Data Collection

For data collection survey, a typical working day was selected. Traffic data was collected on 11th of January 2016 from the selected signalized intersections. The survey was conducted from 9 am to 9 pm so that peak hour can be determined from actual data. Traffic data was collected using video recording technique by mounting camera with an aerial view covering all the movements of intersection. Video cameras were installed at optimum height so that proper view of traffic can be recorded.

2.2 Data Extraction:

The main inputs which were required for this study are traffic flow rates and existing delay. The recorded video of traffic stream was analyzed for data extraction and traffic data was manually extracted from the video. Every movement of traffic was counted mode wise at 1-minute interval using Click Counter software. To convert the heterogeneous traffic into equivalent traffic flow in Passenger Car Units (PCU) the Passenger Car Equivalent (PCE) factors defined by Adnan (2014) were used. The values of PCE factors are described in table 1.

Figure 2 shows the minute wise cumulative traffic flow of all three intersections. By interpreting this graph, peak hour for the selected segment of Khayaban-e-Ittehad was found in between 5:30 pm to 6:30 pm and the least traffic was observed during 3:00 pm to 4:00 pm.

Table 1: PCU factor values of different vehicles

Vehicle	PCU
Car	1
Bike	0.4
Rikshaw	0.909
Truck	3.288
Pickup	1.543
Minibus	3.024

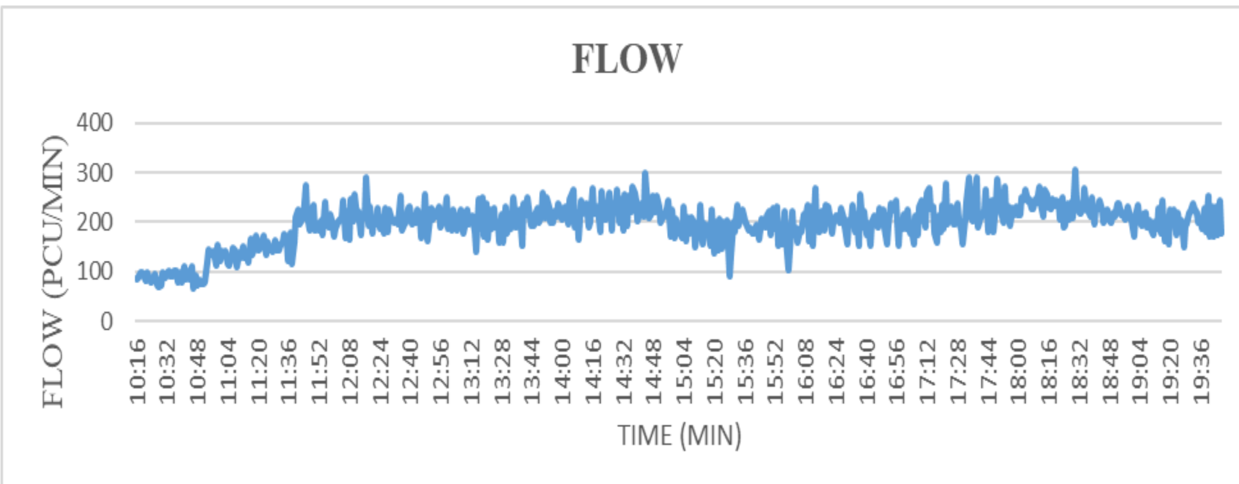

Figure 2: Traffic flow graph

The bar graph below gives useful and comprehensive information about mode wise traffic volume of all three intersection. The maximum number of vehicles encountered were of bikes and the minimum was of minibus.

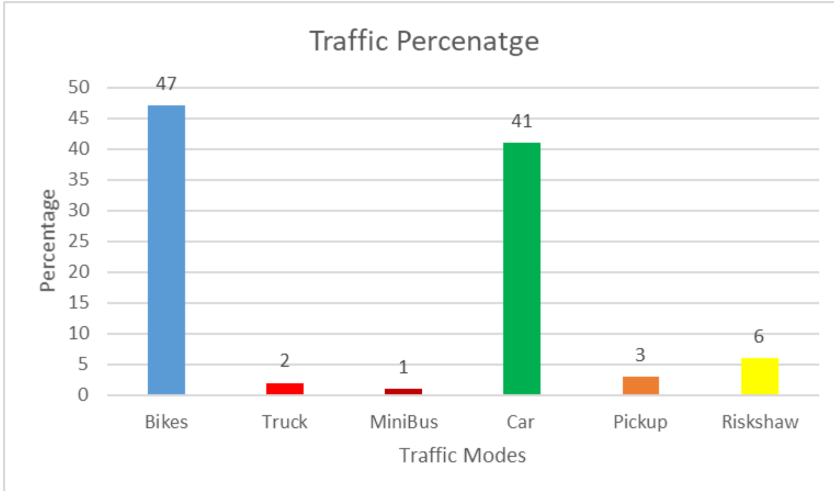

Figure 3: Vehicles composition of study area

2.3 Delay:

The aim of optimization and synchronization is to minimized delay. Therefore, after the extraction of traffic flow. For the delay measurement for all three intersection HCM 2010 method is being used.
For this study, Optimization & synchronization have been conceived for peak hour and for off peak as well. As delay faced by each vehicle can't analyze separately. So, multiply the delay (sec) for each approach with respective peak-hour vehicles and sum up the veh-hr. On station 1 weighted delay is more as compared to station 2 and station 3 because traffic flow is very high on station 1. In broader aspect it can be say that larger the delay more potential toward optimization. Existing delay tackle by all three intersections was found to 274 veh-hr which will be compared with the results of optimization software. The optimized delay for each station will also be compared later.
(Refer table-3)

Table 3: Existing delay of three intersection

		Existing delay		
Station	**Delay (sec)**	**No. of Vehicle**	**Veh-Sec**	**Veh-Hr**
Station1	242.7	1934	469492.0	130.4
	71.0	1081	76779.1	21.3
	60.0	1112	66723.8	18.5
	93.0	1125	104647.3	29.1
Station2	35.0	1501	52546.4	14.6
	34.0	1625	55256.3	15.3
	32.0	1033	33040.4	9.2
Station 3	32.8	1402	45943.3	12.8
	14.3	364	5182.6	1.4
	43.1	1077	46376.9	12.9
	37.4	890	33324.4	9.3
		Total	**989312.5**	**274**

As discussed earlier for the optimization and synchronization of prototype model three software will be used namely DISCO, PTV VISTRO and SIGMIX. These software has ability to simulate the existing condition, predict existing delay and proposed optimal signal timing plan to minimize delay.

The performance and credibility of each software is discussed in the next section.

3 OPTIMIZATION:

3.1 Calibration of Models:

The software should be calibrated for the existing condition, for this existing delay was calculated and geometry of intersection was drawn in such a way that delay predicted by software would be same as existing delay.

3.2 DISCO

The optimization in DISCO is the function of cycle length. The genetic algorithm optimizes the cycle length of intersection with pre-defined stages(Lo et al., 2001).

Table 1: DISCO Optimization Results

Scenario Name	Loading Profile	Existing Delay	Optimized Delay	Improvement
Peak Hour	Hourly	274	163.64	40.27%

3.3 VISTRO

The PTV VISTRO use stage sequence, green time, offset and cycle length optimization to minimize delay at signalized intersection.

Table 2: VISTRO Optimization Results

Scenario Name	Loading Profile	Existing Delay	Optimized Delay	Improvement
Peak Hour	Hourly	274	162.7	40.87%

3.4 SIGMIX

SIGMIX offer optimization in stage sequence, green time and offset optimization (Chen 2016).

Table 3: SIGMIX Optimization Results

Scenario Name	Loading Profile	Existing Delay	Optimized Delay	Improvement
Peak Hour	Hourly	274	180.53	34.11%

3.5 Comparison

Software optimize the signal timing according to their technique, following are the tables showing comparison of results calculated by software.

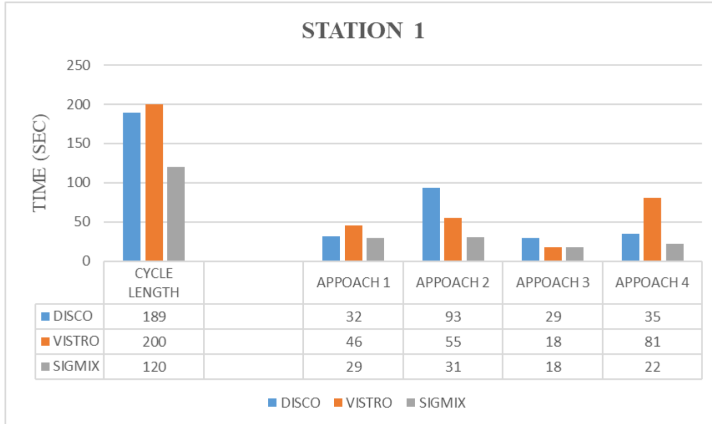

Figure 4: Cycle length and green time comparsion of station 1

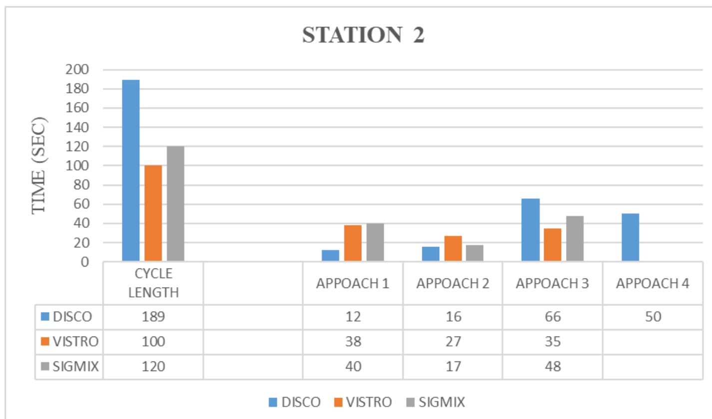

Figure 5: Cycle length and green time comparsion of station 2

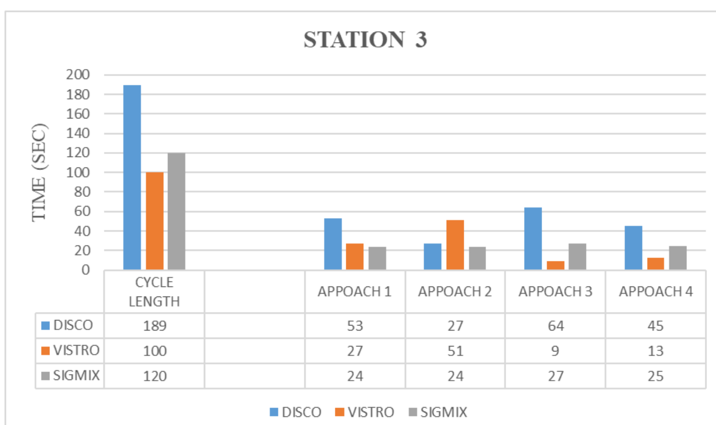

Figure 6: Cycle length and green time comparsion of station 3

The following table shows the comparison of overall optimized delay by this software.

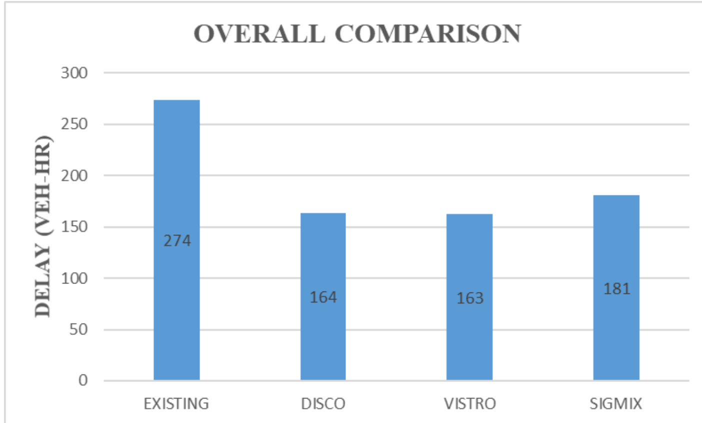

Figure 7: Overall comparsion of delay with existing condition

4 CONCLUSION:

The optimization of the fixed time traffic signal located at the three chosen intersections were performed. Three different software (DISCO, VISTRO, and SIGMIX) were used for the optimization and synchronization of selected intersection. These micro and macro simulation software provided the results in terms of delay. It was observed that delay throughout the study horizon due to the heavy flow of traffic and poor signal timing plan. According to the results obtained by optimization, DISCO, PTV VISTRO minimize more delay as compare to SIGMIX. The problems which were observed during the study are mentioned below:

- The fixed cycles run throughout the day, without reflecting the variation in traffic model.
- Lane discipline not followed by the drivers which resist in the utilization of advanced tools such as loop detectors.
- Violation of traffic signals and signal jumping is common.

Based on findings of this study, it is recommended that the optimal signal timings should be implemented for the selected segment to reduce travel time and delay. Furthermore, all other signalized intersections should be studied and optimized signal timings should be implemented to improve the performance of signal controls. The efficiency and performance of these systems can be further improved by upgrading the signals to ATCS by use of real-time measurements.

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An Investigation of Physical Properties of Plastic Waste Modified Bitumen

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Abstract
The use of plastic product is increasing day by day. The huge increases in plastic products result in a proportionate rise in waste plastic. This study highlights the investigation of modified bitumen by wet process using plastic waste generated in the different industries. The tests were carried out at laboratory on conventional & modified bitumen using different percent of industrial plastic waste. The industrial plastic waste in conventional bitumen is mixed by the mechanical Mixer with speed of 2000 rpm runs for 30, 60 & 90 minutes and also maintaining the temperature to 160-170 degree. A significant number of samples were prepared with varying percentages of industrial plastic waste ranges from (1%-10%) to produce industrial plastic waste modified bitumen. From laboratory results it was observed that the physical properties of industrial plastic waste modified bitumen has enhanced significantly. The results show increase in softening point decrease in ductility & penetration values. The studies conclude that the use of plastic waste in modified bitumen by wet process enhances its physical properties.

Keywords

IPW (Industrial Plastic Waste), Bitumen Modifier, Physical Properties, Laboratory Performances Industrial Plastic Waste Modified Bitumen.

1. Introduction

Plastics are user friendly but not eco-friendly. Nowadays, plastics have become the most favored materials in industrial appliances, office appliances, home appliances, kitchen appliances, automobile parts etc, Due to this proportionate rise in plastic waste in Pakistan. The best sustainable solution for this cause is recycling plastic waste into useful products. So that, studies is going on for new and innovative utilization of waste plastic is extensively encouraged. (Justo & Veeraragavan, 2002).

1.1 Use of IPW in Construction Industry

Applications of plastic waste are growing enomouresily in the construction industry. It is used in construction industry in various forms such as aggregate, filler & also binder. It is economical & easily available that's way it's very effectively used in construction industry.

1.2 Use of IPW in Highway Industry.

Pakistan is having the road network of more than 250,000 kilometers according to the survey of World Bank in 2009(Trading economics pk road).In present year, uses of industrial plastic waste have been considered in highway construction with great interest in many industrialized & developing countries. The use of these materials in highway industry is based on Technical, Economical, and Ecological criteria.

2. Problem Statement

The valorization of Plastic waste affects two major impacts, environmental impact is solved by disposing of such waste and the economic impact is the use of that in industry or in the field of construction. Disposal of plastic waste in an environment is considered to be a big problem due to its very low biodegradability and presence in large quantities. Pakistan is facing the problem of pavement failure like pot holes, rutting, fatigue, cracking stripping, etc. in addition, generation of waste plastic is increasing rapidly. Conventional bitumen has cohesive and elastic material, but does not retain their shape & properties whenever load and temperature is increased. Therefore, it has restricted ability at different temperature range and loads over the life of a pavement. The best option for dispose of the waste material is to Reuse or Recycle. Using industrial plastic waste and domestic additives can easily improving the properties asphaltic materials.

3. Aim and Objectives Of The Study

The aim of this research study is to enhance the properties of conventional bitumen using industrial plastic waste (IPW) as bitumen modifier.

Following are the objectives of the studies.

- To analyze the physical characteristic of IPW and base bitumen.
- Laboratory preparation of industrial plastic waste modified bitumen (IPWMB) by varying proportion (1, 3, 5, 7, 9, 10%) of IPW as bitumen modifier.
- To compare the virgin and modified bitumen with optimum plastic waste for performances purposes.

4. Scope of Study

- This research involves the study of performance of modified bitumen using industrial plastic waste. IPW is added as modifier to the bituminous mixture by carrying out laboratory procedure using the equipment's available in the highway engineering laboratory CED, MUET.
- IPW modified bitumen was prepared by using wet process by blending different percentages (1, 3,5,7,10,12.5,15 & 17.5%) of IPW at 160-170 °C temperatures, blending time 30, 60, 90 mins and blending speed is 2000rpm.

5. Literature Review

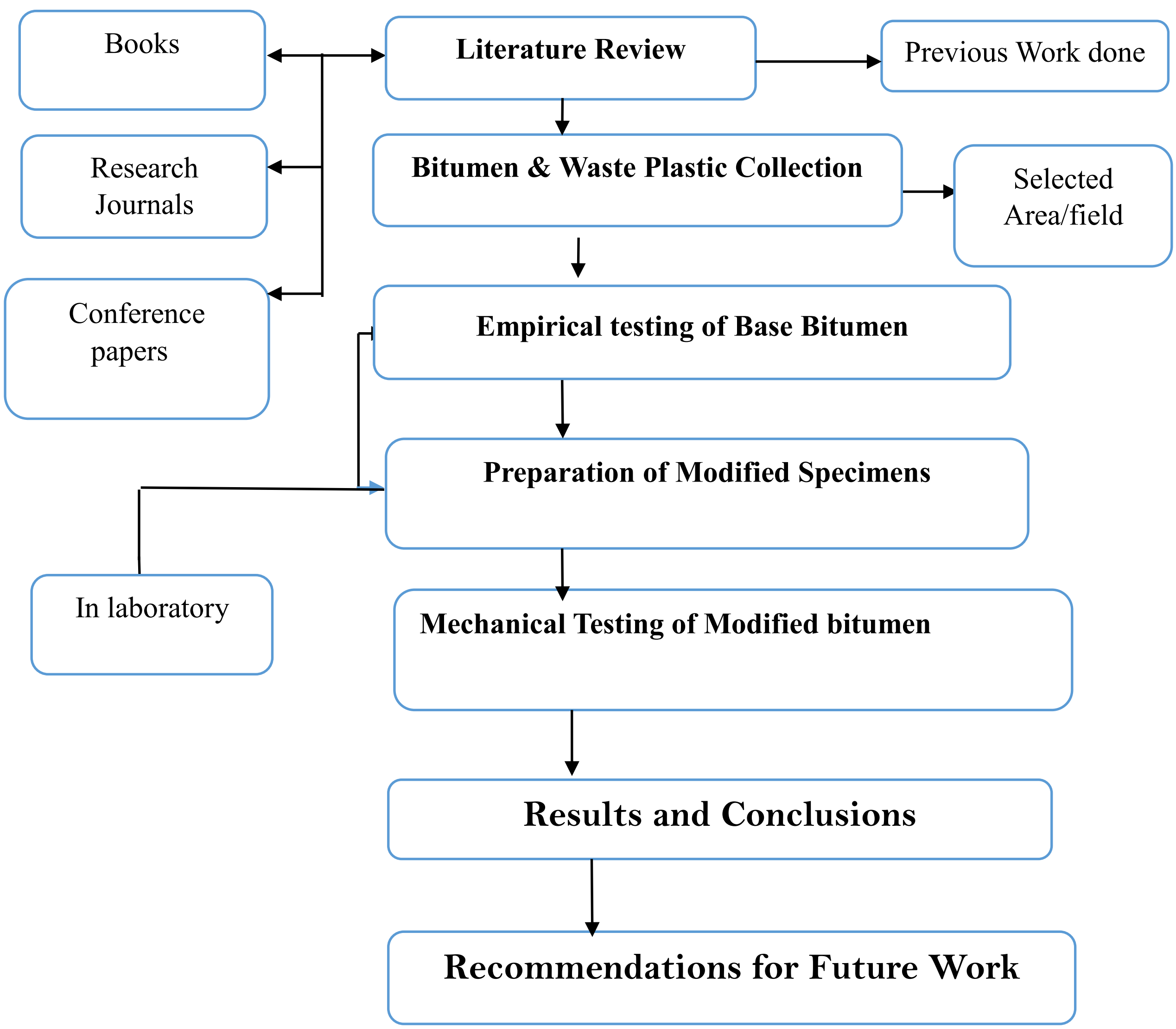
Latest research on plastic waste for improving the properties of bituminous material is being done now a day. These studies help out to enhance the various properties of bitumen to maintain better performance of road (Airey, 1997).

Generation of polymer waste is increasing day by day and necessity to dispose this waste in proper way is arising too. This waste is disposed by using various methods such as incineration, land-filling which affects the environment; but by adding polymer into roads is the eco-friendly process. The addition of polymer into dry bitumen enhances the service properties of bitumen **(**Nemade and Throat, 2013).

Bituminous properties could be modified by using polymer as an additive which is a good choice as well. Additive used in bitumen increase the melting point which makes the pavement stable in hot climate. (Becker,2001).

The bitumen is paving material have the properties of flow ability, stability and cohesion Therefore need for enhancing the bitumen properties various additive are used as a modifier. Polymer diversified bitumen properties such as, brittleness softening point and ductility of the bitumen (Brule and Maze, 1995).

6. Research Methodology



6.1 Bitumen as Base Binder.

Bitumen is complex substances containing heavy hydrocarbons and heavy metal. In IPWMB blends 80-90% of the main portions contain base bitumen of the total blend. The empirical property of base binders and the description used in this experimental program is presented in Table 3.1.

Table 3.1: Empirical Properties of Bitumen

Bitumen Grade	Penetration (mm)	Softening Point (°C)	Ductility (cm)	Specific Gravity
60/70	68	44	138	1.0329

6.2 IPW as a Bitumen Modifier

During preparation of IPWMB, the basic constituent involved is industrial plastic waste (IPW) which serves as a vital part in affecting modifying and controlling blends properties. IPW has been integrated in base binder to change the properties of binder. In this regard lots of trials are going on for enhancing the properties of bitumen in the whole world. The physical bonding of waste plastic and bitumen take place at high temperature.

6.3 Bitumen Testing (Conventional and Modified)

The appropriate understanding of performance of the base bitumen and modified bitumen is enabled by the study of their physical properties. For determine the various properties of bituminous materials usually these test were carried out.

- Penetration test
- Ductility test
- Softening point test

7. Evaluation of the Physical Properties of Modified Bitumen

The base bitumen 60/70 grade is blended with IPW modifier at different speed and different time interval in mechanical mixer to check the variation and evaluate the effects of IPW Modifier on base bitumen.

7.1 Lab Test Results of Penetration Value,

It has been analyzed by laboratory performance that the conventional bitumen enhances their physical properties by the addition of IPW content. Shows that the penetration values is decreasing by increasing the proportion of IPW with different speed and time interval in mechanical mixer.

The value of penetration was found dropping significantly as the proportion of IPW increased. Blending time up to 60 minutes was required to achieve the stable penetration results, when proportion of IPW was up to 10%. However, required blending time was increased with the increase of IPW content.

Table 1.1 Lab Test Results of Conventional and Modified Bitumen

S.No	I.P.W Content (%)	Blending Time (Minutes)	Penetration Value (1/10 mm)	Softening Point	Ductility Value (cm)
1.	0	----	68	44	138
2.	1	30	66	45	88
		60	61	47	85
		90	61	48	84
3.	3	30	58	51	78
		60	53	54	75
		90	52	55	74
4.	5	30	50	60	70
		60	47	64	66
		90	46	65	65
5.	7	30	42	68	62
		60	39	71	57
		90	38	72	56
6.	10	30	34	75	52
		60	30	79	46
		90	29	80	45

7.2 Lab Test Results of Softening Point Value.

The effect of IPW in the ring pall method is directly proportional to softening point value. The IPW is mixed with the base bitumen to increase the melting point of bitumen up to optimum limit, due to mixing of IPW in blend it gets viscous. The results displayed a good improvement in bitumen properties with addition of IPW content up to 10% by mass of bitumen.

7.3 Lab Test Results of Ductility Value.

Ductility is also affected by addition of IPW content in the bitumen .The addition of IPW make bitumen stiffer. The ductility values are decreased by the increased in IPW content. It may be noted that the ductility value of modified bitumen is very low as compare to the base bitumen. Bitumen grade 60/70 is mixed with IPW content up to 10% by the weight of bitumen its ductility value becomes very low.

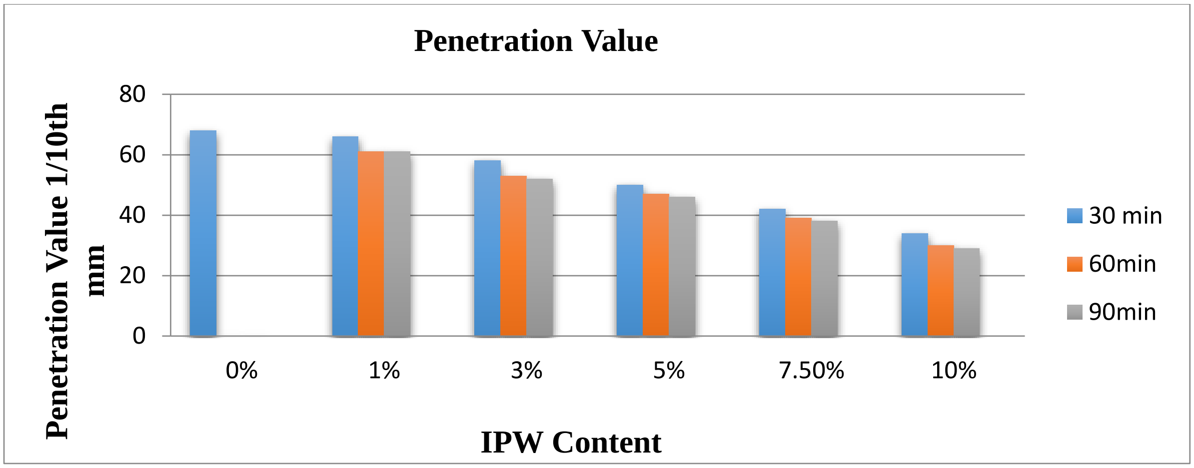


Figure.1.1 IPW Proportion vs. Penetration bitumen grade (60/70)

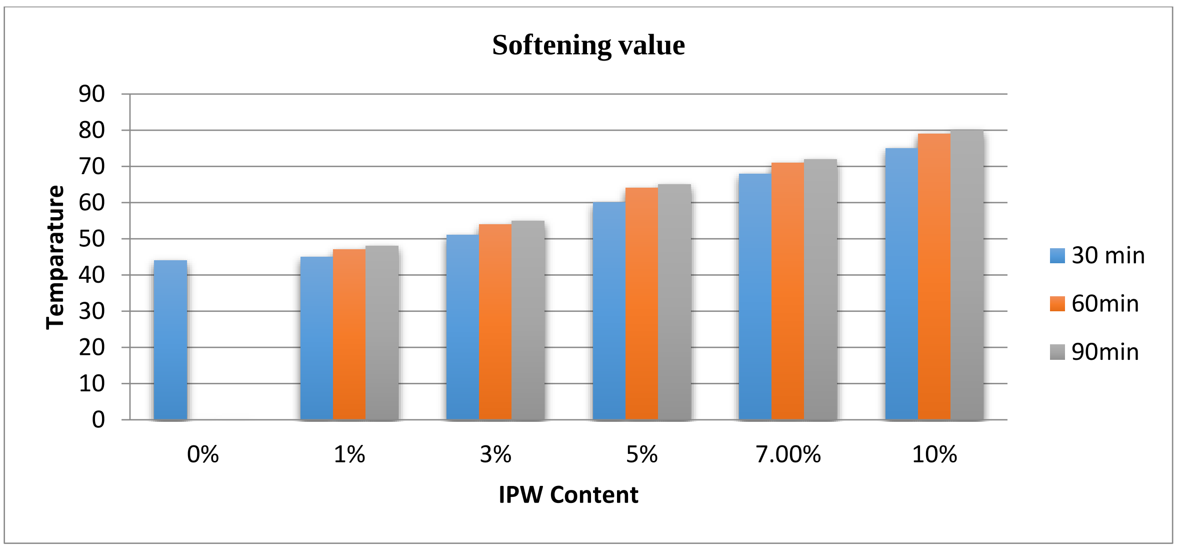


Figure.1.2 IPW Proportion vs. softening bitumen grade (60/70)

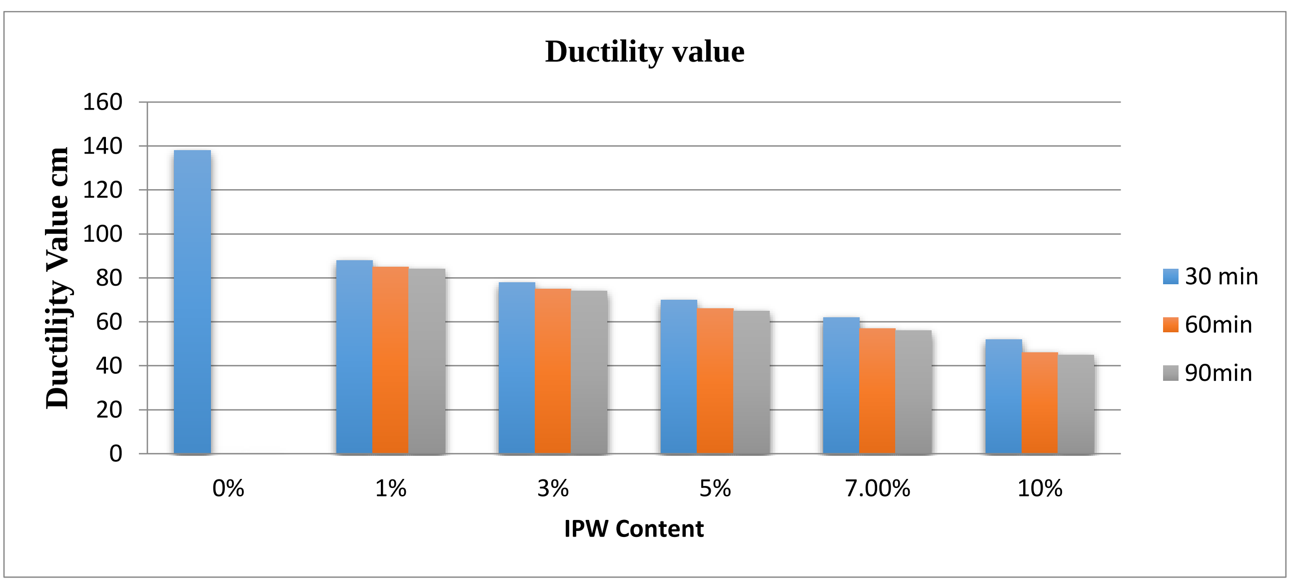


Figure.1.3 IPW Proportion vs. ductility value bitumen grade (60/70)

8. Conclusion

The study contracted to find the practical method to utilize the plastic waste as bitumen modifier in highway construction industry. The effects of addition of this waste on the physical properties of the bitumen have been analyzed. The practices of recycle plastic waste in pavement construction represent a valuable outlet for such materials. The addition of grinded plastic waste about 10% by weight of base bitumen helps in extensively enhancing the properties of base bitumen.

- The conventional bitumen properties have been enhanced significantly with the mixing of IPW content.
- The fundamental practical tests namely penetration point, ductility value and softening point, are within the **tolerance limit** for the IPWMB used for pavement construction. This outcome supports in concluding that the IPWMB shows greater temperature susceptibility and lower deformation due to cracking.
- The addition of IPW content also resulted in greater viscosity value which made the binder harsh and stiffer.

Improving the modification of asphalt properties by using the recycle plastic waste in bitumen is one of the favorable option.

9 Recommendations

- This research is limited to the testing of physical properties and it can be extended at higher levels to find out the rheological properties of the IPW modified bitumen using different size plastic waste at different proportion of PW contents.
- Government and researchers should integrate efforts toward preparing and implementing a sustainable solid waste management plan taking into consideration getting the maximum benefit from the high quantities of industrial solid waste.

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Safety Conscious Planning (SCP) & Safety Performance Functions (SPFs) Indiana – A Case Study

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Abstract

Traffic safety is an important component of highway design and transportation planning. The traditional approach to safety management has been to identify and remedy existing safety problems. The current planning practice addresses safety implicitly as a byproduct of adding capacity and operational efficiency to the transportation system. Transportation agencies are increasingly seeking consideration of how road safety can be proactively incorporated in the short-term and long-range transportation planning processes.

Safety conscious planning (SCP) is a proactive approach to the prevention of crashes by establishing safer transportation networks through integrating safety consideration into the transportation planning process. One of the current major concerns in predicting crashes in transportation networks is the transferability, applicability and accuracy of crash prediction models, called here Safety Performance Functions (SPFs).

The objectives were: (1) evaluation of transferability of selected existing SPFs (other countries) to local city conditions, and (2) modification and recalibration of the existing SPFs or development of new SPFs for cities depending on the evaluation results. All major types of road segments were included. In order to implement the SPFs in long-term planning, a set TransCAD-based tools and the latest state-of-the-art technology are needed for data pre-handling and crash prediction for segments and intersections.

Keywords
Transportation Planning, Safety, Crash, Traffic, Technology

1. Introduction

Safety conscious planning (SCP) is a proactive approach to the prevention of crashes by establishing safer transportation networks through integrating safety consideration into the transportation planning process. One of the current major concerns in predicting crashes in transportation networks is the transferability, applicability and accuracy of crash prediction models, called here Safety Performance Functions (SPF). The objectives of this study were:

- evaluation of transferability of selected existing SPFs to Indiana conditions, and
- modification and recalibration of the existing SPFs or development of new SPFs for Indiana depending on the evaluation results.

The types of road facilities considered are: rural two-lane, rural multilane, rural interstate, urban two-lane, urban multilane, and urban interstate The flow chart shown in Figure 1 illustrates framework.

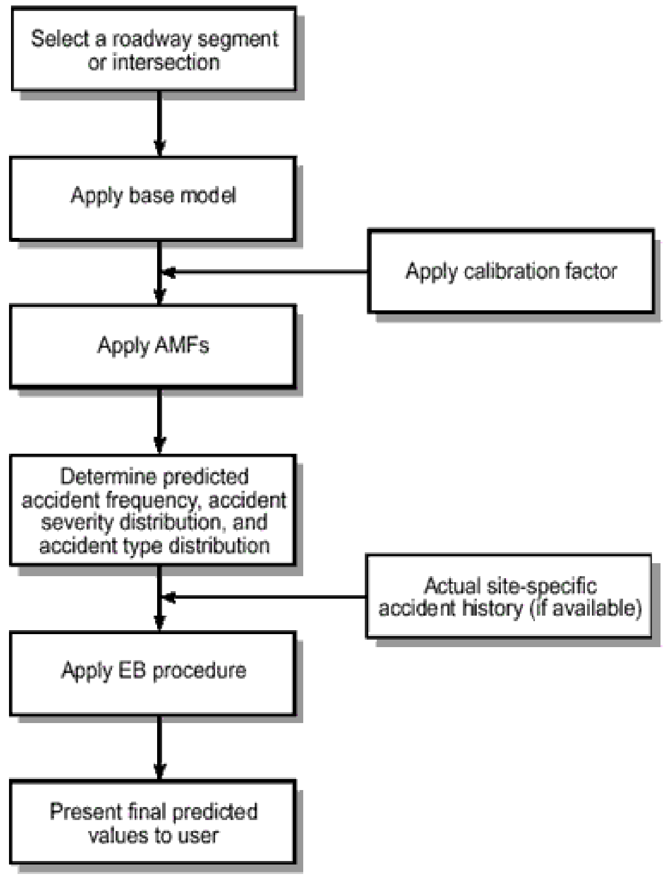

Figure 1: Flow Diagram of the Accident Prediction Algorithm for a Single Roadway Segment or Intersection

2. Literature Review and Need for Updating Existing Models

There was a need to develop a robust model that can address these shortcomings in addition to performing the following tasks: i) The models should be capable of predicting the expected number of crashes in the planning stage when more detailed actual information is not available; ii) The process of crash prediction should become more refined as more data becomes available. Models developed in the past were frequently based on an insufficient number of observations or for regions other than the studied one. These models require calibration and validation, and this paper evaluates two methods of calibrating safety prediction models for user-defined set of network partitions. The methods are tested and evaluated for the Indiana state road network.

The current models available for Indiana employed in practice need to be update due to the following factors:

- Better data is available through TransCAD.
- Some data may be irrelevant; other data may not be readily obtained.
- Current models do not contain enough variables.
- Some are outdated.

The methodology adopted for safety analysis uses SPFs for various types of facilities and CMFs for various safety improvements. However, this study focused on developing CMFs, which were used to predict crashes. As a general note, there is no difference between CMFs and accident modification factors (AMFs).

Different models were analyzed, supplementing and evaluating them for Indiana in order to have a set of SPFs that can be used for planning studies. The models were selected on the basis of region (geography and climate), soundness of the models, quality of the data, applicability to Indiana, completeness of report/study. A set of modified models is then evaluated for transferability after calibration for Indiana conditions.

3. Modifying the Existing Models

Modification of the existing models was done by combining CMFs with Basic Safety Performance Functions (Tarko *et al.*, 2005). Table 1 shows the Basic Safety Performance Functions (BSPFs).

The general form of the BSPF is:

$$A = k \cdot L \cdot Q^{\beta} \tag{1}$$

where:
A = predicted number of crashes on a segment,
L = length of the segment as defined in section,
Q = annual average daily traffic as defined in section,
$k, \beta =$ constants for specific severity level and facility type.

Table 1: Basic Safety Performance Functions for Indiana (Tarko and Kanodia, 2004)

Rural two-lane segment	$a_{IF} = 0.208 \times L \times Q^{0.604}$	0.420
	$a_{PD} = 0.712 \times L \times Q^{0.592}$	0.430
Rural multilane segment	$a_{IF} = 0.107 \times L \times Q^{0.814}$	0.451
	$a_{PD} = 0.634 \times L \times Q^{0.615}$	0.484
Urban two-lane segment	$a_{IF} = 0.105 \times L \times Q^{1.080}$	1.253
	$a_{PD} = 0.603 \times L \times Q^{0.896}$	1.349
Urban multilane segment	$a_{IF} = 0.674 \times L \times Q^{0.435}$	1.588
	$a_{PD} = 2.028 \times L \times Q^{0.460}$	1.946
Rural interstate	$a_{IF} = 0.044 \times L \times Q^{0.917}$	1.053
	$a_{PD} = 0.169 \times L \times Q^{0.943}$	1.604
Urban interstate	$a_{IF} = 0.00048 \times L \times Q^{2.238}$	2.383
	$a_{PD} = 0.0057 \times L \times Q^{1.954}$	2.704

The task of combing crash modification factors included three steps:

- Redefining the original model variables and recalculating corresponding slopes
- Combining the slopes for the same model variables,
- Calculating the average values of the model variables.

The general form of the CMF is:

$$CMF = \exp[a \cdot (X - \overline{X})] \tag{2}$$

where:

CMF = crash modification factor,
a = regression coefficient of the variables (slope),
$\overline{X}$ = average value for the variable.

4. Calibration

The safety performance functions were calibrated using the negative binomial theory. The models were evaluated in order to check the robustness in terms of crash prediction providing confidence and evidence that the developed SPFs were valid in terms of crash prediction from a stability point of view. The most common structures of the models (SPFs) for links (segments) and nodes (intersections) are as follows:

$$A = \exp(k) L Q^{\beta} \exp^{(\sum \gamma_i \chi_i)} \quad (3)$$

where:
A = number of crashes in a year,
L = length of the section in miles,
Q = AADT of the section,
X = explanatory variables,
k, β, γ = constants.

A model structure equivalent to equation 3 is as follows:

$$a = BSFP \cdot AMF_1 \cdot \ldots \cdot AMF_n \quad (4)$$

where:
ao = crash frequency at a specific severity level;
BSFP = basic safety performance function; BSFP=exp(γ0)·E
AMFi = crash modification function i; AMFi = exp[γi·(Xi - Mi)],
Mi = average or default value of characteristic Xi.
Table 2 shows the standard error of prediction was higher for the full-form SPFs than for the BSPFs. Therefore, the CMFs derived from the existing SPFs developed for other states cannot be used to adjust Indiana BSPFs.

Table 2: Standard Error of Prediction for BSPFs and SPFs

Facility Type	Standard Error of Prediction			
	Injury/Fatal		PDO	
	BSPF	SPF	BSPF	SPF
Rural two-lane roads	0.76	0.78	1.78	1.80
Rural multilane roads	1.25	1.22	3.24	3.24
Rural interstate	1.78	2.36	6.77	11.94
Urban two-lane roads	1.05	1.16	2.46	3.12
Urban multilane roads	1.93	2.24	4.83	5.67
Urban interstate	2.54	3.26	11.09	20.66

Figure 2: Indiana State Road Network

The above results indicate that simple calibration by adjusting the predictions with a single calibration factor is insufficient and full calibration of all model parameters (adjusting all the regression coefficients) in the models was needed. Figure 2 and Table 3 depicts the size and density of the example state network available in TransCAD for the year 2004 used for recalibration of various facilities. For each of the six road classes, the past models were recalibrated and compared with the presently developed crash prediction models. Table 4 shows an example of the calibration results for the fatal/injury crashes for Rural Two-Lane Roads in which the present (newly developed) models performed better than the past models as they had good predictive and explanatory ability as indicated by the overdispersion factor

Table 3: Sample Size per Facility and Crash History

Facility	Sample Size	Total Length	Average Length	Total Crashes (No.)	
		(mi)	(mi)	Fatal/Injury	PDO
Rural Two-Lane	11132	7609.37	0.684	7518	15442
Rural Multilane	1313	851.35	0.648	1969	4988
Rural Interstates	307	769.76	2.51	1769	8035
Urban Two-Lane	3221	731.35	0.227	3492	7884
Urban Multilane	2727	666.44	0.244	6276	16216
Urban Interstates	473	401.68	0.849	3532	15313

Table 4: Injury and Fatal Crash Model for Rural Two-Lane Roads

	Present Model		Past Model
	Coefficient	Std.Error.	Coefficient
Constant	0.0002	0.1759	0.2080
L_L	1.0000	0.0000	1.0000
L_Q	1.0582	0.0176	0.6040
LW	-0.1017	0.0170	-0.0831
RSW	-0.0126	0.0045	-0.0442
ST_SURF	0.3076	0.0302	-
ST_COMB	-0.6518	0.0231	-
ACURV	0.1832	0.0071	0.2158
AGRAD	-	-	0.0528
Alpha	0.8519	0.2140	-

5. Conclusions

This study investigated if existing SPFs and corresponding CMFs developed for states other than Indiana could be applied to Indiana conditions for future SCP and to develop CMFs. The BSPFs exhibited better performance than the SPFs, which indicates that the CMFs added additional noise to predictions that reduced the prediction quality. Full calibration of the present (newly developed) models performed better than the past models as they had good predictive and explanatory ability as indicated by the overdispersion factor and the statistics reported in the results. However, an upgrade of this database on a regular basis and adding more years of data is therefore needed to enhance the quality of the SPFs and also for future safety analysis. The calibration has been defined to address the specifics of network modeling in transportation planning. The crash rate may change with different network, weather condition or management strategy. In order to implement the developed SPFs in long-term SCP, a a set TransCAD-based tools and the latest state-of-the-art technology are needed for data pre-handling and crash prediction for segments and intersections.

Karachi being the mega city of Pakistan and with the on-going infrastructure developments throughout the country, there is a need to perform studies like this to develop the SPFs by utilizing the existing available database as a baseline which would be beneficial for the transportation planning and law enforcement agencies in future planning. Studies like this could also be used as a prototype during the planning process.

6. Acknowledgements

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Evaluation of Flexural Strength of Beam with Partial Replacement of Coarse Aggregate with Waste Plastic

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Abstract
The plastic waste is non-biodegradable and therefore to conserve resources, it is need of the day to recycle it. Today, the industry of concrete structures is in search of effective materials with less weight, low cost and less environmental effects. To achieve the aforementioned properties in concrete, waste plastic aggregate may be used as partial replacement of coarse aggregate.
Many researches have been carrying out the experiments to quantify the results while replacing coarse aggregate with plastic aggregate. Replacement of 10 to 20% was recommended. In this research work the process is divided into two stages. In the first stage, concrete cylinders having different percentagewise partial replacement of plastic aggregate were tested and resulted that 20% replacement gives better result. In the second stage, three Reinforced Concrete (RC) beams were constructed. One beam was used as control specimen for comparison of results while two beams were constructed with 20% replacement of coarse aggregate with plastic aggregates. All the beams were designed as per the ACI code and were tested under third point loading as per ASTM C78/C78M. The beams were investigated for flexural behavior at mid span. Beams, partially replaced with plastic aggregate showed almost same flexural behavior while their self-weight was reduced by 14% as compared to their control specimen counterpart.

Keywords
Reinforced Concrete Beam, plastic aggregate, Flexural Behavior, Cracks, Deflection

1. Introduction

Being one of the most versatile building materials, concrete is used in many forms of construction. Usage of reinforced cement concrete started in 1960's, to make structural and non-structural elements while replacing steel columns, girders and steel decking. As it is known, that 70 percent mass of concrete mix consists of fine and coarse aggregate. These aggregate greatly contribute in the self-weight of concrete mass. Using aggregates with low specific gravity may reduce the dead load of structure appreciably with great benefit in terms of reduced loads on supporting elements in a structure and less seismic reactive mass.

For the past several decades, researchers have been trying to introduce aggregate for concrete with low specific gravity. Many types of materials were tested and satisfactory results were obtained in terms of reduction in self-weight and minimal loss of strength, if any. As a result of these efforts, different types of light-weight aggregates were introduced in concrete industry like volcanic pumice, light weight expended clay aggregate, fly ash, rice husk, rubberized aggregate etc.

A number of studies have been carried out to evaluate various properties of these light weight aggregates as a material and their behavior when used in concrete. Industrial and domestic waste consists of significant volume of polymeric materials which are mostly used in landfills as waste or burnt in garbage dumps. These polymeric rubber and plastic have two major impacts, i.e. environmental impact and the economic impact. The first one is addressed by disposing off such waste and latter impact is utilized with their use in industry or in the field of construction

The world's annual consumption of plastic materials has increased from around 2 million tons in the 1950s to nearly 440 million tons in 2015 [3]. This huge amount of plastic waste became a direct cause of environmental pollution and indirectly caused flooding by blocking the drainage system as noticed in Karachi in 2017 [4]. The recycling and use of this waste plastic as coarse aggregate will not only avoid the harmful effects of waste plastic but will also make building structures light in weight. Due to ease of local availability and low density of plastic, researchers are focusing to use plastic as a partial replacement of coarse aggregate.

In the last two decades, use of recycled concrete aggregates and waste plastic aggregates became a common practice in the construction industry. Many researchers focused on the effective use of waste plastic and examined its properties in compression and tension. Most commonly used plastic aggregates are obtained from food plastic wrappers, plastic bottles, shopping bags, rubber extracted from old tires etc. All these plastic aggregates have been used to partially replace the coarse aggregate in concrete mass and satisfactory results have been achieved [12]

The largest component of the plastic waste is low density polyethylene/linear low density polyethylene (LDPE) at about 23%, followed by 17.3% of high density polyethylene, 18.5% of polypropylene, 12.3 % of polystyrene, 10.7% polyvinyl chloride, 8.5% polyethylene terephthalate and 9.7% of other types. These wastes are first collected from disposal area and are then converted into plastic aggregate by recycling it. Recycled plastic is then used as a replacement of coarse aggregate up to a specific percentage in concrete.

In this research work recycled plastic aggregates have been used which were provided by Shazil Pakistan (Pvt.) Ltd Karachi. As the density of plastic is very low as compared to the natural aggregate, therefore the resulting concrete was 18 to 20 percent light in weight as compare to regular concrete

1.1 Methodology

The properties of recycled plastic aggregate provided by Shazil Pakistan (Pvt.) Ltd were not known which requires analysis of material for different properties, necessary for mix design of concrete. The size of plastic aggregate ranges from 12mm to 14 mm. The coarse aggregate were graded such that the plastic aggrade replace the size of corresponding natural coarse aggregate. Other properties were found as shown in the following table 1.

Table 1: Properties of Materials

Material	Size (inch)	Specific gravity
Courser Aggregate	1 (maximum)	2.67
Fine Aggregate	3 FM	2.6
Plastic Aggregate	0.5	0.85

For a concrete of 3 ksi compressive strength, the slump was assumed as ranged from 1" to 2", and with fixed water cement ratio (w/c) of 0.56. Based on above properties and specification, the following ratios have been estimated as per mix design.

Table 2: Estimated values of material ratio

% of Plastic aggregate	Cement (lb./yd^3)	Course Aggregate (lb./yd^3)	Fine Aggregate (lb./yd^3)	Plastic Aggregate (lb./yd^3)	w/c
0	517.24	1240.20	1821.70	0.0	0.56
15	517.24	1240.20	1548.40	141	0.56
20	517.24	1240.20	1457.30	188	0.56

As the complete replacement of course aggregate by plastic aggregate is not possible due to strength and workability property, many researches have concluded that plastic aggregate between 15 to 20 or 22 percent replacement gives good results [1, 2]. Therefore in this research 15 and 20 percent replacements of course aggregate have been examined.

For the said purpose of research, nine cylinders were prepared as per the ASTM standards C31/C31 M and tested under ASTM C39/C39 M. The results are shown in table 2.

Figure. 1: Testing of Concrete Cylinder

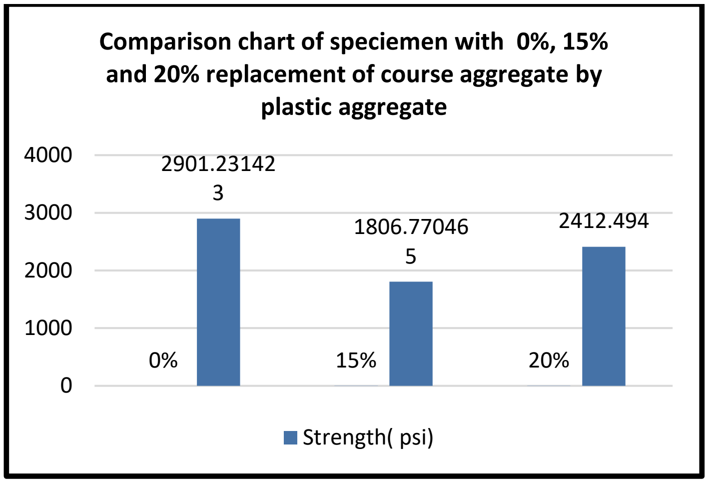

Figure. 2: Compressive strength of cylinders

In figure 1, concrete cylinder with 15% plastic coarse aggregate is being tested, after that cylinder with 20% plastic coarse aggregate was tested and the results in figure 2 above illustrates that the compressive strength of specimen with 20 percent replacement is more than 15 percent replacement. On the basis of this result it was decided to construct beam specimens with 20 percent replacement of course aggregate by plastic aggregates.

1.2 Details of Beam Specimens

For the purpose of this research, three beams were constructed, in which one beam was used as control specimen (without plastic aggregates) while two beams were constructed with 20 percent plastic aggregates. All the beams were constructed with same mix proportional as was used in concrete cylinders. The Beam specimens were 7′ long, 6″ x 9″ in cross section and were provided with minimum flexural and shear reinforcement as shown in figure 3.

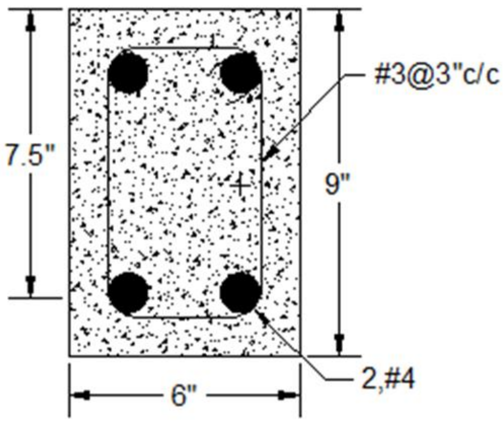

Figure. 3: x-section of beam specimen

1.3 Testing of Beam Specimen

All the three specimens were tested with third point loading as per ASTM C78/78M-10 in the material testing laboratory of UET Peshawar Pakistan.

Figure. 4: Testing of beam specimen

The specimens were placed in the frame to act as a simply supported with a clear span of 6 feet. Three LVDTs (Linear Variable Differential Transformers) were attached at the bottom in the zone of maximum bending for measuring deflections. All the specimens were tested using third point loading criteria as described in sec 2.5. The load was applied at the rate of 0.2 ton/sec

For deflection measurement three LVDTs (Linear Variable Differential Transformers) were attached at the bottom of the beam. Load and deflection data were recorded and analyzed for flexural capacity evaluation as shown in figure 5.

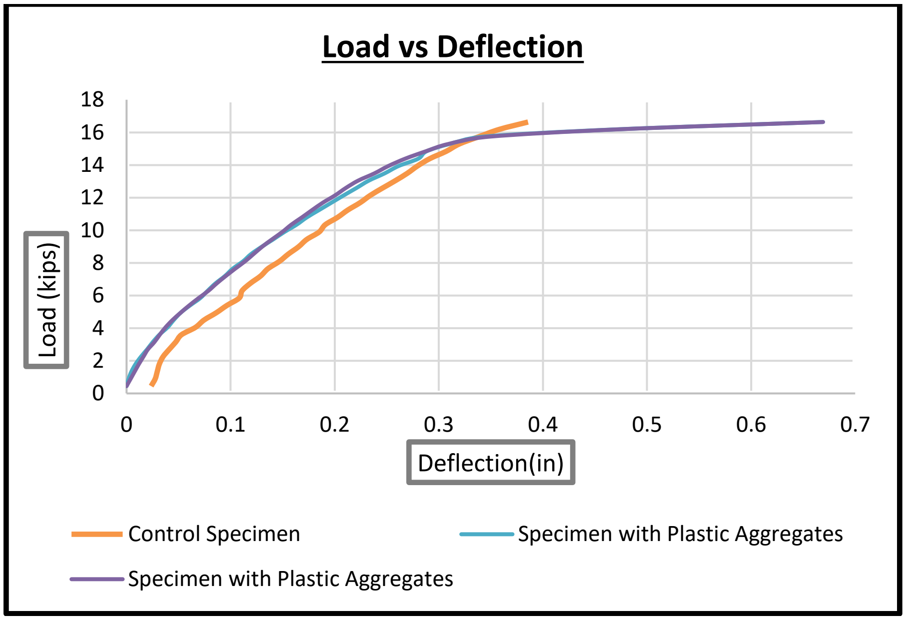

Figure. 5: Load vs Deflection Chart of Beam Specimens

2. Conclusion

The following conclusions were made from this research work.
In comparison with the control specimen it was noticed that the overall flexural capacity of light weight specimens (partially replaced) was same as control specimen

1. The beams showed more ductile failure as compare to control specimen.
2. The flexural capacity of RC beams, replaced with 20% plastic coarse aggregate was the same as control specimen while reducing 18 to 20 percent self-weight.
3. Beams with Partial replacement of coarse aggregate showed uniform distribution of cracks on tension face.
4. The beams were strong enough to resist shear force and did not show shear cracks before the flexural cracks.

3. Future Recommendations

Based on results of this research work and past research recommendations, plastic aggregate as partial replacement of coarse aggregate up to 20 to 22 percent may be practically used in the concrete industry.

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Alkali Reactivity of Aggregates from Pakistan and Control Measures for Alkali Aggregate Reaction

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Abstract
Alkali aggregate reaction is one of the important factors that affect the durability of concrete structures. In this paper, the alkali reactivity of four aggregates for Pakistan KHH 2nd project was tested by petrographic method, and the effects of the content of mineral admixtures (fly ash, slag powder and silica fume) and chemical admixtures (lithium carbonate and barium carbonate) on the expansion of mortars with different aggregate were studied. The petrographic analysis showed that the aggregates from Dor River, Sirran River and Thakot Bridge had alkali silica reaction potential and it was risk to use them directly in engineering. Additionally, it is found that the mineral admixtures and chemical admixtures reduced the expansion of mortars caused by alkali aggregate reaction. Li_2CO_3 and $BaCO_3$ were more effective on the limiting the expansion of mortars. Addition 3% Li_2CO_3 or 5% $BaCO_3$ decreased the expansion of all mortars with alkali-susceptible aggregates below 0.1%.

Keywords
Alkali aggregate reaction, petrographic analysis, expansion, mineral admixture, chemical admixture

1. Introduction

Alkali-Aggregate Reaction (AAR) is an important problem to the durability of concrete, which has caused massive damages of concrete structures and financial loss worldwide (Sellier *et al.*, 2009). AAR is the chemical reaction between certain mineral phases in aggregates and the alkali hydroxides in concrete pore solution (Fournier *et al.*, 2000). Generally, there are two principal types of AAR: the alkali-silica reaction (ASR) and the alkali-carbonate reaction (ACR). Damages of concrete caused by ASR happened more common than ACR. ASR occurs when the aggregates have amorphous or poorly crystallized silica phase, and it generates an alkali-silica gel that swells in the presence of water, leading to volumetric expansion or cracking of concrete. ACR involves the de-dolomitization (decomposition of dolomite into brucite and calcite) of carbonate rocks in strongly alkaline solutions, resulting in expansive products.
The necessary conditions for ASR to occur are the sufficient amount of alkalis and reactive silica, as well as the sufficient moisture level in concrete. In Portland cement based system, alkalis mainly come from the cement. Therefore, low-alkali cement is employed to minimize ASR in many cases. Meanwhile, using non-alkali-reactive aggregates and isolation of moisture are common measures to limit ASR (Krivenko *et al.*, 2014). However, sometimes the alkali-reactive aggregates are unavoidable considering the availability and the cost.

Supplementary cementitious materials are recognized to be efficient in limiting the ASR in Portland cement based systems (Thomas, 2011 and Joshaghani, 2017). They could decrease the alkali concentration in the system, or react with $Ca(OH)_2$ and produce the hydration products with lower Ca/Si ratio which could absorb the alkali. In addition, chemical admixtures, such as lithium salts (LiOH, LiF and Li_2CO_3), are used as inhibitors to ASR too (Zapała-Sławeta *et al.*, 2016).
In this paper, four local aggregates available for KHH 2nd project in Pakistan were tested by petrographic method to identify the alkali reactivity. And the effects of mineral admixtures (fly ash, slag powder and silica fume) and chemical admixtures (lithium carbonate and barium carbonate) on the expansion of mortars were studied. The results will be a technical reference for the project.

2. Experimental work

2.1 Materials

Aggregates came from four different places, including Dor River, Choona Village, Sirran River and Thakot Bridge, were studied in this paper. A PAK (BESTWAY) 42.5 Portland cement with the alkali content of 0.566% was used. The properties of the cement are given in Table 1. A class II fly ash and a local slag powder were used, of which the properties are showed in Table 2 and Table 3. Silica fume was also used in the experiment, details of which are given in Table 4.

Table 1: Properties of the cement

Specific surface area (m^2/kg)	Initial setting time (min)	Final setting time (min)	Flexural strength (MPa)		Compressive strength (MPa)	
			3d	28d	3d	28d
385	180	245	5.9	8.3	29.2	48.6

Table 2: Properties of the fly ash

Density（g/cm^3）	Specific surface area（m^2/kg）	Water requirement (%)	Ignition loss (%)	Water content (%)	SO_3 content (%)
2.21	435	91	3.9	0.1	0.8

Table 3: Properties of the slag powder

Glass phase Content (%)	Density (g/cm^3)	Specific surface area (m^2/kg)	Fluidity ratio (%)	Ignition loss (%)	SO_3 content (%)
96	2.87	461	109	0.9	1.2

Table 4: Properties of the silica fume

SiO_2 content (%)	Ignition loss (%)	Strength activity index at 28 days (%)	Specific surface area (m^2/kg)
90	2.2	105.6	17567

Two chemical admixtures, lithium carbonate and barium carbonate, were used as inhibitors for the alkali aggregate reaction in this study. The properties of the lithium carbonate produced by Shanghai China Lithium Industrial Co., Ltd and the barium carbonate produced by Shanghai Xinguiyuan Chemical Co., Ltd could be found in Table 5 and Table 6.

Table 5: Properties of Li_2CO_3

Li_2CO_3 content (%)	Na^+ content (%)	Ca^{2+} content (%)	Fe_2O_3 content (%)	Cl^- content (%)	SO_4^{2-} content (%)	Undissolved substance in acid (%)
99.0	0.2	0.05	0.01	0.05	0.35	0.005

Table 6: Properties of $BaCO_3$

$BaCO_3$ content (%)	Water content (%)	Undissolved substance in acid (%)	Sulfide content (%)	Fe_2O_3 content (%)
99.35	0.06	0.12	0.045	0.002

2.2 Testing methods

The mineralogical composition and texture of aggregates from four places were analyzed by petrographic method using a microscope.

The potential alkali-aggregate reactivity was evaluated by accelerated mortar-bar method according to Chinese standard "Standard for technical requirements and test method of sand and crushed stone (or gravel) for ordinary concrete" JGJ52-2006. The aggregates were crushed into particles smaller than 5.0 mm and washed. Then 990g aggregates and 440g Portland cement (or binder) was mixed with water to make a mortar with the water-binder ratio of 0.47. Fly ash and slag powder were used to replace cement by 10%, 20% and 30% in mass, while silica fume replaced cement by 3%, 6% and 9% in mass. Li_2CO_3 and $BaCO_3$ were added into mortar at the dosage of 1%, 3% and 5%.

The mortar was cast into moulds with the size of 25mm×25mm×280mm by two layers, and a small testing rod was fixed at each end of the specimen. After being demoulded, the specimens were placed into a curing box with a water bath which kept the temperature at (80±2) ℃ for 24hours. Then the length of the specimen, L_0, was measured and recorded. The NaOH solution with the concentration of 1 mol/L was added into the curing box to replace water. The temperature of water bath was kept at (80±2) ℃ during the process. The length of the mortar specimen was measured after 3d, 7d and 14d immersion. The expansion of a specimen, ε_t, was determined from the equation:

$$\varepsilon_t = \frac{L_t - L_0}{L_0} \times 100\%$$

Where L_0 is the length of the specimen before immersion in NaOH solution; L_t is the length of the specimen at *t* days after immersion in NaOH solution.

3. Results and discussion

3.1 Petrographic analysis

Many rocks are completely or partly composed of reactive silica, such as andesite, chert, granite, gneiss, greywacke, siliceous limestone, quartzite, rhyolite, sandstone and tuff. The alkali-reactive silica is usually the amorphous or poorly crystallized silica phase in a rock, including opal or opaline silica, chalcedony, cristobalite, tridymite, microcrystalline, crypto crystalline and highly-strained quartz (Sims and Nixon, 2003).

The main compositions of aggregates from Dor River, Choona Village, Sirran River and Thakot Bridge are given in Table 7. The results of petrographic analysis showed that samples from Dor River rock contained 61.5% illite/sericite which was deleterious, while the Sirran River rocks contained 58.0% quartz. So they were alkali-susceptible aggregates. The rock sample from Thakot Bridge showed alkali silica reaction (ASR) potential as well. On the contrary, the aggregates from Choona Village had only 1.0% quartz which showed normal optics, so the rock has no alkali carbonate reaction (ACR) potential and no alkali silica reaction (ASR) potential.

Table 7: Petrographic analysis of aggregates

Aggregates	Main mineralogical composition	Content (%)
Dor River	Slate*	38.5
	Dark Grey to Grey Veined Limestone	26.6
	Dark Grey Carbonate Cemented Lithic Arenite	22.9
	Dark Grey Limestone	5.7
	Brownish Limestone	3.7
	Grey Limestone	2.1
	Vain Calcite	0.5
Choona Village	Dark Grey to Grey Veined Limestone	68.2
	Dark Grey Limestone	22.3
	Brownish Limestone	9.5
Sirran River	Dolerites	31.9
	Granites*	17.6
	Schist*	16.6
	Quartz Mica Gneiss*	11.8
	Creamish Grown Quartzites*	6.4
	Mica-Schist*	5.8
	Vein Quartz	5.2
	Gray Quartzites*	4.7
Thakot Bridge	Quartz	42.5
	Amphibolite	8.6
	Quartz Mica Schist*	7.9
	Granite*	7.4
	Biotite	6.8
	Quartz+Polygrainquart	6.7
	Feldspar	5.9
	Epidote	2.5
	Magnetite	2.3
	Carbonate	8.1
	Slate/Argilite*	1.9
	Acid to Intermediate Volcanics	1.3
	Muscovite	1.1

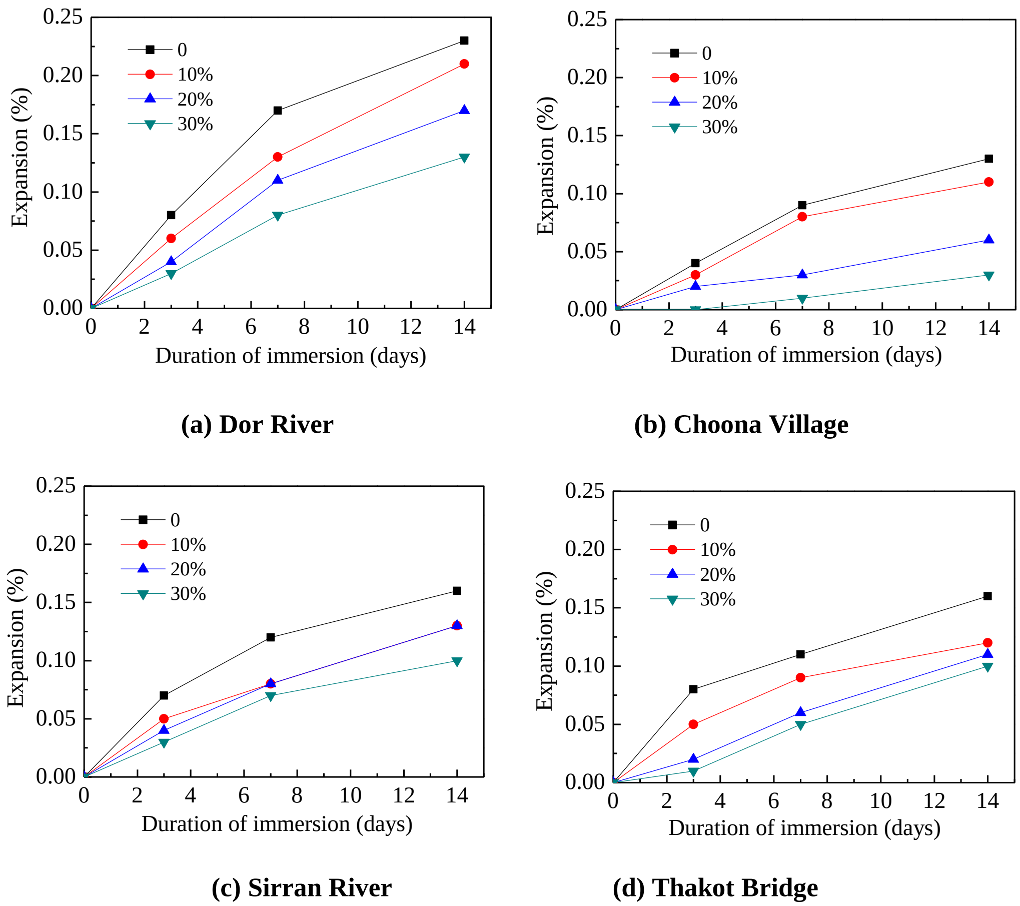
*Potentially deleterious constituents with an ASR Potential.

3.2 Effect of mineral admixtures on the expansion of mortars

The effects of fly ash and slag powder on the expansion of mortars with different aggregates are shown in Figure 1 and Figure 2. The results showed that the expansion of all mortars increased with the increasing of immersion days. And all four mortars with cement and different aggregates showed expansion higher than 0.10% at 14 days. This result indicated that the potential alkali-aggregate reactivity between the cement and four aggregates. The mortar with aggregates from Dor River had the highest expansion since it had the highest reactive silica content. While the mortar with aggregates from Choona Village showed the lowest expansion, which was consistent with the results of petrographic analysis.

Fly ash and slag powder reduced the expansion of mortars remarkably. With the increasing of the dosage of fly ash or slag powder, the expansion of mortars decreased at the same days of immersion. Taking the Choona Village aggregate as an example, when 30% fly ash or slag powder was added to replace the cement in mortar, the expansion was reduced to lower than 0.03% at the 14 days of immersion. In addition, the increasing rate of expansion with immersion days decreased too when the fly ash or slag power was added. At the same content, fly ash and slag powder showed the same level effects.

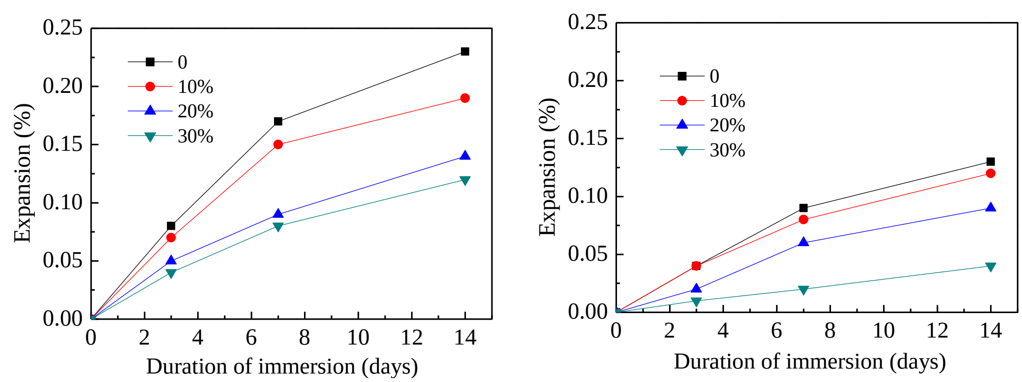
The expansion of mortars with silica fume is given in Figure 3. Silica fume also showed reducing effect on the expansion of mortars, but it was not obvious as fly ash and slag powder. This is due to the lower content of silica fume in mortars. When silica fume was used, the mortar with aggregates form Choona Village still showed the lowest expansion. However, the expansion of mortar with 9% silica fume and aggregates form Choona Village was higher than 0.03%.



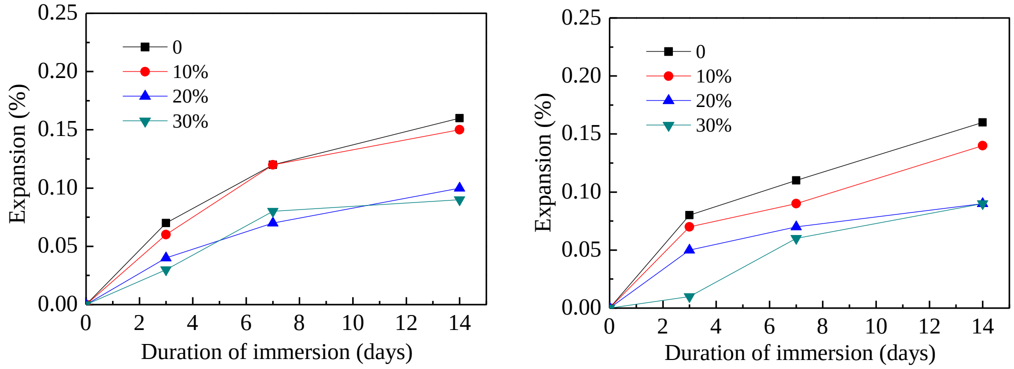
(a) Dor River **(b) Choona Village**

(c) Sirran River **(d) Thakot Bridge**

Figure 1: Effect of fly ash content on the expansion of mortars with different aggregates

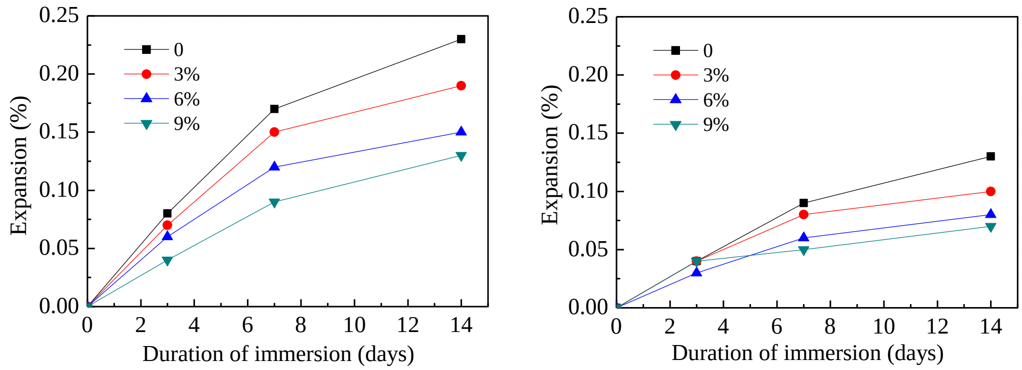


(a) Dor River **(b) Choona Village**

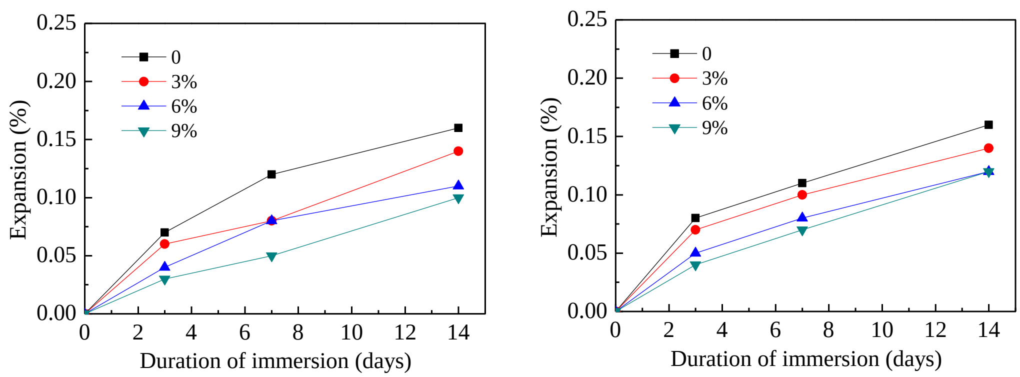


(c) Sirran River **(d) Thakot Bridge**

Figure 2: Effect of slag powder content on the expansion of mortars with different aggregates



(a) Dor River **(b) Choona Village**

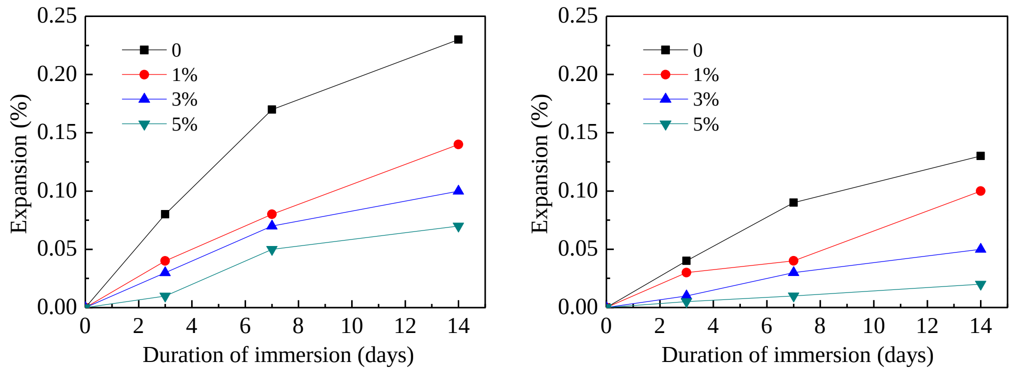


(c) Sirran River **(d) Thakot Bridge**

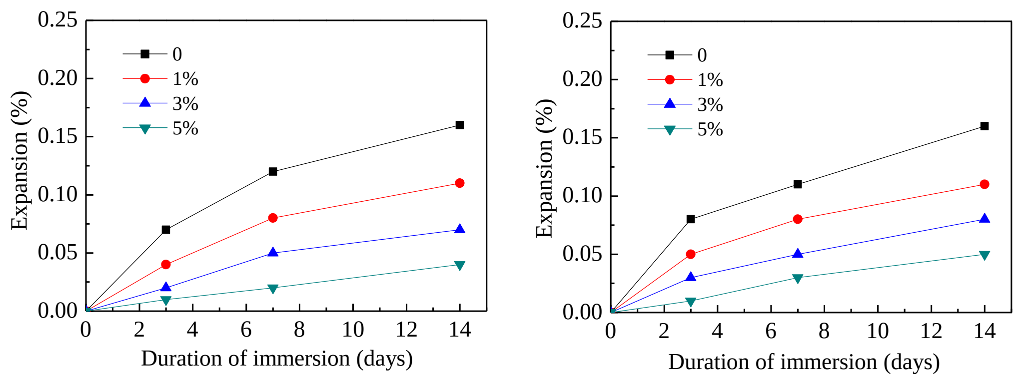
Figure 3: Effect of silica fume content on the expansion of mortars with different aggregates

3.3 Effect of chemical admixtures on the expansion of mortars

The expansion of mortars with different aggregates and chemical admixtures is shown in Figure 4 and Figure 5. The results indicated that both Li_2CO_3 and $BaCO_3$ mitigated the expansion of mortars with alkali-susceptible aggregates. With the increasing of content of Li_2CO_3 or $BaCO_3$, the expansion of mortars reduced obviously. Furthermore, Li_2CO_3 and $BaCO_3$ were more effective than the mineral admixtures in the limiting AAR. It could control the expansion of mortars at 14 days below 0.1% by addition 3% Li_2CO_3 or 5% $BaCO_3$. It is believed that the stability of silica increased due to reduced pH of pore solutions or a change in their chemical composition when chemical admixtures used. The presence of some ions, like lithium ions, also could prevent the gel from swelling (Zapała-Sławeta *et al.*, 2016).

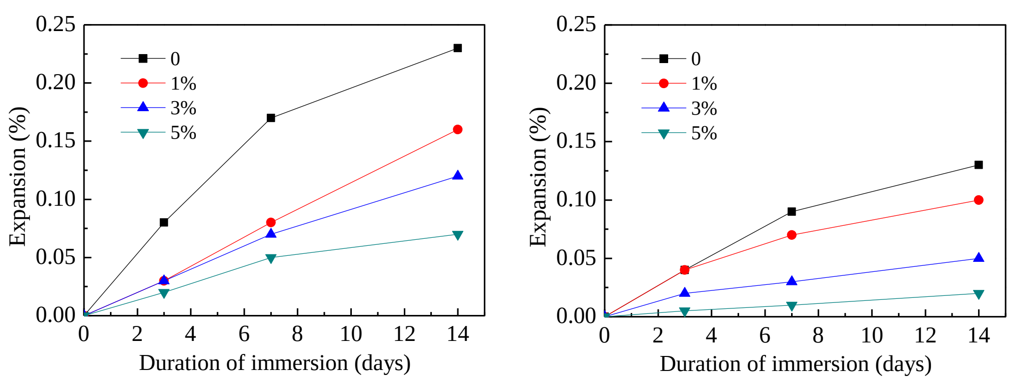


(a) Dor River **(b) Choona Village**

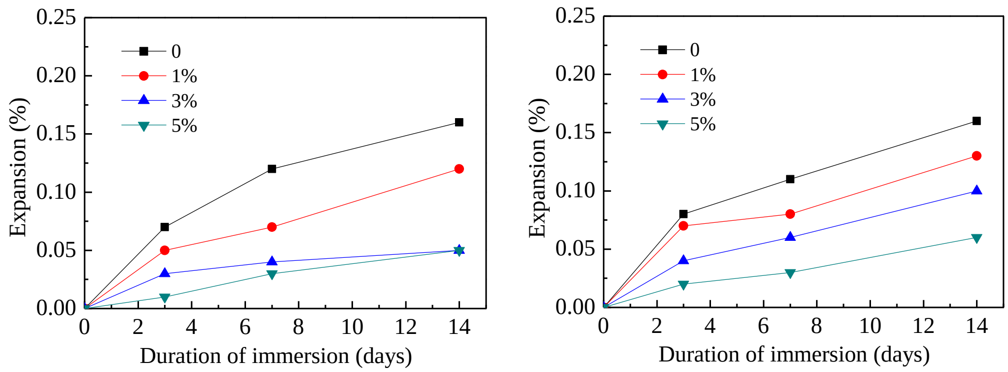


(c) Sirran River **(d) Thakot Bridge**

Figure 4: Effect of Li_2CO_3 content on the expansion of mortars with different aggregates



(a) Dor River **(b) Choona Village**



(c) Sirran River **(d) Thakot Bridge**

Figure 5: Effect of $BaCO_3$ content on the expansion of mortars with different aggregates

4. Conclusions

(1) The petrographic analysis showed that three out of four local aggregates had alkali silica reaction (ASR) potential, inclding aggregates from Dor River, Sirran River and Thakot Bridge. Aggregates from Choona Village were not alkali-susceptible.
(2) Fly ash, slag powder and silica fume decreased the expansion of mortars with different aggregates. For the content used in this study, fly ash and slag powder were more effective than silica fume in reduce AAR.
(3) Li_2CO_3 and $BaCO_3$ were more effective on the limiting the expansion of mortars due to the reaction of AAR. By addition 3% Li_2CO_3 or 5% $BaCO_3$, the expansion of all mortars with alkali-susceptible aggregates was lower than 0.1%.

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Fiber-Reinforced and Rubberized Cement-Based Composite: A Sustainable Repair Material for Thin Bonded Overlays

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Abstract

Concrete has been used in the construction industry from thousands of years and with the passage of time distresses or decrement of load carrying capacity of existing structures has been observed. There can be different approaches for rehabilitation of concrete. Among them, thin bonded cement-based overlay is found to be the most economical solution for large surface areas. The durability of such kind of applications is always a problem and the main reason behind is the cracking of repaired layer followed by the interface debonding. This work is devoted to study the debonding of thin bonded cement-based overlays from the concrete substrate under mechanical loading. Fiber-reinforced and rubberized cement-based mortars were used as repair materials. Although, rubber aggregates incorporation in mortar is detrimental to compressive and tensile strengths, but improvement in strain capacity is also observed. Similarly, the fiber-reinforcement of mortar improves the post cracking residual strength. Moreover, the combine use of rubber aggregates and fibers in repair material can be a suitable solution to delay debonding initiation and to limit the interface debonding propagation. In order to monitor the evolution of cracks and to measure the debonding along the interface, digital 3D image correlation technique was used.

Keywords

Rubber aggregates, fiber reinforcement, overlays, substrate surface preparation, interface debonding.

1. Introduction

A common way to restore the performance of a deteriorated pavement is thin bonded cement-based overlays. Thin bonded cement-based overlays is used to replace deteriorated concrete, to smooth a damaged surface and/or to improve the load bearing capacity of a structure by increasing its thickness, or to provide additional cover for corrosion protection (Bissonnette et al., 2011). This technique of thin bonded cement-based overlays proves to be very effective, especially for large concrete areas (Chanvillard et al. 1989; Granju,1996). With the passage of time, the major problem which occurs in these types of overlays is cracking of this thin repair layer followed by its debonding from the substrate (Granju, 1996; Tran et al., 2006). According to some previous researches (Granju, 1996; Tran et al., 2006;

Bissonnette et al., 2011), the debonding between overlay and substrate is majorly caused by mechanical loading and differential shrinkage. In both mechanisms, this lack of adherence primarily begins from edges, cracks and joints.

Fibre-reinforcement and rubber aggregates incorporation can favorably modify the mechanical behavior of concrete in particular its residual post cracking strength and strain capacity (Turatsinze et al., 2006; Nguyen et al., 2010; Turatsinze et al., 2016). These two properties are without any doubt essential in the durability of thin bonded cement-based overlays. Moreover, (Toumi et al., 2013; Gillani et al., 2016) also concluded that the combine usage of rubber aggregates and of fibre-reinforcement in mortar produce a positive synergetic effect (improvement in strain capacity and in post peak residual tensile strength). So, these benefits encourage the use of rubber aggregates and fibers in the repair material of thin bonded cement-based overlays.

2. Overlay materials

Four mortar mix compositions, i.e. 0R0F, 0R30F, 30R0F, 30R30F were selected as repair materials and cast on top of control mortar substrate. The quantity of each constituent of above mentioned mortar mixes is shown in Table 1 (Gillani, 2017). Before casting the repairs, the substrate surfaces were properly prepared by using the sandblasting technique for ensuring the proper bond between substrate and repair material. The use of above mentioned mortar mixes as repair materials allow us to highlight the effect of incorporating fibers, rubber aggregates and their combination on the structural performance of repaired beams. The rubber aggregates used in this study were obtained by grinding end-of-life tyres. The density of these rubber aggregates is 1.2 which is very less as compared to sand, i.e. 2.7. Also, the maximum dimensions of rubber grains do not exceed 4 mm.

Table 1: Mix design and proportioning (Values in kg/m^3)

Sr. No	Mix Designation	Cement	Sand	Water	Rubber Aggregates	Fibers	Super-plasticizer	Viscosity Agent
1	0R0F	500	1600	250	0	0	1.2	0
2	0R30F	500	1600	235	0	30	5	0
3	30R0F	500	1120	235	215	0	4.5	2.5
4	30R30F	500	1120	235	215	30	10	2.5

Mix designations:

- 30R30F: 30R refers to the mix containing 30% sand replacement with rubber aggregates by equivalent volume and 30F indicates 30 kg/m^3 of metallic fibers.

3. Three point bending test on composite specimens

3.1 Manufacturing of composite specimens

The composite specimens consist of thin repair layer on top of the substrate which simulates that the beam is repaired. Cement-based substrates were used in this study for making condition closer to real application. These substrate bases were manufactured by using control mortar (0R0F) mix composition. The dimensions of these prismatic substrate bases are (100×100×500) mm^3. After casting, these substrate bases were properly cured for 3 months under control temperature of 20 °C and at 100% relative humidity.

The results presented in some previous researches (Silfwerbrand, 1990; Garbacz et al., 2006; Courard et al., 2014; Gillani, 2017) show that substrate surface preparation has a direct impact on the durability of repair system. So, keeping in view the impact of substrate surface preparation on the bond behaviour of repair system, the sandblasted substrates were used in this study. In the light of previous researches (Tran, 2006; Nguyen, 2010), the thickness of repair layer used was 40 mm. So, on top of the sandblasted substrates, a 40 mm thick repair layer was cast. After casting repair layer on top of these sandblasted substrates, these repaired beams were cured for 28 days at a temperature of 20 °C and relative humidity of 100%. In order to localize crack initiation under mechanical loading, a notch is created at mid-span of each repair layer. This notch is created during casting of the repair layer on top of substrate by providing a simple reservation in the formwork as shown in Figure 1. The depth of this notch is 10 mm.

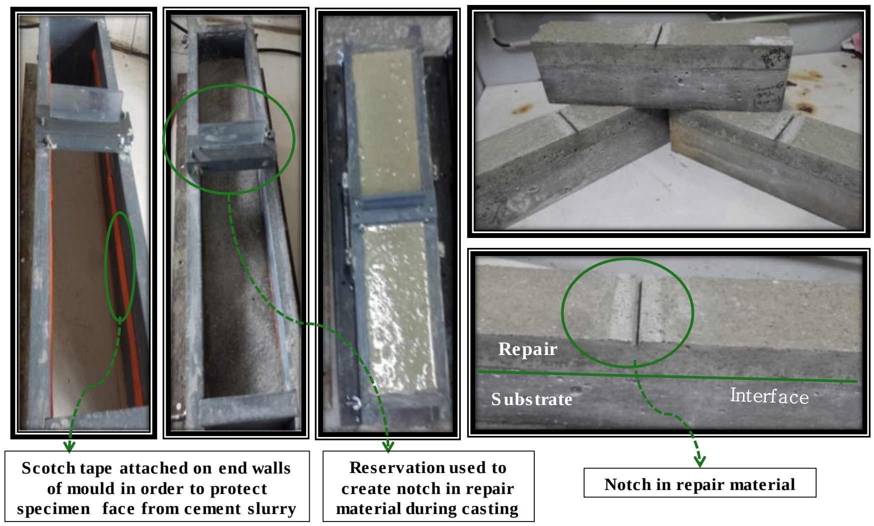

Figure 1: Casting of repair on top of substrate

3.2 Three point static bending test

Monotonic three point bending tests were carried out on MTS machine having loading capacity of 100 kN. Schematic diagram of the composite specimen under three point bending test is presented in Figure 2. Under mechanical loading, the crack initiates from the notched tip in repair layer, that subsequently cause the debonding when this crack reaches the interface between the substrate and the repair layer. These tests were controlled by Crack Mouth Opening Displacement (CMOD) using a sensor called COD. These tests were conducted by using the loading rate of 0.05mm/min up to 0.1 mm notch opening and then increased to 0.2 mm/min until completion of the test (when resisting load is approximately equal to zero). The values of force and opening of the notch (COD) are automatically recorded by the data acquisition system of MTS 100 kN machine. This data is also synchronized with 3D digital image correlation (DIC) equipment.

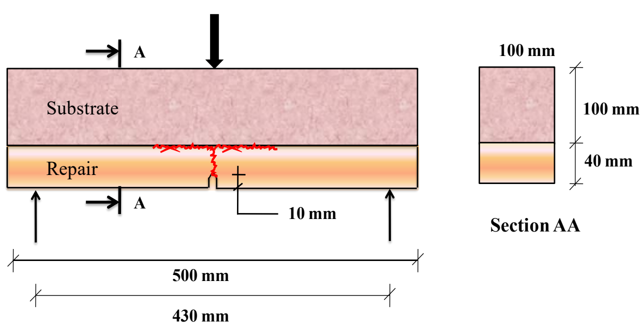

Figure 2: Schematic diagram of a composite specimen under three point bending test

3.3 Flexure testing along with digital 3D image correlation technique

3.3.1 Digital image correlation technique

Digital image correlation (DIC) is an optical and non-contact measurement technique that is used to measure displacements on the surface of an object of interest. This displacement is then used to calculate the surface strain of the object. DIC involves comparing a series of images taken in sequence over a period of time by a digital camera with specific resolution. The distributions of grey scale values in successive images are compared, and their differences are used to characterize the deformation of the surface. For the process to be most effective, the area of interest should be painted with a random speckle pattern prior to the start of the process (Gencturk et al., 2014). 3D digital image correlation technique requires two sets of images of the object taken from separate camera angles at the same time. The system must be calibrated to define the 3D space in which the event or process to be studied will occur. The results of this calibration process are then used to correlate the images from the two cameras to enable the determination of the studied deflection and strain of the material (Pickerd, 2013). In order to carry out 3D image correlation, sample preparation is necessary like painting it white and then black spot on this white painted surface as shown in Figure 3. Figure 4 shows the complete experimental testing arrangements (lights, cameras, etc.) necessary to use this digital image correlation technique. The purpose of conducting three point bending test along with this DIC technique is to follow the crack initiation and propagation and to determine the point at which crack reached at interface location.

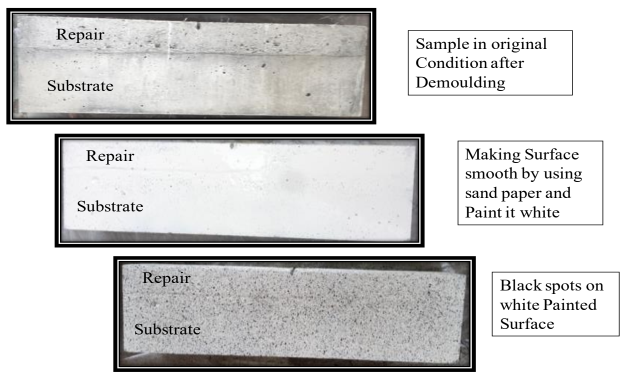

Figure 3: Sample preparation for bending test along with 3D digital image correlation technique

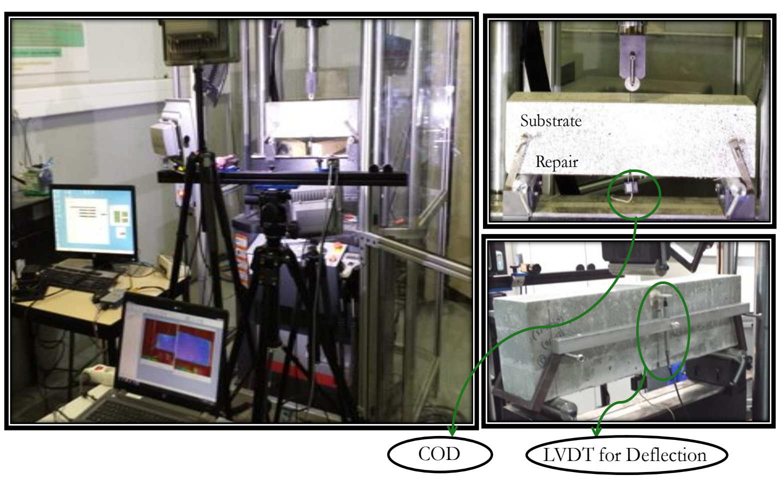

Figure 4: Complete experimental testing setup for three point bending test along with 3D digital image correlation technique

3.3.2 Detection of interface debonding initiation through 3D digital image correlation technique

The primary purpose of conducting three point bending test along with this DIC technique is to follow the cracking pattern and to determine the load at which interface debonding starts. For image processing, all recorded images were treated on the software Vic-3D, 2010. Post treatment of images indicates displacements and strains that occur on surface of the repaired beam during bending test from first image to last image.

This post treatment analysis shows the complete cracking pattern, the image at which crack initiates from tip notch, the load at which crack reached at interface location, etc. Also, it indicates that either debonding at interface occurs or not under flexure loading. In order to identify the load at which crack reaches at interface, an artificial extensometer is used at interface location in post treatment. This artificial extensometer is placed perpendicular to the direction to crack propagation as shown in Figure 5. This DIC technique provides the flexibility to do back analysis of resulting strains obtained by post processing of images. So, after tracing the crack path one can place the artificial extensometer at an exact crack location in order to precisely identify the load at which interface debonding initiates. When crack crosses this artificial extensometer, there is a sudden change in extensometer reading (D1). The load corresponding to the point at which there is a sudden change in D1 value is considered as a load at which crack reach at interface location and initiates the interface debonding. For each repair mix composition, atleast three samples were tested in monotonic loading along with DIC technique. Image processing results also lead us to conclude that interface debonding initiates on the spot when crack reaches the interface level.

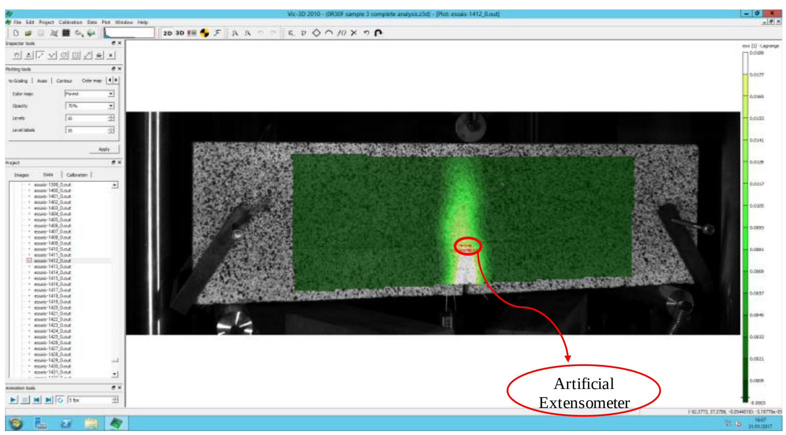

Figure 5: Selected strain visualizations for crack observation and use of artificial extensometer for detecting the load at which crack reach at interface location

4. Results and discussion

Debonding initiation force for composite specimens having different repair mix compositions are listed in Table 2. The results show that these loads are different for each repair mix composition. It may also be noticed that the opening of the notch at which crack reached at interface location is also variable for each repair mix. In the case of non-fiber repair mortars, i.e. 0R0F and 30R0F, the average initiation force at which interface debonding starts is less as compared to repairs reinforced with fibers (with or without rubber aggregates, i.e. 0R30F and 30R30F). Since, the presence of fibers in repair material provides bridging at the crack location and also limits the opening of the crack. In the case of 30R0F repairs, some improvement in debonding initiation force is also observed as compared to control mortar repair. Since, rubber aggregates incorporation in repair material tries to limit micro-cracking phenomena. Also, the debonding initiation force is significantly increased for specimens repaired with 30R30F material. This increase in load is due to the combined synergetic effect provided by rubber aggregates and fibers.

Table 2: Load at which crack reached at interface and debonding started

Mix Designation	Sample No.	Interface debonding initiation force (kN)	Average value (kN)
0R0F	1	7.0	6.6
	2	6.6	
	3	6.1	
0R30F	1	8.4	7.9
	2	8.0	
	3	7.6	
30R0F	1	7.0	6.9
	2	6.7	
	3	6.6	
30R30F	1	8.8	9.0
	2	9.0	
	3	9.2	

For an easy comparison, the set of average curves is grouped in Figure 6. These average curves show the interface debonding length versus load for the composite beams. The results show that the interface debonding propagation is least controlled in beams repaired with 0R0F and 30R0F mortar. It can be noticed that the interface debonding is limited in beams repaired with 0R30F and 30R30F mortar. The interface debonding propagation is closely related to the opening of the crack in repair layer. So, the fiber-reinforcement of repair layer controls the crack opening and thus limits the initiation and propagation of interface debonding. Figure 6 also shows that at any debonding length the corresponding force is always higher for beams with 30R30F repair. For example, in order to achieve 20 mm interface debonding, the force required is 6.5 kN for 0R0F repair, 10 kN for 0R30F and 10.8 kN for 30R30F. Similarly, the interface debonding initiation force is also higher with 30R30F repair material. This shows the positive synergetic effect of the combine use of rubber aggregates and fibers to limit the debonding initiation and its propagation.

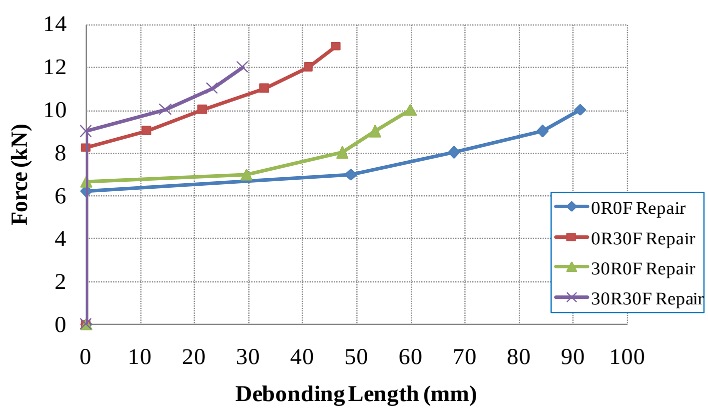

Figure 6: Force vs. debonding length at interface for different repair layers

The propagation of debonding along the interface between substrate and repair is a function of the opening of the notch as shown in Figure 7. The specimens with control mortar repair (0R0F) show a maximum notch opening and the corresponding debonding length is also maximum. On the other hand, opening of notch and debonding length is limited in composite beams repaired with 30R30F mortar.

The notch openings at which debonding initiate in beams repaired with mix compositions 0R0F, 0R30F, 30R0F and 30R30F are 0.015mm, 0.016mm, 0.021mm and 0.022mm, respectively. This shows that incorporation of rubber aggregates in repair material does not only control the cracking, but also helpful to delay the peeling initiation along the interface.

As far as the effect of fiber-reinforcement on the peeling initiation is considered, it is not so significant. Since, the crack opening at which debonding initiates is quite less. Indeed, these types of metallic fibers require a certain crack opening in order to play their role effectively. The debonding lengths are limited in specimens with 0R30F and 30R30F repair materials. So, it can be concluded that fiber-reinforcement of repair material not only control the crack opening but also limit the interface debonding.

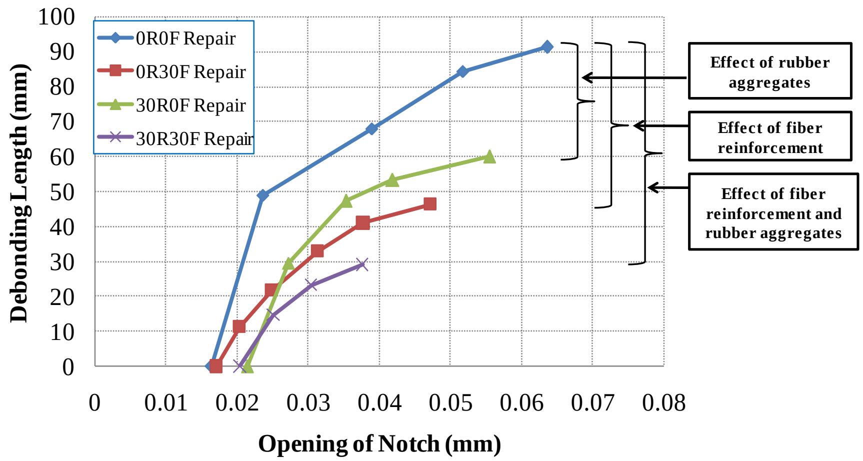

Figure 7: Debonding length vs. opening of notch for different repair layers

5. Conclusions

This experimental work reports the structural behaviour of repaired beams under three point bending. Four mix compositions, i.e. 0R0F, 0R30F, 30R0F and 30R30F were selected as a repair material. The reason for selecting these four repair mix compositions is to better understand the contribution of rubber aggregates, fibers and particularly the combine use of rubber aggregates and fibers towards the durability of the repair system.

- The load required to initiate the interface debonding is the lowest for specimens repaired with control mortar (0R0F). In case of specimens repaired with rubberized mortar (30R0F), the interface debonding initiation is delayed as compared to reference control mortar. Since, rubber aggregates in mortar control the micro-cracking, which as a result increase the debonding initiation force. Along with the debonding initiation force, the notch opening at which debonding initiates is also increased due to the enhanced strain capacity of rubberized mortar.
- The debonding initiation force is always higher for fiber reinforced repairs (with or without rubber aggregates) as compared to composite beams repaired with reference control mortar. The maximum debonding initiation force is observed in composite beams with fiber-rubberized repair material (30R30F).
- The interface debonding length is quite limited for fiber-reinforced repaired composite beams. The minimum debonding length is observed in specimens with 30R30F repair material. So, the combine use of rubber aggregates and fibers in the repair material of thin bonded cement-based overlays can be a suitable solution to delay the debonding initiation and also to limit the interface debonding propagation.
- DIC technique seems relevant and exciting tool in order to detect cracks, the load at which debonding initiates and to follow interface debonding propagation.

- The use of rubber aggregates obtained by grinding end-of-life tyres in cement-based materials can be considered as a contribution to maintain a clean environment by limiting the landfill for residual waste.

Finally, it can be concluded that in thin bonded cement-based overlays, the debonding between repair and substrate is mainly caused by cracking of repair layer and differential shrinkage phenomena between freshly cast repair material and relatively stabilized substrate. Under these conditions, it is recommended to use a repair material having an improved strain capacity and that can also provide post cracking residual strength like fiber-rubberized repair material (30R30F).

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Comparative Study on Seismic Performance of a Fixed Base Building with a Building with LRB and a Building with Shear Wall

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Abstract
The analysis of a structural system has now become a prerequisite to determine the deformations, displacements and forces induced by the applied loads or by the ground movements and to check for the adequacy of the designed structures. In this paper, the seismic response of a residential G+5 RC frame and a G+9 RC building has been analyzed by linear static strength based method using ETABS. Models were analyzed by incorporating a base isolator and a symmetric shear wall along the four corners of the building and compared for lateral forces, displacements, mode period and base shear with those of the fixed base structure.

Keywords
E-TABS, Lead Rubber Bearing, Displacements, Mode period, Base shear

Introduction

Bhutan is a small landlocked country between India and China and is zoned under earthquake zone IV and V as per Indian Standard, so it is essential to incorporate a high level of seismic resistance in the design of buildings to minimize the adverse impacts through proper mitigation and preparedness measures. This paper compares the performance of two most commonly used seismic retrofit techniques (i.e., base isolator and shear wall) in ETABS. One of the major problems faced in seismic resistant design of a building is to minimize the Inter-Story drift and floor acceleration. Inter-Story drifts can be minimized by stiffening the structure, but this leads to amplification of ground acceleration, which in turn leads to high floor acceleration. Floor acceleration can be reduced by making the building more flexible, but this only leads to large Inter-Story drifts.
The earthquake resistant structures are divided as rigid and flexible structures. In rigid structures the inter-Story displacements are reduced by providing diagonal bracings, shear walls, and using composite materials. In flexible structures the excitation input is reduced with the help of using dampers and Isolators (Venkatesh & Arunkumar, 2016). The only practical way to achieve the minimum inter-Story drift and

floor acceleration at the same time is to use base isolation. The isolation system provides the desired flexibility, with the displacements concentrated at the base level. Seismic isolation is an approach to earthquake resistant design that is based on the concept of reducing the seismic demand rather than increasing the earthquake resistance capacity of the structure.

To avoid and minimize the adverse effect of earthquake over its population, the Bhutanese government has restricted its constructional practice to G+4. Considering the limited space for construction and development, vertical expansion would be the best solution. Thus, this paper focuses on the analysis of three different buildings (i.e., fixed base building, a building with base isolation (LRB) and a building with shear wall of 300mm thickness) of varying heights (G+5 and G+9) and compare their performance under zone V earthquake loading and recommend best results for high rise building construction under Seismic zone-V.

Methodology

A building which is symmetric along X and Y directions was analyzed to determine the base reactions using which lead rubber bearing was designed through prescriptive approach. The same building was again analyzed by providing lead rubber bearing and the results were then compared with the fixed base building. The same building was then analyzed by providing a symmetric shear wall on the four corners of the building and compared with the previously analyzed structures. The repeated analysis was carried out for buildings of various heights.

Common constructional materials were adopted for analysis as given in Table 1. Sectional properties given in Table 2 were designed for dead load and live load as per IS: 875 and seismic load as per IS: 1893 considering the frame type to be a Special Moment Resisting Frame (SMRF).

Table 1: Material properties

Element	Concrete	Reinforcement	
		Primary	Secondary
Beam	M20	Fe500	Fe500
Column	M20	Fe500	Fe500
Slab	M20	Fe500	Fe500
Shear wall	M20	Fe500	Fe500

Table 2: Sectional properties

Members	Dimensions (mm)
Beam	500mm×300mm
Column	500mm×500mm
Slab	180mm (Thickness)
Story Height	3300mm (Height)

Building Plans

Shear walls placed in the core of the building are found out to be effective and economical. It can be observed that maximum reduction in drift values is obtained when shear walls are provided at corners of the building (K.O, Ramanujan, Sunil, Kottallil, & Poweth, 2014). Following this literature, shear walls were provided at the corners as shown in Figure 1 (b).

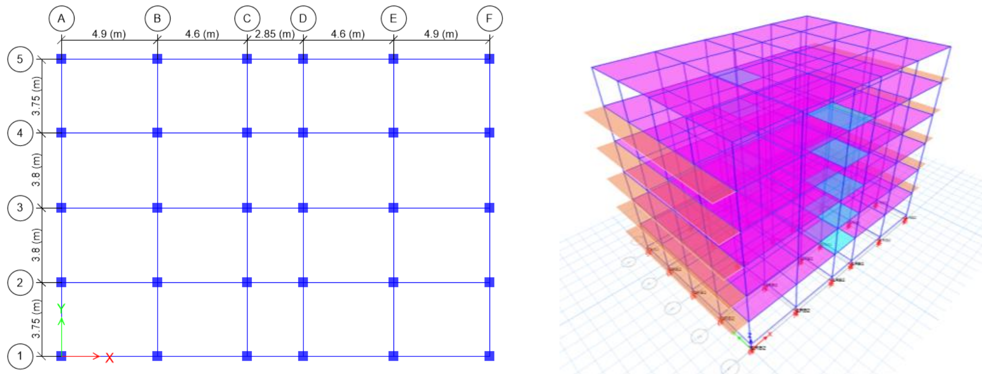

Figure 1(a): Typical G+5 &G+ 9 building plans and model

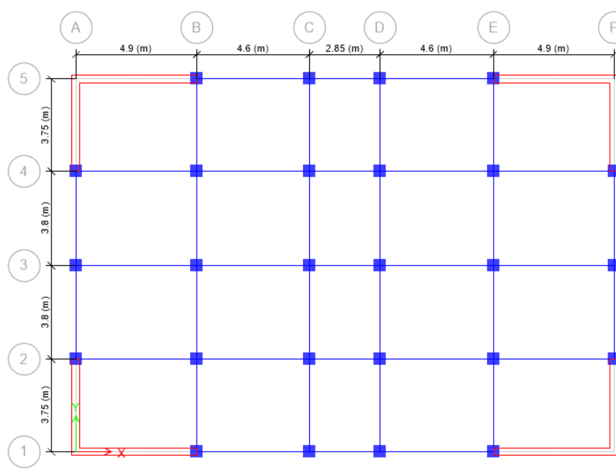

Figure 2(b): Typical G+5 &*G+ 9* building plans with shear wall

Lead Rubber Bearings

Lead Rubber Bearings (LRB) used for isolating the buildings as it has the properties of both damper and isolator. The effective period of vibration of the total system is lengthened sufficiently to reduce the force response, "Substantial reductions in the acceleration are clearly seen when the period of vibration of the structure is lengthened from 0.4 seconds to 2 seconds" (Arya, 1994).

The LRB adopted is designed as per the Uniform Building Code 1997. The data required for LRB design as per UBC 1997 and IS 1893: Part I are as follows:

- The seismic zone factor (Z) = 0.36
- Soil profile type (S_D) = Medium
- Seismic coefficient (C_a) = 0.36
- Seismic coefficient (C_v) = 0.54
- The Importance factor (I) = 1
- Response reduction factor (R) = 5
- Damping (β_{eff}) = 5%

The following properties of LRB has been determined through the above-mentioned design procedure

Table 3: Properties of LRB

Parameters	G+5	G+9
Maximum reaction KN	3402.4306	5287.677
Effective stiffness KN/m	2191.3543	3405.5577
Yield strength KN	58.2646	90.5484
Effective Horizontal stiffness KN/m	2019.1765	3137.9782
Vertical stiffness KN/m	843168.5321	1310358.205
Total height of isolator mm	432	477
Effective damping	0.05	0.05
Post yield stiffness ratio	0.1	0.1

Analysis Results

Story shear and story drift

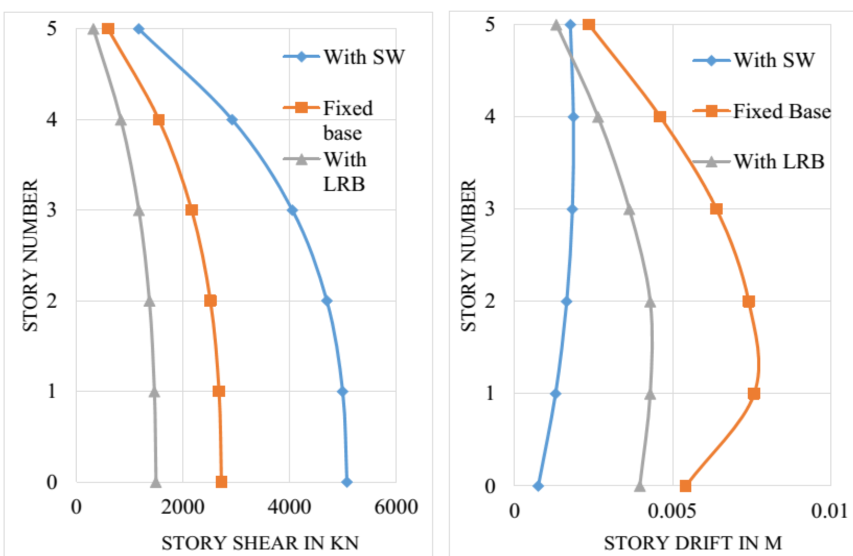

Figure 3: Story shear and story drift in x direction for G+5 story building.

Result observation showed that the story shear for building with shear wall doubled, while that with LRB was reduced by at least 45.14 %. While, comparison of the story drift for the top most story accomplished reduction in drift for building with shear wall by 25% and by 43.82% for building with LRB.

Displacement

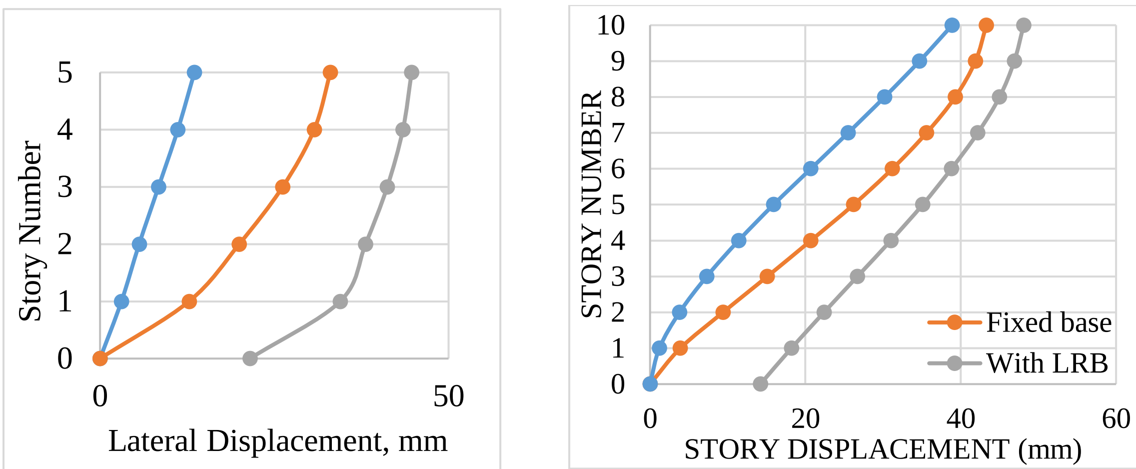

Figure 4: Joint displacements for G+5 and G+9 storied building in x and y-directions

Figure 3 shows that the displacement gets reduced by at least 40.8% while using LRB and by 59.4% while using shear wall for G+5 building. Also, the displacement gets reduced by 22.2% while using LRB and by 34.1% while using shear wall for G+9 building. However, the net displacement is minimum for building with LRB due to displacements taking place at the isolation level.

Mode period

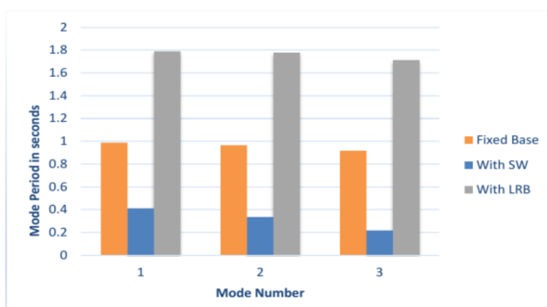

Figure 5: Graph for mode periods

The mode period gets increased by 44.75%, 45.67% and by 46.44% in modes 1,2 and 3 respectively while using LRB and gets reduced by 58.24%, 65.11% and 76.12% in modes 1,2 and 3 respectively while using shear wall.

Base shear

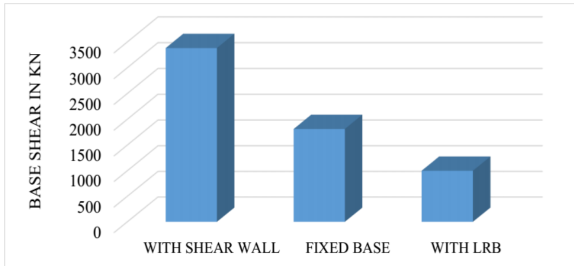

Figure 6: Base shear in x direction

There is an increase in base shear by 86.7% in the building with shear wall and a decrease in base shear by 44.78% when using LRB.

Story shear and story drift

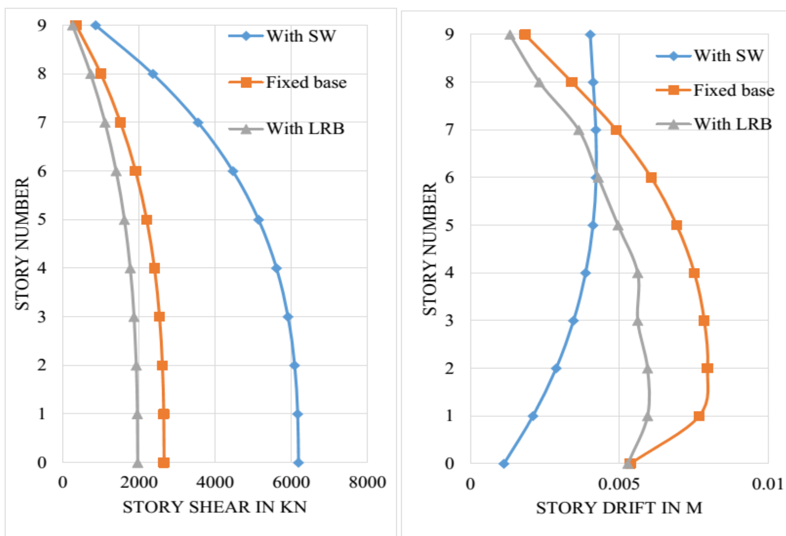

Figure 7: Graph showing story forces and story drift in x direction

On comparing the story shear, it was found that the story shear for shear wall was increased by almost 2.5 times as much and that of LRB was reduced by at least 26 %.

The story drift of the building with shear wall was found to be increasing as the height goes on increasing whereas it was decreasing for the other two building models. The net drift for the building with LRB is lower compared to that of shear wall buildings.

Soil Condition

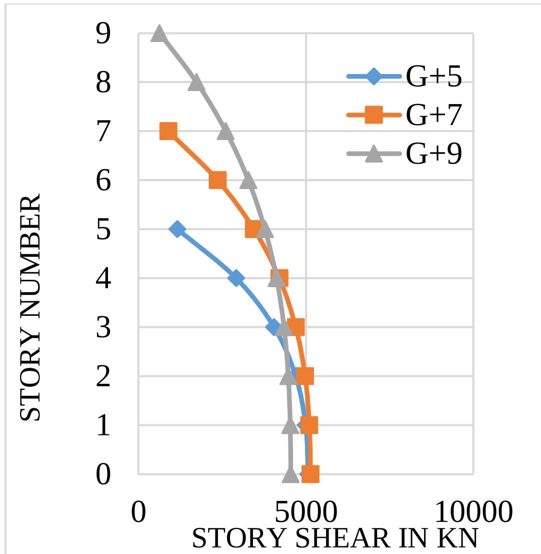

Figure 7: Story shear and Story drift of shear wall building under hard soil

Considering soil to be hard soil with uniform foundation, the building story shear for G+7 and G+9 buildings were found to be reduced by 82.65% and 86.09% respectively. Since, the variation is minimum reduction in shear force will reduce the deformation too. Therefore, shear wall is effective with increase in the building height.

Cost Comparison

The cost analysis is performed to evaluate the cost difference between the building with LRB and building with shear wall. The reinforcement percentage for beams, columns, slabs and shear wall are based on the Indian standard codes. The cost of Lead Rubber Bearing is based on Hengshui Shuanglin Rubber Product Co. Ltd, Hebei, China.

The approximate overall cost of construction for the G+5 fixed base building is Nu. 9285361.86 and for G+5 building with LRB it is Nu. 9939361.86 excluding the cost of transportation and installation. For G+5 building with shear wall is Nu. 9586236.35. Therefore, shear wall building is much economical compared to the building with LRB.

Conclusion

The study and analysis performed on various models concludes that,

- The increase in rigidity of a structure while using shear wall has resulted in increase in story shear, base shear and reduction in mode periods, story drifts and displacement.
- The increase in flexibility of structure by using LRB has resulted in the decrease in story forces, story drifts and base shear with increase in mode periods and displacement.

- The G+5 building performed better with LRB since there is increase in mode period by at least 44.75%.
- The decrease in the displacement in G+9 building model shows that it is more effective for shear wall then for LRB.
- The effectiveness of LRB and Shear wall both tend to decrease with increase in height of the building as viewed in the analyzed results.
- Although it is concluded that the G+5 performs better with LRB by viewing at the increase in Mode period, G+9 building performs better with shear wall as viewing the displacement reduction. Other factors such as cost of construction, installation and availability may also play a major role in determining the effectiveness of LRB and shear wall. Also shear wall is effective for controlling displacement with increase in building height.

This paper highlights that it would be better to use shear wall for vertical extension for a landlocked country like Bhutan where transportation cost is very high. The analysis result also supports shear wall as a structurally liable and effective solution to reduce the story drifts and minimize the adverse effect of earthquake by reducing the shear force, story drift and mode period.

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Assessment of Effective Marketing Strategies in AEC Industry of Pakistan-The Karachi Construction Industry Experience

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Abstract

The survival of construction organization has become difficult in the competitive world due to globalization and advancement in technology. Particularly the actual global economy crisis has the very strong impact on a construction sector, especially in developing countries such as Pakistan. Construction Marketing is very productive tool to improve the construction business. Many construction Professionals face difficulties in selling their services due to failure to embrace marketing concept. This research aims to assess marketing strategies AEC industry of Pakistan. The results presented, are on the basis of a survey in which a questionnaire was distributed among 45 different AEC firms. A statistical technique hypothesis testing was performed on the collected data. It is found that customer relationship and market research are most significant strategies while advertising and service planning are average significant strategies.

Keywords
Marketing, Construction Industry, Assessment

1. Introduction

Marketing is referred to as the activity of getting company to sell what the customer wants in the form of goods and services (Grace, 2011). Marketing has played an important role in the business development of the other industries like manufacturing industries. Because marketing strategy plays an active role that exposes you in market and helps to know the customer needs that what they want. Many benefits of effective marketing to construction have been highlighted by researchers (McNamara, 1999; Ngowi et al., 2000; Stewart et al., 2003; Dikmen et al., 2005; Ganah et al., 2008; Arslan et al., 2009; Polat and Donmez, 2010). This include survival of the firms, growth sustainability, increase in profits, increase in sales, increase in client satisfaction, development of company image, development of products/services, better competitive advantage Shortages of work during the recessions forced AEC industry to take marketing seriously. Recently there is an increasing recognition that marketing has an important role to play in the enhancement of the performance of the construction profession (Arditi et al.2008 ; Chen and muhamed, 2008 ; Arslan et al. , 2009; Polat, 2010; Polat and Donmez, 2010).

The construction industry has been slow to embrace marketing as strategic weapon. Therefore construction industry is lacking in adopting standardized marketing strategies due to its peculiarity. Winter and Preece (2000) share the opinion that a combination of marketing theories would be appropriate for the industry. They recommend a combination of industrial and service marketing theories to construction. Thus, for a contractor, they tend to favor a more extensive use of service marketing compared to industrial marketing as the contractor is usually not involved in any form of design.

David Arditi (2008) describes that business survival and a desire for greater profitability appear to have given construction marketing its greatest impetus. The construction industry is a “crowded” industry characterized by a high entry rate. Any interested person with little capital investment can enter the industry. Therefore, it comes as no surprise that contractors are constantly involved in evaluating ways that will allow them to make better offers than those of their competitors. Contractors also want to explore less crowded areas of construction that may provide increased profits. Marketing therefore becomes imperative to achieve these goals.

Yisa et. al (1995) investigated that the construction contracting and professional firms are increasingly realizing the importance of the marketing activities. Researches show that the in many countries AEC industry is adopting marketing strategies according to the suitability with respect to their demographics.

AEC industry of Pakistan is contributing alot in the economy and development of infrastructure. Therefore there is necessity of research for the assessment of the effective marketing strategies to improve the competitive advantage of the AEC industry of Pakistan.

2. Literature Review

Marketing in construction industry still remains undeveloped and misunderstood and numerous practitioners view marketing and its benefits with skepticism (Mahmood et. al 2017). A systematic approach to marketing is not observed among construction companies (Erdis et. al 2015) . Surveys in the USA, the UK, Turkey, Russia, Indonesia, Malaysia, Croatia, Nigeria, and Jordan indicate that the marketing budget and commitment to marketing management in the construction industry are substantially lower than that in other industries (Yissa et. al , 1995 ; Arditi et. al , 2008 ; Erdis et. al , 2015; Polat et. al, 2010; Tarawneh , 2013; Ojo, 2011 ; Jaafar et. al , 2008 ; Butković et. al 2010)

Specific strategies for the construction sector have been proposed: public private partnership (PPP) (Sobotka et.al , 2007) design and construction (Xu et. al , 2006) pricing strategy (Sullivan et. al ,2009) and social marketing (Barthorpe et. al ,2004). Joseph and Heney (2010) proposed a questionnaire specifically designed for contractors to aid them in arriving at a successful evaluation of their company and its business environment for successful marketing of its services. Gray (2005)recommends a refining of services as a way out of the commodity web. The quality of a contractor's technical performance, the extent of the innovative customization involved in a contractor's contract and the provision of extended services, such as financing and leasing (Naranjo et al, 2005).

According to Winter and Preece (2003) research may constitute valuable tools in marketing a contractor's product. Generally, the level of conformance with governing standards and specifications, the extent to which innovative means and methods of construction are used and the level of professionalism in construction management strategies are usually the differentiating factors in the construction service. Cariaga and El-Diraby, (2013) also shared the opinion that a combination of marketing theories would be appropriate for the industry. They recommend a combination of industrial and service marketing theories to construction. Thus, for a contractor, they tend to favor a more extensive use of service marketing compared to industrial marketing as the contractor is usually not involved in any form of design.

AEC industry has been slow to embrace marketing as a strategic weapon, especially by small to mid-sized firms. This spells "opportunity" for those companies with a solid track record and a willingness to share their experiences, knowledge and successes. Through effective marketing collateral, tactics and communications, marketing helps our AEC clients stay top-of-mind with the companies and contacts that are most important to their future growth. It was stated that goals regarding income are important, but not the only ones to be considered, of course. Other important goals include profitability, number of employees, and the type of work desired by the firm's principals (Yisa et al, 2010).

It is obvious from the literature review that there is a need of assessment of effective marketing strategies specific to AEC firms of Pakistan.

3. Methodology

The first step of the methodology adopted for this study was to go through the literature and get acquainted with the marketing strategies in AEC Industry. After identification of thestrategies, study was moved towards the next step: formulating questionnaire to be used as a statistical data collection method for the study. Questionnaire was formulated by focusing on some major factors on which a questionnaire should be designed which include defining target population and defining the sample size. The target population was mainly the Karachi AEC industry.

The purpose behind centering on the representatives was resulting from the fact that it was ought to root out Architects, Consultants and Contractors view about different marketing strategies. The sample size was of 45 respondents. The questionnaires were distributed among Architectural, Engineering and Construction firms in Karachi; these were done by personal administered surveys.
The next step was collection of these questionnaires among different industry members and later using them for analysis. The last step of study was the data analysis, which was then used to make important conclusions of the study.

4. Data Analysis and Evaluation

The questionnaires were filled and several different responses were received. It was found that the majority of the factors identified and selected for this research were valid; however there were a couple of factors which were rejected by the audience for survey. The data analysis was then done through Hypothetical Analysis (Z-test Method) in relative terms, as to which marketing barriers are most significant, and which ones are the least, or not average significant.

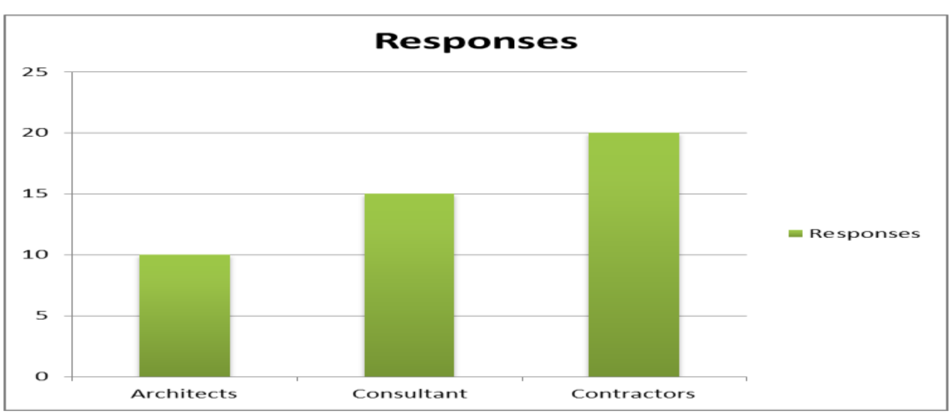

Fig. 1 Respondents Divisions w.r.t Roles in AEC Industry

In above figure it can be seen that respondents include all major key players of AEC industry i.e. 20 responses were from contractors, 15 responses were from consultants and 10 responses were from architects.

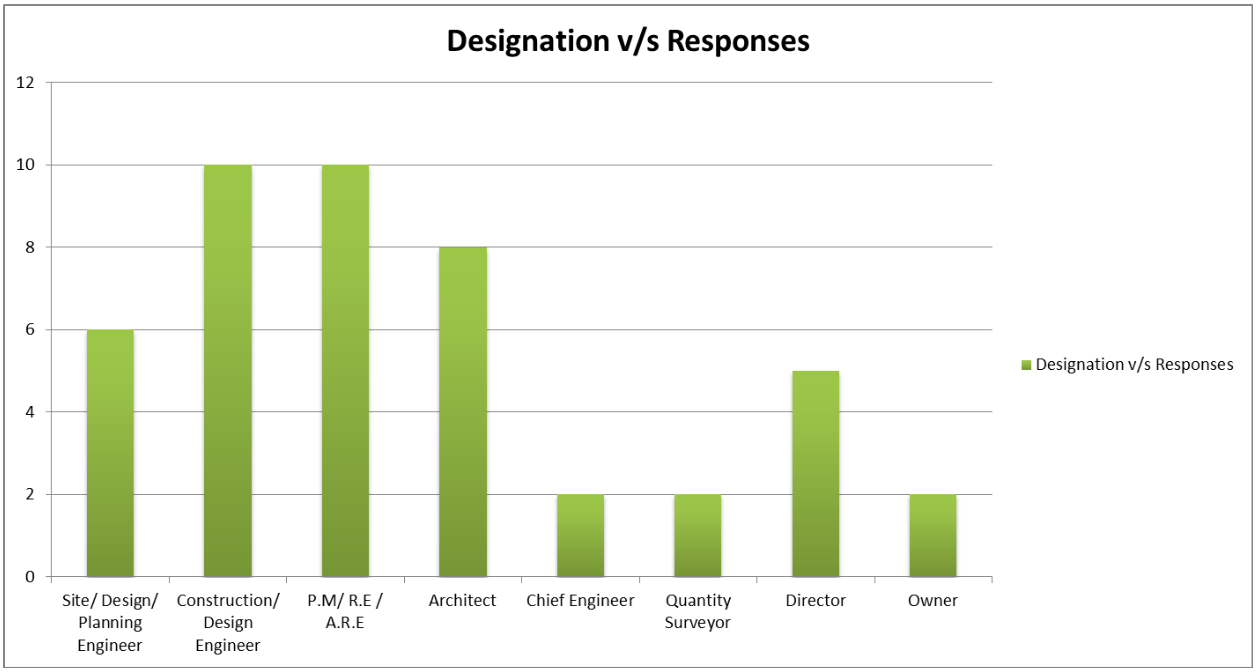

Fig. 2 Respondents Divisions w.r.t Roles in AEC Industry

As shown in above figure, respondents include mostly senior representatives of the AEC firms,

4.1 Hypothetical Test:

Hypothetical testing is used to determine what outcomes of a study would lead to a rejection of the null hypothesis (general or default position) for a pre-specified level of significance; this can help to decide whether results contain enough information to cast doubt on conventional wisdom.
The critical region of a hypothesis test is the set of all outcomes which cause the null hypothesis ($H_{\circ}$) to be rejected in favor of the alternative hypothesis (H1).

4.1 .1 Z Test:

A Z-test is the statistical test for the mean of the population. It can be used when n≥30. Where n = No. of respondents

Formula for Z-Test Analysis

$$Z = \frac{\bar{x} - \mu}{\frac{\sigma}{\sqrt{n}}}$$

Null hypothesis (H_o) and Alternative hypothesis (H1) for Marketing Strategies:
For aforementioned marketing Strategies two hypothetical tests have been performed; one for Most significant Strategies and other for average significant Strategies.
Following are the hypothesis defined for each of the marketing Strategies.

Marketing	**Most Significant**	**Average Significant**
Strategies	H_o= Not Most significant H1=Most significant	H_o= Not Average significant H1= Average Significant

4.1.2 Test value (P – Value) Condition:

After defining the hypotheses and the application of Z-test, the end result is concluded on the basis of final test value (P- value).

The condition was kept constant for every individual test and results were generated.

Condition: If P-value "<" or "=" 0.05, we accept H_o otherwise accept H1

HYPOTHETICAL TEST FOR OF MARKETING STRATEGIES		
Marketing Strategies	**P - Value (Most Significant)**	**P - Value (Average significant)**
Providing free preliminary estimates	6.02E-08	**0.189376163**
Printing brochures and newsletter	5.60E-08	**0.324342222**
Maintaining company website	0.153250105	**9.21E-07**
Marketing gifts with company logo	1.99E-07	**0.366165419**
Issuing news releases	0.139074427	**3.90E-11**
Employing professional marketers	3.25E-13	**0.109031672**
Hosting social events	9.41E-16	**0.881741582**
Setting up scholarships/endowments	3.13E-17	**0.063976506**
Advertising	1.06E-12	**0.85229494**
Usage of Technology	0.7714351	**2.50E-24**

6.Most Significant Marketing Activities

6.1 Maintaining company website:

It is rated as the most significant marketing activities. Presenting your firm's values and services in a clear, meaningful manner by maintaining company website in present AEC industry is of the utmost importance because of the fast availability of the information. The clients/users can easily get information, by navigating the internet, about the services that they wish to acquire, and besides that, they can check the information at any time of the day.

6.2 Issuing news releases:

This is also rated as most significant marketing practice. AEC industry mostly releases news about their current ongoing projects and future upcoming projects through inbound marketing.

6.3 Usage of Technology:

It has been rated as most significant marketing activity and widely used in AEC industry of Pakistan. Usage of technology has made the industry more competitive than ever before.

7. Average Significant Marketing Strategies

7.1 Providing free preliminary estimates:

It has been rated as average marketing practice by the industry. In some cases preliminary estimates could help to get satisfy the client and get business.

7.2 Printing brochures and newsletter:

A large number respondents in AEC industry has rated "Printing brochures and newsletter" as average significant among other strategies. According to them, in AEC industry Printing brochures and newsletter are not as effective as compared to other industries.

7.3 Marketing gifts with company logo:

A large number respondent in AEC industry has rated "Marketing gifts with company logo" as average significant. But in local AEC industry, this activity is taking pace and companies are giving sponsorships.

7.4 Hosting social events:

This is also rated as Average significant marketing activity. AEC industry in Pakistan is getting familiar with the benefits of hosting social events. During discussion with people in industry that with their recent past experience of hosting social events have given them a lot of benefits.

7.5 Employing professional marketers:

A large number of respondents have rated "Employing professional marketers" as average significant. They consider their industry different from other industries. So that is why they don't hire marketing professionals and feels it is not very effective.

7.6 Setting up scholarships/endowments:

It has been rated as average significant activity. In local AEC industry any such thing doesn't exist.

7.7 Advertising:

A large number of respondents in AEC industry have rated advertising as average significant among other Marketing strategies Also, during interaction with people in AEC industry; they disclosed that they do advertising for the following purposes.

8. Conclusion

The objective of the present research was to assess the effectiveness of various marketing strategies in the AEC industry of Pakistan. The study allows us to draw conclusion that the level of practice of marketing among professionals in Pakistani construction industry was very low and inadequate compared to the level and keenness of competition in Pakistani construction industry. Maintaining company website, issuing news release and usage of technology were effective marketing strategies identified by professionals in the Pakistani AEC industry, maintaining company website being the most effective while research and advertising were least effective marketing strategies used. Adoption of appropriate marketing strategies by professionals within the Pakistani AEC industry will result into acquiring more contracts, creating more awareness about their services, building of sales and maintenance of good and continuous relationship with their clients.

9. Recommendations:

This study was carried out in Karachi that is considered to be financial hub of Pakistan but for generalized results about marketing strategies in construction Industry of Pakistan, studies should be carried out in other cities as well. Further there is a dire need of trainings and workshops related to marketing strategies in AEC industry of Pakistan.

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Significant Factors Affecting The Performance of Highway Projects In Pakistan

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Abstract
Construction projects of highway projects located in the Sindh province suffer from many challenges and problems as well as complex issues. Therefore, the main objective of this paper is to find the factors affecting the performance of highway projects and to cause perceptions of their relative importance. A deep literature review was carried to generate a set of factors believed to affect project performance. A total of 120 questionnaires were distributed to 3 key groups of participants; namely consultant, client and contractor. The findings of survey indicate that these 3 stakeholders were agree that the most significant factors affecting project performance are: decision making, financial issues, shortage of advanced equipment's, poor site management, shortage of technical staff, poor designing, poor quality of material and wrong estimation. Same questionnaire was separately distributed to all 3 key stakeholders to identify the significant factor which affects the performance of highway projects. Results from 3 key stakeholders proved that all identified factors are same which were already identified from already carried survey.

Keywords
Performance, Highway projects, Stakeholders, Pakistan

1. Introduction

Every construction project is known as successful if its performance is good. Construction of projects can be divided in three stages, pre-construction stage, running construction stage and post-construction stage. Construction activities itself time taking process and also successful construction projects depends on performance. There are so many factors which barricade the performance of construction activities which cause delay in construction projects and its failure. The survey was carried and findings of survey indicated that the major factors affecting the performance of any projects were, delays of materials at site,

inaccessibility of resources, project leadership skills, change of material prices, shortage of experienced staff; and poor quality of equipments and available raw materials (A. Enhassi. 2009).
This construction industry is known as underperformed as compared to other industries. Not only that, the few construction industries has been complained for not good performing as that of other developed and developing countries. Construction industry is facing critical problems such as poor quality, poor safety, and inferior working conditions etc. These identified problems have been factors that affect construction productivity and that will affect companies over all performance (S. Alwi. 2003). The successful construction project depend on its over all performance, which is knows on based of completion of project within time, within the approved budget, approved standard quality and also customers satisfaction (Omran.2012). In addition, Sohu et al.,(2017) stated that cost. Time and quality are main parameters for successful construction project.

The success of any project can be defined as the performance of project evaluated in a different ways by the different stakeholders like client, while someone used the conservative performance measures, such as quality,time and cost of the projects. Other turns towards non conventional measures. Hence, there is need to find out the measures of performance that commonly used in the construction field and that constructions organizations as well (S. Bhatti. 2013). Cost of the project is known as one of the important criteria of successful project and is of high concern to those who are also involved in the construction projects. Though, previous studies shows that only few projects are completed within approved budget (Memon et al.,2011). Performance of construction projects have received considerable attention from academics and the construction stakeholders since last two decades, therefore awareness of the importance for use of suitable measures and also its role in supporting of the application in construction projects (S. Sarahan. 2013). However, this the reserach identify from a survey targeting client, contractors and consultant in an effort to shed and how construction project improve project performance.

2. Methodology

A questionnaire survey was used to prompt the attitude of client, contractors and consultant towards the factors which affecting the performance of any construction project in highway projects of Pakistan. A comprehensive literature review was carried out and 51 common factors were identified. On based of identified factors a questionnaires were distributed randomly to selected respondents from client, contractors and consultant. The targeted respondents for highway projects of Pakistan are selected from table of Krejcie & Morgan (1970). In total 120 questionnaires were distributed among three parties with formula of 40 questionnaires to each party of construction industry (40 questionnaires to client, 40 questionnaires to consultant and 40 questionnaires to contractor’s respectively). 101 questionnaire were received back from respondents and from 101 questionnaire 7 questionnaire were invalid due to invalid information.

Table 1: Summary of Distributed Questionnaire

Number of distributed Questionnaire	120
Received back questionnaire	101
Invalid questionnaire	7
Response of Respondents	84%

The respondents were requested to specify, each factor based on their experience and knowledge. The relative importance index method (RII) formula was used determine perception about significant factors of performance in highway projects of Pakistan.

RII = summation of W/ A*N,

Where, W indicates the weight given by the each respondents and which ranges from 1 to 5;
A is the maximum weight = 5; N is total number of respondents.

3. Data Collection and Data Analysis

Reliability test of collected data was also analyzed and value of Cronbach's alpha 0.81. As stated that if the value of Cronbach's value is above than 0.7 is known a data is reliable and acceptable (Pallant, 2011). After analysis of data the significant factors affecting the performance are shown in Table 2.

Table 2: Factors Affecting Performance of Highway Projects

Factor	RII value	Rank
Decision Making	0.912	1
Poor Management at site	0.906	2
Shortage of advanced equipment's	0.892	3
Finance Issues	0.871	4
Poor Designing	0.853	5
Poor quality of materials	0.850	6
Shortage of technical staff	0.843	7
Mistakes in time estimation	0.841	8

Results of questionnaire survey are shown in Table 1 which shows that client, consultant and contractors shown their concern about factors which affecting the overall performance in construction of highway projects of Pakistan. After analysis of data total 8 significant factors were identified from 53 common factors which affects the performance of construction projects. Identified significant factors were decision making, poor site management, shortage of advanced equipment, financial issues, poor designing, poor quality of materials, shortage of technical staff and mistakes in time estimation.

4. Perspective of Stakeholders for factors of performance individually

Feedbacks from respondents of each stakeholder are evaluated for significant factors which affect the performance of highway projects. Perspective of respondent from each stakeholder is quite different from each other hence results obtained from each respondent may vary from each other.
Table 3 shows about summarized results of significant factors proposed by Client, poor management at site, poor designing, shortage of technical staff, poor quality of material and decision making were found as significant factors which affects the performance of highway projects of Pakistan.

Table 3: Significant Factors of Performance Proposed by Client

Factor	**RII value**	**Ranking**
Poor Management at site	0.933	1
Poor designing	0.910	2
Shortage of technical staff	0.901	3
Poor quality of material	0.867	4
Decision making	0.862	5

Table 4 shows about summarized results of significant factors sorted by respondents of consultants, from the consultant perspective was that financial issues, poor site management, shortage of technical staff, shortage of new advanced equipment's and poor quality of material were found as significant factors which affect the performance of highway projects in Pakistan.

Table 4: Significant Factors of Performance Proposed by Consultant

Factor	RII value	Ranking
Financial issues	0.918	1
Poor site management	0.903	2
Shortage of technical staff	0.889	3
Shortage of new advanced equipment's	0.882	4
Poor quality of material	0.876	5

Table 5 shows results of analysis about significant factors sorted by respondents of contractors, from the contractors perspective was that financial issues, mistakes in time estimation, Poor designing, decision making and shortage of technical staff were identified as significant factors which affect the performance of highway projects in Pakistan.

Table 5: Significant Factors of Performance Proposed by Contractor

Factor	RII value	Ranking
Financial Issues	0.924	1
Mistakes in time estimation	0.917	2
Poor designing	0.908	3
Decision Making	0.891	4
Shortage of technical staff	0.882	5

5. Conclusion

A questionnaire based survey was carried out from stakeholders of highway projects of Pakistan, the significant factors agreed by the client, consultant and contractors were that main factors affecting the performance were decision making, poor management at site, shortage of advanced equipment's, financial issues, poor designing, poor quality of materials, shortage of technical staff and mistakes in time estimation. Then same collected data from each stakeholders was evaluated by same method RIW. Results showed that client considered that poor management at site, poor designing, shortage of technical staff, poor quality of material and decision making were significant factors which affects the performance of highway projects. From the data analysis of consultant perspective were that financial issues, poor site management, shortage of technical staff, shortage of advanced equipment's and poor quality of materials were significant factors. From contractor perspective the significant factor which affects the performance were financial issues, mistakes in time estimation, poor designing, decision making and shortage of technical staff.

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An Investigation of Real Estate Technology Utilization in Technologically Advanced Marketplace

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Abstract
Recently, digital technology has made its way into the property market, but the applications of new interactive technologies such as Virtual and augmented realities are yet to be explored in real estate sector. The literature in this area is scarce and thus provides an impetus for thorough exploration. This paper systematically reviews the state of the art technologies in the real estate websites of the US and Australia. It presents a SWOT score matrix including the advantages and functionalities of the Real Estate Technology (RET) such as Virtual and Augmented reality, 3D laser scanners, 360 cameras and walk-throughs along with technology adoption capacity of the websites. Based on the scored matrix, the paper discusses future trend in real estate industry taking user's perspectives into account and highlights the need of Technology Adoption Model (TAM) for real estate property management. A total of 10 websites: Top 5 visited websites each from Australia and the US were selected for a systematic analysis based on the SWOT based Rubric of focused disruptive RET use, neighbourhood insights, nearby facilities, virtual tours, total results and minimum price. These assessments are made by comparing search results of Mascot Sydney and Laurel Maryland. The results, based on comparative analyses and novel SWOT scores matrix, are expected to pave the way for developing TAMs in real estate management that so far, is non-existent considering available literature. In future, the matrix can be expanded to include customers and websites from both developed and developing countries and a holistic TAM can be proposed to add both theoretical and practical value to real estate body of knowledge.

Keywords
Technology Adoption Model (TAM), Real Estate Technology (RET), Real estate websites (REW), SWOT Rubrics, SWOT Analysis

1. Introduction

Globally, there has been 11.4% increase in housing market since 2008 (Chandler, 2016). In 2017 alone, there has been 1.8% increase each in the houses and other dwelling such as units, apartments, and condos (Chung, 2017). As per the report of Gold Coast City Council (2003) there will be a growth of as much as 80% residential area into the suburban areas by 2025. Such a high magnitude of residential growth makes real estate management even more important than before. As far as the use of internet in real estates is concerned, A survey of existing homes residents by National Association of Realtors (2017) highlighted the use of internet by 56% people of the age of 36 and below, and 50% from 37 to 51 years. Similarly, a drastic figure of 86% is quoted for usage of internet by new home searchers (Jackson, 2017). This high percentage of usage calls for investigation into technology usage and adoption frameworks in real estate

management and its websites with the websites being pivotal in customers interaction and real estate selection.
With the advancements in construction technologies and era of smart cities, disruptive and advanced technologies are making their way into construction and real instate industries (Ullah et al., 2016, Sepasgozar et al., 2016). These technologies include the use of GIS based Navigation tools, Real time locating systems, 360 visualization and Virtual and Augmented Realities (Guo et al., 2017, Lee and Lehto, 2013). Real estate is gradually adopting the technologies such as 3D videos and 360 imageries, but the state of its websites requires considerable improvements to be termed as state of the art (Richardson and Zumpano, 2012, Arndt et al., 2017, Yang et al., 2017). Currently, as per UABank Australia survey (2015), 33.3% Queenslanders spend 4-6 months for accommodation searching, 14% Victorians spend 7-12 months whereas 36% South Australians spend over 6 months for this search. In general, 9% Australians spend over 2 years for searching a reasonable accommodation. Similarly, in a survey conducted by Zillow.com (Black, 2017) for the US, 51% respondents quoted finding information and use of the website as the most difficult part. These statistics display the need of technology adoption in REW to reduce the search time and help customers find a home quickly, swiftly and as per their needs (Rauniar et al., 2014). Thus, customers' perception is pivotal to the current study along with the types of technologies to be used and their adoption frameworks.

To cope up with modern age technological and internet requirements, real estate needs to transform from traditional to smart real estate management however in its current state, the literature and frameworks for the use of disruptive technologies in real estates are non-existent. In this context the use of sensors to update REW in real time, disruptive visualization technologies: 3D Modelling, Virtual Reality (VR), Augmented Reality (AR), 360 cameras, Indoor Scanning Technologies: Laser scanning, cloud generation and Data generation: Big Data, Internet of things aspects are not explored (Ekman et al., 2016, Ainsworth and Ballantine, 2017, Agrebi and Boncori, 2017, Christensen et al., 2016, Huang and Wang, 2005). Further, due to the growing demands, the theoretical frameworks required for incorporating such advanced and disruptive technologies do not exist in real estate management. This study revolves around the incorporation of disruptive technologies in real estate management. Specifically, it addresses the need of Technology Adoption (TA) frameworks for REW and its dynamic updating using latest disruptive technologies. It uses Strength Weakness Opportunity Threats (SWOT) analysis for introducing SWOT score matrix that assign values to REWs based on its strengths, weaknesses, opportunities and threats to highlight the current state of top 5 REWs for US and Australia.

2. Literature Review

The modern era has seen the uprise of disruptive technologies. These technologies are a gift of the modern computerized world and provides ease of use in different domains. Though, the purpose is to provide ease to the users, this is not always the case with the people involved in a specific industry because of the humongous technological and conceptual requirements associated with such technologies (Flavin, 2017). Ganguly et al. (2017) defined disruptive technology as "the technology that displaces an established technology and shakes up the industry or a ground-breaking product that creates a completely new industry".

Strength Weakness Opportunity Threats (SWOT) analysis is a well-known and established tool used in project management researches. As the name suggests it involves exploring and stating the strengths, weaknesses, opportunities and threats presented to the item under investigation by internal or external factors. It is usually performed to know about the assets, be informed of the feebleness, know the opportunities available and plan vigilantly to avoid threats and make the maximum out of the opportunity at hand. Thus, it is at the core of risk management process that deals with maximizing opportunities and minimizing threats. In construction it has been recently used for prefabricated houses production (Li et al., 2016), promoting off site construction (Jiang et al., 2017) and home inspection (Xiao, 2016). In real estate sector, it has only been used in real estate development planning for promoting eco cultural tourism (Wang, 2016). It has never been used for real estate website assessment previously.

Websites are a key component of information dissemination and one of the trusted source in modern internet led era. Different companies in various sectors are successfully using websites for their product marketing and sales. One of the key advantage of such arrangements is the access to information at ones convince and no travel, and in person visits. Thus, people can buy anything from their homes remotely and can get a better price for the purchase due to the price compare options and discounts offered by different companies (Arndt et al., 2017). Global websites such as *Amazon*, *eBay*, *Walmart*, *AliBaba* and *Shop.com* are well known for their online shopping facilities and global reliability. When it comes to REW, there are plenty of websites existing globally that provides the rent or buy option to the users when it comes to real estate. These websites have transformed the traditional real estate management to more modernized computer based business with remote availability and access. In future, as suggested by different studies, it is expected to incorporate the revolutionary disruptive technologies as well. Figure 1 shows a comparison of traditional and current real estate marketing using websites. It also displays the recommended futuristic tweaks for the website upgradation and inclusion of some disruptive technologies.

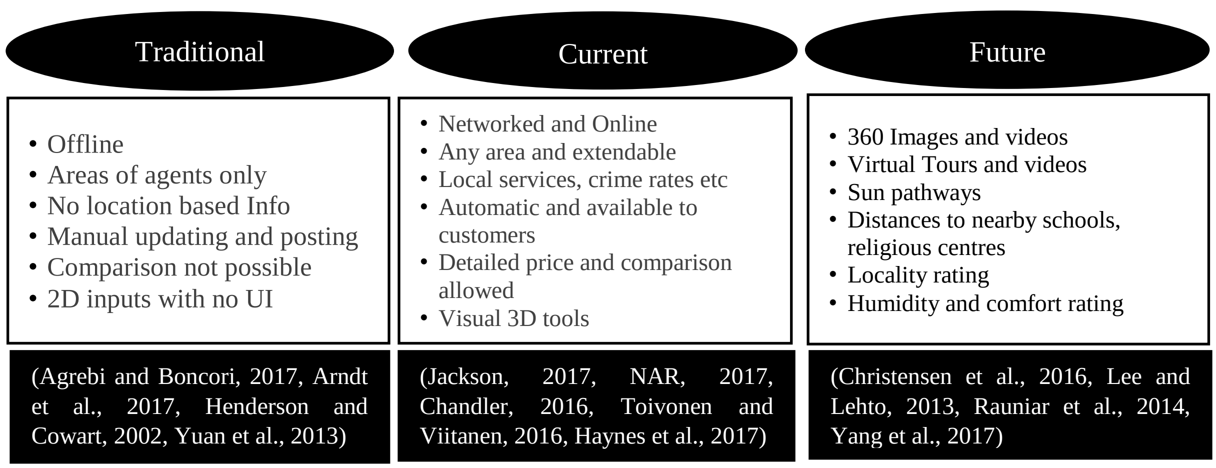

Figure 1: Traditional, Current and Future Real Estate Management

An example website in this context is *AirBnB*, that has revolutionized the hospitality industry and empowered the user to rent or buy short term accommodations including apartment rentals, hostel beds, vacation rentals, homestays, or hotel rooms. This has also empowered the home owners to bypass the traditional rigid system of hospitality services: to register the house as a rental property. Instead, the home owners can rent out a portion or a bed to a user for short time and get finances out of it without declaring their house as a full time rental service (Guttentag, 2015, Liu and Mattila, 2017). In terms of real estate, different technologies have been introduced over time such as VR, AR, Big data and Internet of things (IOT) based technologies. These technologies have revolutionized the real estate markets in terms of visualization but their incorporation in REW is almost non-existent and not explored properly. This gap is targeted in current study: to explore the use of disruptive technologies in REW and propose futuristic ideas about such incorporation. Further SWOT analysis has not been performed for real estate websites and the assessment rubric does not exist presenting yet another research impetus that is targeted in current study.

3. Methodology

In terms of study structure, the paper starts with identifying critical aspects of RET and REW. Further it explores the use of disruptive technologies in top five visited REW of Australia and the US and compares them using SWOT Scores. Figure 2 shows the methodology of current study. From relevant published literature types of disruptive RET are identified. Similarly, from online published data and website statistics, top five REW of US and Australia are highlighted. A rubric for SWOT is established for REW assessment and using case studies and relation assumptions, SWOT analysis is carried out for the REWs to highlight the state of REWs in US and Australia using SWOT Scores. This study starts with the exploration of RET critical aspects and highlights eight such aspects. These aspects are assessed and marketed using three main outlets: exhibitions, conferences and websites. The websites as per previous studies have five assessment criteria: real estate valuation, website evaluation, technology usage, website design and e-commerce. The criterion of technology adoption is focused in current study. The technology has two domains: traditional and disruptive. Disruptive technologies are of different types but specific to RET, four such technologies are focused in current study: AR, VR, IOT, and Drones. In addition, websites are assessed for basic technology adoption domains: information quality, system quality, perceived ease of use and service quality using the assessment rubric.

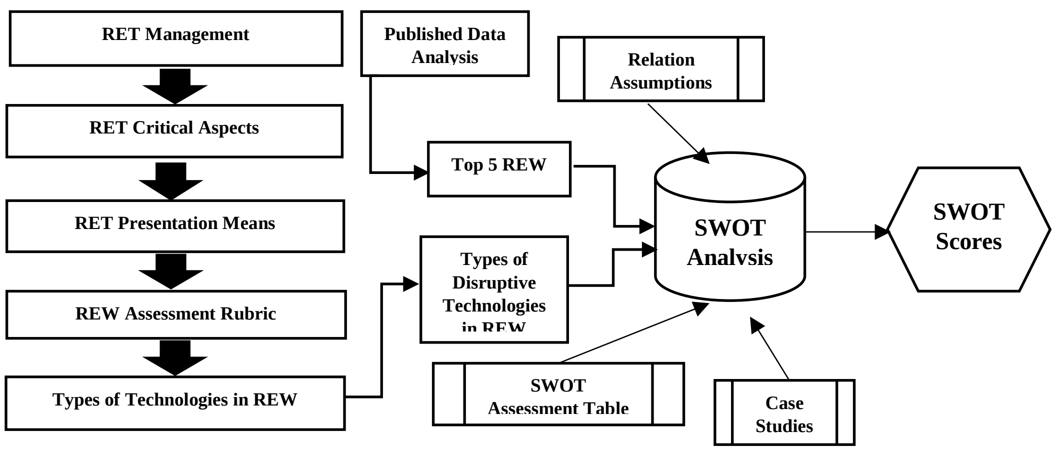

Figure 2: Study Method

For identifying the top 5 REWs of Australia and US, online published data and website statistics were used. Some popular REW include but are not limited to *zillow*, *realtor*, *realestate*, *domain* and others. Table 1 shows the top five REW for the US and Australia in terms of users visiting per month.

Table 1: Top Five REW in Australia and US

Australia					United States				
Code	Website	Users (millions)	Reach (%)	Avg time on site (minutes. seconds)	Code	Website	Users (millions)	Reach (%)	Avg time on site (minutes. seconds)
AU1	realestate.com.au	1.90	11.00	16.2	US1	zillow.com	36.00	9.17	7.1
AU2	domain.com.au	1.00	5.70	13.3	US2	trulia.com	23.00	7.00	4.42
AU3	realestateview.com.au	0.20	1.10	8	US3	realtor.com	18.00	6.09	7.14
AU4	homehound.com.au	0.20	1.10	8.3	US4	rent.com	7.09	5.43	3.92
AU5	homesales.com.au	0.15	0.50	3.12	US5	homes.com	5.00	3.73	3.35

To avoid using the websites names over and over, codes were used as shown in Table 1 with the first two alphabets representing the website country and the digit represent its position. Thus, US1 means the top website of US: *zillow.com.* From Table 1, the most visited website for Australia is AU1 with 1.9 million users followed by AU2 with 1 million users whereas the last among the top 5 lists for Australia is AU5 with 0.15 million users. Similarly, for the US the market leader is US1 with 36 million users followed by US2 with 23 million users and US5 as the 5th highest user attracting real estate website. Another key information displayed by Table 1 is the reach in terms of population percentage. For example, AU1 has 11% reach meaning that 11% population of Australia are exposed at least once to this website or simply, they know about it. The last column shows the average time in minutes spent by a user on the website. This may not exemplify the success or failure of the deal, but it gives an idea about the interactivity and keeping users involved or immersed in a website which is a critical success factor for such websites.

Further to explore the feel and ease of use of these REW, the results displayed by these websites have been explored for 1 bed 1 bath apartments in Mascot 2020, Sydney, New South Wales, Australia and Laurel 20707, Prince George County, Maryland, United States. Table 2 provides the details of the case locations. These locations are selected based on comparable areas, population and authors know how of the suburbs. In future, this ongoing study will result into a matrix for REW assessment to pave the path for TAMs in real estate that will lead to smart real estate management.

Table 2: Case locations details

Details	Australia (Sydney, NSW)	US (Prince George's County, MD)
Suburb	Mascot	Laurel
Zip	2020	20707
Area	9.2 km^2	11.2 km^2
Population	24181	32099

After identification of the top 5 REW for both countries, the next step was to perform SWOT analysis for these REW. For the swot analysis, two major considerations were considered: the assumptions of relations and assessment table for assessing the "how much" of strength or weakness of REW to assign it a SWOT score. In this context, the relations shown in Equations 1 to 3 were assumed. Equation 1 shows that weakness reduces with strength. Thus, an REW with more strength will have less weakness. Equation 2 shows that opportunity is the value of an REW to get the maximum points when strength score is deduced from the total. For example, if the total score is 50 and strength value is 10, the value of opportunity will be 50-10 =40. Thus, there is "40" points chance of improvements. Following the same logic in Equation 3, threats and opportunities go hand in hand, thus an "REW" having "40" points opportunity has the same value of threat as this opportunity if not availed will become a threat according to the basic definition of risk.

$$\textbf{Strength} \propto \frac{1}{\textbf{Weakness}} \quad \textbf{(1)}$$

$$\textbf{Opportuinty} = \textbf{Total Score} - \textbf{Strength} \quad \textbf{(2)}$$

$$\textbf{Opportunity} \propto \textbf{Threats} \quad \textbf{(3)}$$

Once these relations were assumed, the next step was to define and introduce the assessment rubric for calculating the strengths and weaknesses of the REWs. The rubric, as shown in Table 3, use a 5-point Likert scoring philosophy. All the assessment indicators were scored against the 5 points in accordance with the values corresponding to each score. For example, an REW displaying 38 results for the case projects was assigned a value of "4" against "R", thus in the SWOT matrix shown in results section, the "R" was placed in column 4 corresponding to its value from Table 3. Similarly, if a website offered fully digitized inspection, it was assigned a value of "5". The rubric houses 13 assessment factors for REWs: 9

of these are related to disruptive technologies usage while 4 are specific to the website design and qualities.

4. Results

Figure 3, representing the SWOT score matrix, shows the results of the study for the disruptive technologies in top five REW of Australia and the US. In addition to these disruptive RETs, the websites were also assessed for the number of results shown for 1 bed, 1 bath rented apartments or houses in the case areas of Mascot 2020, Australia and Laurel 20707, US. From Figure 3, none of the website is currently using any of the focused disruptive technologies except one only using 360 images and a few planning to use 360 videos in future. Top three Australian REW are moving towards the introduction of 360 videos as shown by the currently inactive 360 video buttons on these websites. Similarly, only one website i.e. AU3 is planning towards using VR in their online websites. Rest of the websites do not use any disruptive technologies. All the websites lack the remote inspection feature due to absence of 360 videos thus the inspection is manual request based where the clients must request for a time to visit the property.

Table 3: SWOT Assessment Rubric

ID	Indicator	Score				
		1	2	3	4	5
R	Search Results	≤ 10	11 to 20	21 to 30	31 to 40	41 to 50
MP	Min Price ($)	1301 - 1500	1101 - 1300	901 - 1100	701 - 900	501 - 700
AR	Augmented Reality	Not present at all	Idea exists about starting the services. Just a mention on the website or news feed	Inactive button is present on the website indicating the sooner start of the service	The services are present in the form of active button. Not fully functional and limited to advertisement videos only	The services are present on the website and are fully Functional
VR	Virtual Reality	Not present at all	Idea exists about starting the services. Just a mention on the website or news feed	Inactive button is present on the website indicating the sooner start of the service	The services are present in the form of active button. Not fully functional and limited to advertisement videos only	The services are present on the website and are fully Functional
360	360 Videos/Images	Not present at all	Idea exists about starting the services. Just a mention on the website or news feed	The services exist but are limited to interactive Images only	The services are present in the form of active button. Not fully functional and limited to advertisement videos only	The services are present on the website and are fully Functional
VT	Virtual Tours	Not present at all	Idea exists about starting the services. Just a mention on the website or news feed	The services exist but are limited to Plain Images only	The services exist in the form of both Videos and Images but are not fully functional	The services are present on the website and are fully Functional
NF	Nearby Facilities	Not present at all	Only education related facilities are shown	In addition to previous, hotels and clubs are also shown	In addition to previous, childcare, entertainment and parks are also shown	Extra facilities are shown in addition to the previous
SI	Suburb Insights	Not present at all	Only Plain/ interactive maps are displayed on the website	In addition to previous, occupation, age, dwelling types are also shown	In addition to previous, neighbourhood insights, crime rate, virtual agent are also shown	Extra facilities are shown in addition to the previous
IT	Inspection type	Not present at all	The traditional paper based request forms need to be filled	An online but manual request form was present	A mixed method existed having manual and digitized forms	The process was fully digitized for easier use
IQ	Information Quality	Just basic level information was present	Some more information existed than basic level but was difficult to use	Information provided was good, comprehensive, rich and somewhat easier	Information provided was better, comprehensive, elaborate, reliable and updated. Additional	Very easy terminology, novelty of information, rich content, more

		and was difficult to understand	and extract the key information	to use with greater improvement prospects	features were missing but was easier to use and get the information	content, accurate, reliable, detailed & updated information images from interior, 3D Models, and mortgage calculator was present
SQ	**Systems Quality**	The website design is poor and very slow speed.	The website design is basic level with normal speed and lesser graphics	The website has an acceptable design and good speed, has a few hyperlinks and normal response time with potential to improve	The website has good design and speed, a more logical information structure and more graphics but lacks customization	The website had quick response time, customization, page location, consistent graphics, hyperlinks, good design and loading speed, more immersion and logical information structure
PU	**Perceived Ease of Use**	The website is difficult to use and learn	The website requires significant efforts to understand and use. It had navigation tools, but usage was not easy	The website is easy to understand and had navigation tools with no option of going to previous pages	The website and information on it was easily discoverable. It has return option as well but lacks customization	Finding required information and learning the website was very easy. It has good navigation tools with easy returning to previous pages and customization
SE	**Service Quality**	The service quality is poor	The service quality is acceptable with lesser and lower quality products. The prices are high.	The service quality is good with normal quality products displayed and higher prices	The product and service quality are good and convenient, but prices are high	The product & service quality is good with greater convenience and lower priced properties

The top three websites show maximum results in terms of number of properties displayed with a maximum of 39 properties displayed by domain.com.au. This site also houses the lowest priced property for the case location. None of the Australian REW uses AR, VR or 360 videos currently. VR is planned to be used by AU3. Currently, normal advertisement based videos exists on the top three websites that are planned to be replaced by 360 videos in near future. The virtual tour facility on all these REWs is based on plain images and none of them use the 360 images. In terms of nearby facilities, maximum facilities are listed down by AU5 with distances. AU4 on other hand enlists the names of places only whereas the top three have special features of interactive maps enabling the users to scroll around and know the locality. In terms of suburbs insights, all the websites entail basic details such as occupation, age, family types and income of the locals. Additionally, AU3 offers the advantage of an online agent for answering specific queries and AU4 has the statistics of greenhouse gas contribution for houses.

The US REW although having more users due to the greater population of US, seemingly take a rather casual approach towards disruptive RET. None of the websites except US4 is using such technologies. US4 on the other hand is the only REW among all studied websites that is using the 360 images for its properties display. That is one of the reason for its larger user attraction despite being the newest website among its competitors. It also displays the most property results in terms of number: 30 properties. All other websites use the traditional 2D images based virtual tours. None of the top five US REW are using or otherwise indicating the use of disruptive RET including VR, AR and 360 videos. US4 plans on incorporating 360 videos in future as evident from the 360-tour button on their website. All the US REW focus schools in the nearby facilities with US1 and US2 mentioning clubs, markets, universities and businesses and hotels respectively. US4 like AU3 has the facility of online agent. US1 has unique features of transit and walk scores for suburb insights. Walk Score measures how walkable an address is based on the distance to nearby amenities whereas Transit Score measures how well a location is served by public transportation. Similarly, US2 and US4 have the unique features of displaying crime rates on their websites to give an idea about the suburban safe passages and dangerous zones. These two along with US3 have the features of interactive maps as well. US5, unlike above, displays and uses plain 2D maps only. In terms of minimum price, US1 displays the cheapest property at $815 per month. In terms of the website design and usefulness factors, as expected, the top 2 REWs for both countries are getting maximum points followed by medium marks for US3 and AU3 whereas the last two REWs are getting lowest marks for website design and features related to information, service and systems quality, and ease of use.

SWOT score, a key contribution and novelty of the current study was calculated in such a way that an REW can get a maximum of 50 points in its strength score. It is calculated in such a way that each indicator has a value of 0.77. This value is assigned so that the indicators which are 13 in number should get a maximum value of 10 (13*0.77). Thus, the maximum value an REW can get is possible only when all the 13 indicators get a maximum score of 5 each. The corresponding value in this case will be 13*0.77*5 = 50, where 13 is the number of indicators, 0.77 is the conversion factor and 5 is the corresponding value column. As an example, consider US1, where 3 indicators have value of "1", next 5 have value of "3" whereas 5 has value of "4". So, the score for this REW is ((3*1) + (5*3) + (5*4)) *0.77 = 29). Similarly, values for all REW are calculated using the above concepts. In terms of the SWOT score, AU2 and AU3 stands out of the studied REW with a strength score of 31/50. All the indicators are placed in the corresponding value columns based on Table 3 SWOT rubric. The lowest strength score "16" is obtained for US5 whereas none of the indicators are getting a value of above 3. Similarly, weaknesses are calculated by inverting the value of strengths. For example, a factor placed at value 2 corresponds to value 4 if the minimum score is 1 and maximum is 5 otherwise, the highest points are inverted. Consider US1, where the maximum value is 4 and minimum is 1, thus the boundaries for inversion are 1 to 4. Therefore, factors AR, VR, 360 placed in 1 are inverted into 4 in weakness column to get the corresponding weakness value of 21. It can also be done simply by subtracting the strength values from total possible score: 50-29 = 21. But, understanding the concepts needs logical underpinning, therefore logically all the values must be inverted. Thus, US5 the minimum strength scorer gets the highest weakness score. This is in line with the assumptions that greater the strength score, lower will be the weakness and vice versa. Further, pertaining the logics of assumptions, opportunities and threats are calculated as equal to weakness or the weakness score subtracted from the total strength score. This is since more weakness presents more opportunity and at the same time presents equivalent threats if things go wrong or corrective measures are not taken.

REW	Strengths					Score	Weaknesses					Score	Opportunities	Threats
	1	2	3	4	5		1	2	3	4	5		50- Strengths	50- Strengths
US1	AR, VR, 360		R, VT, NF, SI, IT	MP, IQ, SQ, PU, SE		29		IQ, SQ, PU, SE	R, VT, NF, SI, IT		AR, VR, 360	21	21	21
US2	AR, VR, 360	R	MP, VT, NF, IT	SI, IQ, SQ, PU, SE		28		SI, IQ, SQ, PU, SE	MP, VT, NF, IT	R	AR, VR, 360	22	22	22
US3	AR, VR, 360	NF	R, MP, VT, IT, IQ, SQ, PU, SE	SI		25		SI	R, MP, VT, IT, IQ, SQ, PU, SE	NF	AR, VR, 360	25	25	25
US4	AR, VR, NF	SI	R, IT, 360, IQ, SQ, PU, SE	MP, VT		24		MP, VT	R, IT, 360, IQ, SQ, PU, SE	SI	AR, VR, NF	26	26	26
US5	AR, VR, 360, PU	R, NF, SI, IQ, SQ, SE	MP, VT, IT			19			MP, VT, IT	R, NF, SI, IQ, SQ, SE	AR, VR, 360, PU	31	31	31
AU1	AR, VR	MP, NF	VT, IT, 360	R, SI, IQ, SQ, PU, SE		30		R, SI, IQ, SQ, PU, SE	VT, IT, 360	MP, NF	AR, VR	20	20	20
AU2	AR, VR		SI, NF, 360, MP, IT, PU	R, VT, IQ, SQ, SE		31		R, VT, IQ, SQ, SE	SI, NF, 360, MP, IT, PU		AR, VR	19	19	19
AU3		PU	R, MP, VR, AR, 360, NF, IT, IQ, SQ, SE	VT, SI		31		VT, SI	R, MP, VR, AR, 360, NF, IT, IQ, SQ, SE	PU		19	19	19
AU4	R, MP, AR, VR, 360	NF, PU	VT, SI, IT, IQ, SQ, SE			21			VT, SI, IT, IQ, SQ, SE	NF, PU	R, MP, AR, VR, 360	29	29	29
AU5	R, AR, VR, 360, PU	MP, IQ, SQ, SE	VT, SI, IT	NF		20		NF	VT, SI, IT	MP, IQ, SQ, SE	R, AR, VR, 360, PU	30	30	30

Figure 3: SWOT Score Matrix

5. Conclusions

This study focuses the investigation of technology adoption and subsequent use in REW of Australia and the US. The results based on SWOT scores for case studies of Mascot 2020 Australia and Laurel 20707 US, show that none of the top five REW for each country is using any disruptive RET in its current state. The only REW to use 360 images is US4, a US based REW. Three Australian REW: AU1, AU2 and AU3 are planning of introducing the 360 videos soon as indicated on their websites but none of the US REW is inclined towards it except US4 which is attracting larger audience. The Australian REW provides more details of the nearby facilities as compared to the US counterparts. None of the REW are providing 360 videos based virtual tours so far which can act as a crowd and potential buyers' attraction for upboosting the online markets of REW. Some useful features such as Transit scores and Walk scores introduced by US1 are a good step towards more information of the suburbs and users comfort but till date there is no remote tour facilitation on any REW which if introduced can lead to more customer satisfaction and attraction. Currently, all REWs are using the manual request based inspections of the properties which is time consuming and often ends in disappointment for customers due to excessive travel and no useful results. The SWOT scores for the top 5 US and Australian REW confirms that in its current state, the websites are below par in terms of strengths. Only three Australian REWs are getting 60% and above score which is alarming as these are the top 5 used websites in developed countries. These lower scores, displaying a poor current state of REW, present an opportunity at the same time to improve and uplift the REW state to state of the art. In total, 5 out of the 10 REWs are displaying 50% and above improvement opportunity which can act as an incentive for investors, property developers and researchers to explore and implement counter measures.

This study has both theoretical and practical implications. The theoretical implications include better understanding of technology adoption, risks and opportunities in real estates. Development of multiple Theoretical TAM frameworks for smart real estate management based on website design, VR & AR, Laser scanning and 360 cameras. Each can be taken as a research task and steps should be taken towards formulating theoretical frameworks. In terms of practical implications of this study, customers or end users can enjoy VR & AR based virtual tours and walkthroughs of homes. Have an idea about Sun pathways, Humidity levels and Defects in homes for smart decision making. Get access to location based services: Nearby schools, Hospitals, Police stations, and Religious centres when making a home rent/buy decision. Realtors will have improved business and more sales. Cost saving due to no inspection arrangements as well as time saving due to customers remote tours will also be achieved. Website Developers will have more business growth and can hire VR Technology aware people in time and exploring real estate market for business growth. This will also create job opportunities for people equipped with disruptive technology knowledge and subsequent software based incorporation in real estate website. This study is part of an ongoing investigation into the customers perception of REW and tweaking them in such a way as to achieve more customer satisfaction and attraction. The current study lays the foundation for futuristic TAM formations for each of the disruptive technologies. Further, the SWOT scores introduced in current study is the novelty of this research and can act as launching pad for many related and relevant exploratory researches.

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Assessment of Accuracy Improvement Measures and Internal Review Mechanisms for Construction Project Cost Estimation in Pakistan

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Abstract
The cost estimating practices have vital role in establishing effective project control. The pioneer product of the estimating exercise i.e. budget estimate is required to be used as a baseline for enforcing control mechanism from very early in the project till the project closeout. However, the accuracy of cost estimates is only revealed, once the project is complete. Thus, from the construction industry perspective, not only it's important to measure and identify the cost accuracy influencing factors, but emphasis should also be there to elicit information regarding the nature of accuracy improvement measures being adopted and internal review mechanisms established for reducing the inaccuracy of cost estimates. Realizing the need, this paper will present an assessment of both aforementioned aspects, as an extension of previously published study (by the same author) that explored accuracy of cost estimates of construction projects, in the construction industry of Pakistan. Total of 25 accuracy improvement measures (AIM) and internal review mechanisms (IRM) highlighted through the literature were utilized in this regards. Both AIM and IRM were made the part of same survey where the assessment of cost estimate accuracy and influencing factors was done so as to understand the link among these aspects. The paper provides short background of the overall study, then focuses on the analysis of the AIM and IRM aspects, and finally provide integrated conclusions and recommendations based on the results of the assessments. It is concluded that although, the construction industry is aware of the measures required for improving the cost estimate accuracy, however, their internal review mechanisms do not support this awareness through practice. The main findings are aligned with those of the earlier study i,e, lack of a systemized database, that hinders strategies such as benchmarking and effective use of the cost estimation software. The paper will act as a preliminary benchmarking study for the developing construction industry of Pakistan as well as can provide a foundation on which further extensive research plans could be developed in future.

Keywords
Accuracy improvement measures, internal review mechanisms, cost estimates, Pakistan.

1. Introduction

The cost estimating practices have vital role in establishing effective project control. The pioneer product of the estimating exercise i.e. budget estimate is required to be used as a baseline for enforcing control mechanism from very early in the project till the project closeout. However, the accuracy of cost estimates is only revealed, once the project is complete. Thus, from the construction industry perspective, not only it is important to measure and identify the cost accuracy influencing factors, but emphasis should also be there to elicit information regarding the nature of accuracy improvement measures being adopted and internal review mechanisms established for reducing the inaccuracy of cost estimates. Realizing the need, this paper will present an assessment of both aforementioned aspects, as an extension of previously published study (by the same author) that explored accuracy of cost estimates of construction projects, in the construction industry of Pakistan. Realizing the need, this paper presents an assessment of both aforementioned aspects of cost control in construction projects of Pakistan.

The paper will first discuss context of the study by defining its relationship with the prior study, importance of studying AIM and IRM, and source plus reasons for adaption of AIM and IRM List for this study. Thereafter, it will discuss the research methodology, followed by analysis and discussions, and finally the paper will derive some conclusions and present recommendations w.r.t. the context of the study. The paper will act as a preliminary benchmarking study for the developing construction industry of Pakistan as well as can provide a foundation on which further extensive research plans could be developed in future.

2. Study Context

2.1 Relationship with Prior Study

The study and analysis presented this paper is basically part of a comprehensive research that was conducted to evaluate the extent of cost estimate inaccuracy, identify the influencing factors that create such inaccuracy, determine the relationship between the inaccuracy and influencing factors, elicit information regarding accuracy improvement measures taken by the construction companies, and the kind of internal review mechanisms that have been established by the construction firms for construction projects in Pakistan.

In the already published part of this study, a thorough analysis was conducted to determine accuracy influencing factors in construction project cost estimates. Cost data was collected for more than 40 construction projects. A two-tier strategy founded on statistical analysis framework was adopted. This analysis resulted in determining the "perceived and real factors" influencing the construction project cost estimates' accuracy. The statistical framework was based on factor analysis (principle component analysis) and regression analysis. Project team-related factors category was found to be the influential category. Analysis resulted in determining that the most influential factors were "involvement of the contractor in the estimation procedure", and "labour rates" (Arif et. al, 2015).

However, since it's not only important to measure and identify the cost accuracy influencing factors, but emphasis should also be there to elicit information regarding the nature of accuracy improvement measures (AIM) being adopted and internal review mechanisms (IRM) established for reducing the inaccuracy of cost estimates.

2.2 Importance of studying AIM and IRM

The accuracy improvement measures (AIM) are considered to be those strategies that the construction project professionals deem necessary to improve the cost estimates' accuracy, i.e. decreasing the estimate bias. The internal review mechanisms (IRM) for cost estimates are considered to be certain cost estimate review mechanisms that are employed by the construction firms in order to ensure that the prepared cost estimates have the least errors, omissions and are as per the required quality of an accurate estimate.

The accurate prediction of construction costs is heavily dependent upon the availability of quality historical cost data, strong estimating procedures, the level of professional expertise and extent of use of information technology among other things (Liu and Zhu, 2007). Molenaar et. al. (2013) found that relationship among project participants (i.e. CM-contractor relationships) and involvement of designer in construction had strong correlations with the prime aspects of project success i.e. schedule and budget. Sriprasert (2000) points out those cost overrun problems are caused by ineffective construction management and poorly established cost control systems. If we combine the aforementioned three thoughts, it is pertinent to state that it is not only important to measure the accuracy of estimates but also exploration of such factors w.r.t. their implementation and effectiveness is also important.

2.3 Source and Reasons for Adaption of AIM and IRM List

The accuracy improvement measures (AIM) and internal review mechanisms (IRM) for this study were adopted from similar studies conducted by Ling and Boo (2001), and Aibinu and Pasco.[5,6] The study object is presented in this paper was to understand the awareness level of the industry professionals regarding the measures adopted by the construction firms to improve the accuracy of cost estimates, as well as the nature and extent of their internal review mechanisms to improve the estimate accuracy. Furthermore, the idea was to link the findings of this study with conclusions from an earlier study (Arif et. al, 2015). Since, the stated objectives also partially aligned with the objectives of the aforementioned studies, author found the AIM and IRM used in these studies relevant enough to be used for the current study.

3. Study Methodology

A thorough literature review was performed to identify of 25 accuracy improvement measures (AIM) and internal review mechanisms (IRM). The literature review included review of books, conference proceedings, the internet, and leading construction management and engineering journals.

A structured questionnaire survey was conducted in which the respondents were asked for the personal information (like experience, experience as cost estimator etc.), project cost information, rating of the accuracy influencing factors. Moreover, both AIM and IRM were made the part of same survey where the assessment of cost estimate accuracy and influencing factors was done so as to understand the link among these aspects. The analysis the responses regarding AIM and IRM were then conducted and weighted indices were developed to measure to draw comparisons and conclusions. Furthermore, relationships among the outcomes of the accuracy influencing factors study and the current study were analyzed qualitatively to provide a critical discussion alongside recommendations on the same.

4. Analysis and Discussions

4.1 Survey and Respondents' Characteristics

The questionnaire was disseminated among general contractors, specialty contractors, construction management contractors, and design-build contractors. The acceptable sample for this survey was Pakistan Engineering Council (PEC) licensed construction contracting companies, as PEC is the sole regulatory body for the engineering profession in Pakistan. The respondents were requested to provide responses with respect to their most recently completed project. The project cost data and influencing factors related data was intended to be filled by the cost estimating personnel related to the project surveyed. However, the data related to AIM and IRM was provided by the project managers and/or project personal. The details on the survey responses are shown in Table 1.

Table 1: Survey Response

Total Surveys	95
No Responses	35
Potential Responses	60
Valid Responses	46
Valid Response %	48

Total percentage of valid responses received was approximately 48%. This is an acceptable for a construction industry questionnaire survey. Abiinu & Passo (2008) collected cost-related data for 56 projects and received a response rate of 41% for a similar study. Therefore, reliable conclusions can be obtained from the collected data. A majority of the respondents were either civil engineering managers or project managers. The average experience of the respondents was approximately fifteen years in cost engineering.

4.2 Accuracy Improvement Measures (AIM)

A list of improvement measures has been developed through literature review and expert opinion. The respondents were asked to rank the probable effectiveness of these factors on a five point scale (1 = very low; 2 = low; 3 = medium; 4 = high; and 5 = very high) in order to improve the accuracy of cost estimates on their construction projects. The effectiveness index of each item was calculated using the following formula:

$$\textit{Effectiveness Index (EI)} = \sum \frac{(\textit{Level of effectiveness X Response frequency at that level})}{\textit{Number of Response}}$$

Table-2 provides the detail results of the analysis. According to ranking of the EI, the most effective improvement measure considered by the project managers is to "establish effective communication and co-ordination between members of the project team." An important conclusion that was drawn based on the results of already published part of this study. Arif et. al. (2015) emphasized that "the early involvement of the contractor in the cost estimation phase can result in early involvement of subcontractors, materials suppliers, and manufacturers." Therefore, result of current analysis reveals that the construction industry personal are already aware regarding the significance of this strategy and consider this to be very highly effective AIM. However, it needs to be explored further that to what extent this strategy is currently being employed by the construction industry and how can such a strategy be incorporated in the construction industry practice if not significantly employed at this time. A case study based research (already planned by the author) will be helpful in this regards. The second most effective improvement measure factor is to "ensure availability of sufficient information for estimating."

4.3 Internal Review Mechanisms (IRM)

A list of Internal Review Mechanisms (IRM) to reduce inaccuracy was developed through literature review and expert opinion. The respondents were asked to rank the level of usage frequency of these measures on a five point likert scale (1 = very low; 2 = low; 3 = medium; 4 = high; and 5 = very high) for their construction projects. The frequency index of each item was calculated using the following formula

$$\textit{Frequency of Use Index (FUI)} = \sum \frac{(Level\ of\ usage\ frequecy\ X\ Response\ frequency\ at\ that\ level)}{Number\ of\ Response}$$

Table 2: Effectiveness Indices of Accuracy Improvement Measures

AIM	EI
Establish effective communication and co-ordination between members of the project team.	4.19
Ensure sufficient information is available for estimating	4.08
Check all assumptions with clients and consultants.	3.96
Ensure proper design documentation	3.90
Provide a realistic timeframe for estimating activity.	3.86
Incorporate future market trends into the estimate	3.75
Establish formal feedback for design and estimating activities.	3.73
Improve methods of selection, adjustments and application of cost data.	3.69
Increase cost planning and control activities during the design stage.	3.56
Update cost database with new cost analyses and provide feedback for improving estimate accuracy.	3.54
Use a more rigorous method of estimating.	3.53
Incorporate market sentiments and economic conditions into the estimate by way of simulations, probability and utility functions.	3.44
Incorporate other market sentiments and economic conditions into the estimate.	3.29

The results are shown in Table-3. The results show that all of the FUI in each of the IRMs lies in the range of medium-to-high, to be more precise, half of the IRMs lie more near to medium than high FUI. According to FUI ranking the most frequently used IRM to Reduce Inaccuracy is "identification of project specific needs or risks", and the second most frequently used IRM is to "internal quality assurance procedures." Understanding the project specific needs is always a mandatory requirement for any kind of construction project, and although it is one of the IRM for any given project, the high FUI for this IRM is not that significant in showing the strength of IRM. The second significantly used IRM i.e. internal quality assurance is important information elicited through the survey. However, the FUI is not high enough to conclude that overall IRM system is strong. Infact, quality assurance itself is a subjective term and only upon further investigation (through case studies), can conclude whether or not the process of such assurances is a beneficial or not. On the other hand, there is lack of "internal peer review" in the construction firms as shown by the results. "Communication with the market forces" is also on the lower side while construction firms do not practice "benchmarking" to improve their review mechanism. All three of these are the key IRMs which should be the habitual part of any cost estimate development process. Not relying on benchmarking is also a significant lacking, and shows that generally, construction firms do not have a formal cost related database that can help in benchmarking and thus improving the cost estimates' accuracy. This is specifically true w.r.t. lack of productivity database, which was also one of the conclusions drawn in the earlier study by the author (Arif et. al, 2015).

Table 3: Frequency of Use Indices of Internal Review Mechanisms

IRM	FUI
Identification of project specific needs or risks	3.81
Internal Quality Assurance procedures	3.71
Bulk Checking / Self Checking procedures	3.70
Ensuring proper communication / information flow on the project	3.65
Comparisons with received tenders for future estimates	3.56
Review with final costs on projects	3.48
Use of external price information	3.42
Elemental review	3.38
Use of computer estimating software	3.31
Internal Peer review	3.14
Communication with the market	3.10
Benchmarking	2.96

5. Conclusions and Recommendations

It is worth mentioning here that in the earlier study, Arif et. al. (2015) found that "involvement of the contractor in the estimation procedure, and labor rates were the most influential cost accuracy influencing factors" (Arif et. al, 2015). The results of the analysis related to AIM and IRM presented in this paper are significantly aligned with these earlier conclusions.

The results of analysis for AIM show that the project managers/construction managers are aware of the fact that effective communication among project team members and availability of sufficient information for estimating could be the two most significant and effective accuracy improvement measures when we deal with construction projects. Both of these measures could only be assured when there is early involvement of the contractor in the estimating process. This can be achieved through utilizing project delivery methods that ensure early involvement of project participants such as; such as Construction Manager at Risk (CM @ Risk), Design-Build (DB), Public-Private Partnership Projects (PPP), Early Contractor Involvement (ECI), and Integrated Project Delivery (IPD) etc.

The results of analysis for IRM show an overall weakness in the review mechanisms or system. More significantly, the results (as also discussed earlier) advocate that there is a lack of formal cost related database both at the company level, as shown by the results of this analysis, and at the construction industry level, as shown by the analysis in Arif et. al (2015). At company level, this gap can only be fulfilled through an established cost engineering and control system, while at industry level through establishment of commercially available databases. A related finding is that the use of estimating software was also rated relatively low, which is also an indication of the fact that a strong cost related database would be required as such a database will benefit and encourage the use of the estimating software(s) to their intended effectiveness.

The author is currently conducting research on both aforementioned aspects and is also seeking for the construction industry's partnership (some of which already in-place) from both logistic and financial perspectives. A long-term partnership is between academia and construction industry would certainly result in high benefits for improving the standards with regards to cost control in the Pakistan Construction Industry. The research discussed in this paper would help as catalyst for establishing such partnerships by creating awareness among the construction industry professionals as the major findings of current and earlier published research will be disseminated through lectures and workshops.

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CBR Improvement of Road Subgrade Clayey Soil with Geotextiles

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Abstract

The presented study aimed to evaluate the effectiveness of using different types of geotextiles in order to reinforce weak subgrade soil of a road. The construction of highway pavements becomes uneconomical on weak subgrade such as on clayey soil since it can't support the axle load of the vehicles and results in different types of structural failures. Conventionally the weak subgrade soil is either replaced or stabilized to make it workable through different types of engineering processes or it is reinforced by different types of natural and conventional fibres. The contents of this study present one of those techniques.

Four different types of geosynthetics (Typar Sf-37 Geotextile, Typar Sf-56 Geotextile, Needle Punched Geotextile and Fiber Glass Geogrid Composite Geotextile) were used to reinforce the road subgrade clayey soil and their CBR values were evaluated. It was found that the CBR values of weak subgrade soil were improved 1.6, 1.2, 1.3 and 1.5 times when the weak road subgrade soil was reinforced with Typar SF37, Typar SF56, Needle Punched and Fiber glass geogrid composite geotextiles respectively. The CBR values were improved when the geosynthetics were placed at 0.2H (H is the depth of sample taken from top). The Geosynthetics TYPAR SF-37 and Fiber Glass Geogrid Composite Geotextiles have shown better results.

Keywords

Geotextiles, CBR, CBR increase ratio, Road subgrade, clayey soil.

1. Introduction

The significance of stabilized subgrade in overall construction of highway flexible pavement is very crucial to be considered because it is the base layer that decides thicknesses for all layers constructed over it and even increases the service life of the highway. The prime strength parameters of highway subgrade are its CBR values, higher CBR values the more economical with higher service life of highway will be. The construction of highway pavements becomes uneconomical on weak subgrade such as on clayey soil since it can't support the axle load of the vehicles and results in different types of structural failures. Conventionally the weak subgrade soil is either replaced or stabilized to make it workable through different types of engineering processes or it is reinforced by different types of nature and conventional fibres. (Benson and Khire 1994) improved the soil properties such as CBR values, Shear strength and resilient modulus of the soil by using the strips of plastic containers of reclaimed high-density polyethylene (HDPE). (Babu and Chouksey 2011) improved the strength and compressibility of soil by using recycled plastic waste from water bottles. Although several investigations were performed in order to find the reasonable depth for embedding the geosynthetics. (Nagrale, Sawant et al. 2010; Iliescu and Ratiu 2012) improved CBR values of sub-grade soil from 2.9% to 9.4% and 4.15% to 6.83% respectively by using geogrids reinforcement. The separator layer of geotextile between the gravel layer and soft subgrade interface prevent the intermixing and also improves the CBR values (Dembicki and Alenowicz 1990; Chakravarthi and Jyotshna 2013).

Materials

In this study Clayey soil was selected as base material; while four different types of geosynthetics were used as reinforcing agents in the clayey soil.

Soil

Clayey soil was used as a base material. The selected clayey soil was sieved through wet and dry sieve analysis as per specifications of ASTM D6913-04 (2009) and ASTM D1140 (2017). The index and physical properties of the base material were measured as regulated by ASTM D4318 (Standard 2000) and are given in

Table **Error! No text of specified style in document.**-1. As per USCS classification while using A-line chart for further categorization of clay, the soil may be classified as low plastic clay and/or silty clay. The Maximum dry density (MDD) and optimum moisture content (OMC) were found through modified proctor test as per specifications of ASTM D1557 (Standard 2012).

Table **Error! No text of specified style in document.**-1 Properties of Soil

Property	Soil
Liquid Limit (%)	37
Plastic Limit (%)	21.67
Plasticity Index	15.33
Specific Gravity	2.73
Clay (%)	73
Silt (%)	24
Sand (%)	3
Mean Grain Size (mm)	0.0061
Coefficient of Uniformity (Cu)	2.777
Coefficient of Curvature (Cc)	150.32
OMC (%)	10.25
MDD (lb./ft3)	122.3

2. Geosynthetics

In this study four different types of geosynthetics were used for the reinforcement of clayey soil in order to compare their strength contribution to the unreinforced weak subgrade soil (Clayey Soil) and these four types of geosynthetics were easily available in the market. The detailed engineering properties and characteristics of these four different types of geosynthetics are given in the Table Error**! No text of specified style in document.**-2. The visual view of different four geosynthetics trimmed to the diameter of the mould are shown in Figure Error**! No text of specified style in document.**-1.

Table **Error! No text of specified style in document.**-2 Properties of four different types of Geosynthetics

S. No	Geosynthetics	Properties	Units	Value	References
1	TYPAR SF-37 Geotextile	Area Weight	g/m^2	125	EN ISO 9864
		Tensile Strength	KN/m	8.5	ASTM D4595
		Grab strength	N	725	ASTM D4632
		Puncture CBR	N	1200	EN ISO 12236
		Tear strength	N	320	ASTM D4533
2	TYPAR SF-56 Geotextile	Area Weight	g/m^2	190	EN ISO 9864
		Tensile Strength	KN/m	13.1	ASTM D4595
		Grab strength	N	1100	ASTM D4632
		Puncture CBR	N	1850	EN ISO 12236
		Tear strength	N	460	ASTM D4533
3	Needle punched geotextile	Area Weight	g/m^2	140	EN ISO 9864
		Tensile Strength	KN/m	10.1	EN 1SO 10 319
		Grab strength	N	1100	EN ISO 12237
		Puncture CBR	N	1720	EN ISO 12236
		Tear strength	N	500	EN 13433
4	Fiberglass Geogrid Composite Geotextile	Area Weight	g/m^2	210	EN ISO 9864
		Tensile Strength	KN/m	18	ASTM D4595
		Grab strength	N	1500	ASTM D4632
		Puncture CBR	N	1800	EN ISO 12236
		Tear strength	N	600	ASTM D4533

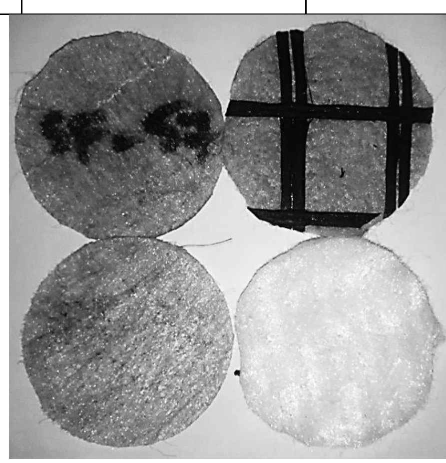

Figure **Error! No text of specified style in document.**-1 Geo synthetics samples cut to the diameter of the cylindrical mould

3. Experimental Setup

In order to investigate the bearing capacity of reinforced clayey soil and unreinforced clayey soil with geosynthetics, we need to develop a reliable experimental setup so that it should give the desirous results. The developed experimental setups basically involved the series of CBR tests that were performed on reinforced and unreinforced clayey soil with geosynthetics.

CBR test

This study involved a series of CBR tests that were performed on geosynthetics reinforced and unreinforced clayey soil in order to evaluate the effect of geosynthetics on CBR values. As no specific and standard procedure for investigating the effect of geosynthetics on CBR values is currently available. However thorough review of the literature where the CBR values were improved by using different types of fibres and geosynthetics (Khan and Rahman ; Mishra ; Yetimoglu, Inanir et al. 2005; Naeini and Mirzakhanlari 2008; Nair and Latha 2010; Sivapragasam, Vanitha et al. 2010; Chakravarthi and Jyotshna 2013) and guidelines suggested by ASTM D1883-16 were followed for this purpose.

Test procedure

A Series of CBR (Unsoaked and One point) tests was carried out in the conventional CBR mould available in the laboratory of 152 mm internal diameter and 178 mm of total height. A oven dried 5.5kg each clayey soil sample without geosynthetics were prepared and compacted in CBR moulds at its OMC as per procedure of ASTM D1557 and ASTM D1883-16 in five layers and 56 blows each layer. While moulds with geosynthetics are prepared and compacted as per standard procedure by placing the geosynthetics in different layers of the mould as shown in the Table Error**! No text of specified style in document.**-3. During the preparation of CBR mould with geosynthetics, a geosynthetic sample of the same diameter of the mould was cut and placed over the each compacted layer from the bottom. Details and a schematic view of the CBR mould preparation and layers are given below.

Table **Error! No text of specified style in document.**-3 Arrangement of layers in the mould

S. No	Layers	Height (%)
	1st Layer	0.2H
	2nd layer	0.4H
	3rd layer	0.6H
	4th layer	0.8H

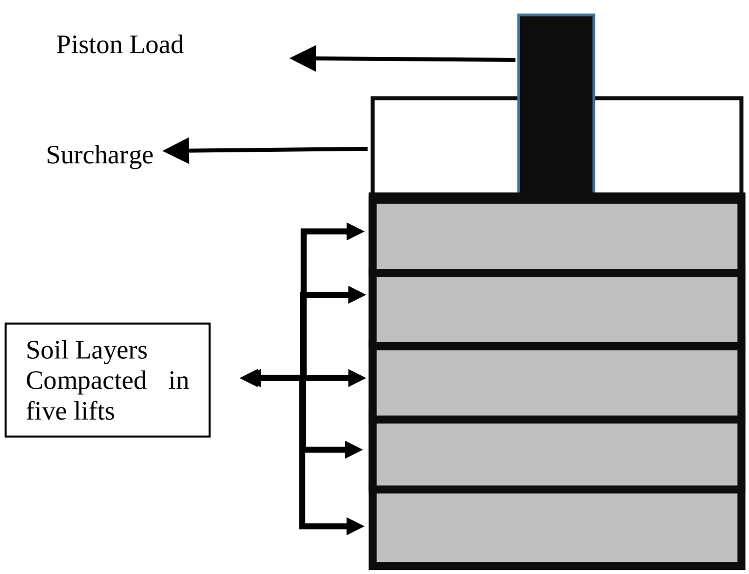

Figure Error! No text of specified style in document.**-2 Sample Compacted in five lifts**

Figure **Error! No text of specified style in document.**-3 Placing geo synthetics sample in the CBR mould.

The compacted CBR mould at its MDD & OMC than was placed in available manual CBR loading machine equipped with a movable base that travelled at a uniform rate of 1.27 mm/min and a calibrated load indicating device was used to force the penetration piston with a diameter of 50 mm into the specimen. The loads were carefully recorded as a function of penetration at 0.025in, 0.05in, 0.075in, 0.1 in, 0.125in, 0.15in, 0.175in, 0.2in, 0.3in, 0.4in and 0.5in to observe the post-failure behaviour as per standard procedure.

Results and discussions

The clayey soil without any reinforcement was compacted to its MDD and OMC as shown in Figure **Error! No text of specified style in document.**-4 in CBR moulds in order to achieve its mean CBR values. The CBR values of four controlled samples 8.67%, 8.3%, 8.3% and 9.05% respectively were calculated from a series of four CBR tests and their mean was further compared to the reinforced soil with four different types of geotextiles.

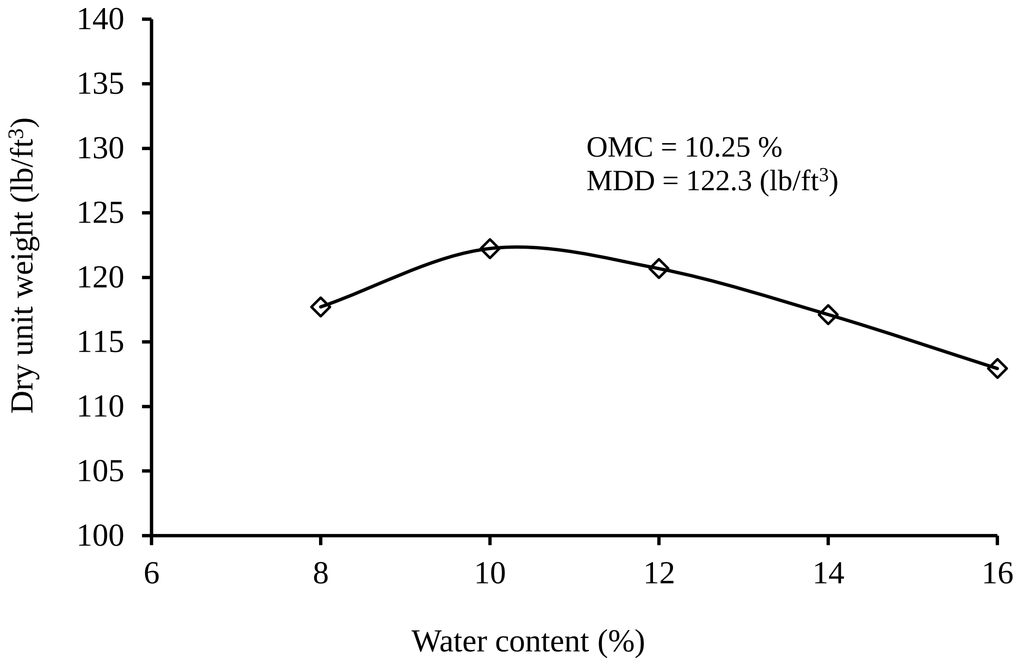

Figure **Error! No text of specified style in document.**-4 Compaction Curve for MDD and Water Content

4. Effect of geotextiles on CBR values of road subgrade clayey soil

As all four geosynthetics were used at different locations in the CBR specimens and resulted in different CBR values. Geosynthetics have performed and improved the CBR values and the greater increment in CBR values were investigated at 0.2H depths and the effect decreased to the bottom of the specimen. The improved CBR values and their improvement ratios are given in the Table Error**! No text of specified style in document.**-4 and

Table **Error! No text of specified style in document.**-5. Typar SF37 and Fiber Glass Geogrid composite Geotextiles have shown better performance than others two types of geosynthetics in terms of CBR values and load bearing capacity (stresses). Their performances and stress-penetration curves are shown in the

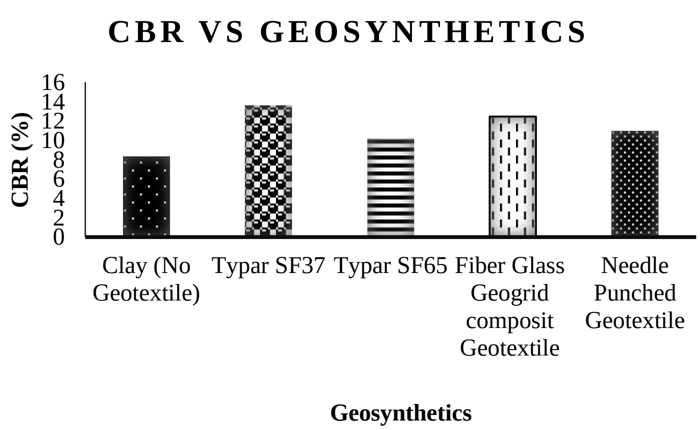

Figure **Error! No text of specified style in document.**-5 and
Figure **Error! No text of specified style in document.**-6.

Table **Error! No text of specified style in document.**-4 Improved CBR values

S/No	Geosynthetics	CBR (%)
1	Clay (No Geotextile)	8.3
2	Typar SF37	13.58
3	Typar SF56	10.18
4	Fiber Glass Geogrid composite Geotextile	12.45
5	Needle Punched Geotextile	10.94

Table **Error! No text of specified style in document.**-5 Improvement ratios of CBR values

S. No	Geosynthetics	Improvement ratio
	Typar SF37 Geotextile	1.6
	Typar SF56 Geotextile	1.2
	Fiber Glass Geogrid composite Geotextile	1.5
	Needle Punched Geotextile	1.3

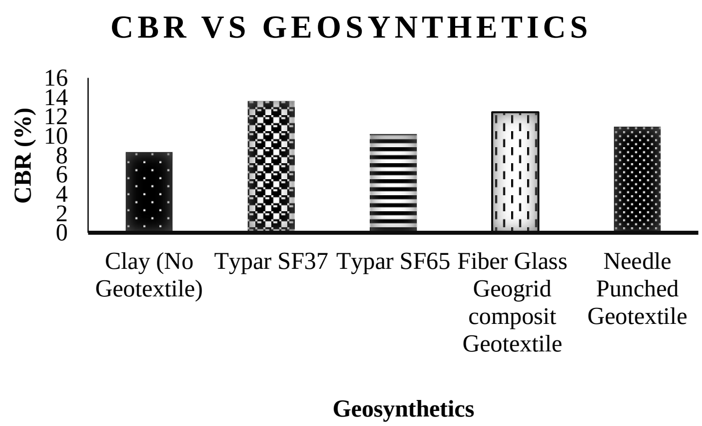

Figure **Error! No text of specified style in document.**-5 Performance of geosynthetics in terms of CBR values.

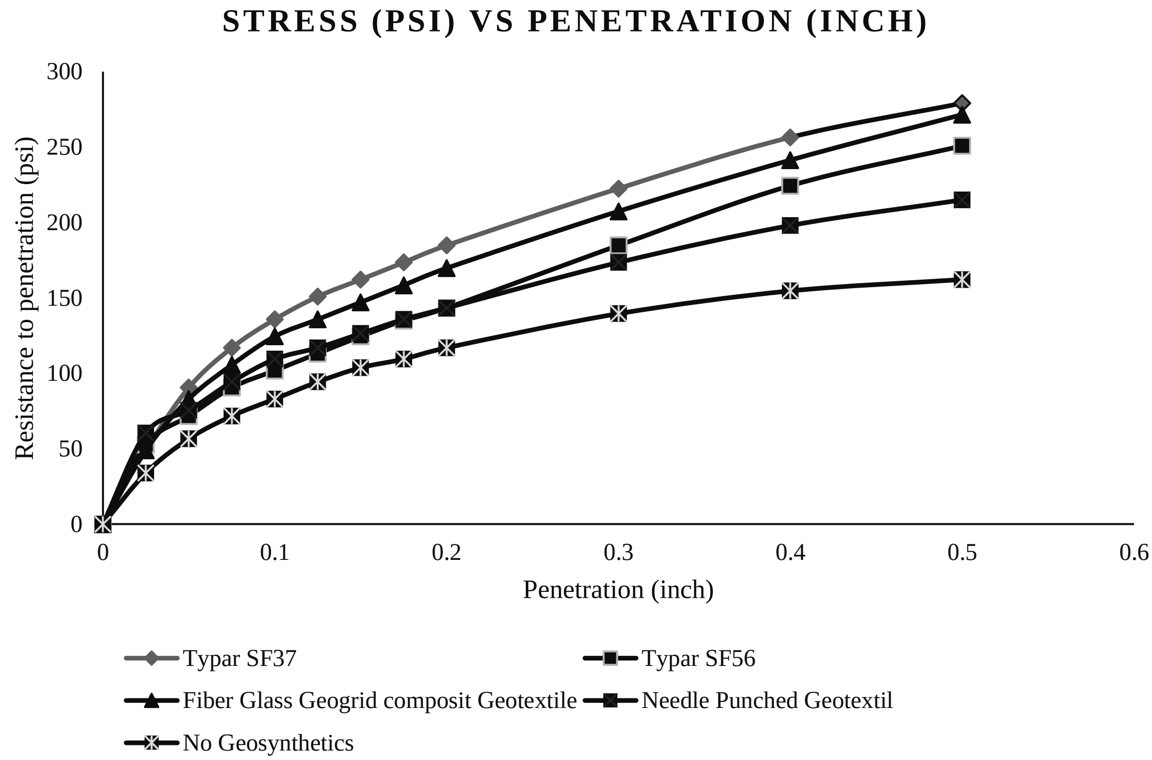

Figure **Error! No text of specified style in document.**-6 Performance curve of geosynthetics in terms of stresses & penetrations

5. Conclusions

From experimental data, the following conclusions were drawn:

It's concluded that the geotextiles samples may be placed at 0.2H from the top of the specimens in CBR moulds in order to improve the CBR values.

It is concluded that the CBR values of weak subgrade soil can be improved 1.6, 1.2, 1.3 and 1.5 times when the weak subgrade soil is reinforced with Typar SF37, Typar SF56, Needle Punched and Fiber glass geogrid composite geotextiles respectively.

It was concluded that it needs proper precautions to apply the geosynthetics in the soil in the shape of their accurate location, smoothness and compacted surface so that proper compaction may be done on the geosynthetics layer in order to achieve its MDD.

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Experimental study on the Effect of Permeation Grouting on Bearing Capacity of Sandy Soils

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ABSTRACT

The soil profile in coastal areas often consists of very loose sandy soils extending to a depth of 3 to 4 m from natural ground level, underlain by clayey soils of medium consistency. The very low shearing resistance of the foundation bed usually causes local as well as punching shear failures. Hence structures built on these soils can have settlement problems.

Granular soils can be improved effectively by variety of ground improvement techniques such as vibro-flotation, compaction piles, densification with explosives, weak soil replacement, drainage by well point system, reinforced earth and grouting etc. Ground improvement techniques are often used to target important engineering properties of sub soil like: bearing capacity, shear strength and permeability etc. Permeation grouting is an effective and state to the art technique of improving important engineering properties of granular soils. In this paper permeation grouting is evaluated as one of the possible mechanism for improvement of bearing capacity and increasing relative density of loose sandy shallow depth coastal soils.

In this research permeation grouting phenomenon was evaluated in laboratory by injecting grouts of different viscosities (different moisture contents) in a loose sand bed while a series of load tests were carried out to determine the bearing capacity of improved soil.
Results of comprehensive investigations carried out during this research clearly shows that permeation grouting can be used as an effective technique for improving the shear strength and relative density of loose sandy coastal soils which consequently reduces excessive settlements in these soils.

Keywords

Ground Improvement, Permeation Grouting, Vibro-flotation, Bearing Capacity, Granular Soils

1. INTRODUCTION

Ground improvement refers to any technique undertaken to increase shear strength, bearing capacity and decrease permeability and compressibility of in-situ weak soil, hence makes the soil suitable for the intended purpose. A large number of methods have been developed for ground improvement over the years.

Excavating the deficient soil and replacing it with improved soil having desired properties is normally economical only when soil has to be treated down to a depth of 3 m and the water table is below 3 m. If the water table is high, lowering of water table prior to excavation has to be carried out by dewatering techniques, which are expensive. *T. G. Santhosh Kumar et al. {2011)* Permeation grouting technique is generally used to reduce ground permeability and control ground water flow, but it can also be used to strengthen and stiffen the ground. *Suat Akbulut et al. (*2002) reported that the soil particle size and the cement maximum particle size have important effects on the successful grouting. *Purbi Sen et al. (2011)* concluded that, it is desirable to use a mixture of lime and cement or pretreatment of soil with lime before use of cement or use of lime only if possible. *Costas. A. Anagnostopoulos, (2005)* stated that the combination of such material with cement in grouting purpose with respect to water cement ratio contributes significantly to the improvement of grouted soil. Grouting technique finds applications in case of seepage control in rock and soil under dams, advancing tunnels, cut off walls etc. *(Nonveiller, 1989). P. Dayakar et al.* have concluded that, permeation grouting is an effective way to inject the grout into the ground without disturbing the soil structure. The increase in cement content increases load carrying capacity of the sandy soil.

The degree of improvement in loose sand by grouting will depend on the binding ability of the grout, the grout sand adhesion (bonding), and the properties of the sand. The physical or chemical interaction, or both, of two materials at their interface is known as adhesion or bonding. The strength and type of this bond plays an important, though poorly understood, role in the mechanical behavior of chemically grouted geo-materials. *Ata.A, and Vipulanandan.C, (1998)*. Cement grouting by impregnation in granular media is a widely used technique in civil engineering, applied in order to improve the mechanical characteristics of soils. The idea consists in incorporating a pressurized cement grout in the pore space of the soil. The setting of cement grout in the pore space increases both the strength and stiffness. Grouting is mainly responsible for the gain in cohesion by the material and only marginally affects the friction angle. The cohesion linearly varies with cement content, the magnitude of the cohesion gained by grouting and also the friction angle is a slightly increasing function of cement content. The increase in angle of friction is negligible with respect to cohesion. *(Maalej.Y, Dormieux.L, Canou.J, and Dupla.J.C; (2007))*. The Mohr-Coulomb cohesion varies between 0.1 and 0.5 MPa, depending on the cement content of the grout, the relative density of the soil and increases in proportion with cement to water ratio *Dano,C., Hicher,P.Y., and Taillez,S.(2004)*.

Addition of a cementing agent into sand imparts two components of strength, one due to the cement itself and the other due to friction. The friction angle of cemented sand is similar to that of uncemented sands *Clough,G.W., Sitar,N., et al. (1981)*. In the process of cement grouting, cement is used to fill the voids of soil mass and to render it impervious to percolating water and improve the strength and elastic properties of soil. The strength of soil increases with increase in cohesive strength and angle of internal friction arising from the bonding between soil grains and hydrated cements. Unconfined compressive strength of micro fine slag cement grouts increases with increase in curing time from 7 to 60 days and decreases in water cement ratio from 2 to 0.8 *Sinroja,J.M., Joshi,N.H., and Shroff,A.V.(2006)*. The weakly cemented sand shows a brittle failure mode at low confining pressures with a transition to ductile failure at higher confining pressures. The shear strength parameters - cohesion and angle of internal friction increase when grouted with cement. The water cement ratios have much influence in the control of strength gain of sandy soils *Glory.J., Abraham,B.M., Jose,B.T., and Paul,B.(2001)*.

2. MATERIALS

The materials used in this research were sandy soil and cement grouts, the soil was collected from "Sar-daryaab, Charsadda", along the bank of Kabul river in KPK Pakistan. Ordinary Portland cement (PS 232-2008) was used to prepare the grout. The gradation curve of soil is illustrated in figure 1.

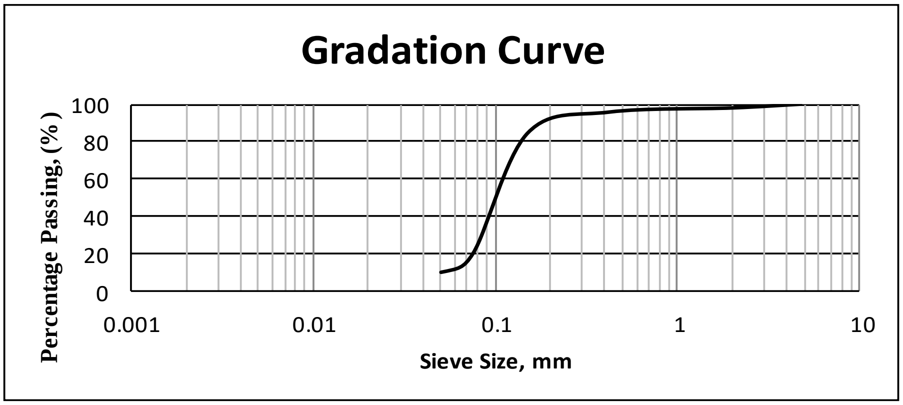

Figure 1: Gradation Curve of Sandy Soil

The physical characteristics of soil and cement are summarized in table 1 and table 2 respectively.

Table 1: Physical characteristics of Sandy Soil

Moisture content (%)	Specific Gravity	D_{60} (mm)	D_{30} (mm)	D_{10} (mm)	C_c	C_u	Classification
23.17	2.62	0.10	0.080	0.065	0.98	1.53	SP

Table 2: Properties of Cement

Grade	Specific Gravity	Standard Consistency (%)	Fineness (%)	Initial Setting time (min)	Final Setting Time (min)
43	3.13	28	91	38	112

3.TEST SAMPLES PREPARATION

The test samples were prepared in a specially fabricated cubical transparent tank (25cm x25cm x25cm) with poly-carbide sheet. The sandy soil was filled in tank in loose state with unit weight of 16.78 KN/m^3. Figure 2 shows the tank and pipe setup for grouting process. Four PVC Pipes of 25 mm diameter were used for grouting. The bottom of PVC pipe was plugged so as to disperse the grout circumferentially and make the grouting more effective.

(a) (b)

Figure 2: Test Specimen

After filling the sandy soil into tank and installation of PVC pipes in the soil, the grout was poured into the soil through the pipes. The viscosity of the cement grout is determined with Marsh funnel viscosity test (ASTM D691004). The grouted sample was kept under moist condition for a curing period of 3 days, 7 days and 28 days as displayed in figure 3(a). The notations used for the mix proportion is given in table 3.

Table 3: Mix proportions of Cement Grout and their notations

Notation	Water content	Cement content
G-1	7	1
G-2	6	1
G-3	5	1
G-4	4	1

4. TEST SETUP

The test setup consists of a loading frame, base for placing the soil tank and loading platen as shown in fig. 3b. The grouted soil in acrylic tank is placed in the loading setup and the axial load is applied to the sample through the center of the plate, via the load cell and the loading frame. The axial displacements are measured by the Linear Variable Displacement Transformer (LVDT) which is mounted on the tank. It was ensured that the load should be applied centrally on the surface of test specimen.

(a)

(b)

Figure 3: (a) Samples during curing, (b) Sample in compression testing machine

5.RESULTS AND DISCUSSIONS

The effect of permeation grouting on sandy soils was investigated in this research by using load tests applied on untreated and treated samples in compression testing machine. Fig 4 illustrates the load vs settlement curve of untreated sample. The ultimate load carrying capacity of sample without grouting is 1.6 kN. Fig. 5 shows the deformation against the applied load after treatment with cement grouting after 3 days curing. It was observed that 13 % load carrying capacity of treated samples increased after 3 days of curing, while significant reduction was noted in settlement. The treated sample with G4 reduced the settlement by 53% and G1 reduced 16% settlement after 3 days of curing.

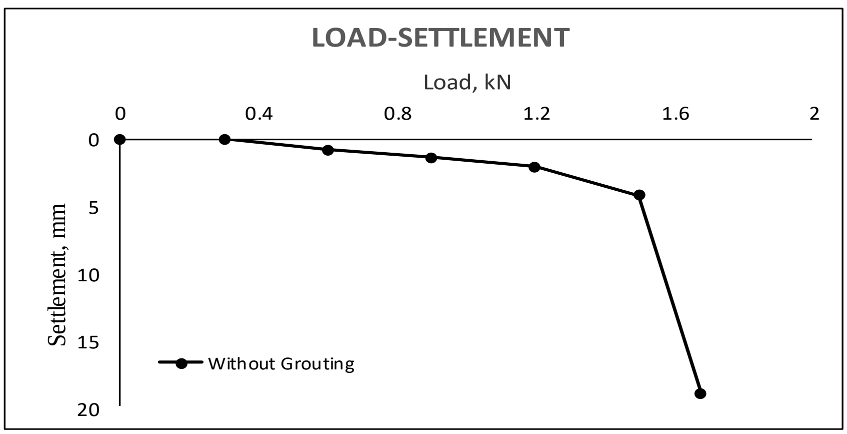

Figure 4: Load vs Settlement curve – untreated Sample

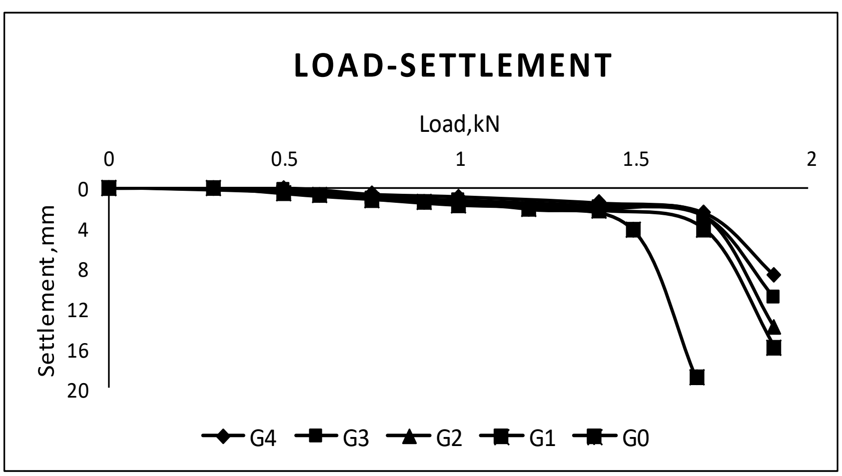

Figure 5: Load vs Settlement curve – Treated Sample after 3 Days Curing

Fig 6 illustrates the relationship of deformation against the applied load after 7 days curing period. It was observed that, with an increase in curing period the grouted sand achieved more strength as compared to 3 days curing, hence the ultimate load carrying capacity of composite samples increased. The ultimate load carrying capacity increased by 42% whereas, 85% reduction in settlement was noted after 7 days curing in G4, while settlement reduced 78% in test sample treated with G1. This increment in load carrying capacity is due to higher cement contents and early strength of grout.

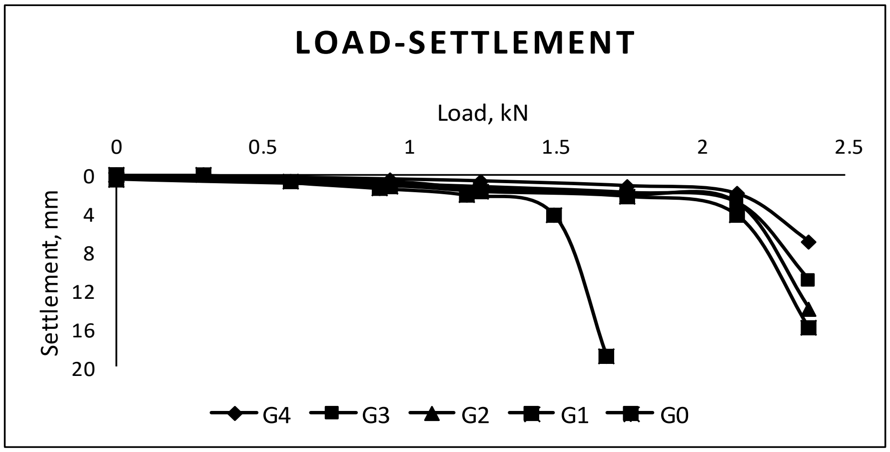

Figure 6: Load vs Settlement curve – Treated Sample after 7 Days Curing

Similarly, Fig 7 depicts comparison of the load deformation curves of untreated and treated samples with different grout ratios with 28 days curing period. The ultimate load taken by the composite grout is 2.66 kN. The 59 % load carrying capacities of composite samples were observed after 28 days whereas, 93% and 88% reduction in settlement was observed in G4 and G1 respectively.

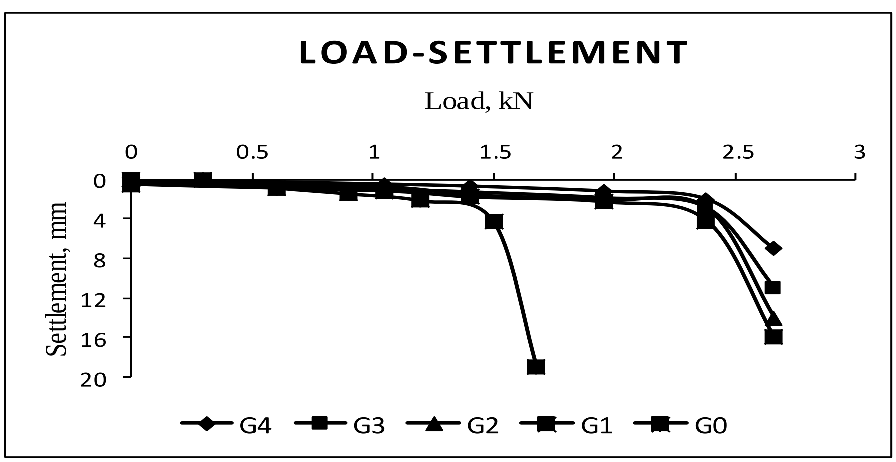

Figure 7: Load vs Settlement curve – Treated Sample after 28 Days Curing

6. CONCLUSIONS

Due to rapid technological advancements over the years, planners and designers can now take liberties of targeting even the most vulnerable soil deposits like loose coastal sands and make it useful. Permeation grouting is one of the modern and effective technique of improving important engineering properties of loose sandy soils. This study was carried out to study the effectiveness of permeation grouting in loose sandy soils. The efficiency of grouting mainly depends upon the penetration of cement grout through the pores of loose sand. The following conclusions were made on the bases of load tests.

1) The shear strength of loose sandy soil significantly enhanced with increasing the cement contents. The rate of strength improvement is very with higher percentage of cement.
2) The efficiency of cement grout is more in initial days due to early stage reactions of cement with water which gave rapid strength gain in initial days. With increasing curing period, the rate of strength attainment is comparatively low.
3) The influence of water content on the cement grout is very significant. As the grout ratio decreases the ultimate load carrying capacity increases and subsequent settlement decreases.
4) It was also concluded that, the effectiveness of permeation grouting increases with an increase in curing period of the treated samples, 7 days cured samples shows more improvement then 3 days cured samples.
5) It is concluded that G4 is more effective as compared to G1. Furthermore, it was concluded that, sample treated with G4 reduced settlement by 53%, 85% and 93% after 3 days, 7 days and 28 days respectively. While 16%, 78% and 88% settlement reduction was observed after 3 days, 7 days and 28 days curing in treated sample with G1 respectively.

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Estimation of Shear Strength parameters of A-4 soil by Dynamic Cone Penetrometer (DCP)

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Abstract

This research paper aims to develop the correlations between DCP and Shear Strength parameters (c and phi) of commonly used A-4 soil. Shear Strength of soil is one of the most important characteristic of soil. One must understand the nature of shearing resistence in order to analyze soil stability issues. The problems associated with conventional Shear testing is that it is time consuming, laborious, expensive and has low repeatability. The dynamic cone Penetrometer (DCP) is the most versatile, fast, inexpensive, portable and lightweight in-situ device, currently available for evaluation of sub-grade soil strength. Therefore, an appreciable alternative device, sush as DCP was used to predict the Shear Strength parameters of A-4 soil. In the present study, several field and laboratory tests were performed on soil samples, which have been collected from different locations. From the tests, the Atterberg limits (PI, LL, and PL), Natural Moisture Content, Gradation Analysis, Maximum Dry Density (MDD) and Optimum Moisture Content (OMC), Cohesion, angle of internal friction and Dynamic Cone Penetration Index (DCPI) values were acquired. Based on these results, correlations have been proposed in the study to predict c and phi values for A-4 soil from DCPT Results.

Keywords
Dynamic Cone Penetrometer (DCP), Cohesion (c), Angle of internal friction (ϕ), A-4 soil, AASHTO Soil classification, SPSS.

1. Introduction

Soil is naturally occurring un-cemented or un-aggregated deposits of minerals and fragments, covering large portion of earth's crust. It includes boulders, gravels, sand, clay, silt organic matter etc depending on the size of grains. As the soils are the naturally occirring materials, thus these are the highly variable, complex and possessing different engineering properties. The properties of soil vary from point to point, thus sampling of representative soil for laboratory and field testing is very difficult as well as evaluation of their geotechnical properties is cumbersome.

Dynamic Cone Penetrometer (DCP) is one the instruments which is recently getting popularity to evaluate the complex geotechnical properties like compaction, CBR and Shear Strength indirectly. DCP is basically instrument used to measure the resistivity of the soil. it is used to determine the soil properties without any excavation and is the one of the least expensive and easily transportable tools that can be used to charecterized soil properties.

DCP, is also getting used in road projects like on-site measurement of the road material thickness. The DCP test provides a measure of the penetration resistance by the material and that can be utilized to indirectly estimate the strength properties of materials as mentioned earlier. Since Shear Strength parameters (c-phi) cannot be determined on site and we need sample of that soil to determine the required paramters in laboratory. On the contrary, once we have correlation between DCP and shear strength parameters(c-phi) we can easliy determine the values of c-phi using that correlation on site. To obtain correlations between DCP and Shear strength parameters this reseach was conducted.

2. Experimental Work

Firstly, the soils clay and sand were collected from different locations of near jamshoro, From the obtained soil samples of clay and sand, Soil samples were classified as per AASHTO soil classification system, after performing the L.L, P.L and gradation tests. The groups of clayey soil and sand were obtained as A-7-6 and A-3. After obtaining the groups, soil samples were mixed with trial basis to get the required group of soil, i.e A-4. Only six soil samples of A-4 soil were selected for further soil tests. These samples were tested at Geotechnical Engineering laboratory in Civil Engineering Department of MUET for liquid limit, plastic limit, plasticity index, gradation, cohesion, angle of internal friction, DCPI, maximum dry density and optimum moisture content, The modified proctor compaction test was used to find MDD and OMC. Each Shear Strength test mould for direct shear box test was compacted in five layers of soil at predetermined OMC. The compaction effort used to prepare moulds for direct shear box test was generated by 25 blows per layer. The diameter of mould was approximately 4 inches.

DCP test was performed on each water content percentage of modified compaction test as well as OMC. The required depth of penetration was 100mm. From results, Dynamic Cone Penetration Index (DCPI) values were worked out at MDD and OMC.

All the tests were performed according to AASHTO and ASTM specifications. Table.1 shows the summary of laboratory test results of soil samples taken for present study.

Table 1: Summary of Test Results

Sample No.	LL (%)	PL (%)	PI (%)	% Finer (#200 passing)	MDD (gm/cm^3)	OMC (%)	DCPI (mm/blow)	Cohesion (Kg/cm^2)	Phi	AASHTO Soil Group
1	27	18	9	50.97	2.02	11.31	13.60	0.82	42.2	A-4
2	26	17	9	48.67	1.99	11.34	13.90	0.79	42.5	A-4
3	25	17	8	46.38	1.97	11.33	13.80	0.77	42.6	A-4
4	25	16	9	44.08	1.98	11.18	12.60	0.64	43.0	A-4
5	24	16	8	41.79	1.98	11.27	12.80	0.56	43.1	A-4
6	23	15	8	39.49	1.97	11.07	12.10	0.51	43.8	A-4

3. Results And Discussions

3.1 SPSS Regression Analysis

Accordingly, the results of six laboratory tests are used in regression analysis, to determine the effect of variables, the progressive linear regression was analyzed, and as a result, the correlation coefficient and the significance level were determined as shown in Table 2. (Pearson correlation coefficient)

Table 2: Pearson Correlation Analysis

	LL	Passing % (#200 Sieve)	PL	PI	MDD	OMC	C	Phi	DCPI
LL	1	.983	.944	.775	.832	.736	.924	-.952	.787
		.000	.005	.070	.040	.095	.009	.003	.063
	6	6	6	6	6	6	6	6	6
Passing %	.983	1	.968	.683	.771	.790	.973	-.966	.871
(#200	.000		.001	.135	.072	.062	.001	.002	.024
Sieve)	6	6	6	6	6	6	6	6	6
PL	.944	.968	1	.522	.764	.848	.945	-.980	.880
	.005	.001		.288	.077	.033	.004	.001	.021
	6	6	6	6	6	6	6	6	6
PI	.775	.683	.522	1	.683	.277	.575	-.582	.347
	.070	.135	.288		.135	.596	.232	.225	.500
	6	6	6	6	6	6	6	6	6
MDD	.832	.771	.764	.683	1	.435	.612	-.701	.435
	.040	.072	.077	.135		.388	.197	.121	.388
	6	6	6	6	6	6	6	6	6
OMC	.736	.790	.848	.277	.435	1	.822	-.895	.943
	.095	.062	.033	.596	.388		.045	.016	.005
	6	6	6	6	6	6	6	6	6
C	.924	.973	.945	.575	.612	.822	1	-.952	.932
	.009	.001	.004	.232	.197	.045		.003	.007
	6	6	6	6	6	6	6	6	6
Phi	-.952	-.966	-.980	-.582	-.701	-.895	-.952	1	-.906
	.003	.002	.001	.225	.121	.016	.003		.013
	6	6	6	6	6	6	6	6	6
DCPI	.787	.871	.880	.347	.435	.943	.932	-.906	1
	.063	.024	.021	.500	.388	.005	.007	.013	
	6	6	6	6	6	6	6	6	6

It can be observed that the relation of cohesion (C) suits best with DCPI and passing % (#200 sieve), having high reliability coefficient. Similarly the relation of angle of internal friction (phi) can be more reliable with DCPI and passing % (#200 sieve) (as observed form Table 02)

3.2 Single Linear Regression Analysis:

Correlation Between Cohesion (C) and DCPI

C = 0.165(DCPI) - 1.480,with R^2=0.868 (Linear Equation)

C = 2.146Ln(DCPI) - 4.841, with R^2=0.868 (Logarithmic Equation)

C = 2.812 – 27.908(1/DCPI), with R^2=0.868 (Inverse Equation)

C = 0.294(DCPI) – 0.005$(DCPI)^2$ – 2.321, with R^2= 0.868 (Quadratic Equation)

Correlation Between Angle of Internal Friction (Phi) and DCPI

Phi = 51.983 – 0.694(DCPI), with R^2=0.820 (Linear Equation)

Phi = 66.277 – 9.096Ln(DCPI), with R^2=0.829 (Logarithmic Equation)

Phi = 33.784 + 118.963(1/DCPI), with R^2=0.838 (Inverse Equation)

Phi = 140.553 – 14.303(DCPI) + 0.521$(DCPI)^2$, with R^2=0.910 (Quadratic Equation)

3.3 Multiple Linear Regression Analysis:

Correlation Between Cohesion (C), Passing % (#200 Sieve) and DCPI

C = 0.062(DCPI) + 0.020(Passing % #200 Sieve) – 1.045, with R^2=0.977

Correlation Between Angle of Internal Friction (Phi), Passing % (#200 Sieve) and DCPI

Phi = 49.917 – 0.205(DCPI) – 0.096(Passing % #200 Sieve), with R^2=0.95

4. Conclusions

The recommended best correlation developed to estimate shear strength parameters of A-4 soil-cohesion (C) and angle of internal friction (Phi) is through multiple linear regression analysis as given below

C = 0.062(DCPI) + 0.020(Passing % #200 Sieve) – 1.045, with R^2=0.977

And

Phi = 49.917 – 0.205(DCPI) – 0.096(Passing % #200 Sieve), with R^2=0.95

5. Recommendations

This study is purely on a single soil, i.e. A-4 Soil. It can be noted that the several researches are to be needed for further study on different soils.

This study is only based on AASHTO Soil Classification System, it can be further extended to other soil classifications such as Unified Soil Classification System (USCS).

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Stabilization of Jamshoro Soil by Marble Powder

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Abstract
Soil investigation results are of great importance before constructing any structure. Soil stabilization techniques are applied if the soil is not found in satisfactory condition in accordance with the structure constructed over it, like Jamshoro soil. Among such techniques, one of them is stabilization by marble powder. This paper expresses variation in the properties of natural soil samples and samples prepared through stabilization by marble powder. The soil sample is collected from USPCASW department in MUET Jamshoro and is brought to laboratory for performing basic tests including specific gravity, sieve analysis, Atterberg limits, direct shear box test, unconfined compression test, modified proctor test and California bearing Ratio to obtain its natural properties. After this, two different soil samples are prepared by adding marble power in specific amount of 10% and 20% respectively and same tests are performed over them. The results obtained are then compared with natural soil properties and inferences are drawn towards the usability and effectiveness of marble powder stabilized soil as replacement of natural soil for deep or raft foundations, from economical point of view.

Keywords
Jamshoro Soil, Soil Stabilization, Marble Powder, Soil Properties, Foundations

1. Introduction

Since soil is the important material for designing of structural foundations which bear load according to their capacity; it proves to be a fundamental part of engineering work which is utilized below the foundation. It is used in pavements, buildings, embankments, below bridge piers, railways foundations, earthen dams, and as a filler material for several purpose in engineering work. Thus, its properties like swelling, compaction, contraction, and bearing capacities play vital role during design of engineering works.

Soil is an engineering material whose properties vary from place to place. Unlike other materials, it has many variations in properties which need to be investigated properly for specific project location. This is because in case of general soil survey of a city we may be misled for specific project location. Considering the bearing capacity parameter of soil, it usually shows much variation along the vertical and horizontal profile of land. At some locations, it may have good capacity for less depth whereas, it may not have enough capacity at considerable depth for which it becomes necessary to take engineering improvement steps for its stabilization.

Soil stabilization may be defined as "The process of alteration of soil by mechanical or chemical method to improve its properties which may serve the desired engineering properties". Generally, soil is stabilized to improve its strength, durability, erosion, and dust formation by different methods to achieve required properties for design period of building as well as an economical design. For this purpose, in this research work, marble dust is utilized to improve soil properties by mechanical method which involves physical mixing of materials.

2. Problem Statement

Many Researchers have found a problem in Jamshoro soil that it has very irregular behavior of bearing capacity at various depths as well as along horizontal plane and also, it shows less bearing capacity for depth which is required for shallow foundations. Therefore, this problem is left with only one option for foundation which is deep foundation. Usually, pile foundation is adopted for high rise institutional buildings in Jamshoro. Since the problem is highlighted by researcher for bearing capacity at low depth, this concept is carried further and soil stabilization with marble dust is done to improve its bearing capacity for low depth profile so that shallow foundations may be adopted for construction which ultimately results in economical and safe design of Buildings.

3. Research Methodology

In this Research, soil samples were collected from USPCASW Department of MUET Jamshoro (Fig 1). The methodology employed for this research work was to divide the samples in two portions, one for soils at natural stage and second with marble dust. Initial Soil tests were performed on samples (Fig 3) after which, they were mixed mechanically (physical mixing) with marble dust (Fig 2). Marble dust was collected from marble market Hyderabad at the rate of 10% and 20 % by soil weight. After mixing with said proportion, tests were performed as shown in Fig 4. Finally, Comparison between soil at natural state and soil mixed with marble dust at rate of 10% and 20% was drawn for basic tests shown in fig 3 and 4.

Fig 1: Natural Soil Sample

Fig 2: Marble Dust Sample

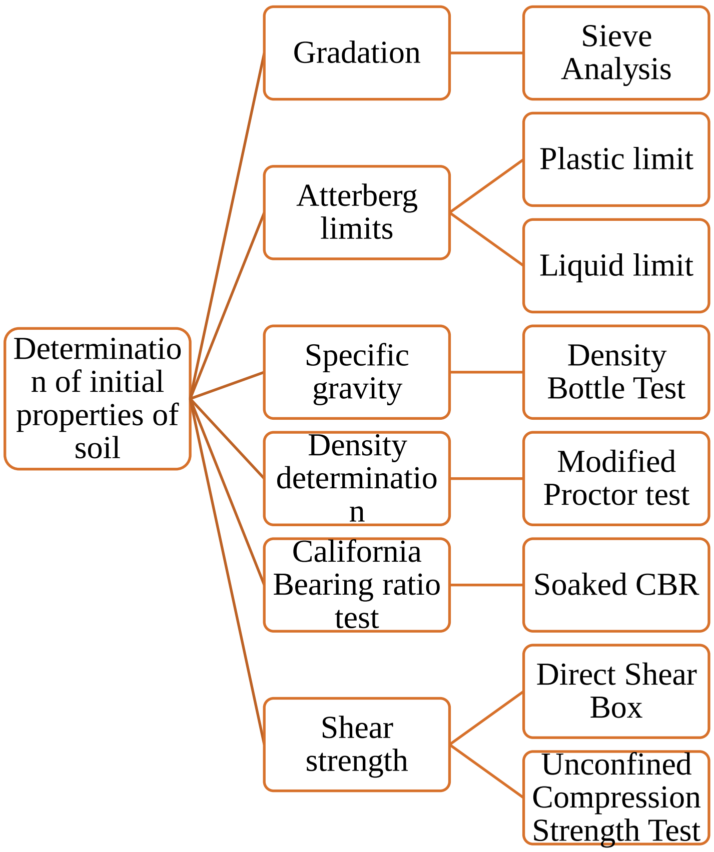

Fig 3: Initial Soil Tests (Soil in Natural State)

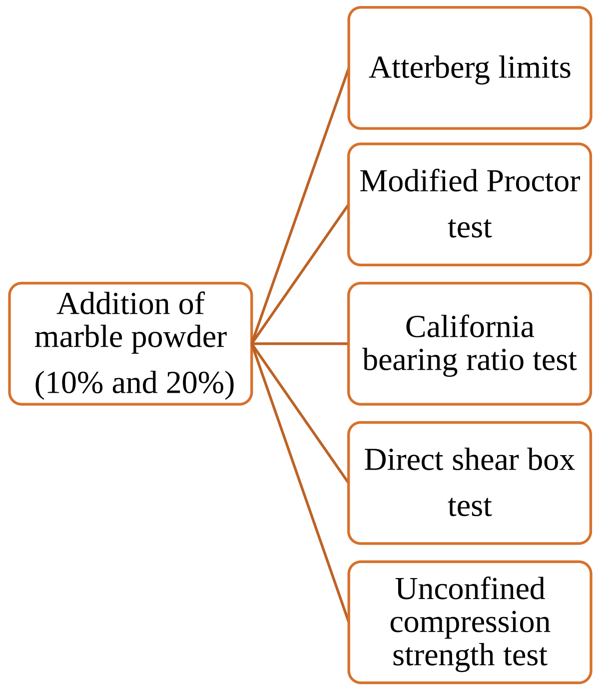

Fig 4: Soil Tests With Marble Dust

4. RESULTS AND DISCUSSION

Table 1 shows the results of different tests which were carried out on soil samples in natural state, with 10% marble addition and with 20% marble addition separately.

Table 1: Summary of Test Results

Test No	Test Name	Results		
		Natural Soil Sample	Soil Sample - 1 (10% Marble Powder Addition)	Soil Sample - 2 (20% Marble Powder Addition)
1	Liquid Limit (LL)	31%	29%	30%
2	Plastic Limit (PL)	20%	20%	20.35%

3	Plasticity Index (PI)	11%	9%	9.65%
4	Optimum Moisture Content (OMC)	11.30%	8.25%	10.44%
5	Maximum Dry Density (MDD)	1.94 gm/cm3	2.00 gm/cm3	1.962 gm/cm3
6	CBR @ 95% Compaction	10.50%	15%	21%
7	CBR @ Maximum Dry Density	15.80%	18.47%	28.42%
8	Cohesion (c)	1.2 kg/cm2	2.1 kg/cm2	1.7 kg/cm2
9	Angle of Internal Friction (ϕ)	34.99	36.25	38.65
10	Unconfined Compression Strength	2.229 kg/cm2	2.401 kg/cm2	2.337 kg/cm2

The main aim of this research work was to show the comparison of OMC, MDD, Cohesion, Angle of internal friction, CBR at 95% compaction and unconfined Compression strength of soil with and without marble powder addition. The graphical representation of these comparisons is discussed separately in coming sub-sections.

4.1 Comparison of Modified Proctor Test Values (OMC and MDD)

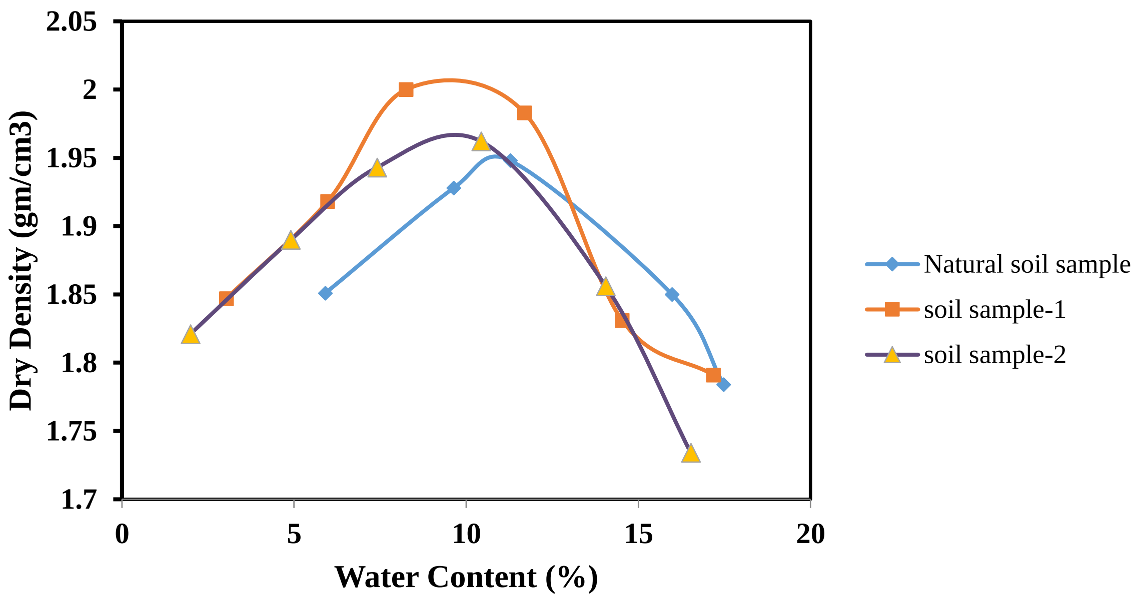

Fig 5: Comparison of OMC and MDD For Soil Sample With and Without Marble Powder Addition

Figure 5 shows that in case of natural soil sample, by performing modified proctor test, the values of OMC and MDD comes out to be 11.3% and 1.94 gm/cm^3 respectively. On addition of 10% marble powder, OMC decreases to 8.25% whereas the maximum dry density increases to 2 gm/cm^3. On the other hand, on 20% marble powder addition, OMC slightly decreases to 10.44% whereas maximum dry density slightly increases to 1.964 gm/cm^3.

4.2 Comparison of California bearing Ratio Value (CBR) @ Maximum Dry Density:

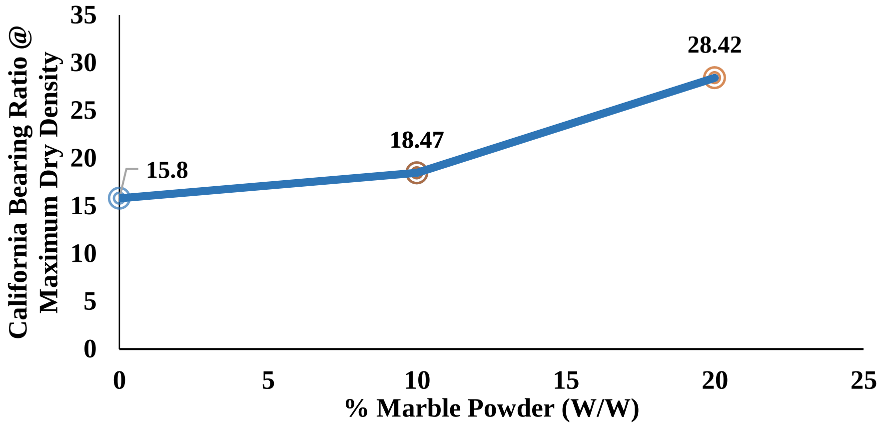

Fig 6: Comparison of CBR @ Maximum Dry Density For Soil Sample With and Without Marble Powder Addition

Figure 6 shows that while performing California Bearing Ratio test on natural soil sample, the CBR value @ maximum dry density comes out to be 15.8% which increases to 18.47% on 10% marble powder addition. On further addition up to 20%, its value increases to 28.42%.

4.3 Comparison of Direct Shear Box Test Values (Cohesion and Angle of Internal Friction):

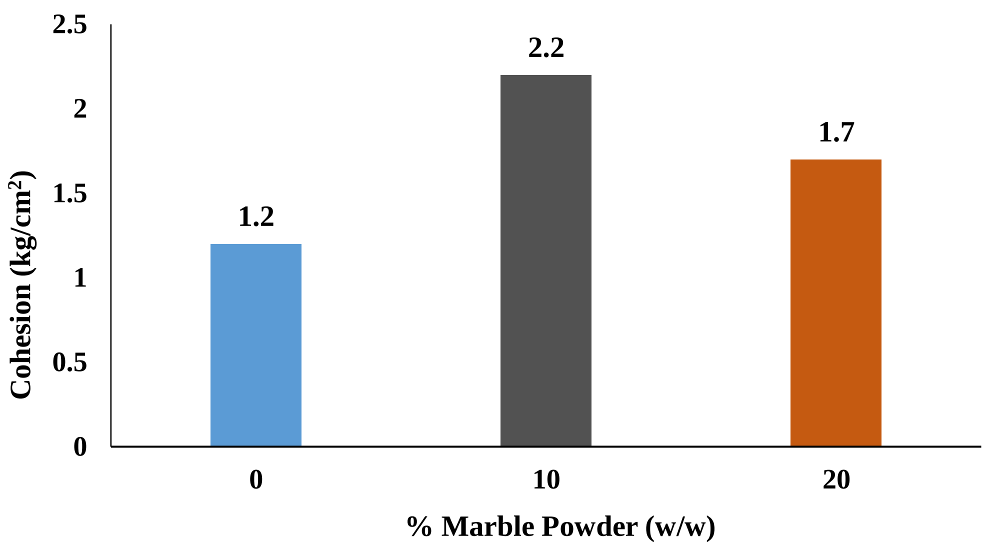

Fig 7: Comparison of Cohesion Value (c) For Soil Sample With and Without Marble Powder Addition

Figure 7 shows that while performing direct shear box test on natural soil sample, the value of cohesion (c) comes out to be 1.2 kg/ cm^2. On addition of 10% marble powder, C value increases to 2.2 kg/cm^2, however, on 20% marble powder addition the C value increases to 1.7 kg/cm

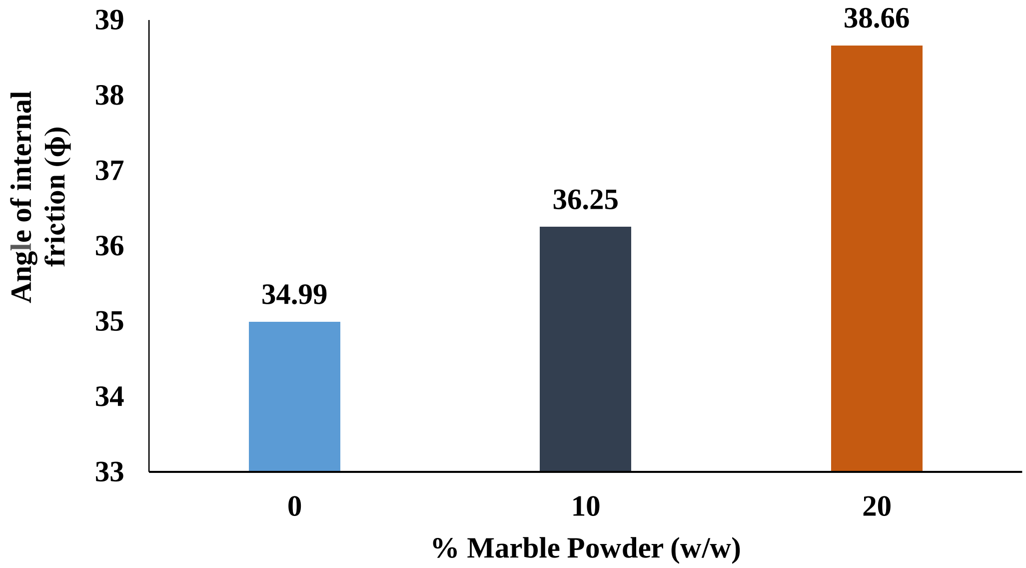

Fig 8: Comparison of Internal Friction Angle Value (ϕ) For Soil Sample With and Without Marble Powder Addition

Figure 8 shows that while performing direct shear box test on natural soil sample, the value of internal friction angle (ϕ) comes out to be 34.99°. On addition of 10% marble powder, ϕ value increases to 36.25°. Moreover, by increasing marble powder addition up to 20% friction between the particles further increases as ϕ value increase to 38.66°.

4.4 Comparison of Unconfined Compression Strength (UCS) Value:

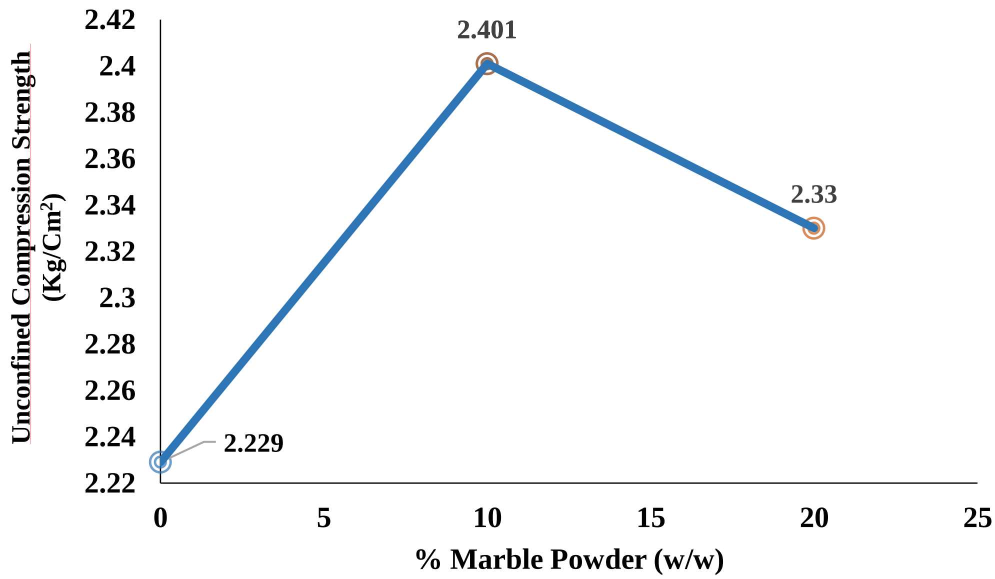

Fig 9: Comparison of Unconfined Compression Strength For Soil Sample With and Without Marble Powder Addition

Figure 9 shows that while performing unconfined compression strength test on natural soil sample, the value of cohesion (Cu) comes out to be 2.229 kg/ cm^2. On addition of 10% marble powder, Cu value increases to 2.401 kg/cm^2, however, on 20% marble powder addition, the Cu value slightly increases to 2.33 kg/cm^2.

4.5 Comparison of California bearing Ratio Value (CBR) @ 95% Compaction:

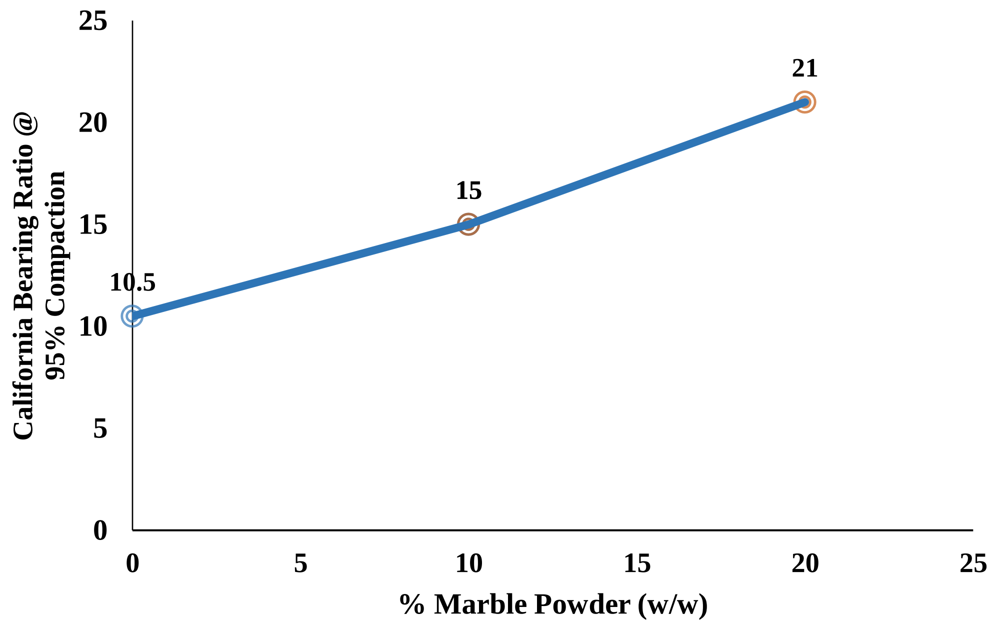

Fig 10: Comparison of CBR Value @ 95% Compaction For Soil Sample With and Without Marble Powder Addition

Figure 10 shows that while performing California Bearing Ratio test on natural soil sample, the CBR value @ 95% compaction comes out to be 10.5% which increases to 15% on 10% marble powder addition. On further addition up to 20%, CBR value further increases to 21%. This means that by increase in marble powder addition, the strength of soil to be used as sub-grade below pavements further increases.

5. CONCLUSIONS

From the results of this research work, following conclusion are drawn:
Liquid limit of the natural soil is decreased by some percentage with the addition of the marble in it.
If the percentage of marble added is more than 10%, little variation occurs in the plastic limit.
The value of Plasticity Index decreases with the marble addition.
Strength of soil varies in negative and positive direction with the addition of marble powder.
By addition of marble content, optimum moisture content decreases whereas the maximum dry density increases. By 10% marble content addition, optimum moisture content decreases up to 26.99% whereas increase in maximum dry density is about 3.09%. On the other hand, on 20% marble content addition, there is 7.61% decrease in OMC and 1.13% in maximum dry density.
CBR value at @ maximum dry density increases with addition of marble content. On 10% marble powder addition the CBR content increases from 15.8% to 18.47%. Thus, the net percent increase is

16.90%. On the other hand, on 20% addition of marble content, it increases from 15.8 to 28.42% resulting in about 79.87% increase in CBR value of soil.

The values of Cohesion (c) increases by addition of marble powder. On 10% marble powder addition, 83.33% increase in cohesion value is observed whereas on 20% addition, increment in cohesion value is about 41.67%.

The value of internal friction angle (ϕ) also increase with addition of marble powder. On 10% marble powder addition, 3.60% increase in ϕ value is observed whereas on 20% addition, increment in ϕ value is about 10.49%.

Unconfined Compressive Strength also increases by addition of the marble powder. On 10% marble powder addition, net increase is about 7.72% whereas on 20% addition, increase in strength is about 4.84%.

CBR value at @ 95 compaction increases with addition of marble content. On 10% marble powder addition the CBR content increases from 10.5% to 15%. Thus, the net percent increase is 42.86%. On the other hand, on 20% addition of marble content, it increases from 10.5 to 21% resulting in about 100% increase in CBR value of soil.

The strength of soil increases up to specific amount of marble powder required to fill the pores in the soil. As the pores filled, the soil gets densified and further addition will try to lose the strength of soil.

6. RECOMMENDATIONS

Due to time constraints, this study is limited to 2 types of marble powder addition that is 10% and 20%. Conduct more tests with small increments about 2 to 3% in order to determine the behavior of soil more accurately on marble powder addition.

More number of samples should be taken from different areas of Jamshoro so as to find out variation in soil parameters at different places of the district.

Conduct 2 to 3 trials for each test for determination of soil parameters more accurately.

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Improvement of Soil Properties of Baleli and Its Vicinity With Lime And Cement

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Abstract

This study was carried out with an intention to observe any sign of improvement of clayey soil due to addition of lime and cement. This paper focuses on the primary research of using Blends of Lime and cement on the soil of Baleli and its vicinity region to evaluate the features of Cement and Lime at the shear strength of unsaturated soil through carrying out direct shear test. Soil admixed along with natural or artificial admixtures is one of the techniques to improve its physical and mechanical characteristics. Soil residences vary decently and formation of structures can be encouraged a lot on the bearing capability of the soil, consequently we need to alter some features of soil that could make it expedient to calculate the burden bearing ability of the soil and even stabilize the weight bearing potential. Here, in this endeavor, soil stabilization has been accomplished with the assist of randomly allotted probabilities of lime and cement obtained from Baleli and its vicinity. The values obtained are in comparison with unblended soil samples at 5% admixed soil with cement and lime showed maximum angle of friction and moderate cohesion but shear strength 30.32 psi achieved at this percentage of lime and cement was greater than other percentages. Soil sample admixed with 7% lime and cement presented less cohesion than those blended with 3% and 5% lime and cement but greater shear strength as compared to all of the samples including simple soil. As a significance, its miles better to combine distinctive types of soils collectively to improve the soil vitality back ground.

Keywords

Stabilization, Lime, Cement, Unconfined compression test, Direct shear test

1. Introduction

For any land-based structure, the foundation may be very crucial and has to be strong to maintain the entire structure. So as for the foundation to be sturdy, the soil close by it plays a completely acute role. Soil stabilization is the way toward enhancing the execution of a soil as a development material by utilizing added substances with soil to enhance its volume stability, quality, penetrability and strength. Baleli and its vicinity soil is clayey silty soil. Therefore it possesses some plasticity, low shear strength, which are main reasons of failure in different civil engineering structures.

Therefore this soil cannot be used directly for the civil structures without stabilization, Raft footing construction or with using borrowed soil but borrowed soil costs much Hence, we need to stabilize the soil which makes it less complicated expecting the load bearing capability of the soil and even enhance the load bearing potential with help of Chemical additives (cement, fly ash, bitumen, cement clay dust, and lime). The major goal and objective of the studies described herein is to gain an expertise of the number one outcomes of lime and cement on the material of Baleli soil. Specifically this studies is restrained to a research of the impact of lime and cement on the strong point characteristics of Baleli soil. The material set forth in this report is fundamental in serving as a foundation for subsequent research in this field.

2. Review of literature

According to a study conducted by E.A. Basha R. et all (2004). The subsequent conclusions may be drawn on the idea of take a look at outcomes acquired from cement–RHA (Rice Husk Ash) stabilized soils. Cement and RHA reduce the plasticity of residual soil. The MDD of cement-stabilized residual soil slightly decreases with the growth in cement content material. Including RHA and cement, the OMC is extended steeply.

Organic soil poses significant challenges to civil engineering constructions because of its unsatisfactory strength characteristics. This study was done by N. Chowdhury, et all (2016) reports the finding of a laboratory investigation into the effect of cement and fly ash stabilization on the strength properties of organic soil of a selected location of Bangladesh. Cement was added at a percentage of 1.5, 3, 5 and fly ash was added at a percentage of 5, 10, and 20 of dry weight of organic soil and compacted to desired densities. Compacted stabilized specimens were cured for 7, 14 and 28 days. Unconfined compression tests were carried out on specimens prepared with the stabilized soil. The results illustrate that the unconfined compressive strength of organic soil was increased while cement was used as stabilizer. The increase in compressive strength of soil-fly ash used specimens was, however, found insignificant.

The study conducted by Amin Esmaeil Ramaji (2012) talked about lime treated rent types display diminished versatility, lessens volume change qualities and enhanced workability. It ought to be noticed that the properties of lime soil blend are more relies on the dirt sort; curing conditions and %of lime. Lime is white or grayish white, uneven, when water is sprinkled over that structure the slaked lime known as hydrated lime or slaked lime which is for the most part utilized as a part of mortars, concretes and as a dirt stabilizer.

A study B.Le Runigo, et all (2011) examined that the change was carried by blending the lime with the silty soil. When soil was treated with lime was then opened for long haul conditions (long haul water contact and water course. The point of this paper was to know the effect of water powered conditions on the mechanical shear quality of a lime treated silty soil and the impact of beginning conditions (lime substance and compaction conditions on lime settled toughness.

3. Methodology

Distinctive techniques of all of the laboratory tests were done to acquire the targets of this study. The soil samples were collected from specific locations at Baleli and its vicinity, and all the samples used in this study were examined inside the laboratory by means of American Society of Testing and Materials.
The experimental process consists of the subsequent steps:

- Natural moisture content
- Sieve Analysis
  Determination of Atterberg Limits
- Liquid limit by Casagrande's apparatus
- Plastic limit
- Determination of the maximum dry density (MDD) and the corresponding optimum moisture content (OMC) of the soil by Standard Proctor compaction test
- Preparation of admixture containing soil samples.
- Direct shear test (DST)

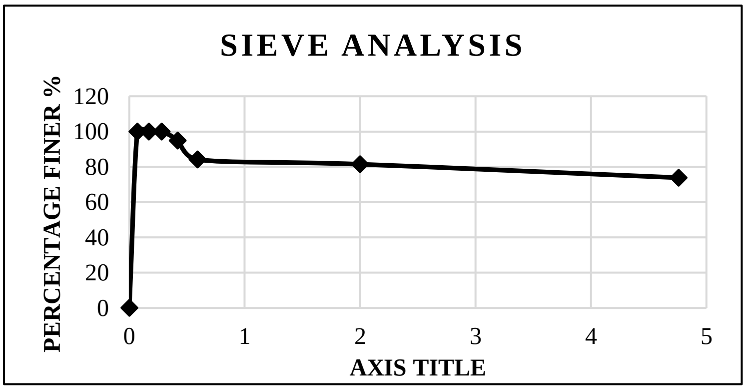
4. Result:

Table 1 Basic Geotechnical Properties

S.no	Determination No	
1	Moisture content (%)	**13.54**
2	Liquid Limit	**24**
3	Plastic Limit	**22.366**
4	Plasticity Index	**1.64**
5	Standard Proctor Test • MDD (kg/m^3) • OMC (%)	**118.215** **14.7 %**

Figure 1 Sieve Analysis

Figure 2 Direct Shear Test

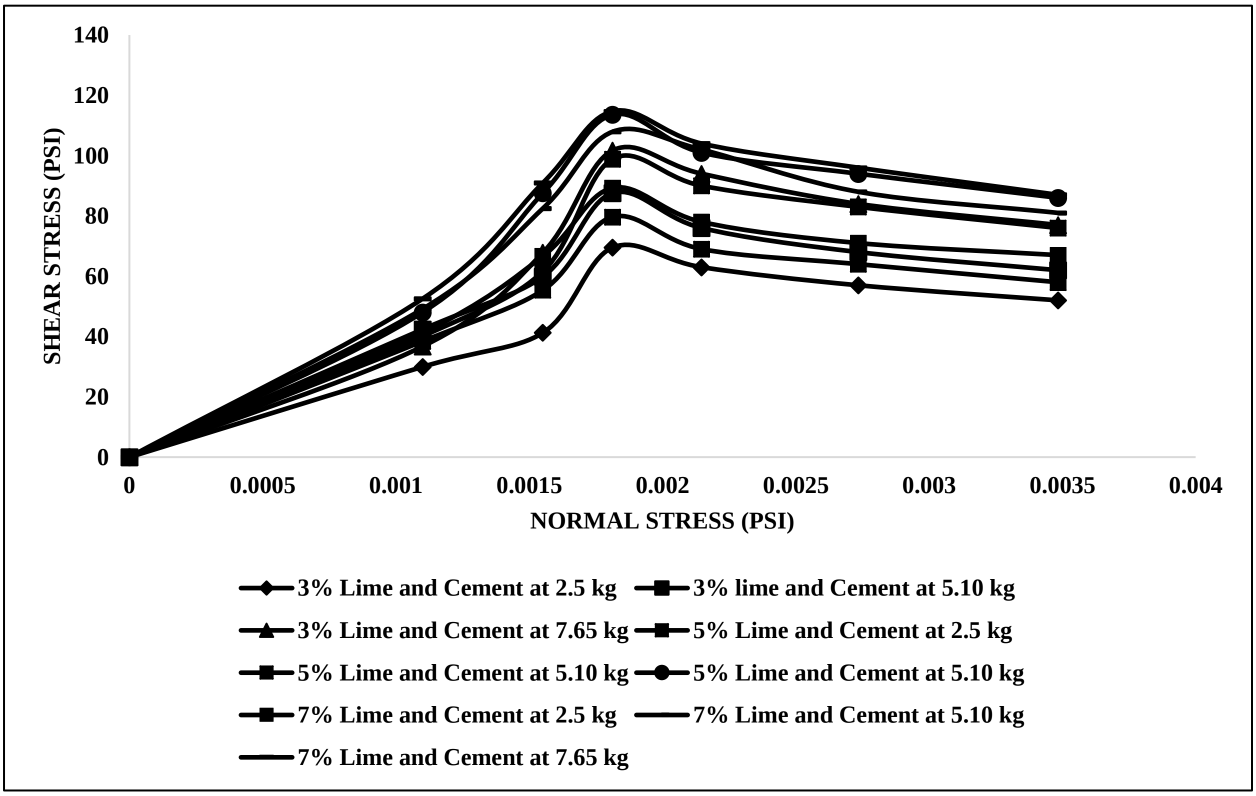


Graph 1 Sieve Analysis

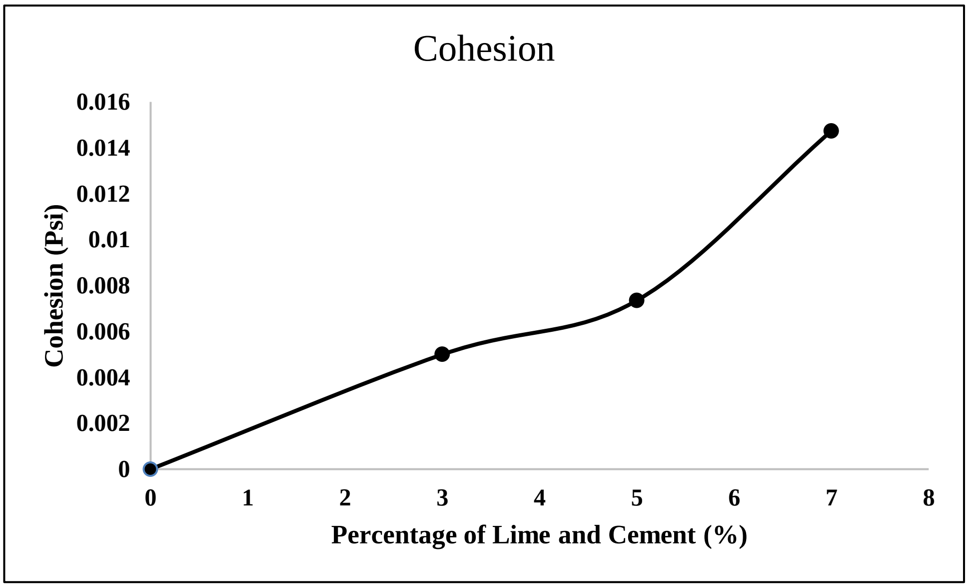
4.1 Direct Shear Test

4.1.1 Describing Cohesion, Angle of Internal Friction and Shear Strength

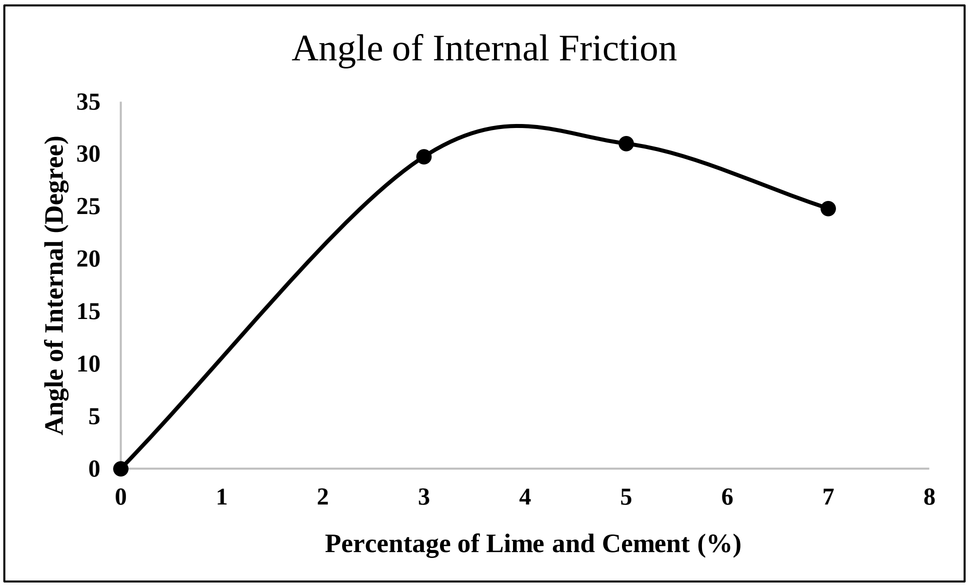
Direct shear box test results of blended soil with 2.5% lime and 2.5% cement shows greater amount of increment in shear strength and cohesion as compared to that of simple soil sample, 1.5% lime and 1.5% cement and 3.5% lime and 3.5% cement. The shear strength obtained from this percentage of lime and cement is the greatest according to this research.



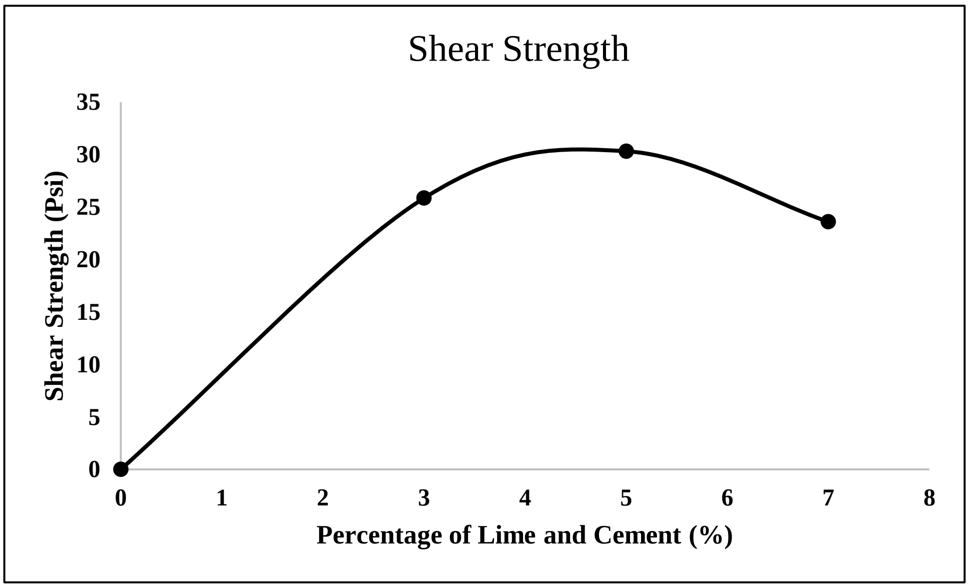
Graph 2 Direct Shear Box Test



Graph 3: Relationship of Lime and Cement with Cohesion



Graph 4: Relationship of Lime and Cement with Angle of Friction



Graph 5: Relationship of Lime and Cement with Shear Strength

5. Conclusion

Based on present study and the laboratory experimental work carried out in the research, the following conclusions are drawn.

- Outcome of 3.5% lime and cement showed greater cohesion than 2.5% admixed soil sample while less angle of friction and shear strength characteristics.
- 2.5% admixed soil with cement and lime showed maximum angle of friction and moderate cohesion but shear strength achieved at this percentage of lime and cement was greater than all of the above percentages

6. Recommendations

- Triaxial test for these blends of cement and lime is recommended as it is a more consistent and accurate shear measurement method

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The Effect of Corrosion on Bond Strength of Steel and Concrete

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Abstract
In this research the reduction in bond strength between concrete and steel due to corrosion has been studied. In the recent studies it is quite clear that the bond strength between concrete and steel is affected heavily due to corrosion. Reduction in bond strength causes the reduction in flexural strength and serviceability of reinforced concrete members. Corrosion is the main cause due to which this reduction in bond strength and deterioration in concrete members happens. Corrosion causes the volume of steel to increase while the effective area of steel decreases consequently reducing the bond strength. In the present research, efforts have been made to study the phenomena of this reduction in bond strength due to corrosion. In total 24 specimens were prepared and passed through an electro chemical process that caused artificial accelerated corrosion up to 7%. The bond strength was tested using the modified pull-out technique. An empirical equation is then proposed based on the experimental results.

Keywords
Bond Strength, Corrosion, electro chemical process, modified pull out technique

1. Introduction

Corrosion of reinforced concrete was first recognized early in the twentieth century. But it has become worse in recent years with the widespread use of de-icing salts. Usually concrete provides an ideal protective environment for the reinforcing steel. However when salts (chlorides or sulphates) penetrate the concrete and reach the steel rebars, corrosion normally commences. The corrosion products of the steel reinforcement will swell up to seven times its original size, developing pressures as high as 34.5 MPa (5000 psi) within the concrete, which cause cracking and spalling of the concrete cover and expose the rebar to further corrosion activity. Corrosion of reinforcing steel in concrete has caused catastrophic failures in some specific cases, resulting in injury and death, such as the collapse of the Berlin Congress Hall and of a parking garage in Minnesota [1].

Chloride-induced corrosion of reinforcing steel in concrete bridge decks, parking garage slabs and marine structures has been identified as the primary cause of concrete deterioration. The distress in concrete is caused basically by several interactive factors and characterized mainly by severe environment, unsuitable materials, inadequate construction practices and specifications in conjunction with structural weakness.

The repair and maintenance of reinforced concrete structures is becoming increasingly important and extensive. In order to increase the reliability of the structure and to reduce maintenance costs, eliminating or at worst impeding the corrosion problem is very important. Also, to design new concrete structures and to repair existing deteriorated concrete structures requires an understanding of the various causes and mechanisms of corrosion of reinforcing and prestressing steel [1][2][3][4].

The concrete cover acts as a physical barrier to the access of aggressive agents because of its hardness and resistance to wear and tear, and to permeation of fluids containing harmful compounds. The high alkalinity of concrete normally provides excellent protection to the reinforcing steel. Despite the "interest" in the protective qualities of concrete, corrosion of steel is the most common cause of distress in concrete structures. Use of deicing salts in cold climate countries aggravate this situation. Concrete structures subjected to sea water spray in the zone in marine structures and carbonation of concrete in industrial environments also lead to the depassivation of the protective oxide layer on the reinforcing steel. These distresses have also occurred from errors in concrete mixes, lack of quality control in mixing, placing, consolidating and curing of concrete resulting in permeable concrete. In addition, incorrect use of the different types of cements, supplementary cementitious materials, superplasticizers and other additives available commercially and used without full understanding of their properties have also resulted in deterioration of concrete structures because of steel corrosion. Corrosion of the reinforcing steel causes a decrease in the bar diameter which affects adversely the mechanical properties of the steel bar in terms of its ultimate strength, yield strength, ductility, etc. Furthermore, when reinforcement corrodes, the corrosion products occupy much larger volume than the original steel, and eventually exerts a large force on the concrete surrounding it to cause cracks which grow slowly as the reinforcement continues to corrode followed by spalling of the concrete cover. Also, corrosion of the reinforcing steel causes changes in the surface conditions of the reinforcement steel, and layer of the corrosion products causes loss of cohesion and adhesion at the steel-concrete interface. As corrosion continues, it finally leads to changes in the profile of the bar rib. Eventually, all of the concrete around the steel bar is forced off by the growing corrosion products, and the reinforcement loses not only any remaining protection against corrosion, but also loses a significant part of the bond resistance to transfer the force from the reinforcing steel to the surrounding concrete and vice versa [1][4][5][6].

Corrosion is very serious in the case of post-tensioned prestressed concrete due to the fact that the reinforcement is deliberately stressed in tension prior to any loading on the structure. In addition, the post-tensioning tendons are free to move within the concrete over the beam length as they may be anchored only at the ends: such movements are prevented in grouted tendons. Therefore, rusting of tendons may take place without any visible sign on the concrete surface causing a sudden structural failure without any advance warning [1][6][7].

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A considerable amount of research has been undertaken to study the bond characteristics of deformed bars in concrete. It has been reported by Abrams (1951) that the earliest published tests on bond of reinforced concrete with “iron bars” was carried out by Hyatt in 1877 [1][5][6][7].

Concrete in sea water is a topic which has been in the literature for generations [Gjorv (1975)]. Some papers were presented as early as 1909 in the International Association for Testing Materials in Copenhagen, which reported on corrosion mechanisms, procedures to reduce or prevent corrosion in the early investigation based on the results of the various experiments and observations [1][7][8][9][10].

Several researchers have investigated the bond characteristics of steel bars in reinforced concrete by studying the behaviour of reinforced concrete tension elements. Houde and Mirza (1979) have obtained new and basic information on the bond-slip characteristics of deformed bars at the various load levels using two widely different types of specimens which are anchorage and transfer type of specimens, along with the variation of the concrete strength from 20.5 MPa to 44.0 MPa (2980 psi to 6390 psi).
Al-Sulaimani et al. (1990) studied the influence of reinforcing bars corrosion and the associated longitudinal cracking on the steel-concrete interface bond behavior using the standard pullout and beam tests. They used pullout tests to simulate severe local corrosion conditions and the beam tests to simulate relatively uniform corrosion conditions along the bar length [8][9][10][11].

Houde and mirza (1979) worked on bond-slip characteristics of deformed bars. They used two different types of specimens. Concrete strength of 20.5 MPa to 44.0 MPa were used. No. 2, No. 4, No. 8 bars were used. They found that the bar starts to slip linearly with an increase in the steel stresses.
Page et al (1978) conducted research on the effect of mix characteristics and steel surface conditions on the bond between the steel reinforcement and different mortars. It was found that the changes in the properties of the steel oxide film influence the bond strength. The relationship between bond strength and the potential resembles an electro-capillary curve [10][11][12].

Al-Sulaimani et al. (1990) studied the influence of reinforcing bar corrosion and the associated longitudinal cracking on the steel-concrete interface bond behavior using the standard pull-out test. There specimens were of 150mmx150mmx150mm cubes reinforced with 10, 14, and 20mm diameter bars. They induced corrosion by producing a direct current of $2\frac{\mathrm{mA}}{\mathrm{cm}^2}$. He concluded that the bond strength decreases linearly with the increase in corrosion on the steel bars.

Cabrera and Ghoddoussi (1992) investigated the influence of reinforcement corrosion on bond strength of deformed bars using pullout specimens made by OPC. A voltage of 3V was induced to accelerate the corrosion on the bars. The experimental results were used to obtain the relationships between bond stresses and corrosion rate. The ultimate bond stress was related to the crack width.

In this research the effect of corrosion on the bond strength of steel and concrete is taken into account. Corrosion effects the bond strength, as the corrosion takes place in the reinforcement bars, the volume of the bars starts to increase, and this increase in volume decreases the effective area of steel consequently decreasing the bond strength of steel and concrete. In this research corrosion is induced in steel bars by using electro-chemical process, in which a 2.25Amp direct current was provided to steel bars for a particular amount of time, and then bond strength at each corrosion level was noted down.

2.1 Aim

The aim of this research is to investigate the effects of corrosion on bond strength between steel and concrete and to formulate a simple equation between corrosion and bond strength.

2.2 Objectives

Objectives of this research are:

1. To find the change in weight of steel due to corrosion.

2. To find a relationship between corrosion and bond strength.

2.3 Materials and Methods

2.3.1 Materials

Materials used in this research are

1. Cement: Ordinary Portland cement was used as a binding material for the concrete mix.
2. Fine aggregate
3. Coarse aggregate
4. Steel bars (3/8"diameter): Ringed steel bars were used as in figure 3 to obtain better bond strength between steel and concrete so that proper results can be obtained

2.3.1.1 Instrumentation

1. Instruments used in this research are as follows:
2. Voltmeter: Volt meter was used to know that the voltage that is being applied during the process I constant.
3. Ampere meter: Ampere meter was used to know the amount of current being applied during the process.

Rectifier: rectifier was used to convert the alternative current into direct current.
By using these instruments circuits were made. These circuits included a rectifier which converted the alternative current into direct current.

2.3.2 Methods

In this research corrosion is induced in the steel bars using electro-chemical process. A total of five steel bars were taken, and each steel bar was weighed. Once the steel bars were weighed they were then rusted using the electro-chemical process. A direct current of 2.5Ampere was induced on the steel bars. The steel bars were attached on the negative side of the circuit, and on the positive side the stainless steel plates were attached, and the process of rusting was initiated. The steel bars were rusted for different time intervals of 1 hour, 2 hour, 3 hour, 4 hour and 5 hour.

After the rusting of steel bars was completed, the change in weight of the steel bars was calculated, and the percentage change in weight was calculated. Then a graph of change in weight (y-axis) against Time (hours (x-axis)) was plotted. The results of the rusting of steel bars are shown in table 1.

Table 1. Change in Weight Of Steel Bars

Sample Number	Time(hr)	Weight of steel bars(kg)	Weight of steel bars after rusting	Percentage of rusting (%)
1	1	.105	.110	4.70
2	2	.105	.115	9.50
3	3	.105	.119	13.3
4	4	.105	.123	17.1
5	5	.105	.128	21.9

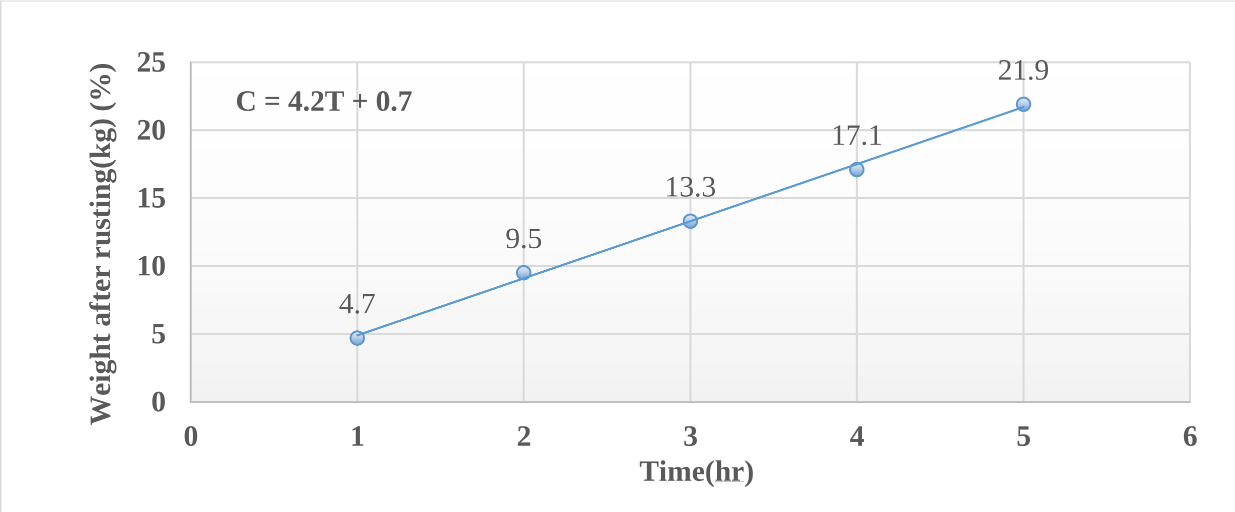

Figure 1 Graph Plotted Of Weight After Rusting Against Time

The equation obtained from this graph of $C = 4.2T + 0.7$ was rearranged making time as the dependent variable and corrosion as the independent variable. By reassembling the equation we get:

$$\boldsymbol{T = \frac{C - 0.007}{100 \times 0.042}}$$

Now corrosion for the steel bars were assumed to be $1\%, 2\%, 3\%, 4\%, 5\%, 6\%\ and\ 7\%$ and the values were kept in the equation to obtain the time needed for the steel bars to be induced with direct current to obtain the assumed corrosion levels. After using the following equation the results obtained for time needed for the desired corrosion levels are in table 2:

Table 2. Corrosion To Be Achieved In Required Time

Corrosion (%)	**Time(minutes)**
0	0
1	15
2	30
3	45
4	60
5	75
6	90
7	105

Now the specimens were prepared using 1:2:4 mix ratio. Using ordinary Portland cement as the binding material. Twenty-four specimens (cylinders) were prepared for this research. Each specimens was embedded with two steel bars, one from the top and the other from the bottom. The steel bars were embedded in such a way that 3 inches were inside the cylinders and 4 inches were outside the cylinders. The steel bars were 4 inches oil painted, and the rest 3 inches were not pained. The painting of steel bars was done in order to ensure that no corrosion occurs on the steel bars during the curing process. The steel bars were embedded in the cylinders in such a way that they both were on the same vertical axis, after the specimens were prepared they were cured for 28 days to achieve maximum bond strength between steel bars and concrete. After the curing process was done the process of rusting was initiated.

3. Results

The twenty-four specimens prepared were marked for each time intervals. Each time interval was given three different specimens so that an average value of bond strength can be calculated and the errors can be eliminated as much as possible. The bond strength of each specimen was calculated using the modified pull-out test, in the modified pull-out test the specimens are pulled from top and bottom al-together and the bond strength values are noted down. Every specimen in this research went through the modified pull-out test and the desired bond strength values were obtained. The results for each specimen are as show in Table3, as follows:

Table 2 Average Bond Strength Obtained At Different Corrosion Levels

Specimen 1 at 0% corrosion	
Time of Corrosion (minutes)	**Bond Strength (MPa)**
0	4.6
0	4.3
0	4.4
Average of 0% corrosion = 4.40 MPa	

Specimen 2 at 1% corrosion	
Time of Corrosion (minutes)	**Bond Strength (MPa)**
15	3.8
15	3.7
15	3.7
Average of 1% corrosion = 3.75 MPa	

Specimen 3 at 2% corrosion	
Time of Corrosion (minutes)	**Bond Strength (MPa)**
30	3.4
30	3.4
30	3.4
Average of 2% corrosion= 3.4 MPa	

Specimen 4 at 3% corrosion	
Time of Corrosion (minutes)	**Bond Strength (MPa)**
45	3.00
45	3.00
45	3.02
Average of 3% corrosion = 3.00 MPa	

Specimen 5 at 4% corrosion	
Time of Corrosion (minutes)	**Bond Strength (MPa)**
60	2.45
60	2.5
60	2.4
Average of 4% corrosion = 2.45 MPa	

Specimen 6 at 5% corrosion	
Time of Corrosion (minutes)	**Bond Strength (MPa)**

75	2.14
75	2.13
75	2.14
Average of 5% corrosion = 2.14 MPa	

Specimen 7 at 6% corrosion	
Time of Corrosion (minutes)	**Bond Strength (MPa)**
90	1.67
90	1.70
90	1.70
Average of 6% corrosion = 1.69 MPa	

Specimen 8 at 7% corrosion	
Time of Corrosion (minutes)	**Bond Strength (MPa)**
105	1.20
105	1.20
105	1.17
Average of 7% corrosion = 1.19 MPa	

Average Bond Strength Obtained At Different Corrosion Levels		
Time (minutes)	**Corrosion (%)**	**Bond Strength (MPa)**
0	0	4.40
15	1	3.75
30	2	3.40
45	3	3.00
60	4	2.45
75	5	2.14
90	6	1.69
105	7	1.19

The average bond strength values obtained from the modified pull-out test are given in table 3. The values obtained from the modified pull-out test provide us with the knowledge that the bond strength decreases as the corrosion increases. It indicates that the bond strength is in an indirect proportional relationship with corrosion. After this a graph of Bond strength (MPa(y-axis)) against Corrosion (%) was plotted.

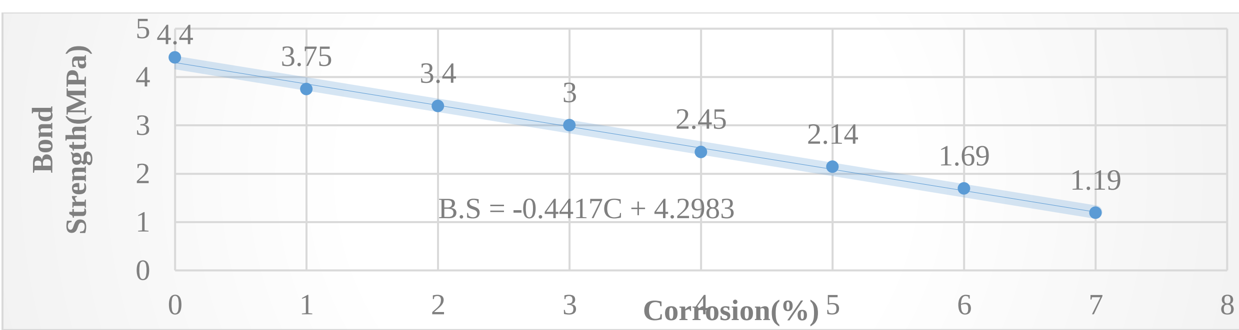

Figure 2 Graph Plotted of Bond Strength Against Corrosion

Figure 2 shows the linear relationship between bond strength and corrosion. As we can see that the bond strength decreases as corrosion increases and vice versa. The equation obtained from the following graph is as follows:

$$\boldsymbol{B.S = -0.4417C + 4.2983}$$

B.S = Bond Strength
C= Corrosion

4. Conclusions

1. Bond strength is effected by corrosion
2. Bond strength is indirectly proportional to corrosion
3. The equation to determine bond strength between steel and concrete using corrosion is as follows:
4. $\boldsymbol{B.S = -0.4417C + 4.2983}$

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Compressive Strength Evaluation by Using Waste Fly Ash and Marble Dust in Cement Mortar

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Abstract
Pakistan being one of the most urbanized countries of the region having a large number of industries which have been known to produce large quantities of byproducts and wastes and currently facing the major environmental issues. The aim of this research is the utilization of fly ash and marble dust waste as partial replacement of cement and fine aggregate in cement mortar production to overcome the environmental and ecological challenges concerned with the use of industrial waste. This study examines the mechanical properties of hardened cement mortar by using fly ash as partial replacement of cement and marble dust waste as partial replacement for fine aggregate in cement mortar at various percentages (0%, 5%, 10%, 15% and 20%) by weight of the cement and fine aggregate. Cubes samples with fly ash and marble dust waste replacement as cement and fine aggregate were used to determine the compressive strength of hardened cement mortar. The test results showed that addition of fly ash and marble dust waste into cement mortar mixture significantly increased its compressive strength as compared to conventional cement mortar.

Keywords
Industrial Waste, Fly Ash, Marble Dust, Cement Mortar and Compressive Strength.

1. Introduction

In the construction industry, cement mortar plays an imperative role in plastering, block and brick masonry. Cement mortar is a composite material which is a mixture of cement, fine aggregates, and water. Among which the cement plays an important role in strength of cement mortar. In construction world cement mortar is the mostly used as a binding material of bricks, concrete blocks and as a plaster for covering the walls, roofs etc. (R. Krishna et al. 2015).

Various industries such as marble industry, steel mills etc., uses materials that result in the production of various by-products and waste such as marble dust, fly ash and many others. In some countries these materials are dumped in open fields as of no use without knowing about their cementitious properties. Thus by doing so they are polluting the environment and also reducing the natural resources by cutting mountains. Some of these by-products and wastes have cementitious properties, so they can be used as replacement with cement while some can be used as a replacement of sand. (B. Rai et al. 2014).

Out of various cementing materials worldwide the mostly used material is fly ash. In thermal power plants when pulverized coal is combusted a byproduct is produced this byproduct is basically called fly ash. American Concrete Institute (ACI) defined the fly ash as "The finely divided residue transported from the combustion zone to the particle removal system by flue gases resulted from the combustion of powdered coal." Worldwide, in 1998 above than 390 million tons of coal ash annual production was estimated. The utilization rate of this fly ash was only 14 percent while remaining amount was dumped in landfills (A. A. Ramezanianpour, 1994).

Marble Dust is a grounded marble fine particles also another byproduct during process of marble cutting and finishing in marble industry. Marble has been widely used in structures since ancient times. Most of the monuments and ancient sculptures were made with the help of marbles. Nowadays, marbles are used for the decoration purpose, which increases its demand in the market. With the increase in production of marbles it increases the waste that obtained from it. Many of marble industries in world generate several thousand tons of marble dust annually. As marble powder is the waste product this marble dust is considered to be solid waste material, obtained during the process of sawing and shaping of marble by parent marble rock, contains heavy metals which makes the water unfit for use. Marble powder also creates environmental problems. Due to environmental problems, it has a great impact on human health as well as on nature. (P. Chaurasia, 2015).

2. Review of literature

Cement Mortar is a composite material that includes basically of a binding medium (cement) and fine aggregate (Sand) mixed with water. It is also known as Portland cement mortar. In the mid nineteenth century it was invented. Its use was propagated during nineteenth century and in 1930 for new construction cement mortar was used instead of lime mortar. Its versatility, durability, sustainability, and economy have made it the world's most broadly utilized construction material. It can be used as binding material for masonry work of bricks and concrete blocks. It is also used as a plaster for walls and ceiling of structures. In past years many research work has been done on the bonding and strength of bricks or blocks with cement mortar. (F. Ogbonna, 2009).

(G. Bumanis and D. Bajare, 2016) stated in their research that traditionally fine sand is used as inert mineral filler to the mortar mix to make material structure compact in micro level. To obtain powder mineral filler material often milling is used to reduce particle size distribution and morphology of obtained particles. Traditionally planetary ball milling is applied, however this method is ineffective if large quantity of material is to be prepared. Grinding by collision is more effective method for refining of brittle material and one of the few machines for material grinding by collision is disintegrator. They dealt with natural quartz, dolomite screening and natural quartz-dolomite mixed sand milling by collision in disintegrator at different energy rates and tested as micro filler in portland cement mortar as partial sand replacement.

(B. Rai et al. 2014) concluded that the excellent performance due to efficient micro filling ability and pozalinc activity quarry dust and fly ash can be used together in mortar when they evaluated the compressive strength (3, 7, 28 and 50 days) and tensile strength (28 and 50 days) of 1:3 mortar mixes in which the cement was partially replaced with fly ash of 15%, 20%, 25% and 30%. Whereas the fine aggregate was replaced with 20%, 50% and 100% quarry dust by weight.

(K. Deepankar et al.2016) stated in there research that increase in compressive strength of concrete can be achieved if waste marble powder is used as a replacement of sand up to 15% whereas decrease in compressive strength of concrete is caused if waste marble powder is used as a replacement of cement up to 15%. Where slightly increased in the durability with reference to the control mix can be noted with waste marble powder replacement at various percentages.

3. Methodology

Cement mortar of 1:2 was prepared by replacing fly ash by weight of cement and marble dust waste by weight of fine aggregate with percentages of 0%, 5%, 10%, 15% and 20%. All the materials were mixed thoroughly in pan with water cement ratio of 0.5 by trowel. Once the mortar ingredients were mixed thoroughly it was placed in a large rigid pan where the quality of cement mortar was observed and then it was poured in the moulds of 2 X 2 inch cubes. The inner surfaces of the moulds were coated with oil before pouring so that they can be easily demolded after 24 hours. Each layer was tamped 25 times with tamping rod and then vibrated for sufficient time. The top surface was leveled with a trowel and finished properly. Once the cement mortar was set thoroughly, the cubes were demolded and were covered in wet clothes at normal temperature for the required time period of 7, 14 and 28 days. After curing period of 7, 14 and 28 days, the specimens were allowed to dry the surface for about one to two hours. Then they were tested in universal testing machine (UTM) for studying the compressive strength.

4. Result:

4.1 Compressive Strength

Compressive strength test was conducted on universal testing machine in accordance with ASTM C109/C109M − 16a to evaluate the strength development of cement mortar containing various fly ash and waste marble dust contents at the age of 7,14 & 28 days as shown in Figure 1. From the obtained results as shown in Table 1 and Chart 1, it was observed that the compressive strength of all mortar samples was increased significantly with adding the percentages of fly ash and waste marble dust. After 28 days compressive strength of cement mortar with fly ash and waste marble dust was increased by 10% as compared to normal cement mortar with 20% replacements of fly ash in cement and waste marble dust in fine aggregate in the mortar mix with a constant water - cement ratio of 0.50. The reason for this was due to the cementitious properties of fly ash and waste marble dust. The bond between fly ash, waste marble dust, cement and fine aggregate were incredible and acted much more effectively due to which there was a decrease in voids.

Figure 1: Measuring Compressive Strength by Universal Testing Machine

Table 1: Results of Compressive Strength Tests

S. No.	**Days**	**Sample No.**	**0 % Fly Ash & Marble Dust**	**5% Fly Ash & Marble Dust**	**10 % Fly Ash & Marble Dust**	**15 % Fly Ash & Marble Dust**	**20 % Fly Ash & Marble Dust**
			Strength (Psi)	**Strength (Psi)**	**Strength (Psi)**	**Strength (Psi)**	**Strength (Psi**
1	7	1	1458.318	1516.65	1545.817	1574.983	1604.149
2		2	1488.085	1547.608	1577.370	1607.131	1636.893
3		3	1435.994	1493.433	1522.153	1550.873	1579.593
4		**Average**	1460.799	1519.230	1548.446	1577.662	1606.878
5	14	1	2030.749	2111.978	2152.593	2193.208	2233.823
6		2	2071.964	2154.842	2196.281	2237.721	2279.160
7		3	1999.837	2079.830	2119.827	2159.823	2199.820
8		**Average**	2034.183	2115.550	2156.233	2196.917	2237.601
9	28	1	2236.824	2326.296	2371.033	2415.769	2460.506
10		2	2282.16	2373.446	2419.08	2464.732	2510.376
11		3	2202.821	2290.933	2334.990	2379.046	2423.103
12		**Average**	2240.602	2330.226	2375.038	2419.850	2464.662

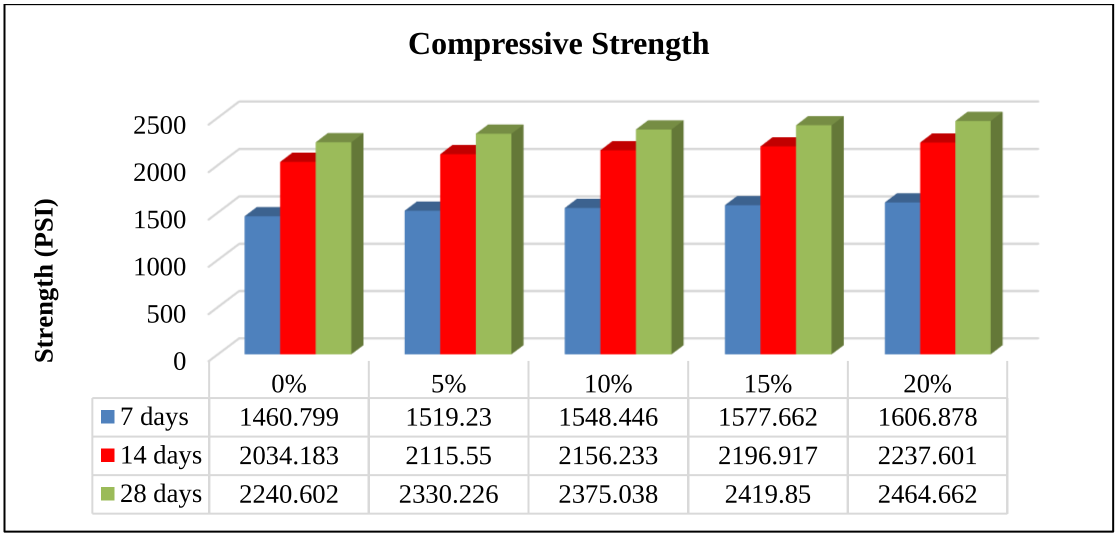

Chart 1: Compressive Strength of Tested Specimens

5. Conclusion

Based on present study and the laboratory experimental work carried out in the research, the following conclusions are drawn.

- Positive effect was observed by using fly ash as a replacement in cement and waste marble dust as a replacement in fine aggregate in cement mortar. Compressive strength and workability of cement mortar were increased by increasing the quantity of fly ash and waste marble dust.

- This research work is the basis for further experiments to establish the effect of fly ash and waste marble dust on the durability of cement mortar and on long term compressive strength after 28 days

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Experimental Investigation of Natural Waste Fiber (Coir and Jute) Reinforced Concrete in Term of Compressive Strength and Workability

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Abstract

For this study, natural waste fiber (coir and jute) were used as they are freely available in large quantities. The properties of natural waste fiber (coir and jute) reinforced concrete was compared with conventional concrete based on experiments performed in the laboratory. The use of coir and jute fiber will also lead to better management of these natural waste fibers. Coconut and jute fibers were cut into desire length of 71.67 mm and 96.48 mm per aspect ratio of 75. 1:2:4 ratio concrete mix was produced which contains coir and jute fiber of 0.5, 1 and 1.5% by volume of concrete. Engineering properties such as workability and compressive strength of coir and jute fibers reinforce concrete specimens were determined following standard procedures at curing ages of 7, 14 and 28 days. The results showed that the workability decrease with increase of fibers, while the compressive strength increased up to 4.5% but dropped afterwards when compared with conventional concrete.

Keywords

Natural Fiber, Coir Fiber, Jute Fibers, compressive strength and workability.

1. Introduction

Concrete is inexpensive, durable, and readily moldable into complicated shapes and has good compressive strength and stiffness due to which it is most frequently used man made material in the world. However, it has low ductility, low energy absorption and low tensile strength, Due to its lack of tensile strength, it is reinforced with reinforcement bars or mesh (rebar's) in structures. But this kind of reinforcement is crude and ineffective for micro crack control. Also, this reinforcement gets decayed and

corroded in abusive environments. Concrete technology now includes reinforcement in the form of fibers, Fiber-reinforcement in concrete is not used for structural strengthening, and rather it reduces the requirement of amount of rebar's or mesh and adds to the improvement of durability by delaying the crack propagation.
The concept of reinforcement in the form of fibers is not new. In ancient times horsehair was used in mortar and straw in mud bricks as a fiber reinforcement. Considerable efforts have been made world-wide to add various types of fibers to concrete so to make it more strong, durable and economical. Natural fiber such as coir and jute has certain mechanical and physical characteristics due to which it can be utilized effectively in the development of reinforced concrete material. In most cases, these fibers are dumped as agricultural waste, so can be easily available in large quantity hence making them cheap.

2. Review of Literature

(Noor Md. Sadiqul Hasan et al, 2012) from Malaysia, have investigated the physical and mechanical characteristics of concrete after adding coconut fiber on a volume basis. They conducted a micro structural analysis test using a scanning electron microscope for understanding the bonding behaviour of the coconut fibers.

(Domke P. V., 2012) from Nagpur, Maharashtra has investigated the use of natural and agricultural waste products such as coconut fibers and rice husk ash to enhance the properties of concrete. The study also emphasizes on the fact that coconut fibers and rice husk ash not only improve the properties of concrete, but it also leads to proper disposal of these waste materials and reduces their impact on the environment.

(Baruah P & Talukdar S A, 2007) Investigated coir fiber reinforced concrete with the volume fraction of 0%, 0.5%, 1.0%, 1.5%, 2.0% by the volume fraction of concrete shows compressive strength, split tensile strength, modulus of rupture, shear strength and toughness continuously increases up to 2% volume fraction of concrete.

(Ali Majid et al, 2012) from New Zealand investigated the mechanical and dynamic properties of coconut fiber reinforced concrete (CFRC) members were examined. A comparison between the static and dynamic moduli was conducted. The influence of 1%, 2%, 3% and 5% fibre contents by mass of cement and fiber lengths of 2.5, 5 and 7.5 cm is investigated. CFRC with higher fibre content has a higher damping but lower dynamic and static modulus of elasticity. It is found that CFRC with a fibre length of 5 cm and a fibre content of 5% has the best properties.

3. Methodology

In this research 1:2:4 ratio concrete mix with w/c ratio of 0.6 was used. The experimental investigation was carried out for both plain concrete and natural waste fiber Reinforced Concrete. Normal and natural waste fiber reinforced concrete by varying the percentage of addition of 75 Aspect ratio Natural fibers by 0%, 0.5%, 1% & 1.5% to the total volume of the concrete were prepared (British Standard, 1983b). In each percentage, the addition of coir and jute were same proportion. Once the concrete was mixed thoroughly it was placed in a large rigid pan where the test for fresh concrete was conducted and then it was poured in the moulds of 6" x 6" cubes. A set of three samples for different percentages and proportions of fibers added to

concrete was casted as shown in Table 1 and after 24 hours the cubes were demoulded and immersed in curing tank containing water at normal temperature for the required time period. After curing the cubes were tested for hard properties of concrete for 7, 14 and 28-day strength as shown in

Figure 1 and Figure 2. Finally, the results of workability and strength obtained by the methods of various percentages of fibers in concrete were compared.

Table 1:Details of specimen casting

% Fiber	Days	No. of Cubes
0	7	3
	14	3
	28	3
0.5	7	3
	14	3
	28	3
1	7	3
	14	3
	28	3
1.5	7	3
	14	3
	28	3

Figure 1: Testing of cube specimens

Figure 2: Tested cube specimens

4. Results:

4.1 Workability

The workability was tested by slump test in accordance with ASTM C 143-03 (ASTM International, 2003). The results are shown in figure 1, From the obtained results, it was observed that there was decrease in the concrete workability with increase in the percentage of fiber in the concrete. The reason was due to the absorption capacity of fibers which absorb the water from the concrete mix which lowers the workability furthermore the fiber in concrete prevented the concrete ingredients from flowing which also decreased the workability.

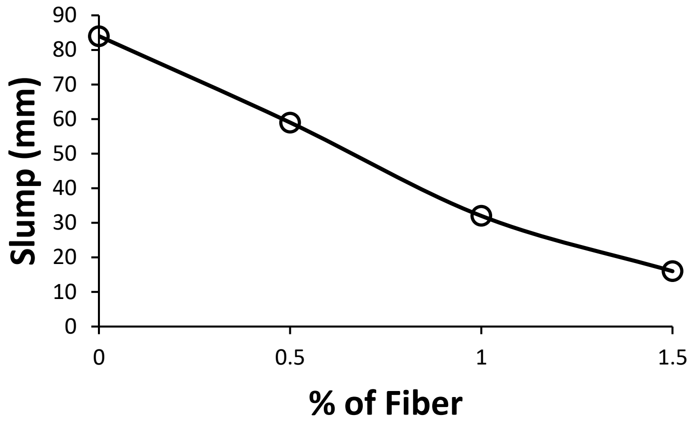

Figure 3: Workability of concrete mix with different % of fiber

4.2 Compressive Strength

Compressive strength test was conducted on compressive testing machine in accordance with BS 1881-101 (British Standard, 1983a) to evaluate the strength development of concrete containing various percentages result are shown in figure 2. From the obtained results, it was observed that the compressive strength of all concrete samples was increasing up to addition of 1% fiber and after that, strength reduced with the increase in fiber Percentage. After 28 days compressive strength of 1% fiber reinforced concrete increases 4.5% as compared to plain concrete.

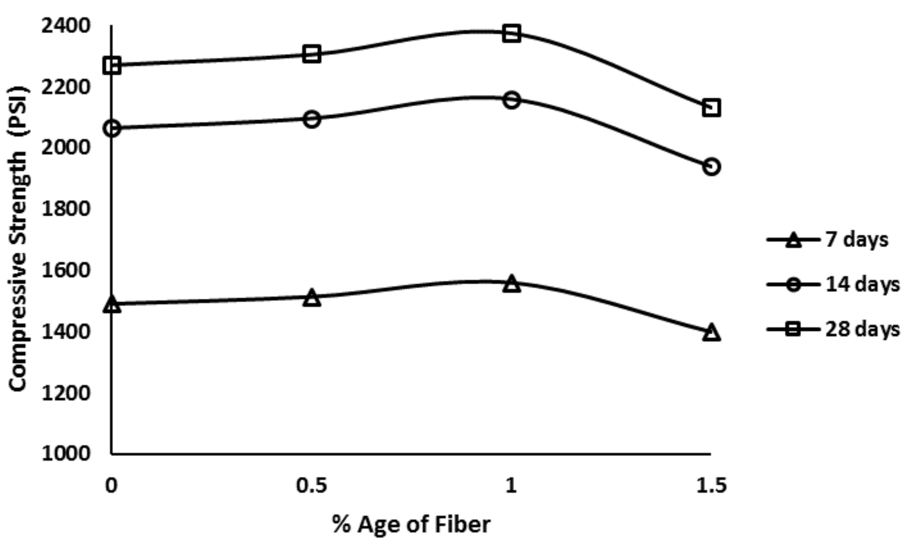

Figure 4: Average compressive strength

5. Conclusions

Based on present study and the laboratory experimental work carried out in the research, the following conclusions are drawn.

- Workability reduced as the fiber content increases. Because as the fiber percentage was increased, the mix became more cohesive.
- The compressive strength has an increasing trend up to 1%. After that, strength reduced with the increase in fiber Percentage.
- The compressive strength of 1% of coir and Jute fibers by volume of concrete gave optimum results with increase of 4.5% compressive strength as compared to normal concrete.

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Experimental Analysis on Tensile Behaviour of ECC using Polypropylene Fiber

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Abstract
Concrete is the most popular construction material used all over the world. Tensile strength of concrete is about 10 – 15% of the compressive strength, which is not sufficient. A modern kind of material is studied in this research i.e. Engineered Cementitious Composite (ECC) from previous research by Professor Victor Li, from the University of Michigan. The composite replaces coarse aggregates and fine aggregates by sand and fly ash respectively. Using Cement OPC, sand (passing from 250 µm and retained on 150µm), Fly Ash (Class F) with addition of Polypropylene fiber on different percentages i.e. 0%, 0.25%, 0.5%, 0.75%, 1.0% were studied. Tensile Strength of ECC was measured by casting & testing cylinders of 4"x 8" in UTM. Results revealed that 111.40% increment in tensile strength was found at 0.5% of fiber. The study concludes that this composite could substitute the normal concrete where high tension is the ultimate requirement.

Keywords
Engineered Cementitious Concrete, Tensile Strength, Polypropylene Fiber, Universal Testing Machine.

1. Introduction

Concrete is the most popular construction material used all over the world because of its special properties such as versatility, durability and easy handling. Due to these special properties, more than 11.4 billion tons of concrete consumed annually worldwide (Chethan et al., 2015). The tensile strength of normal concrete is within the range of 15 – 20 % of the compressive strength of the concrete, for a concrete whose compressive strength is around 20 – 28 MPa has the tensile strength of around 4 – 6 MPa, however it is not sufficient for such cases where the tensile strength has higher priority. For that purpose, there is a need of such tehniques which results in better carrying tensile loads. Keeping in view the drawbacks of normal concrete, a modern type of composite has developed by Prof. Victor Li known as Engineered Cementitious Composite (ECC). According to him, Engineered Cementitious Composite (ECC) is an easily molded mortar-based composite reinforced with polymer fibers. ECC is similar to Fiber Reinforced Concrete which contains water, cement, fine aggregates, coarse aggregate, fiber and some common admixtures. But the only difference between FRC and ECC is that, in ECC coarse aggregates are not used.

In this study, ECC is made by reinforcing Polypropylene (PP) fibers instead of polyvinyl alcohol (PVA) fibers. Regardless of its geometry, polypropylene has several unique properties that make it especially suited for use in concrete. The fibers are chemically inert and stable in the alkaline environment of concrete, with a relatively high melting point, and low cost. Polypropylene fibers do not absorb water, due to a hydrophobic surface, which prevents any chemical adhesion with the concrete matrix. Fibers bond in the concrete matrix through interfacial adhesion and mechanical anchoring. The disadvantages of polypropylene fibers are that they are sensitive to fire, sunlight, they have a low modulus of elasticity, and they bond poorly with the concrete mix. Despite the above demerits, PP fibers can be used successfully, especially to control plastic shrinkage cracking.

The purpose of this research is to develop a composite which is better in tension and can be used in places where the normal concrete fails in tension. For this purpose, the various changes to be carried out. Furthermore, various types of fibers are used in concrete for strengthening the composite. The fiber does not let the concrete deteriorate when it fails, whereas the normal concrete deteriorates and it breaks into small pieces.

2. Literature Review

Over the last few decades, material engineers have improved concrete mix designs using technology that has increased strength, durability, placing and improved environmental aspects. It has been a dream of materials engineer to produce a composite having better mechanical properties. For achieving the dream, different researchers had carried out different studies on ECC by incorporating PVA/PP fibers.

During the past thirty years there has been considerable progress in concrete technology. Through the reduction of typical drawbacks of plain concrete (i.e. cement matrix micro-cracks resulting from shrinkage or excessive loading, destruction of the material in a brittle manner etc.) new types of cement composites have been obtained. Teresa Zych (2014) done various changes in an experimental work (i.e. the significant reduction of w/c ratio, excluding coarse aggregates in order to obtain a material of much higher homogeneity, the addition of super-plasticizer to achieve good workability of composites and inclusion of fibers in order to strengthen the matrix. The core aim was to get a better composite which is more durable and strong (Teresa Zych, 2014). En-Hua Yang et.al, (2009) in their study included four factors like Class C Fly ash ratio to Class F Fly ash ratio, water to binder ratio, amount of High-range water reducer and amount of viscosity modifying admixture to investigate the composition effects on fresh and hardened properties of ECC (En-Hua Yang et.al, 2009). Another study done by Srinivasa, C. H., Dr. Venkatesh (2014) on Engineered Cementitious Composites for structural applications which concludes that the various investigations carried out by several authors related to the development of Engineered Cementitious Composite (ECC) and its applications in the real field proves to be one of the best alternative and sustainable concrete materials of the future decades (Srinivas, C.H., Dr. Venkatesh, 2014).

Recently, a ductile concrete material – Engineered Cementitious composite (ECC) has been designed and developed to the point where it is emerging in full scale applications, including on bridge decks and tall buildings. An experimental study done by E. Ramya et al. (2015) of Polypropylene fiber in Engineered Cementitious Composites in which an effort was made to study the effect of PP fibers on the mechanical properties of mortars incorporating silica fume. From the experimental investigation it was noted that, in addition of fiber with silica fume to cement mortar at lower volume fraction (0.2%), the strength of mortar achieves 2% higher than normal cement mortar and further concluded Formation of crack is arrested by using polypropylene fiber (E. Ramya et al.,2015).

A.W. Dhawale, V. P. Joshi (2013) published a literature review on Engineered Cementitious Composites for structural applications. The study suggests the need for developing a new class of Fiber reinforced concrete (FRC) termed as Engineered Cementitious Composites (ECC), can be designed based on micromechanical principles. The result is a moderately low feiber volume fraction (<2%) composite which shows extensive strain-hardening, with strain capacity of about 3 to 5% compared to 0.01% of normal concrete [A.W. Dhawale, V. P. Joshi 2013).

A.P. Sathe, A.V. Patil (2013) carried out experimental investigation on Polypropylene fiber reinforced concrete with artificial. This study presents the effect of polypropylene (PP) fibers on various properties of concrete such as compressive strength, tensile strength, workability, and fracture properties with various content of fiber (0% ,0.5%,1.0%,1.5%). This study concludes that up to 0.5% adding of concrete with polypropylene fiber there is optimum increase in all mechanical properties and strength enhancement in splitting tensile strength due to polypropylene fibre addition up to 22%. The compressive strength, split tensile strength increase with the addition of fiber content as compared with conventional concrete (A.P. Sathe, A.V. Patil, 2013).

Varsha Bhamare et.al. (2017) did research on comparative study of ECC and CC on the basis of compression and tensile strength. According to the results, the compressive strength of ECC and CC both is nearly same. Also without the coarse aggregate it is possible to gain strength with the use of PVA fiber volume fraction less than <2%. Nearly 65.70% strength is developed within 7 days for compression. The results show the 27% higher tensile strength for ECC (Varsha Bhamare et.al., 2017). In past, the development on ECC in terms of tensile strength is not much mature. In some cases, the tensile strength of ECC remains same as of PCC while in other cases, it increases but up to little extent (A.P. Sathe, A.V. Patil, 2013 - Varsha Bhamare et.al., 2017). Hence, this work focuses on substantial increment in tensile strength of ECC using Polypropylene (PP) fibers.

3. Specimen Details

In this research, 45 cylinders were cast at mix design ratio 1:1:1 on various percentages of fibers (i.e. 0%, 0.25%, 0.5%, 0.75% and 1%). Also 9 cylinders were cast for normal concrete (1:2:4 @ 0.5 w/c ratio) at 7, 14 and 28 days for reference purpose. The specimens were test in UTM. The hierarchy to work on this research is shown in Figure 1 as follows:

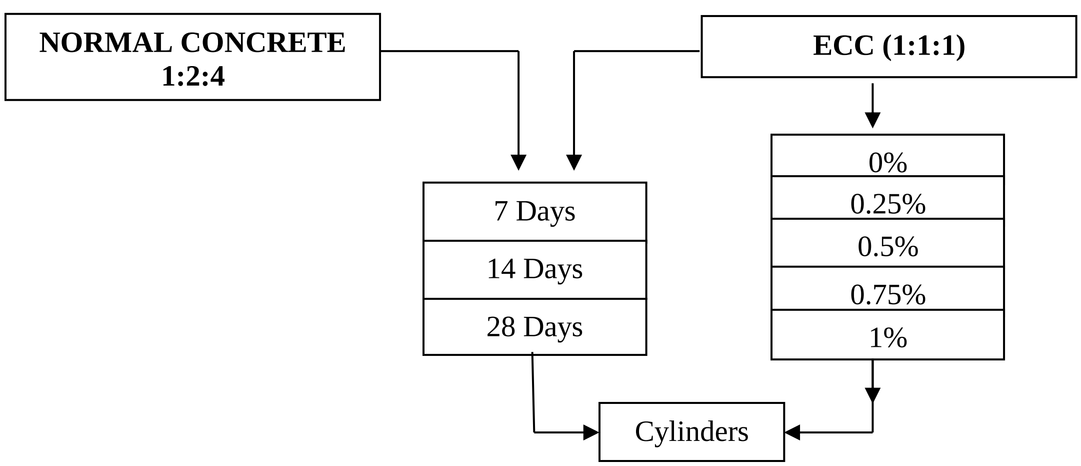

Figure.1. Flow chart of work done

4. Materials

4.1 Cement

Ordinary Portland cement (OPC) which is locally available in the market was used as binding material. It was used in both ECC and PCC as binding agent. The lab tests conducted on cement were Unit weight, Fineness of cement, and initial & final setting time. The Unit weight of cement was measured as 1400kg/m^3 whereas Fineness of cement was 12.5% retained on No. 200 sieve. The initial and final setting time of the cement was 40 minutes and 148 minutes respectively.

4.2 Fine aggregates

It was used in the preparation of normal concrete only (PCC). These aggregates were passed through 4.75mm sieve. The lab tests conducted on fine aggregates are mentioned in table 1.

4.3 Coarse aggregates

In PCC coarse aggregates were of nominal size 13mm and 10mm respectively. In this study, 13mm nominal size aggregates were used. The tests conducted on coarse aggregates are given in table 1 with their results.

4.4 Sand (used in ECC)

The sand which is used in casting ECC is fine sand is basically passed on 250µm sieve and retained on 150µm sieve. Its pictorial view is shown in Figure 2.

Figure 2: Sand used in casting of ECC

For this work the sand was manually sieved. The top sieve was 250µm sieve from which the sand was passed and retained on 150µm sieve placed after 250µm sieve and at last pan was placed which carried dust. After manual sieving, retain of 150µm sieve was taken for research work. The lab tests done on sand are given in table 1 with their results.

Table 1: Properties of fine aggregates, coarse aggregates & sand

Lab Tests	Fine Aggregates	Coarse Aggregates	Sand
Specific gravity	2.69	2.66	2.66
Unit weight (kg/m^3)	1597	1256	1381
Water Absorption	2%	0.25%	3-4%
Fineness Modulus	3.63	3.68	100% passing of 250µm

4.5 Poly propylene fiber

Polypropylene (PP) fibers (as shown in Figure 3.) with length of 19mm were mixed with the ECC. The fibers were used in different percentages. The fibers were purchased from Matrixx Company (Duracrete), Karachi Pakistan.

Figure 3: Polypropylene Fiber

4.6 Fly ash

The Fly Ash used in the research work was Pozzocrete 40 type and shown in the Figure 4. It confirms to ASTM 618 fly ash for use as a component of cement with Portland clinker. The properties of fly ash are provided by the Matrixx Company is mentioned in table 2.

Figure 4: Fly Ash used for casting ECC

Table 2: Properties of Fly Ash

Presentation	Finely divided dry powder
Color	Light grey
Bulk Weight	1.0 tone/ m^3
Specific Density	2.3
Fineness	< 25% retained on 45 micron sieve
Loss-in-ignition	< 2.5%
Particle Shape	Spherical

The material which are used in ECC an PCC are clearly mentioned in table 3.

Table 3: Index of materials of ECC and PCC.

PCC	ECC
Cement	Cement
Fine Aggregates	-
Coarse Aggregates	-
-	Sand
-	Fly Ash
-	Polypropylene Fiber
-	Admixture (Superplasticizer)
Water	Water

4. Laboratory Testing and Results

For lab testing, cylinders of dimensions 4 in. diameter x 8 in. depth of having total volume of 100.6 in^3 were cast. The method adopted to determine the tensile strength of the specimen was indirect tensile or splitting tensile strength method. The mixer which was used in this research was not the normal traditional/ drum mixer, but of high shear mixer which is usually used for high mortar mixing. It was utilized for proper mixing of fiber so to have a homogenous form of mixture. The pictorial view of the mixer is shown in Figure 5.

Figure 5: High Shear Mixer

4.1 Comparison of ECC with PCC

The comparison of splitting tensile strength results of PCC with ECC at 0% of PP fiber shows that, ECC at 0% of the fiber obtained more ultimate strength as compared to PCC as shown in Figure 6.

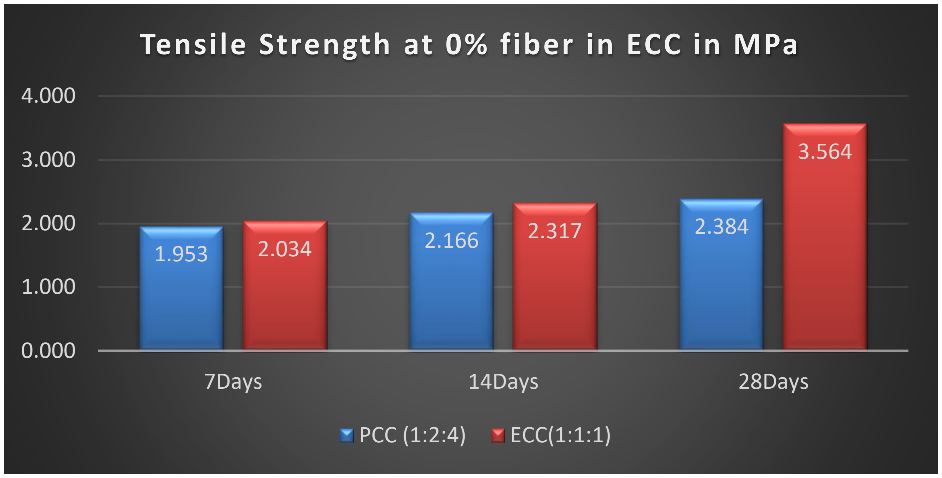

Figure 6: Graph showing comparison between Tensile Strength of PCC (1:2:4) and ECC (1:1:1) @ 0% fiber

Further, by increasing the percentage of PP fiber in ECC by 0.25% the ultimate strength has sufficient increased as shown in Figure 7.

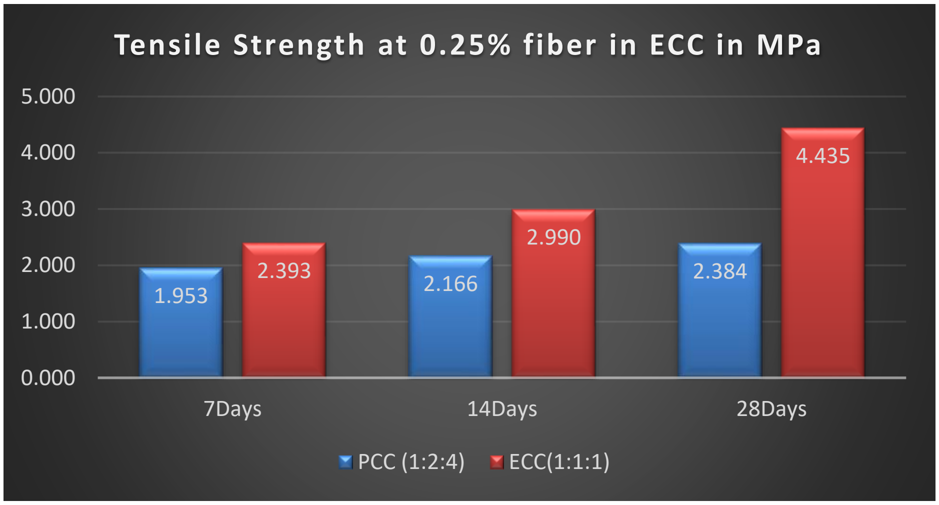

Figure 7: Graph showing comparison between Tensile Strength of PCC (1:2:4) and ECC (1:1:1) @ 0.25% fiber

At 0.5% of PP fiber, more ultimate strength was achieved as compared to ECC at 0.25%. Whereas the early strength results (i.e. at 7 and 14 days) are also increased as shown in Figure 8. Here the cracks were minor as compared to the cracks appeared in the 0.25% of fiber in ECC.

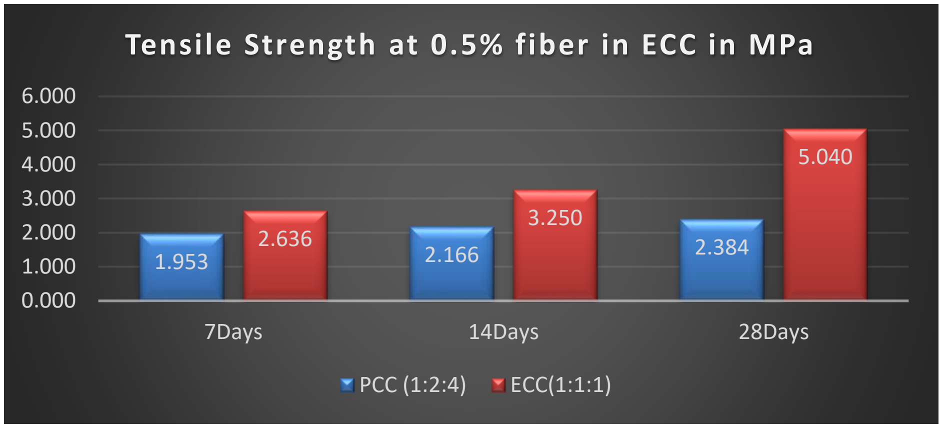

Figure 8: Graph showing comparison between Tensile Strength of PCC (1:2:4) and ECC (1:1:1) @ 0.5% fiber

Now, as fiber % is increasing above 0.5% (i.e. at 0.75% & 1%) the tensile strength of ECC are getting decreased as compared to ECC at 0.5% of PP fiber. The early strength results are shown in Figure 9 & 10.

The major reason is at 0.75% fibers the lumps of fibers were developed & creating difficulty in the proper mixing.

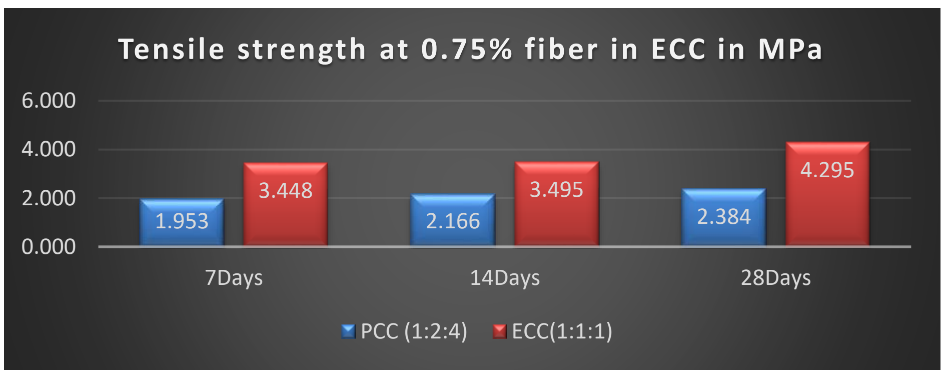

Figure 9: Graph showing comparison between Tensile Strength of PCC (1:2:4) and ECC (1:1:1) @ 0.75% fiber

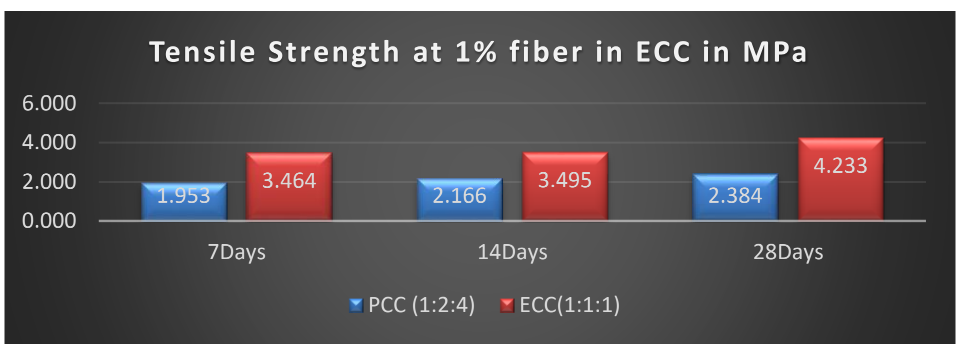

Figure 10: Graph showing comparison between Tensile Strength of PCC (1:2:4) and ECC (1:1:1) @ 1% fiber

5. Conclusion & Recommendations

Based on results of experimental investigation conducted on PCC and ECC, the following conclusions are drawn:

- ECC achieve highest strength at 0.5% PP fiber than normal concrete 1:2:4. The results shows an increment of 111.40% than PCC 1:2:4
- Tensile strength is increasing from (0% - 0.5%) of fibers by weight of cement and above 0.5% (such as, 0.75% and 1%) is decreasing as compared to other percentages (i.e. 0% - 0.5%).
- Workability of fiber reinforced concrete is also an appreciable issue as satisfactory workability was observed with use of chemical admixture with dosage of 1%.
- It is recommended to do proper mixing of concrete for not only achieving good workability but to reduce the chances of improper distribution of fibers leading to the failure of concrete.

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Brief on construction planning of high rise projects in Pakistan (Case study)

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Abstract
Bahria Icon Tower is a mixed used commercial sky scraper complex that consists of three Towers with a total plot area of approx. 14000 sq. meter. Planning of this project has played a vital role throughout the project lifecycle. The focus of this paper will be on construction planning of the tower; initiation of the project, construction methodologies used, types of equipment's and moreover will give a brief description on structural design system and construction techniques which are integrated from the early design concept. Furthermore methodologies, software's used for planning and monitoring will be discussed.

Keywords
High rise construction, construction methodologies, planning, monitoring, project lifecycle.

1. Introduction

One of the most significant tools in a project life cycle is planning of a construction project. (Newitt, et.al. 2005). For smooth execution of works sequence has to be planned accordingly. (Mubarak, 2010). BT-Icon Project consist of multi-use development towers with the total floor area of 3,472,789 sq ft that includes retail , shopping , commercial , leisure , apartments, restaurants, offices. BT-Icon is designed to be the center piece of the large scale development of Karachi that rises into the sky to an unprecedented height; two towers that exceeds 300 meters and 200 meters with 62 and 42 floors respectively.
The client of BT-Icon is Bahria Town that is a major real estate developer in Pakistan and AAA-Partnership is the Project Managers while WSP Middle East is the designer and ASA are the Project architect's with more than thirty international designers and contractors involved in the various stages of the project.
The tower is design on the method of Core and outrigger system, at level 48 of BT- Icon tower and at level-14 of BT-Apartment tower one outer bay peels away as the structure spirals into the sky. The 62 Floors BT-Icon tower is designed as composite decking slab (CDS) system with a center mounted core wall while the other tower is a reinforced concrete structure building with two core walls at corners and two outrigger floors. The tower massing is also driven by wind engineering requirements to reduce dynamic effects of wind pressure. Integrating wind engineering principles in architectural design and design of façade works results in taming the powerful wind forces.

Figure 1: BT-Icon Tower render image

2. Structural System Brief Description

2.1 Lateral Load Resisting System

In both towers lateral load resisting system consist of high performance reinforced concrete core walls linked to the exterior reinforced concrete columns through a series of reinforced concrete shear walls and steel girders at the mechanical (outrigger) level.
The core walls vary in thickness from 1600mm to 550 mm. The core walls are typically linked through a series of 600mm to 1200 mm periphery beam at every level. In CDS structure system periphery beams typically consist of steel shear plates with shear studs embedded in concrete section which are then connected through girders with the center core wall.
At the top of the center reinforced concrete core wall, a very tall spire (mast) tops the building, making it the tallest tower in Pakistan. The lateral load resisting system of the mast consist of both reinforced concrete and steel structure as well.

2.2 Floor Framing System

The BT-Icon Tower floor framing system consist of 125mm to 375mm two-way composite decking slab approximately 12 meter spanning between center core wall and exterior columns. The floor framing system within the interior core consist of a two way reinforced concrete flat plate system. While BTI-Apartment tower consist of a two-way flat plate slab system with 225mm to 500mm slab and with beams at the periphery with 1000mm depth.

2.3 Foundation System

The tower is founded on a 3000mm thick high performance reinforced concrete pile supported raft foundation at -26500mm, for raft layout plan (see figure-2). The reinforced concrete raft foundation utilizes high performance self-compacted concrete and is placed over a minimum 50mm binding slab over water proofing membrane (geo-textile membrane and bituminous layer). The bottom of raft foundation and all sides are protected with water proofing membrane.

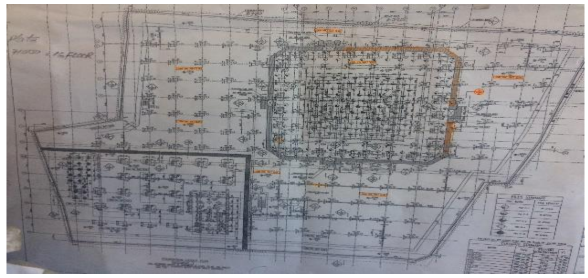

Figure 2: Raft layout plan

The piles are 1000mm, 1500mm and 2000mm in diameter depending upon the area of construction, high performance reinforced concrete pile extending approximately 25 meters below the base of the raft. All piles utilizes self-compacting concrete (SCC) with water cement ratio of 0.35 placed in one continuous concrete pour using tremie method.

3. Construction of Tower Super Structure

Currently BT-Icon tower super structure is completed while BT-Apartment tower is in progress. The original construction program is very tight. The following plans were followed by Project Managers:

- Achieve a 14 days cycle for structural works
- Develop optimum transportation system with large capacity high speed equipment's.
- Utilize optimum formwork system to accommodate various building shapes along the building height. (sequence shown in figure 3)
- Developed organized logistics and procurement plan through-out the construction period.

Since construction planning is extensive and cannot be covered in detail in this paper, so only a brief summary on construction planning will be covered in this paper.

Figure 3: Construction of super structure

3.1 Planning for Concrete Work

Prior to the construction of the tower, extensive concrete testing and quality assurance and quality control programs were put in place to ensure compliance and all the work is done in agreement with all the consultants involved including owners independent testing agency and concrete was owner supplied item. These programs started from the early development of the concrete mix design until the completion of all tests and verification programs. Summary of concrete pumping is (shown in table 1)

Grade of concrete varies at different floors levels and between horizontal and vertical elements; on higher levels concrete grade was less as compare to lower floor levels and area of steel varies with the same ratio too. Cement, concrete admixtures and aggregates were used of high quality with the strong compliance to all quality assurance and quality control protocols and in harmony with project specifications and drawings.

The testing programs included:

1. Trial mix design for all concrete used.
2. Mechanical properties including compressive strength test and modulus of elasticity.
3. Durability test which includes initial surface absorption test.
4. Creep and shrinkage test program for all concrete mix design.
5. Water permeability and penetration test.
6. Pump simulation test for height above 200m.
7. Heat of hydration analysis and test, which includes cube analysis test.

Table 1: Summary of Concrete Pumping (Horizontal & Vertical line)

Pumping Length	200m , 300m
Pipe Diameter	6 inch
Grade of Concrete	C-64, C-48, C-32, C-25, C-20
Concrete Testing	Flow, Temperature, Strength
Pressure measuring	Strokes and Hydraulics (gauge)

Figure 4: Concrete pump

4. Site Plan and Construction Areas

4.1 Site Logistics Plan

Total site area of the plot is 14,495 sq. meter that is divided into A (1673 sq.m.), B (523 sq.m.), C(2300 sq.m.), D (2844 sq.m.), E (2981 sq.m.) and F (2500 sq.m.) respectively. The job site management plan is ever evolving and dynamic to cater ever changing site conditions. The batching plant is located at 3 km away from site and ready mix concrete is transported by transit mixture and pumped by the powerful stationary pumps of capacity 30 cum/hr. Specialized transit routes, ramps and parking lots were developed and adequate traffic management plan was laid out to cater flow of traffic around the site area. The existing site plan after 7 years of construction is attached below in figure 5.

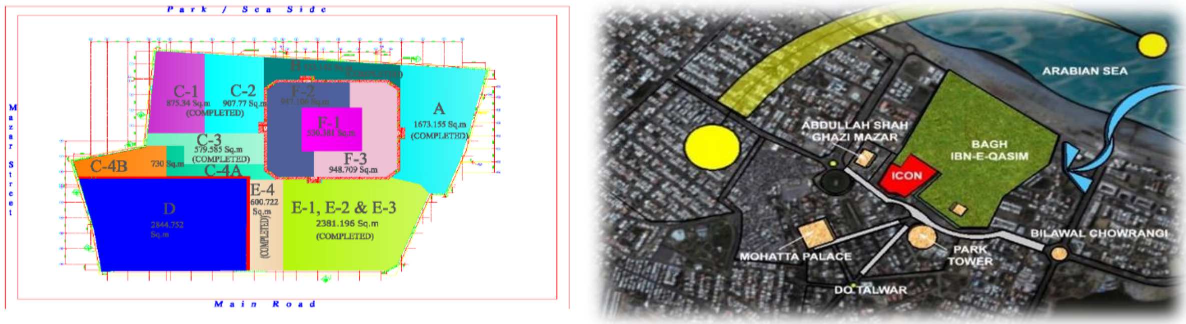

Figure 5: Site layout plan

4.2 Major Equipment's, Vertical Transportation System and Façade

There are three tower crane of capacity 3 to 6 tons, two self-erecting crane and one specialized luffing crane (tip load capacity of 2 tons) are being used to optimize vertical movement of goods and material. Winch machine, temporary passenger hoist, forklifts, hanging cradle for façade works, vertical concrete pumps are being used for temporary transportation of material and hoist cabin for labor vertically.
Sixteen high speed elevators are under the process of installation to ease vertical transportation of residents and visitors along with cargo. Twenty five escalators are being installed at retail area provided by German Vendor and skilled Turkish installers as local capacity wasn't enough as per desired work. As seen in the riding schedule in figure 6

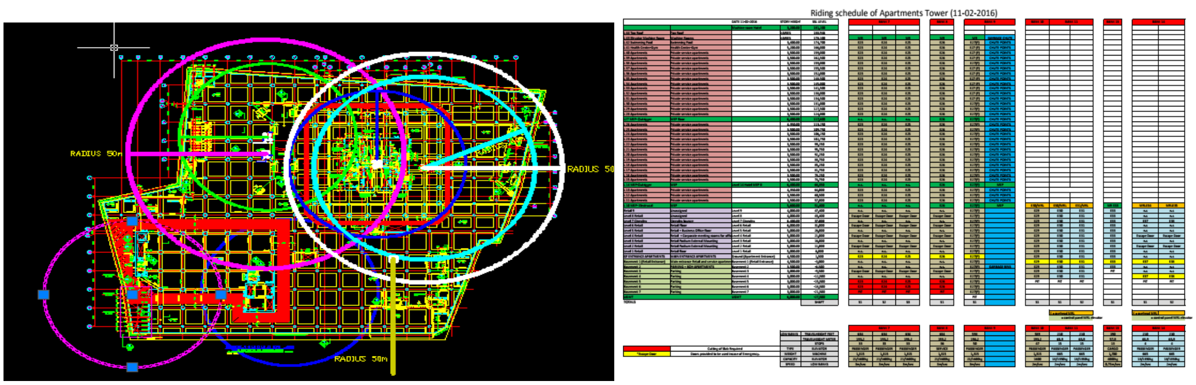

Figure 6: Tower crane layout and Riding schedule of vertical transportation system

For Façade works Unitized curtain wall, Solid aluminum (Aluminum Composite Panels) and Granite are being installed by specialized Chinese and Middle East contractors, while BMU and monorails are made integral part of the towers for its facility management.

4.3 Sequence of construction, ACS (automatic climbing formwork system) and Outrigger Floors

In high rise building due to typical floors, sequence of construction is repetitive which can be further clear by figure 7 and shows the auto climbing formwork designed by Meva for the Core Wall; which was specially used to reduce human effort and time. This formwork was specially designed on the measurement of BT-Icon Tower Core Wall, it takes a single day to jump between floor levels; all which was done automatically using hydraulic jacks. This automatic climbing formwork system eased the execution and reduce time up to 30 percent then the conventional formwork. In BT-Icon Towers following sequence was followed:

Table-2 Construction Sequence

BT-Icon Tower (CDS) 62 storeys	BT-Icon Apartment Tower (RCC structure) 42 Storeys
1- Core wall is poured	1- Columns are poured with its slab (pour-1)
2- Columns are poured	2- Corewall-1 is poured and then its RCC slab is poured (Pour-2)
3- Beams are poured with embedded plates	3- Corewall-2 is poured and then its RCC slab is poured (Pour-3)
4- Girders are welded between beam and core walls	All these pours are together with construction joint at L/3 of slab.
5- CDS slab is joined with girders and poured with placement of 8mm steel mesh	

Furthermore, Project has two mechanical floors at both towers (L-24 and L-35 of BT-Icon and L-14 and L-27 of BTI-Apartment Tower) which are used as outrigger floors; basically this outrigger braces the entire super structure with the interior core wall that transfers load to the pile-raft foundation. These outrigger floors have greater height than typical floors due to operations of mechanical and electrical equipment's during the project operations phase. And these floors also contains project mid-way water tanks.
Moreover the construction initiation for different project areas as highlighted in figure 5 site layout plan was started on different time, so therefore a delay pour strip was designed by WSP (structural designer) in between these areas which was poured on later stage, the main reason for this delay pour strip was to leave the provision of initial settlement of different areas.
Similarly the aforementioned outrigger floor construction was planned to be started after the completion of structural work of individual tower as instructed in the Engineering drawings and BT-Icon Tower has the Co-Generation Plant at Floor Level-5 which is itself an unique feature in a multi used development building and open to sky restaurant at level-48, viewing deck are some of the unique feature of BT-Icon Tower.

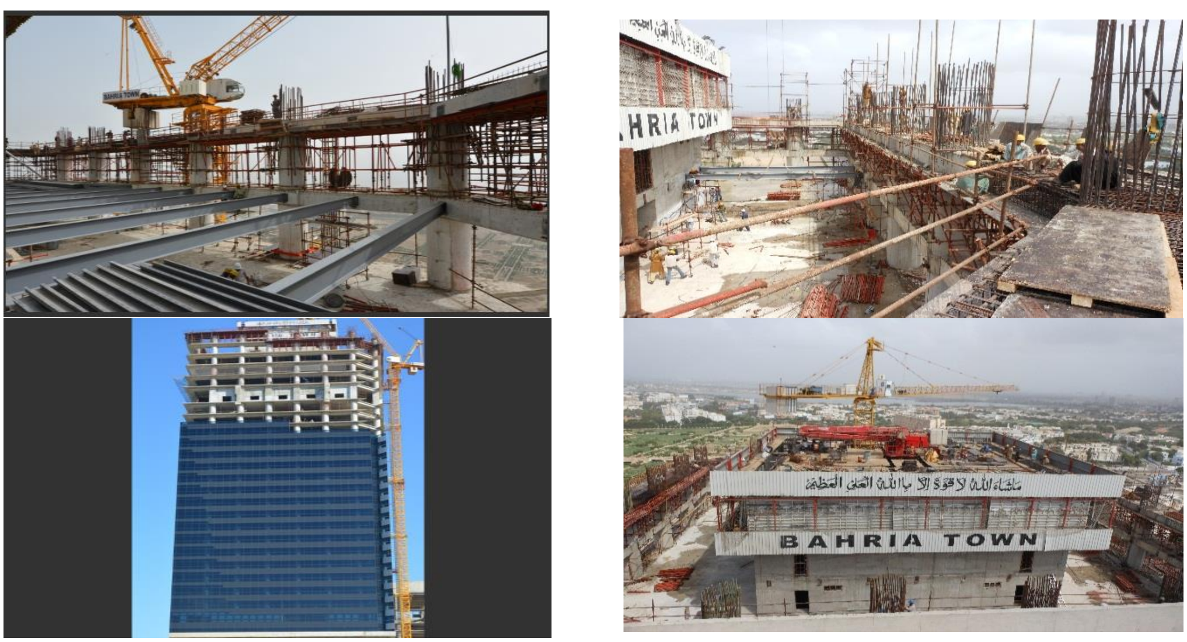

Figure 7: Construction sequence

5. Tools used for Planning, Monitoring and Controlling

Initially basis of design was developed by principle design consultant using User Requirement Standard provided by client. Upon which AAA Partnerships (Pvt.) Ltd, project management consultants developed a top down initial feasibility report and methodology to achieve the desired timelines under the stated budget by client. Furthermore developed work break down structure, organizational structure and jobsite management plans.

Initial phase developed a master baseline using Critical Path Method on Excel as per the desired deadlines and milestones. As the detailed construction drawings started to develop. We formulated construction methodology and used primavera p6 for floor and area specific mini-plans of integrated works of different trades.

To monitor the work progress we use earned value analysis using two major tools. Cost Performance Index and Schedule Performance Index to depict either our project is over budget or under budget and the physical completion is as per planned or not, depicted using S-curves, histograms and bar charts.

The delay in the project was majorly due to scope change by client and unfreeze design due to which frequent changes and reworks are being carried out on regular basis. To mitigate it we use recovery plans as to control delays and control cost overruns to the maximum extent while implementing changes.

6. Conclusion

The trend of high rise building construction is on its apex in Pakistan especially in big urban cities where rural to urban migration is most. In this crucial time this BT-Icon Tower encompassing both types of structure i.e. RCC and CDS will serve as a model for all the near-term civil engineering projects. The Icon Tower project reflects the advancement in civil engineering technology and initiation of multibillion projects in Pakistan. The construction methodologies, planning techniques and architectural apparition will surely be a guideline for future high-rise projects.

As of today BT-Icon Tower is tallest man made structure in Pakistan and imitates as an only model on the coast of Arabian Sea which is surely a source of attraction for other developers and international client to Pakistan.

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Replacement of In-situ Bathrooms with POD Bathrooms to Save Time & Money with in construction of fast track projects (A Project Case study)

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Abstract

Perception of success in projects is strongly interrelated with its context and industry. This study is purely based on the construction 3-star hotel project of Premier inn hotel Dubai, UAE. Which is constructed by Khansaheb Civil Engineering LLC, Dubai, a partner company of Interserve Group London, UK. The project team were faced with an extremely challenging program to deliver a 389-bed room hotel within 14 months, including holy month of Ramzan and two summers where working hours are restricted.The hotel was originally designed using traditional construction method which included in- situ bathrooms however bathrooms are usually challenging and costly due to many trades are in a confined space. Where the project team decided to replace the in-situ bathrooms with POD bathrooms. The objective of this study to produce high production, high quality product with less cost and time to meet the requirement of the project

1. Introduction

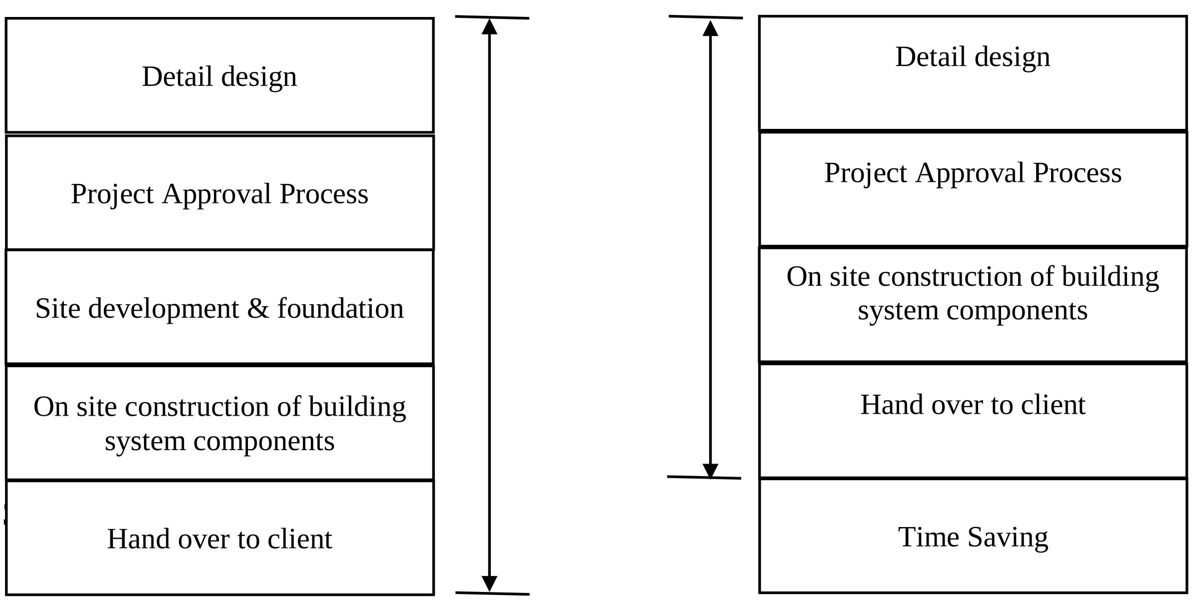
Off-site construction is an application of modern methods of construction where building sector meets the industrial sector.construction is where any of building components, or even a whole building, manufactured in factories away from the actual site where the building will be sited, or simply is where construction site is different from the building site. Off-site construction has different terms, terms like (prefabrication, off-site assembly, factory assembly, pre-assembly, off-site manufacture, etc.). The term off-site construction is generally used nowadays to any part of the construction process that takes place in factories. Offsite construction has many optimistic advantages like Less time in construction process, cost predictability, higher quality, helping the society and the environment, resolving skilled labour shortage, reduce health & safety risks, and helping the business & the economy.

2. Project Case study

The cost of project is a AED 137,449,210.00 million Lump-sum three-star hotel. Duration of the project is 425days. The building consists of basement + G.F +8+GYM+Roof in the heart of the Dubai, UAE. The hotel has 10 structural levels with 389 rooms and consists of a conference room, private dinning space, breakout space, hotel restaurant (with kitchen and beer cellar) and staff areas/offices. As the Construction industry of UAE is booming for Expo 2020, No of new innovative ways in the construction industry are also increasing day by day. Offsite construction of bathrooms, prefabricatedform work used for construction of stairs, other elements of infrastructure, and other time taking activities are going to take place off site construction to save time and money.

The case study presented examines offsite construction application applied to the construction of high specification bathrooms for the construction of a prestigious projects. The client required a high specification of finishes and fittings and it was deemed that an offsite manufactured solution would provide assurance in relation to the programme, quality and cost of the construction. one mockup was carried out to construct one bathroom onsite as well as off site.

Site build construction schedule   off site construction schedule



Construction industry of United Arab Emirates is progressing with full swing to meet the requirement of expo 2020. There are 5276 projects of worth $99.4 billion (dh364.79 billion) are currently under construction & tendering phase in the uae. Most of these buildings are excepted to be completed by october 2020, before the world expo 2020 as the country gears up to handle the biggest inbound tourist rush. All the construction companies which are involved for the construction projects of expo 2020 has already a lot of hurdles during their construction and are in hurry to complete the project within specified time & cost.

3. Research Methodology

This research methodology is based on project case study, every engineer of this project was given different assignments to carry out the offsite construction, so that time and cost could be saved. Every engineer of the site came with new different ideas, but two ideas were highly appreciated and implemented on the project one was prefabricated formwork for the stairs and other was off site construction of bathrooms. But most of time and cost was saved on offsite construction of Bathroom.

4. Results & Conclusion

Offsite construction is fastest, safest & greener way to carry out the construction activities with high production rate, less cost, less time, safer & quicker way. Except the offsite construction of bathroom, a lot of other infrastructure elements can be constructed which can also lead to save time and money.

Table 1: Results of Offsite Construction

On site Construction	Offsite construction
Time Estimated time for onsite construction of 389 bathrooms was 182 days to complete the job	**Time** with offsite construction we finished our job 73 days before which shows that 40% of time was saved.
Cost The estimated cost per bathrooms for onsite construction was AED 19,000. Total Estimated cost of 389 Bathrooms is AED 7391000	**Cost** The utilized cost for offsite construction of bathroom was AED 13680. Total utilized cost of 389 bathrooms aed 5321520. By doing off site construction we saved 28% of cost.
Quality On site construction we did rework due to some quality issues and some damages like waterproofing damages also	**Quality** While doing offsite construction quality issue was not much concern.

occurred.	
Wastage On site construction has a lot of wastage of material.	**Wastage** In off site construction wastage was reduced upto 25% (wastage is calculated for civil materials only like concrete, block, plaster, waterproofing & tile)
Safety On site construction was not safer due to limited confined space.	**Safety** Off site construction was safer due to huge space for working.

Conclusion

To conclude,off-site construction is on the rise and gaining popularity. The rise is stilllower than the desired figures, the industry would have liked to have seen.by widely displaying the impact that new off-site technology can have into the wider world, by showing companies who have profited most successfully from the sector, companies have been able to promote these new methods to the boardroom with more confidence. We need to share, we need to spread, and we need to display the glowing benefits of the off-site industry to the wider masses involved in construction, thus impacting its growth for the better.

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Implimentation of Lean Construction

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Abstract
Lean construction is a philosophy adopted in construction industry to increase the value of the project by reducing all types of the waste and improving construction performance. It emphasizes on maximizing value of the project with less expenditure. Gloablly, lean philosophy has got wider popularity in construction sector. Lean construction has supported the practitioners with several tools and techniques to implement at various stages of the project.. The findings of this analysis highlighted that pull approach, work standardiztion, just in time, increase visualization tools, integrated project delivery method and fail safe for quality are common lean techniques implemneted in local construction industry. While reduction in waste, client's satisfaction, improved communication, visual control and proper task management are major benefits of the lean construction application.

Keywords
Lean construction, lean tools and techniques, lean benefits, waste reduction, Pakistan

1. Introduction

Lean construction has been an important topic amongst researchers all over the world, and is considered as an approach or concept that should be introduced, specifically to increase productivity by reducing waste in the construction process. As stated by (Koskela, 1992), Lean construction (LC) as a philosophy and a set of principles was introduced in construction to maximize customers' value through waste reduction and continuous improvement. Literature has many case studies describing the effective application of lean construction on real projects. According to (Salem et al., 2006), From such case studies, the advantages of applying lean were visible, as projects completed within the budget and ahead of targeted schedule (i.e., three weeks earlier than the completion time); the relationship between subcontractor and the main general contractor was improved, comparatively less injuries and improved safety standards were observed. Garnett et al., 1998 reported a 25% reduction in construction time, an increase in client satisfaction, and a decrease in the overall project cost. (Conte, 2001) showed that the project construction time was reduced by 20% to 30% and cost was reduced by 5% to 12%. Similarly, (Mota et al., 2008) presented the beginning of a Lean journey of a small-sized company in

Fortaleza/Brazil and found that results achieved in the case study were, project's favorable cash flow, incentives given to early completion of tasks, high level of repetitiveness, use of actual productivity vs. planned to re-schedule the project.

Lean construction approach tries to manage and improve construction processes with minimum cost and maximum value by considering customer needs (Koskela et al. 2002). Lean construction concept is keen to reduce uncertainty and variability in the execution of project plans. The application of lean in construction industry has been evolving in past two decades and many new techniques have been observed since then. Among such techniques some are practical and are in practice, while many are still in their theoretical development phase and require deep computer programing skillsAnsah et al., 2016. However, many lean construction tools and techniques are still in an embryonic state, lean construction techniques are gaining popularity because they can affect the bottom line of projects (Salem et al., 2006).The absence of suitable tools, improvement and performance of a lean project program is not possible, which may further result in poor decisionsAnsah et al., 2016.

There has been very little experimental and theoretical research on lean construction in Pakistan. Therefore, there is a necessity of quantitative research that concentrates on the identification of status of level of Implementation of Lean Tools and benefits of lean construction for construction projects. Hence, this study aims to assess the current status of lean construction management and its benefits and the tools used to minimize waste in Construction industry of Pakistan. Following objectives were set to carry out this research.

- Identifying status of level of execution of lean tools &techniques in construction projectsin Pakistan.
- Identifying Benefits achieved by using Lean Construction Management
- Analyze the data and discuss the results

2. Literature Review

It is certain that lean is the next generation of construction industry, aims at reducing many fundamental problems, e.g., time delays. Hannis-Ansah et al. (2016) has reported loss of almost 57% of productive time in most of the construction projects; that further beacons the dire need of further research. For this purpose, many researchers studied about the lean construction tools and techniques which can be implemented in construction industry. Such as, Ansah, (2016) presented 30 new lean tools focusing construction projects Last Planner System (LPS) and Concurrent Engineering and Daily Huddle Meetings. Similarly, Salem et al., (2006) also discussed some lean tools and techniques which were just-in-time JIT, Last planner, Automation(Jikoda), Poka-yoke, single-minute exchange devices SMED, Kaizen, five S's, increased visualization , First-run studies, PDCA plan, do, check, and act, huddle meetings and Fail-Safe for Quality.

There are many more such examples in the literature, some past research papers were studied and 20 lean tools and techniques were mapped with the highest frequency of 15 as shown in table 1.

Table 1: Mapping of Lean Tools and Techniques

S. No	Tools & Techniques	Mota et al. (2008)	Ansah et al. (2016)	Pheng and Shang (2011)	Mehany (2015)	Alarcón et al. (2006)	Bertelsen (2004)	Emdanat et al. (2016)	Seppanen et al. (2010)	Koskela (1993)	Enshassi&Zaiter (2014)	Pan & Pan (2016)	Ingle &Waghmare (2015)		Lim et al. (2006)	Hamzeh et al. (2016)	Awada et al. (2016)	BALLARD & HOWELL (1997)	Salem et al. (2006)	Seppanen et al. (2015)	Dave et al. (2015)	Ballard et al. (2002)	Frequency
1	**5 S's**		1								1	1	1				1		1				**6**
2	**Concurrent Engineering**		1															1					**2**
3	**Creative Thinking**		1							1									1				**3**
4	**Daily Huddle Meetings**											1							1				**2**
5	**Fail Safe For Quality**		1									1										1	**3**
6	**First Run Studies**	1	1									1							1				**4**
7	**Increased Visualization**		1								1	1					1		1			1	**6**
8	**Integerated Project Delievery**																						**0**
9	**Just In Time (JIT)**	1	1	1	1					1				1								1	**7**
10	**Kaizen(continuous Improvement)**		1																1				**2**
11	**Last Planner**	1	1			1	1	1	1		1	1			1	1	1	1		1	1	1	**15**
12	**One-Piece Flow**																						**0**
13	**Poka-Yoke**		1																1				**2**

14	**Pull Approach**																						**0**
15	**Re-engineering**		1							1													**2**
16	**The Kanban System**		1																				**1**
17	**Total Productive Maintaince**									1													**1**
18	**Total Quality Management**									1													**1**
19	**Value Stream Mapping**		1																				**1**
20	**work standarization**		1																				**1**

2.1 The Five Steps Plan (5s) Process

The five step plan (5s), also known as the Visual Work Place, is technique associated with inventory that focuses a designated place for each material that is to be used on the site. This has process has 5 level of implementation, which can significantly reduce the loss of resources Ingle &Waghmare, (2015)

2.1.1 Sort (Seiri)

Separating material by reference and placing materials and tools close to the work areas with consideration of safety and removing unnecessary items from site location.

2.1.2 Straighten or set in order (Seiton)

Piling materials in a regular pattern and placing tools in gang boxes. Keeping important materials nearby for improving efficiency.

2.1.3 Shine (Seitso)

Cleaning and looking for ways of cleaning and organizing job site and construction equipment, because cleaning is regular activity.

2.1.4 Standardize (Seiketsu)

Maintaining and monitoring first three categories to implement standardized methods for improving efficiency.

2.1.5 Sustain (Shitsuke)

Sticking to the rules and regulations for long term operation.

2.2 Concurrent Engineering

As evident from the name, it is about the parallel execution of works by multi-disciplinary teams, aiming at optimizing the efficiency, workability of the labor and the quality of finished products (Aziz and Hafez, 2013; Rahman et al.,2012).

2.3 Creative Thinking

Creative thinking offers continuous improvement through feedback and supports the continual improvement of a production line's daily tasks (Salem et al., 2006).

2.4 Daily Huddle Meetings

This technique is used for communication between project actors that include routine interactions and daily meetings of the project team. Saleem et. al. (2005) identified it very useful for on- site problem solving, and suggested some training to the project partners may further enhance the satisfaction of job (i.e., sense of growth and self-esteem).

2.5 Fail Safe for Quality

This technique depends on the development of concepts that may point out possible defects. This is much more similar to "Poka-Yoke" techniques; however, it can be further extended to safety as well. Nevertheless, the concentration in safety is on probable hazards rather than possible defects, and therefore it is usually used with the risk assessment procedures. It also requires action plan that avoids bad outcomes (Ansah, 2016).

2.6 First Run Studies

First-run studies are utilized to remodel important task (Ballard and Howell, 1997). Operations are scrutinized thoroughly, bringing ideas and suggestions to explore alternative of doing the task. The PDCA (plan, do, check, and act) cycle is used to build up the first-run study (Muhammad et al., 2013).

2.7 Increased Visualization

Visual control is considered a prerequisite for continuous improvement and process control. It includes posting signs for safety, hazards, schedules, and quality standards. The purpose behind increased visualization is communicating key information effectively to the workforce by posting various signs and labels around the construction area (cited by Awada et al., 2016)).

2.8 Integrated Project Delivery

"A project delivery approach that integrates people, systems, business structures, and practices into a process that collaboratively harnesses the talents and insights of all project participants to optimize project results, increase value to the owner, reduce waste, and maximize efficiency through all phases of design, fabrication, and construction" (Cited by Kent et al., 2010).

2.9 Just In Time (JIT)

This technique aims mainly at reducing flow times within a production as well as response times from suppliers and to end users. In any case, JIT is a way of thinking, working and managing to eliminate wastes in processes (Ansah, 2016).

2.10Kaizen (continuous Improvement)

It is a philosophy for continuous improvement originated from Japan (cited by Ansah. 2016). This is an approach that seeks to create an environment in which responsibilities area assigned to each worker and also encourage and motivate the workers to identify loopholes to improve quality and efficiency through the reducing waste.

2.11 The Last Planner System (LPS)

The Last Planner System (LPS) is one of the tools used in LC which produces anticipated and dependable work flow and fast learning in programming, design, construction and commissioning of projects (Muhammad et al., 2013).

2.12 One-Piece Flow

One-Piece Flow system produces many products in medium volumes; on equipment arranged in cells in which material flow is regular and paced by a cycle time; and provides high levels of the flexibility and innovativeness outputs (like the batch flow system) and high levels of the cost and quality outputs (MILTENBURG, 2001).

2.13 Poka-Yoke

Poka-yoke is a Japanese word which can be defined as "error-proofing". Pokayoke devices was introduced by Shingo as new elements in order to avoid defective parts from flowing through the process. It is a lean tool that engages all forms of activities and devices that could avoid an error from happening (Cited by Muhammad et al., 2013).

2.14 Pull Approach

A tool that strengthens our current scheduling process by helping risk management through detailed collaborative planning and continuous improvement. Pull Planning, in simplest terms, is a technique that is used as part of the Last Planner System to develop a coordinated plan for one phase of a project. However, with practice, Pull approach will become much more.

2.15 Re-Engineering

In reengineering whole workflow for each activity/process is redesign considering constraints, variances non-value activities and then taking appropriate solution to remove or minimize these to make uniform workflow. Reengineering can be used from design phase to final execution phase by defining value of each step or process.

2.16 The Kanban System

In the Japanese language, Kandban stands for "billboard or signboard". Kanban is a toll that regulates the information circulation across the organization pertaining to the flow of resources so as the required parts and supplies are procured and released to the utilization sites as they are needed (Ansah, 2016).

2.17 Total Productive Maintenance

TPM is a powerful program for planning and achieving minimal machine downtime. The aim is to divert from fixing breakdowns. For this purpose Machine operators take far greater responsibilities of monitoring the function of machines and warning the constructors regarding the multifunction of machines.

2.18 Total Quality Management

Total Quality Management uses a combination of Statistical process control and problem solving teams to improve process capability and ensure that external factors do not negatively affect the process driving it out of control (4squareviews.com, 2017).

2.19 Value Stream Mapping

This is technique used to visually analyze, document and improve the flow of a process in a way that highlights improvement opportunities (Ansah, 2016).

2.20 Work Standardization

It is one of the most powerful but least used lean tools. By documenting the current best practice, standardized work forms the baseline for kaizen or continuous improvement. As the standard is improved, the new standard becomes the baseline for further improvements, and so on. Improving standardized work is a never-ending process.

3. Methodology

This research is carried out through quantitative research method involving questionnaire survey for data collection. Survey was conducted through interviews, posts, by hand and through Emails to professional respondents (client, consultant, contractors and material suppliers) and were asked to fill the questionnaire forms in order to identify the status of level of implementation of Lean Construction Tools & Techniques in Construction Industry of Pakistan, and to identify the benefits achieved by Lean Construction tools.

The questionnaire was classified into close form or restricted type. Closed questionnaires often require short responses in the form of Yes or No, Agree or Disagree, Important or Not Important, etc. Closed-ended questions are easy to ask and quick to answer, they require no writing by either respondent or interviewer, and their analysis is straightforward (Naoum, 1998).

The questionnaire was consisted of three parts which are Part 1, Part 2 and Part 3. Part 1 is focused on Respondent Demography which is seeking respondent's information such as education, years of experience, position in the organization and also the type of their organization. Part 2 of questionnaire is to assessing the current status of tools used in construction industry, where 20 tools and techniques were enlisted. While part 3 is on benefits achieved from lean construction tools and techniques, where 21 different types of benefits of lean construction were given.

A total of 100 questionnaires were distributed randomly in many construction Companies of Pakistan and 34 completed questionnaire sets were received back

.
Collected data analyzed by using Microsoft Excel spread sheet program and SPSS statistical software package.

4. Results and Discussion

This section presents the results and discussions of analysis for the data collected through questionnaire survey. The questionnaire survey was aimed to identify the current status and benefits of Lean Construction Management tools & techniques in Pakistan.

4.1 Characteristics of Respondents

A total of 100 questionnaire sets were distributed to the respondents (client, consultant, contractors and material suppliers) involved in construction industry and 34 completed questionnaires were received back.
Respondent's Organization
The table 2 shows that majority of respondents are consultants i.e. 13 out of 34 with a percentage of 38.2% of these, a significant number of contractors i.e. 12 0f 34 with the percentage of 35.3%, Clients are 5 with the percentage of 14.7%. And 4out of 34 are material suppliers with percentage of 11.8%.

Table 2: Characteristics of Respondents

Type of Organization	Frequency	Percent
Consultant	13	38.2
Contractor	12	35.3
Client	5	14.7
material suppliers	4	11.8
Total	34	100.0

4.2 Respondents Experience

Table 3 shows a significant number i.e. 13out of 34 with 38.2% respondents have experience within 5 years of handling large projects, the respondents having experience 6 – 10 years are 7 with 20.6%, the 4 respondents with 11.8% having experience 11 – 15 years, and 10 respondents with 29.4% having experience above 15 years.

Table 3: Respondents Experience

Experience	Frequency	Percent
0-05 Years	13	38.2
6-10 Years	7	20.6
11-15 Years	4	11.8
>15 Years	10	29.4
Total	34	100.0

4.3 Cost of Project

The table 4 shows that 1 projects are of the <RS 20 M , with a percentage 2.9%, likewise between 20 M-50 M are 2 with 5.9 %, RS 50 M- 150 M are 5 projects with 14.7 %. RS 150 M- 400 M are 11 with 32.4 %,RS 800 M- 1800 M are 7 with 20.6 %, RS 1800 M- 3000 M are 2 with 5.9 % and > RS 3000 M are 6 with 17.6 %.

Table 4: Cost of Project

Cost of project	Frequency	Percent
< 20M	1	2.9
20M-50M	2	5.9
50M-150M	5	14.7
150M-400M	11	32.4
800M-1800M	7	20.6
1800M-3000M	2	5.9
>3000M	6	17.6
Total	34	100.0

4.4 Respondents Qualification

Table 5 shows the level of education of respondents, i.e. (Diploma, Degree, and Masters).Results in table 5.4 shows educational level of respondents that 10 out of 34 are diploma level with 29.4, most of the respondent's i.e. 22 respondents with percentage 64.7% possess bachelor degree. While 2 respondents has obtained master's degree with percentage 5.9%.

Table 5: Respondents Qualification

Level of School	Frequency	Percent
Diploma	10	29.4
Degree	22	64.7
Master	2	5.9
Total	34	100.0

4.5 Tools and Techniques

From questionnaire survey, respondents perspective were collected towards implementation of lean tools, respondents were asked to rate the implementation of lean construction tool with respect to their implementation on a Likert point scale ranging from 1 to 5, where 1 = Not Implemented, 2 = Rarely Implemented, 3 = Sometime Implemented, 4 = Oftenly Implemented and 5 = Mostly Implemented. 20 tools of lean construction were examined through literature mapping from past research papers towards their implementation. Each tool shows respondents frequency with respect totheir level of implementation. Then the collected data is analyzed by Weighted Analysis.

Weighted opinion averages of each tool of lean construction were determined to assess their perceived significance. The average weighted perceived significance was then computed using the formula adapted from (Kaliba et al., 2009):

$$WA = \frac{1}{5} \times \frac{\sum_1^5 F_i R_i}{\sum_1^5 F_i} \times 100$$

Where WA is the average weighted perceived significance;$R_i$is the response type on the Likert scale, i ranging from 1 to 5 on the Likert scale; F_i is the frequency or total number of respondents choosing response type i on the Likert scale, with i ranging from 1 to 5 as earlier described.
As an example, WA, the average weighted perceived significance was computed as follows:

$$WA = \frac{1}{5} \times \left\{ \frac{(F_1 \times R_1 + F_2 \times R_2 + F_3 \times R_3 + F_4 \times R_4 + F_5 \times R_5)}{(F_1 + F_2 + F_3 + F_4 + F_5)} \right\} \times 100$$

It is noteworthy that the possible values of average weighted perceived significance, WA, ranged from 25% to 100% because each factor identified through literature or interviews had some level of significance that would not amount to zero. The factors whose WA score was 50% and above were categorized as major tools which are mostly implemented in construction industry of Pakistan.

By applying weighted average formula in the collected data, result is shown in following figure 1.

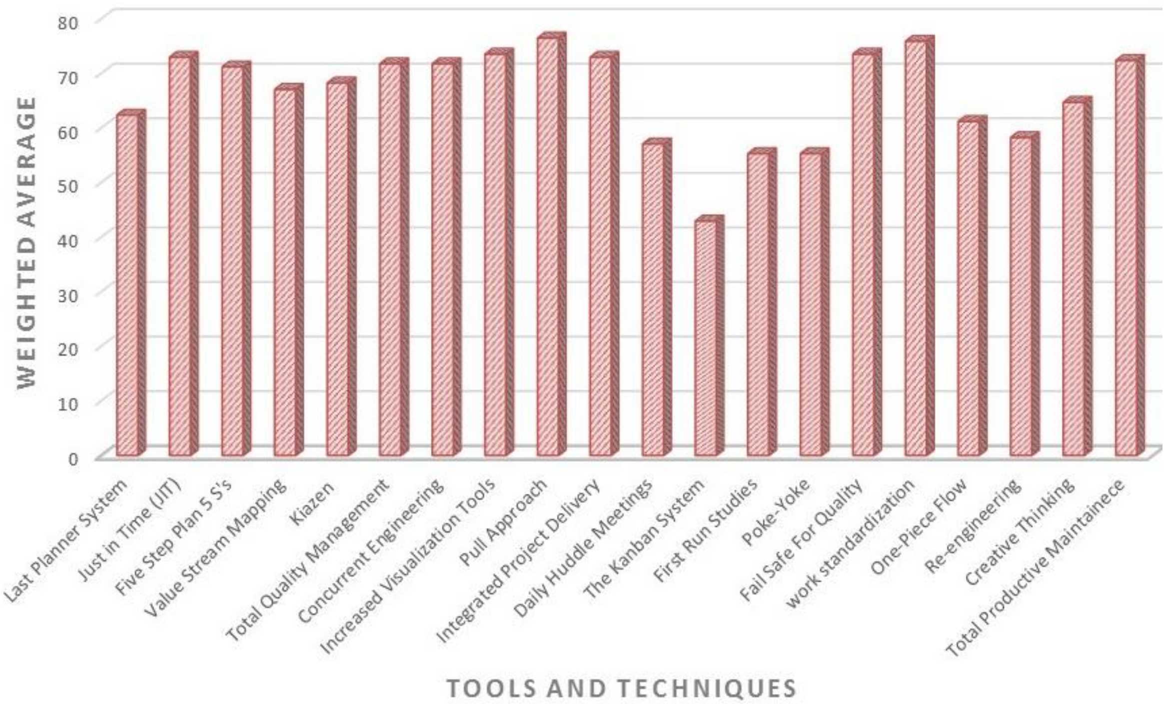

Figure 1: Respondents Qualification

4.6 Benefits of Lean

Data collected from the 2nd part of questionnaire (where respondents were required to fill short responses in the form of Yes or No) were analyzed by software SPSS V.20. The results are shown in following table 6.

Table 6: Respondents Qualification

S.No	Benefits	Yes	No	Total
1	Reduce Project Duration	21(61.8)	13(38.2)	34(100)
2	Reduction in waste	30(88.2)	4(11.8)	34(100)
3	Smooth Work flow	25(73.5)	9(26.5)	34(100)
4	Visual control	28(82.4)	6(17.6)	34(100)
5	Improved safety	25(73.5)	9(26.5)	34(100)
6	Improve Project Quality	24(70.6)	10(29.4)	34(100)
7	Client's Satisfaction	29(85.3)	5(14.7)	34(100)
8	Improved flexibility	21(61.8)	13(38.2)	34(100)
9	Proper Scheduling	26(76.5)	8(23.5)	34(100)
10	Defects reduction	19(55.9)	15(44.1)	34(100)
11	Standardizing work	22(64.7)	12(35.2)	34(100)
12	Simplifying Work	22(64.7)	12(35.2)	34(100)

13	Reduction in cost	20(58.8)	14(41.2)	34(100)
14	Proper Estimation	26(76.5)	8(23.5)	34(100)
15	Greater Profit	15(44.1)	19(55.9)	34(100)
16	Improved Communication	29(85.3)	5(14.7)	34(100)
17	Minimize Inventory	17(50.00	17(55.0)	34(100)
18	Control Budget	24(70.6)	10(29.4)	34(100)
19	Proper Task Management	28(82.4)	6(17.6)	34(100)
20	Simplify Data organizing	23(67.6)	11(32.4)	34(100)
21	Stress Free Working Environment	27(79.4)	7(20.6)	34(100)

Results shows that Reduction in waste is on 1st rank, Client's Satisfaction and Improved Communication are on 2nd rank and Proper Task Management and Visual controls are on 3rd rank.

5. Conclusions

This study consisted of two objectives, evaluating the current and beneficial status of Lean Construction Management techniques. These objectives are achieved and explained in the earlier chapters. The summary of all the findings is explained in following sections.

5.1 Identifying Status of Lean Techniques

The first objective was to identify status of lean tools & techniques. This objective was achieved through literature review. With a review of 80 published research papers, a total of 20 lean techniques were selected. From the analysis of data collected it is concluded that Pull Approach is ranked on 1st position with respect to their implementation, it means in construction industry Pull approach planning (a part of last planner system) is widely used. On 2nd position lean tool Work standardization is most implemented & on 3rd position Fail Safe for Quality is most used in construction sector.

5.2 Identifying Beneficial Status of Lean Construction

Results shows that Reduction in waste is on 1st rank, Client's Satisfaction and Improved Communication are on 2nd rank and Proper Task Management and Visual controls are on 3rd rank. Waste is extra activities that consume time with non-value added activities that are the main reason of extra project cost rather than their actual budget that's why Profit content is greater in lean construction management techniques than the traditional methods of construction management.

5.3 Limitations ofthe Study

Limitations of the study are related with respondents, the more the respondents, more accurate results. The data collected were gathered keeping in view all the sections of the construction projects such as design, structural, material, survey and management section, so all the respondents were not expected to have full understanding of every section because every respondent will be expert in the factors which he daily exercises only that is why he will be giving accurate response to those factors which are related to his section.

6. Recommendations

From the results it is clear that almost all lean tools & techniques are used in construction industry, but due to lack of supports from construction organizations and less researches in lean construction they are not fully implemented as they are functioned. Many respondents using these techniques are not fully aware of their specified functions for that they are made. Although still Lean construction is in its beginning stages so large efforts are required to implement Lean techniques properly. For future recommendations to assess the Lean benefits fully, organizations are suggested to organize Lean workshops, seminars and meetings to achieve the theoretical functions and benefits of lean techniques.

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Evaluation of Chilled Beams System as a viable Alternative to the Variable Air Volume System in the United States

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Abstract

This paper provides a thorough understanding of chilled beams system as a highly efficient, low maintenance and energy saving alternative to the conventional variable air volume (VAV) systems. Chilled Beams system consists of the chilled water pipes which are installed to the ceiling to provide cool air in a building. As opposed to the conventional HVAC system water is used instead of the air for cooling. This paper presents different types of the chilled beams systems, discusses the history of the chilled beams system briefly, and explores its advantages and disadvantages.

Keywords
HVAC, Variable Air Volume Systems, Chilled Beam System

1. Introduction

In most projects being built in the United States the conventional Variable Air Volume (VAV) system is being used. While the conventional VAV system is the most popular and commonly used HVAC system in the United States, this does not mean that it is easy to implement. Many projects during the construction period go through various challenges due to the conflicts between the ductwork layout, precast joists, plumbing piping, fire protection system, as well as light fixtures, and some other scopes of work. To accommodate one or the other system and avoid the conflict between them, relocation of the piping, lowering ceiling heights and complete redesign from the architect may be required.

The main objectives of this paper are to introduce the chilled beams system and compare it to the conventional VAV system; demonstrate advantages and limitations from the constructability, energy efficiency, cost analysis and schedule impact perspectives in comparison with the conventional VAV system.

2. Brief Background on Chilled Beams System

Chilled Beams system consists of the chilled water pipes, which are installed to the ceiling to provide cool air in a building. As opposed to the conventional HVAC system, water is used instead of air for cooling.

Chilled Beams can be passive or active. Also, multifunctional (or multiservice) chilled beam system, which is a chilled beam system combined with various other systems (lighting, smoke, occupancy detectors, fire alarm, fire protection, etc.) is available on the market. Chilled beams can be installed exposed or can have a lay in application.

Passive chilled beam system does not have an air supply and is intended just for the heat removal from the space. Figure 1 below demonstrates the principle how the passive system works and shows how the passive chilled beam look like. Warm air from heat sources (people, equipment, etc.) go up naturally to the ceiling space. As the warm air contacts cooling coil, the air is cooling down and is dropping back to the space, heat is removed, which causes the air to drop back into the space as it is becoming more dense.

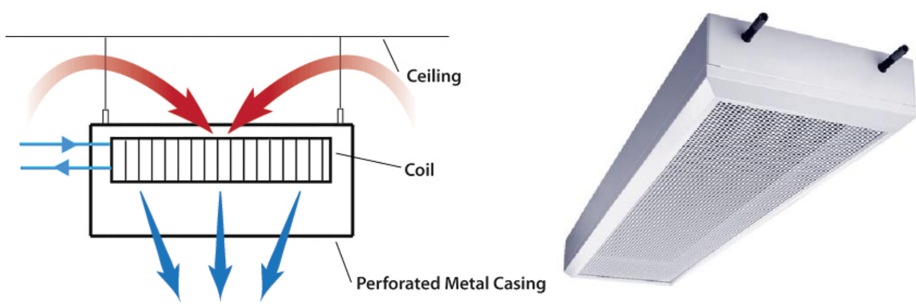

Figure 1: Passive Chilled system (Passive Chilled Beams)

Active chilled beams system, as opposed to the passive, use a ducted primary air supply (1), which is coming through a series of nozzles(2) ,creating the induction of room air(3) that is coming through the cooling coil (4) . The primary air and the air in the room are then mixed and discharged to the space. Figures 2 illustrates the principle of operation and show the look of a typical active chilled beam.

To give an understanding that the system is not relatively new, brief course to the history of the chilled beams introduction is needed. Chilled beams system is relatively new to the United States; even it has been used in Europe since 1975, so it is known for more than 40 years. In various resources, authors state that chilled beams were developed in Norway in 1975, but the history goes back to the 1940s-1950s, when Gunnar Frenger, Norwegian Engineer, created and patented a device, which consisted of the pipe attached to the aluminum profile to provide radiant temperature control. In the late 1960s the first application of the radiant ceiling took place in Gothenburg, Sweden. Chilled beams started resemble current chilled beams system after the concepts for the radiant cooling by Gunnar Frenger and HVAC induction units by Willis Carrier were combined and installed for the first time in 1972 in Gothenburg. In 1986, in Stockholm, Sweden, the chilled beam system that could be considered as the predecessor to current passive chilled beams was installed. (Chilled Beams: What They Are, Why You Should Use Them)

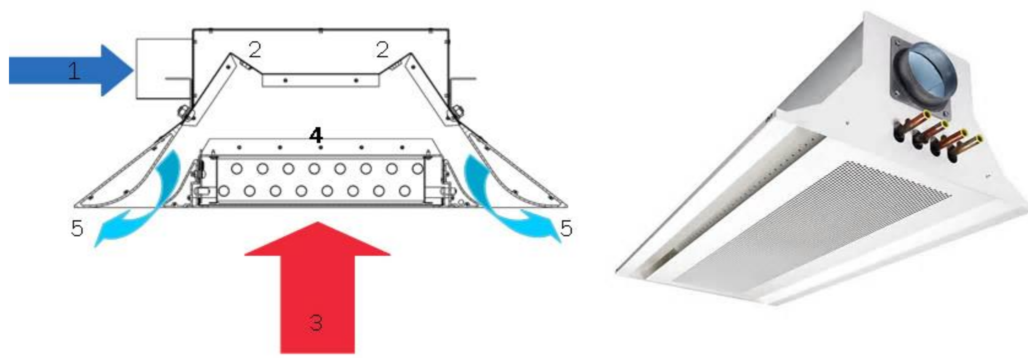

Figure 2: Active Chilled system (Active Chilled Beams)

Through the years since the system was implemented it became rather popular in Europe. It is not widely known in North America, but in the past 10-15 years it was implemented in some of the projects, and various advantages were noticed and system is becoming utilized as an alternative to the conventional HVAC system.

3. Advantages and Limitations

As it was stated before the system is relatively new to North America, but it is known in Europe for decades, so based on the experience of European countries and North America, the following advantages of the chilled beams system were discovered:

First of all, as far as energy efficiency, it was determined that chilled beams system requires less energy for the fans to operate, as the supply air flow rates are reduced compared to the conventional HVAC system (Murphy).

Secondly, chilled beams are significantly smaller in size than the traditional system of the ductwork, fans, FCU(fan coil units), AHU(air handling units), which drastically can reduce the mechanical rooms floor space, allows to reduce the above ceiling space, which is normally used for the mechanical, electrical, plumbing, fire protection systems, where mechanical system is taking more than 50%; and overall chilled beam system can decrease the height of the structure, as there system is rather compact and does not need significant amount of space above ceiling. So system can significantly reduce the overall construction price. (Kevin M., Jason Leffingwell and Ken)

Also, through the evaluation of the project life cycle it was evaluated that maintenance cost for the chilled beams system is much lower, as chilled beams do not require motors, fans, damper actuators or control devices for the operation. Most chilled beams systems available on the market do not require filters and therefore will not need regular filter changes and coils and surfaces are easy to clean, only once in every 1-5 years(depending on the use of space), as stated in the Chilled Beams application guidebook by Federation of European Heating, Ventilation and Air-conditioning Associations. (Chilled Beam Application Guidebook Presentation)

Last, but not least, chilled beam system provides more comfort for the occupants, as system noise levels are lower due to lower rate of airflow and no VAV boxes are required for the operation. (Energy Design Resources)

Even there are the advantages to incorporate chilled beam system in the project there are certain limitations and resistors:

One of the main limitations is that system is not well known by the engineers and there is a lot of the resistance to incorporate it in the United States.

Also, building need to be properly sealed in order to maintain the humidity levels, in other cases if the higher humidity is observed in the building it can potentially lead to the condensation of the chilled water pipes. Especially, extra care needs to be taken in the humid climates. (Murphy)

Besides the other two aspects already mentioned, there is a limitation of the heating in the active chilled beam system; passive system does not even have a feature of heating. So this system need to be considered for moderate or warm climates, where excessive heating is not required or the system can be used just for the cooling during the warmer periods and another system will need to be introduced for the heating.

As soon as the entire system is composed if the piping filled with water, the system need to be completely sealed to make sure that there will be no leaks in the system, as it will cause damage to the building and mold growth.

4. Chilled Beams System in Comparison with the conventional HVAC system

If two systems will be compared it can be noticed that primary diffence between the two is the energy consumption and better performance, as chilled beam system uses less energy because, first of all, the duct is significantly smaller(by 75%), as the cooling efficiency of water is higher than of the air , and as illustrated on the figure 5 below water pipe, 1” in diameter, can transport the same cooling energy as the 18” square air duct. (Rumsey);

Secondly, all the air supplied to be building at 68 degrees F, which avoids the need for the reheat coil, which does exist in the VAV system, as mixed air is supplied at 55 degrees F.
Third, as the smaller duct is used in the chilled beam system, less horizontal space is needed for installation, it allows to reduce the size of the equipment and permits to reduce the floor-to-floor height, as the above ceiling space will be reduced.

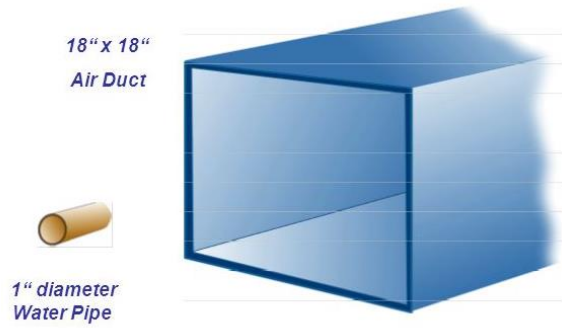

Figure 3: Cooling capacity comparison(air duct vs. water pipe) (Gebrüder Trox GmbH)

All the differences described above for two systems can be summarized in Figure 4 below.

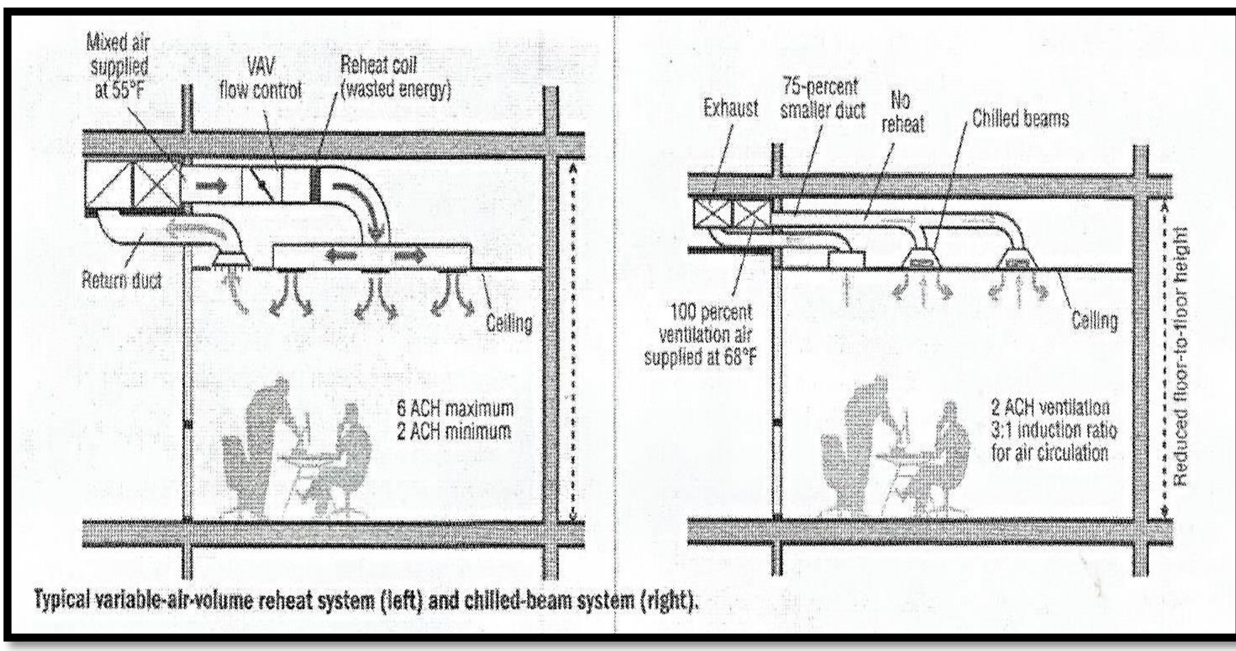

Figure 4: VAV systems vs. Chilled Beam System (Rumsey 2010)

Chilled Beam system is not widely spread in the United States due the resistance of engineers to incorporate the system in the design and in some cases higher upfront costs to install the system.

Even the chilled beam system technology can be more expensive than a VAV system per the sq. ft., but the system need to be evaluated through the entire life cycle of the building and the building structure and cost savings can be found in the following aspects:

First, chilled beam system utilize less ceiling space and the equipment size is smaller than the VAV system, so it allows to reduce the floor space for the mechanical rooms and the overall floor-to-floor height (savings on the materials – concrete, masonry block, stucco, paint, etc)

Second, maintenance costs are lower than the all-air system. As illustrated in the Figure 5 below retrieved from Chilled Beams application guidebook by Federation of European Heating, Ventilation and Air-conditioning Associations, life cycle maintenance costs are almost 19 times lower, assuming twenty (20) years life cycle for the Fan coil units and chilled beams. (Chilled Beam Application Guidebook presentation)

	Fan Coil Unit	Active Chilled Beam
Filter Changes:		
Frequency:	Twice Yearly	NA
Cost per Change:	$30.00	
Cost over Lifetime (20 Years):	$1,200.00	$0.00
Clean Coil and Condensate System:		
Frequency:	Twice Yearly	Every four Years
Cost per Event:	$30.00	$30.00
Cost Over Lifetime:	$1,200.00	$150.00
Fan Motor Replacement:		
Frequency:	Once during life	NA
Cost per Event:	$400.00	
Cost Over Lifetime:	$400.00	$0.00
Life Cycle (20 years) maintenance cost:	$2,800.00	$150.00

Source: REHVA Chilled Beam Application Guidebook (2004)

Figure 5: Life Cycle Maintenance Costs Active Chilled Beams vs. Fan Coils

Also, Chilled Beams System use less energy to operate, as they do not have any motorized parts or fans, so the building will be using less energy for the mechanical system and operational costs for the entire life cycle will be reduced.

Chilled Beams system became an alternative to the conventional HVAC system and the main applications, where chilled beams are a better application are:
First of all, spaces with high space sensible cooling loads, relative to the space ventilation and latent cooling requirements. These spaces can be represented by heat driven laboratories (biological, pharmaceutical, electronic and forensic), brokerage trade areas(heat gains from multiple computers and terminals), broadcast and recording studios, hospitals, where patient rooms require higher ventilation rates and constant air flow.

Also, buildings with the limited height, as there are zoning restrictions which need to be followed. Chilled Beams would be a great application for such a problem, as they help to reduce the above ceiling height and reduce the overall floor height.

5. Main Findings, Conclusions and Future Outlook

To summarize the main findings presented in the paper, there are various advantages of the system and it should be considered as an alternative to the traditional HVAC system. To recap the advantages and disadvantages, chilled beam system has lower operational costs and allows saving on the energy up to 30 percent. Such a system is quiet, efficient and requires minimal maintenance to the system. System is occupying minimal space above the ceiling and allows reducing the overall building height, and due to the size and easier installation requires less coordination between the trades and construction process face with fewer issues. The system is not perfect and disadvantages should be considered as well before the implementation of chilled beams system to the project. The system is typically 5-15% more expensive than the traditional VAV system and in various cases is not considered, especially when the project will be turned over to the tenants who will be the primary users and will face the benefits. System cannot produce a lot of heating and in various climate zones will require additional system to achieve heating effect, also various engineers are concerned with the humidity in certain occupancies and climate zones, as proper insulation and seal of the building is required to make sure the system will be operational and no condensation will occur. Chilled beam systems can be a great system in the right setting; however, the construction team, architects and engineers need to be certain that the system will work as intended when selected for an area. If this system is placed in an area that it is not designed for, the expected savings will quickly diminish.

The benefits of the system described in the above sections will bring a better performance of the HVAC system and will help reduce the energy consumption of the building, which will reduce the CO2 emissions. There are certain limitations to the system, but various manufacturers are developing new technologies to improve the existing ones and make them better. Chilled Beams System is a common practice in Europe and Australia, and in some countries, such as Germany, known for the best sustainable buildings; chilled beams are mandated for the HVAC application. Chilled Beams System is not widely popular in the United States yet, but is implemented on some of the projects. If the practice of incorporation of the chilled beam in the design will be more common and designers will be willing to get more familiar with the system and analyze the benefits, it can become a widely used system, oriented on the energy efficiency, sustainability, cost savings and schedule acceleration. Also, taking into account current conditions with the environment and threat that human activity is causing a lot of harm, every owner developing a new or renovating existing facility need to take into account alternative construction methods, equipment and technology, which will be more environmentally friendly.

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HYDRAULIC MODELING OF K-IV WATER CONVEYANCE SYSTEM (PHASE-I)

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Abstract

Karachi faces shortage in water supply due to an exponential increase in population. Therefore, K-IV project is designed for supplying additional 260MGD water. The objective of the study includes estimation of losses associated with the supply structures and determining optimum flow by varying supply conditions. PCSWMM (SWMM 5.0.012) model was used to estimate losses induced due to evaporation, and to determine change in flow due to varying geometry. Model was calibrated using depth and velocity data from Hub canal. The results obtained included maximum variation in depth of 0.163m and velocity up to 0.209m/s, due to roughness, and entry and exit losses. An evaporation rate of 1.21mm/day resulted in a loss of 1.3MGD from the system. The results also show that increase in manning's roughness above 0.021 will cause flooding at the inlet channel. Proper maintenance of the channel should be kept for avoiding flooding. The study will also help in determining any additional amount required, and taking measures for maintaining a constant flow of 13.68m^3/s available at the outlet to cater the demand of the city.

Keywords
Hydraulic Modeling; PCSWMM; Evaporation; Expansion-Contraction Losses; K-IV

1. Background

One of the alarming issues in Karachi city is water shortage and with rising temperatures of the city due to climate change and urbanization have enhanced the effects of shortage in water supply. Water demand for the city is 1,100 MGD (KW&SB), only 650 MGD is supplied to it. In 2008, KW&SB proposed a new water supply project Greater Karachi Bulk Water Supply Scheme K-IV stretched over a length of 121 km with 90 km of open channel, while the rest is a combination of pipelines, siphons and conduits.

As the system comprises mostly of canal sections and is based mainly on gravity flow, it is essential to determine whether the system would be efficient enough to cater its designed supply. Thus, creating a hydraulic model of the system provided an assessment for the designed parameters of the canal and information on designing cross drainage, siphon flow, and flow through unpressurized conduits.

The main objective of the study is Hydraulic modeling of K-IV water supply project (Phase I), estimation of different losses associated with the supply structures. The modeling process in the study included determination of evaporation losses. Determining effectiveness of conduits at transition points for frictional and other losses. To check compliance of final capacity to the designed capacity and to check velocity at each section for breaching and self-cleansing.

2. Input Data

PCSWMM (SWMM 5.0.012) model the drainage system under different environmental compartments; Our study area focuses only on the transport compartment which uses the predefined direct flow to model the system. The input data required includes visual data such as junction node, outfall node, conduits, alignment and pump. This data is essential for the formation of the K-IV system. The alignment file of K-IV developed through ArcGIS is added to the system interface. This line included points indicating change of slopes, structures, sections and other facilities like storage tanks and transition chambers. The centreline also shows each chainage throughout the system. Junctions are used at siphons bends and at varying slopes, surcharge depth is provided at junctions to act as bolted main holes to prevent pressure loss. The principle input parameters for a junction are Invert Elevation, Height to ground surface and Surcharge depth. At the starting junction, a user defined direct inflow was introduced. The deviating factor was assumed “1”, as the system had to be modeled against constant flow. Non-visual objects are necessary to describe the additional characteristics and processes in the system like evaporation, infiltration etc.

3. Computational Method

PCSWMM uses SWMM engines to model the system, it implies conservation of mass, momentum and energy equations to simulate flow.

3.1 Flow Routing

In SWMM flow routing is governed by Saint-Venant flow equation (Rossman, 2010) which used the conservation of mass and momentum equation for gradually varied and unsteady flow. Further SWMM provide user a choice for different flow routing options which are differentiated according to their complexity. Our study will go through dynamic wave routing.

3.1.1 Dynamic wave routing

Dynamic wave routing account for all such phenomena that can occur in our conveyance system such as backwater, flow reversal, pressurized flow and exit/entrance losses. For flow along a conduit the Saint-Venant equations are shown, equation 1 shows the continuity and equation 2 shows the momentum.

$$\frac{dA}{dt} + \frac{dQ}{dx} = 0 \qquad \text{(Eq. 1)}$$

$$\frac{dQ}{dt} + \frac{d(\frac{Q^2}{A})}{dx} + gA\frac{dH}{dx} + gAS_f + gAh_L = 0 \qquad \text{(Eq. 2)}$$

Where,
x = Distance along the conduit
A = Cross-sectional area
Q = Flow rate
H = Hydraulic head of water in conduit
S_f = Friction slope (head loss per unit length)
g = Acceleration due to gravity
t = Time of flow
h_L = Local energy loss per unit length

3.2 Flow Rate (Q)

Manning equation is used to generate the flow rate relationship. The flow rate equation using manning equation is shown in equation 3, While for the pressurized flow that is force main circular pipe, Hazen-William formula is used. The flow rate equation using Hazen-William formula is shown in equation 4.

$$Q = VA = \left(\frac{1.49}{n}\right) AR^{\frac{2}{3}}\sqrt{S} \qquad \text{(Eq. 3)}$$

$$Q = AV = 1.318\ C\ A\ R^{0.63}S^{0.54} \qquad \text{(Eq. 4)}$$

Where,
A = Cross-sectional area
S = Conduit slope or friction slope
n = Manning coefficient
R = Hydraulic radius
C = Hazen-William C-factor

4. Modeling Process

The excel file was produced and used as data source for the GIS interface, the input parameters were referenced to form a centre chainage line for the system. The developed centre line was split into elements of known length according to the system requirements. Next step was to Position each element to be marked on the centre line for determining the precise location of it in the system. The shape file developed, was used as an input for the PCSWMM Engine. The file was used as a background for proper marking the system lengths. The junctions were marked on designated locations, followed by the conduits, siphons and canals. Cross sections details of all conveyance system elements were added. Junctions invert elevations were allotted accordingly to attain the designed slope and elevations. Contraction and expansion coefficients were assigned to every conduit expected to experience the phenomena. Evaporation value in (mm/day) was calculated from the metrological data using Dalton's law of evaporation. Which was further entered in climatology editor of PCSWMM to get total evaporation in MGD. Direct inflow of 13.68 m^3/s was assigned at inlet (0+00) junction in uniform pattern with period of simulation varying between 96, 48 and 24 hours. The routing step throughout was set to 5 seconds to reduce the routing error. Sensitivity analysis was conducted to determine the parameter sensitive to flow, velocity and depth. Sensitivity-based Radio Tuning Calibration (SRTC) was used for calibration and validation of model. Scenarios were created for optimal condition and by varying manning coefficients between 0.01 and 0.014. Flooding at inlet junctions was checked for the maximum manning coefficient of 0.022 and flow loss from the system at inlet junction was computed. Flow, depth and velocity were computed at nodes and links throughout the system.

4.1 Model Development

The data required for model development such as the centre line coordinates, the cross-section geometries, invert elevations, surcharged, depth, slopes and inflow parameters of intake were extracted from Design report of K-IV conveyance system.

4.2 Model Simulation

The simulation period is set at different intervals by varying routing step from 1sec to 5sec to control the routing error. The inlet baseflow was set to a constant value of 13.68 m^3/s with a uniform pattern to account for the constant supply available at all intervals. The flow characteristics through the junction were made like the actual system, the simulation method was set to dynamic wave routing while Hazen-

Williams equation was used for force-mains. The entry and exit losses accounting for variation due to expansion and contraction were set as 0.1 and 0.3 respectively.

4.3 Sensitivity Analysis, Calibration and Validation

The model was simulated to check sensitivity against manning coefficient, exit and entry losses parameters. The calibration process synchronized the model output, in which parameters sensitive to the model were adjusted so that it can reasonably represent the actual conditions. The model calibration is a process of parameter refinement, through comparing simulated and observed values of interest. The SRTC tool was used to calibrate the model against observe data of interest, collected from Hub Karachi canal. Data collected for calibration included Velocity and Depth. The automatic tuning scale adjusts the related parameters and performs multiple SWMM runs to validate the model. Model validation ensures that the calibrated model assesses all the variables and conditions affecting the model results. The PCSWMM auto calibration helps in reducing the extra effort requirements of model calibration.

5. Model Performance Criteria

PCSWMM was also used to compute, analyse and grade errors for statistical analysis of observed and simulated data. The following performance criteria were calculated,

- ISE Rating {Excellent (0-3), Very Good (3-6), Good (6-10), Poor (10-25)}
- Nash–Sutcliffe efficiency (NSE)
- Coefficient of Determination (R^2)
- Root-mean square error (RMSE)

NSE quantifies model result on basis of simulation and is found by subtracting ratio of sum of square of model errors to the total sum of squares, from unity. both NSE and R^2 range from 0 to 1, with higher values indicating a better agreement between the observed and simulated data. For RMSE lower values indicate better fit with zero indicative of a perfect fit.

6. Evaporation

The evaporation through open canal will result in loss of water from the system. The average value of evaporation in mm/day was calculated using Dalton’s law of evaporation for open surface as presented by Huffman et al. (2012), the result of which were utilized as an input value for PCSWMM. Hence determining the total loss of water from the system in million gallons per day (MGD). Equation 5 shows the evaporation in mm/day. The value of C is evaluated by Rohwer et al., (1931).

$$E = C(e_s - e_a) \qquad \text{(Eq. 5)}$$

Where,

E = rate of evaporation (mm/day)
C = constant (mm day^{-1} kPa^{-1})
e_s = saturation vapor pressure at the temperature of the water surface (kPa)
e_a = actual vapor pressure of the air (e_s of the air times relative humidity) (kPa)

7. Results

The results through the simulation of PCSWMM model for K IV system are presented. The interpretation of these results and the behaviour of the system for the assigned flow condition are also discussed. All the parametric values are input in the model for simulating balanced flow. The coefficients were so adjusted to maintain the flow conditions such that the physical conditions of the system should remain intact.

7.1 Flow Lag

Multiple simulation was carried out by varying routing time and sweeping period. The results so obtained showed greater accuracy in terms of flow destabilization with decrease in routing time while increase in sweeping time does not seem to have any effect on the flow condition except for it becoming steady over time. The water arriving at the outfall is found to be 31 hours and 37 minutes short in time for the first flow. which becomes comparable to the principle inflow at a lag of 39 hours and 30 minutes and becomes equal at about 51 hours and 30 minutes pass the initial intake, although it will be unstable throughout.

7.2 Variation of Manning's coefficient

The variability of different flow components with roughness are discussed

7.2.1 Outflow

The outflow results by varying manning's coefficient for the concrete and steel sections showed variation at the outfall. While the inflow was kept constant at 13.68 m^3/s. outflow variation with manning's coefficient are shown in Table 1.

Table 1: Variation of Outflow in Manning's Coefficient

Manning's coefficient		Inflow (m^3/s)	Outflow (m^3/s)
Concrete	Steel		
0.014	0.01	13.68	13.67
0.014	0.014	13.68	13.636
0.01	0.01	13.68	13.671
0.015 (calibrated)	0.01	13.68	13.561

7.2.2 Velocity

The velocity profile for the inlet and outlet links showed at a sizeable decreased in the velocity under normal conditions of manning's coefficient the velocity dip was about 0.199 and a drop of 0.201 under increased manning's coefficient. The change in drop was recorded at 0.233 when the coefficient was decreased to 0.01. The results show a maximum velocity of 1.415 m/s at inlet while 1.216 m/s at outlet links.

7.2.3 Depth

A gradual increase in depth of flow throughout the system was observed. The quantitative difference in depth at inlet and outlet by the manning coefficient of optimum conditions and at extreme high and lows are calculated and found that the maximum depth will be attained at optimum conditions. The average increase in depth between the both end links is 0.448m.

7.2.4 Gravitational Flow Analysis

The results determined from the model shows appropriate amount of energy head available throughout the system. The Energy Grade Line (EGL) and Hydraulic Grade Line (HGL) plots also confirms the reach of the gravitational flow. The height of EGL determines the surcharged depth that is to be provided for the junctions to be considered as bolted in the model, both EGL and HGL attains maximum point and becomes stable for the further simulation period.

7.3 Working of Siphons

The velocity profile of the siphon shows a sudden increase from the previous element in the system due to abrupt change in gradient this increase in velocity results in full flow through the siphon. There is enough EGL and HGL available at transition chambers located at both ends of the siphons to cause the flow of water against gravity.

7.4 Expansion and Contraction Losses

The expansion and contraction losses which were assigned in terms of entry and exit losses in the software are ranged from 0.1 to 0.3 in the cases of contraction and expansion respectively. The results obtained exhibited an increase in velocity with simultaneous decrease in depth with respect to optimum condition Variation in depth, flow and velocity with losses are shown in Figure 1, Figure 2 and Figure 3 respectively.

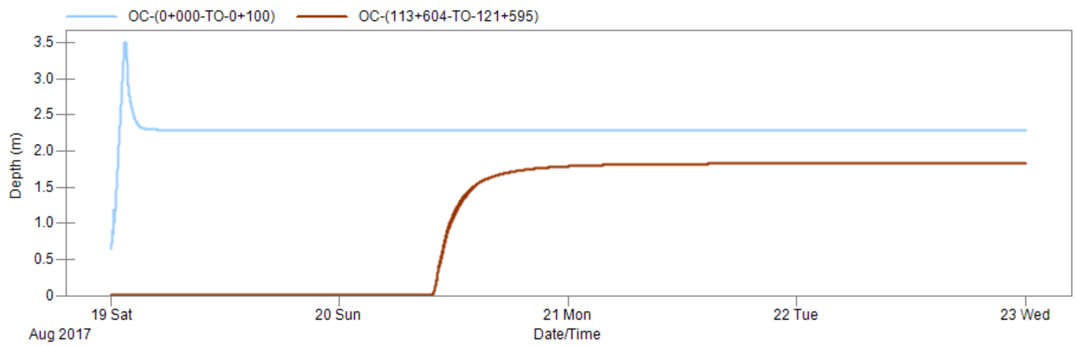

Figure 1: Depth Profile Considering Exit and Entry Losses

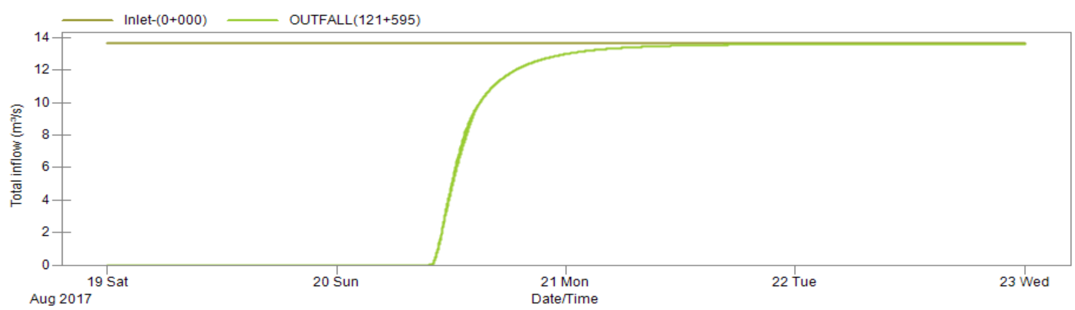

Figure 2: Flow Routing and Roughness Lag Considering Exit and Entry Losses

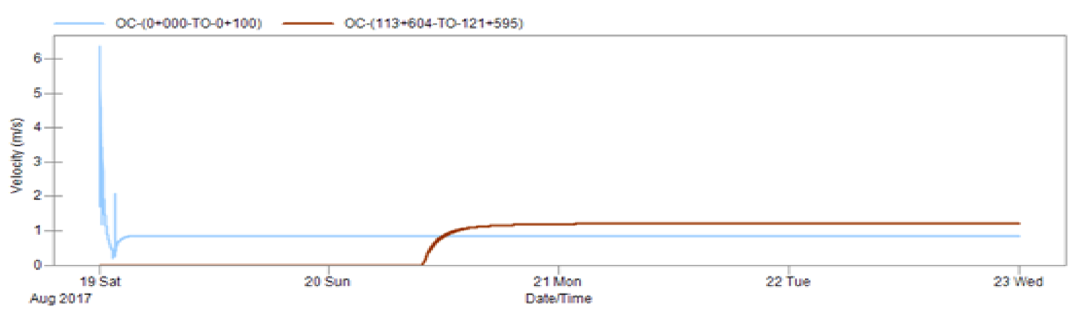

Figure 3: Velocity Profile Considering Exit and Entry Losses

The quantitative difference in depth, flow and velocity at inlet and outlet by the exit and entry losses are calculated. The maximum decrease in depth to 1.826 m and maximum increase in velocity to 1.214 m/s is observed at outlet link when considering expansion and contraction loses.

7.5 Evaporation Estimation

The calculation of average evaporation of 1.21 mm/day by Dalton's law of evaporation at an average temperature of 36ºC. The final total evaporation losses are estimated at an average 1.33 MGD.

7.6 Calibration Results

The model is calibrated against manning coefficient, exit and entry loss coefficients. The data is collected from Hub canal but due to the poor management and maintenance issues canal bed is mostly deteriorated which leads to a 2 MGD water is loss from the Hub canal system. The observe results from SRTC tool are shown in Table 2 and the generated performance criteria before and after calibration are shown in Table 3.

Table 2: Calibrated Input Parameter Value

Manning Coefficient	0.015
Entry Loss Coefficient	0.103
Exit Loss Coefficient	0.308

Table 3: Performance Criteria Results

Error	Current (Simulated)	Calibrated
NSE	0.346	0.898
R^2	0.96	0.96

The calibration results were replaced and the model was run again. The variation in flow, velocity and depth at inlet and outlet after calibration are shown in Figure 4, Figure 5 and Figure 6 respectively.

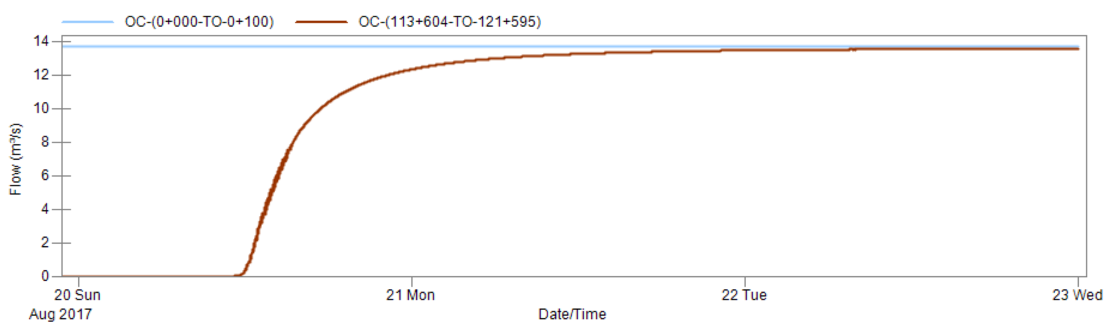

Figure 4: Flow Profile after Calibration

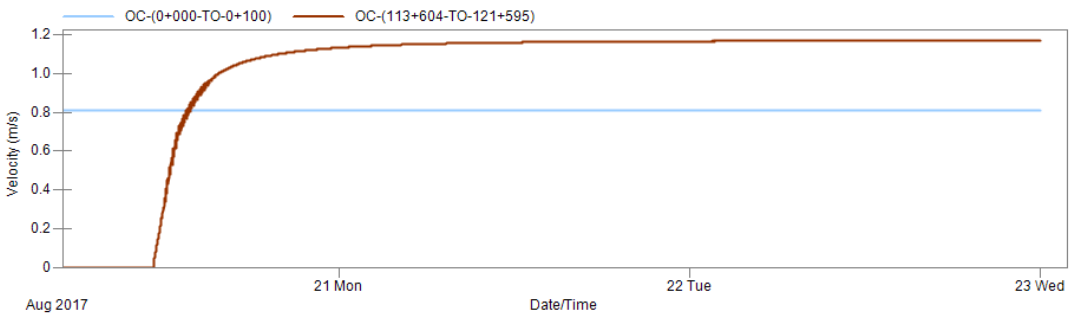

Figure 5: Velocity Profile after Calibration

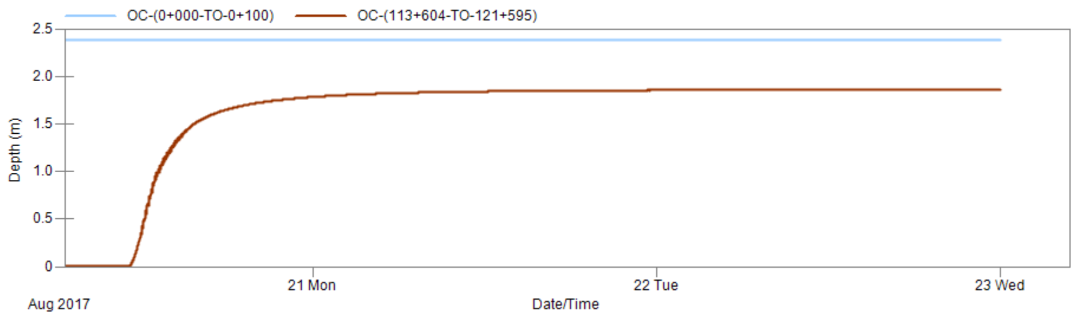

Figure 6: Depth Profile after Calibration

8. Conclusions

The flow through the system is unsteady and will cause variable outflow. The expansion and contraction causes the flow to decrease in depth with increase in velocity to a point where self-cleansing velocity is attained, but within the range that will not cause any deterioration of canals or bursting in conduits or siphons. The roughness of the system will only cause flow variation with time and would not be enough to cause any flooding at intake point. The evaporation losses may range from 1.3-1.8 MGD and should be incorporated at the inlet, Deviation from optimum conditions may cause a variation of up to 0.121m^3/s at the outlet resulting in a variable supply.

9. Recommendations

Maintenance of the conveyance system should be done so that the roughness may not raise to a level to cause flooding in the open channels causing minimum or no water at the outlet. Maintenance will also help in stopping the growth of algae and plants in the canal which will obstruct the flow hence reducing the velocity up to a critical level causing silting and hence reducing the capacity of the canal. Pumping should be maintained at constant head to avoid any bursting of conduit due to extra flow or backflow due to low head. Flow should be increased at the inlet to account for the evaporation and line losses thus maintaining a flow of 260 MGD at the outlet.

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Influence of Sulphate Reducing Bacteria on Corrosion Rates of Buried Cast Iron Pipes

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Abstract
Microbial corrosion is a severe concern for owners and asset managers of buried pipes. Among the various types of bacteria, sulphate reducing bacteria (SRB) are by far one of the most extensively reported in relation to microbial corrosion. The influence of SRB on corrosion rates of buried pipes is a matter of prime significance among researchers due to their peculiar localized impact in the form of pits. Several research studies have been conducted on finding the effects of SRB on buried pipes. However, the literature suggests that these studies are conducted in culture medium where bacteria can easily be grown, but not replicating the soil environment. Hence, a gap exists in finding the influence of SRB in the real soil medium where most of onshore pipelines are buried. This paper presents the outcomes of an innovative experimental setup developed for simulating the external corrosion caused by sulphate reducing bacteria (SRB) for buried cast iron pipes. Coated cast iron specimens with one surface remained uncoated are exposed to SRB in clay soils using well-controlled experimental setup. Results revealed high corrosion rates for the cast iron specimens buried in the soil having SRB compared to the specimens immersed in culture medium as reported in the literature. The experimental approach presented in this paper is first of its kind, and the corrosion rates obtained from the test can be used for accurate prediction of failure of buried metal pipes.

Keywords
Cast iron, Coating, Clay soil, Baar's medium, and Corrosion rates.

1. Introduction

Microbiologically influenced corrosion (MIC) is defined as the change in the corrosion behavior of material/metal in the presence of microorganisms (Costerton et al. 1987). Bacteria are attached to the metal surface and form biofilm which degrades the metal surface by changing its physical and chemical characteristic due to their biochemical activities associated with their metabolism, growth, and reproduction (Hamilton 1985, Little et al. 2007). Several researchers have reported MIC as a cause of significant economic loss to maritime, oil and gas, power generation, water distribution system, etc. (Videla 1996, El Hajj et al. 2013).

MIC caused by sulfate-reducing bacteria (SRB) is well reported in the literature (Booth, 1964, Little et al., 2007). SRB are anaerobic bacteria that can be found in oxygen-deficient saturated soils, with a pH from 6 to 8, containing sulfate ions, organic compounds, and minerals and grow in soils at a temperature of 20-30 C^o. Anaerobic bacteria have been reported to increase the corrosion process and convert non-corrosive soil to a very aggressive environment by generating H_2S (Javaherdashti, 1999). However, in well-aerated soils, the effect of anaerobic bacteria on corrosion of buried pipes is not significant. The microbial effect on the corrosion of metals is due to the removal of electrons from the metal, and the formation of corrosion products principally (Javaherdashti 2008) by:

a) The formation of sulfuric acid, inorganic or organic sulfides and organic acids due to the chemical action of metabolic products.

b) Formation of electrochemical corrosion cells due to differential diffusion, concentration gradients, etc. caused by varying oxygen concentration, salt concentration, and pH that increase the possibilities for differential diffusion and concentration gradients, etc., establishing the local electrochemical corrosion cells**.**

Many researchers have investigated the effect of SRB on mild steel corrosion in the saline environment (Melchers and Jeffrey, 2008, Javaherdashti et al. 2006, Raman et al. 2005). Recently, Javed et al. (2015) investigated the role of ferrous ions in initial SRB bacterial attachment and corresponding corrosion of mild steel specimens in both mediums containing Fe and without Fe. The experiments were continued for 28 days, and then corrosion caused by SRB in the medium containing Fe and without Fe was investigated by comparing the weight loss measurements and microstructural studies. The results indicated that there was a little effect due to the presence of Fe in the culture medium and the corresponding corrosion of specimens at the early stage. However, on reaching 28 days, the corrosion of specimens in the presence of Fe was very significant as compared to the specimens immersed in without Fe medium. The bacterial attachment was also investigated at various time periods during 28 days of testing. The study revealed that in conditions where SRB mediates MIC attack, the monitoring of ferrous ion levels may be practically more critical than the enumeration of bacterial cells typically undertaken in field studies. However, the study was conducted only for 28 days, and hence there remains a gap to understand the exact behavior of Fe in culture medium and corresponding corrosion of SRB in long-term exposure conditions.

Moreover, the research related to corrosion of high carbon steel (such as cast iron) in soil is limited. The first MIC of cast iron pipes due to SRB in anaerobic soil was reported in 1934 (Wolzogen Kuhr and van der Vlugt, 1934). Then, after a lapse of almost 70 years, some investigation related to the effect of SRB on corrosion of carbon steel in the soil was conducted, but this study was undertaken for very short time interval (Li et al., 2001). Moreover, they used the ER probe, polarization techniques, and EIS to find the effect of SRB on metal corrosion. These studies concluded that the corrosion rates can increase 20 times than the normal control case in soils. Although this study involved some electrochemical techniques for finding the corrosion of SRB in soils, the results are not well established as the research was undertaken only for few days. Therefore, a gap exists in the studies of corrosion of high carbon steel such as cast iron caused by SRB in soils for longer durations. The objective of the current research is to address this knowledge gap by testing cast iron specimens using an innovative designed anaerobic test setup in the presence of SRBs.

2. Test Setup

2.1 Metal Sample Preparation

Cast iron specimens (dimensions 60x12x6 mm) with composition as shown in Table 1 were used in the experiments conducted in this research. After initial polishing of the samples, the specimens were ground sequentially up to 1200 grit silicon carbide paper for a smooth surface finish. Then the specimens were further polished through a sequence of fine polishing with 9 and 3 µm diamond suspensions to a final 0.04 µm finish using silica suspension (Struers OPU, Australia). The polished specimens were ultrasonically cleaned with acetone for 10–15 min, rinsed with distilled water and ethanol, and then dried under warm air in the mechanical specimen preparation lab. After recording initial weights for mass loss measurement, the specimenspecimens were coated with epoxy except one surface to simulate external corrosion. Before testing, specimens were ultrasonically cleaned with acetone followed by sterilization via immersion in absolute ethanol (100%) and then aseptically dried within a level 2 physical containment (PC2) cabinet.

Table 1: Chemical Composition of the Cast Iron Specimens

C %	Mn %	Si %	S %	P %	Ni %	Cr %	Mo %	Cu %	Ti %	Al %	Mg %
3.58	0.74	2.48	0.06	0.067	0.01	0.01	<0.01	0.02	0.18	<0.01	<0.01

2.2 Preparation of Culture Medium

For the growth of the SRB ATCC 1249 Modified Baar's (MB) medium was used. This medium was prepared using the components: 4.1 g $MgSO_7H_2O$, 1.0 g NH_4Cl, 1.26 g $CaSO_42H_2O$, 0.5 g K_2HPO_4, 5.0 g tri-sodium citrate, 5.67 g sodium lactate, and 1.0 g yeast extract were added for one-litre solution. The pH was adjusted to 7.5 ± 0.1 using a 3.5 M KOH solution and sterilized at 121 Celsius and 103 KPa for 20 min in an autoclave. After cooling, 0.1 ml of 5% $(NH_4)_2Fe(SO_4)_2$ solution which was sterilized by filtration using syringe filters (0.02 lm in pore size), was added to 5.0 ml of the previously sterilized medium. This was equivalent to 195 mg/L of iron ions concentration in the medium. The culturing procedure is shown in Fig 1. The culture was prepared after every 5 days. One ml of inoculum was poured in 50 ml glass tube and then it was sealed from the top to avoid oxygen diffusion. Cultures were incubated at 37 Celsius on a rotary shaker at 110 rpm for four days. 4-5 test tubes were prepared after every 5 days till the quantity reached 500ml of culture with inoculum.

Figure 1: Culturing Procedure

2.3 Preparation of Test Setup

A plastic container with two outlets was used for simulating anaerobic external corrosion. One valve designed for purging nitrogen into the test container and the second valve was used to remove oxygen from it. The container was sealed to avoid or minimize oxygen diffusion into it. First 250 gram of soil (after being autoclaved) was added into the container and then four test specimens were buried into the sterilized soil. Then, 100 ml of the culture medium was introduced into the soil where specimens were buried. One of the specimens was connected with wire and reference, and auxiliary electrodes were also buried in the soil for electrochemical measurements. Soon after introducing the cultured bacteria from the

test tube into the cell containing soil and metal specimens, new soil (~ 250 grams) was added to the container to simulate the underground corrosion. The size of the container along with the experimental assembly is shown in Fig 2. The culture and test setup were put in incubator at 37 Celsius as shown in Fig. 2.
After every 7 days, new soil and fresh culture medium with bacteria were added to the test container. Also, nitrogen was purged into the test container to make the environment anaerobic for the growth of the SRB till 30 days.

Figure 2: Experimental Setup

3. Results and Discussion

After 30 days of corrosion, specimens were removed from the test setup. Upon visual observation, specimens were found to be heavily corroded with some signs of pitting (Fig. 3). As can be seen from Fig. 3, black colored biofilm and the corrosion products have been resulted in the specimens. In order to investigate the significance of bacteria on the resulted corrosion, specimens were analysed under SEM for bacterial attachment. Specimens were then cleaned using Clerk's solution, and mass loss measurements were undertaken. The electrochemical measurements performed are not presented in this paper. The bacterial attachment and the corrosion rates obtained from mass loss measurements are discussed below.

Figure 3: Cast Iron Specimens after 28 Days of Corrosion in Soils Contaminated With SRB

3.1 Bacterial attachment

After visual observation of the specimens taken out from the test setup, 70 % ethanol was sprayed on the specimens to make the bacteria dead. Then, coatings were removed from the specimens. After eliminating coatings, the test specimens were analysed for the bacterial attachment under scanning electron microscope (SEM). FEI Quanta 200 SEM was used for bacterial imaging. Using the spot size of 5, HV of 30 kV and the working distance of 10mm, the image was taken at the magnification of 1500. Fig. 4 indicates the black colored biofilm and white colored objects are the bacteria along with soil particles on one of the specimens. Beech and Gaylarde (1991) found that extracellular polymeric substance (EPS) plays an important role in the initial attachment of Pseudomonas fluorescens and Desulfovibrio

Desulfuricans to mild steel surfaces. Javed et al. (2015) found that the intial SRB cells attachment and their colonies formation were responsible for the corrosion of steel in the culture medium (Javed et. al 2015). From Fig. 4, it can be seen that black colored biofilm is well spread on the specimens and bacteria are in the forms of colonies.

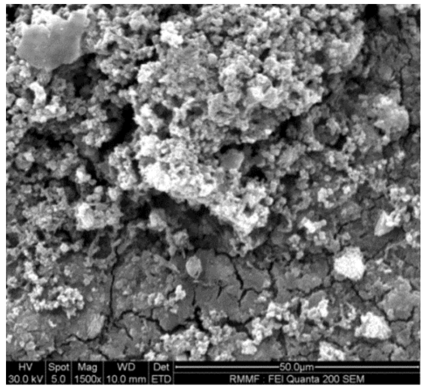

Figure 4: Bacterial Attachment

3.2 Corrosion Rates

The corrosion rate is an indicator of sectional loss due to corrosion, and it is calculated from the mass loss after a certain period of corrosion as describe in ASTM G01 (2014). First, the corrosion products are removed, and then the difference of the mass before and after corrosion is determined. Then, the mass loss is converted to corrosion rates using ASTM G01 (2014). In the current research, the corrosion products on the specimens were removed by immersing them in Clerk's solution for 1 min following ASTM G 01 (2014). Clark's solution was prepared using 20 g antimony trioxide (Sb_2O_3), 50 g stannous chloride ($SnCl_2$) and 1000 mL hydrochloric acid (HCl, specific gravity 1.19) stirred at 23°C for 15 minutes. Specimens were immersed in Clark's solution for 1 min and then are taken out of the solution for corrosion products removal by brushing. Soon after corrosion products removal, the specimens were dried and then weighed to observe the reduction in mass. Mass loss was calculated using Eq. 1 as the reduction in mass of each specimen after immersion.

$$\text{Mass loss} = M_1 - M_2 \qquad (1)$$

where M_1 is the initial mass of the specimen and M_2 is the mass after and rust removal.
After undertaking the mass loss measurements, the corrosion rates were determined using ASTM G 01 (2014). Measuring corrosion rates depends on various factors which are explained by the corrosion rate expression Law as follows;

$$\text{Corrosion rate (mm/year)} = K * W/(A * T * D) \qquad (2)$$

where K= a constant=8.76 x10^4, W= mass loss in gram, T= exposure time in hours, D=density in g/cm^3 of material, A= area in cm^2.

The mass loss measurements were converted to corrosion rates using the Eq. 2. The corrosion rates of 30 days of observations are shown in Fig. 5. It can be seen that the corrosion rates of two specimens out of three are relatively similar. The highest corrosion rate of 138 µm/yr was obtained in the short span of 30 days indicating that destructive impact of SRB on the corrosion of metals. Javed et al. (2015) performed similar experiments on steel using the same SRB as used in the current research in broth culture medium. The maximum corrosion rates obtained from their research was 80 µm/yr almost half of 138 µm/yr obtained in the current research. Although the material was different in their study from the current study, the difference of corrosion rate between the two studies can still verify that the corrosion is caused by SRB. Thus, from the outcomes of the corrosion rates of the current research, it can be noted that the test setup designed for simulating external corrosion due to SRB in soil is effective. The test setup developed in the current research is first of its kind and the contribution in the field of underground corrosion is novel as most of the reported literature was limited to culture medium rather using soil as a realistic medium of corrosion.

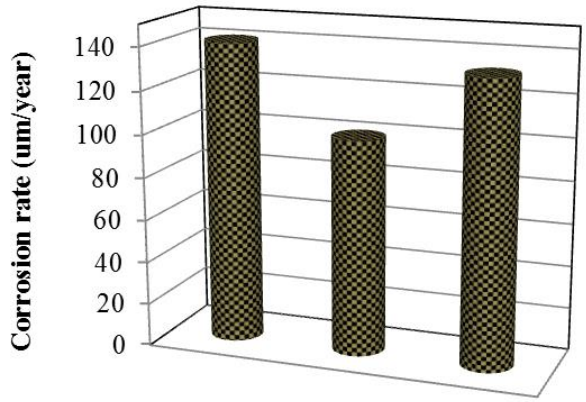

Figure 5: Corrosion Rates Of Cast Iron Specimens In Clay Soil Contaminated With Desulfovibrio Desulfurican

4. Conclusion

This paper presents an innovative test setup and methodology for simulation of external corrosion caused by SRB. In the current research, real soil was used for external corrosion simulation by SRB rather a culture medium which was extensively used in the literature. The bacterial attachment and the high corrosion rate of 138 µm/yr obtained after a short span of 30 days from the specimen indicated the validity of the proposed test setup. The test setup and methodology can be effectively utilized to simulate the corrosion of buried metals in realistic soil medium. The current research is being extended for long duration testing at present to investigate the long-term corrosion and the influence of SRB on the mechanical properties of cast iron metals. These studies will assist in the accurate prediction of external corrosion and the whole life assessments of buried metal structures.

Acknowledgements

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Assessment of Hydropower Potential and Economical Alternatives-A case study of Khanpur Dam Canal

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Abstract
This research considers using the available hydropower potential at canal of Khanpur Dam for generation of electricity and proposes the three alternatives for the installation of machinery required for power generation. Prevalent energy scenario is the biggest problem faced by Pakistan .This was the main motivation behind this research. Problem statement includes the energy crises and the effective ways to encounter the problem. Location of research site and its geological features are included along with the salient features. Topographical sheet for the site is also available. Related data was collected from various agencies. Discharge data was collected for the past 23 years from Khanpur dam authorities. Data was processed using methods like spread sheet analysis and Flow Duration Curves. From Flow Duration Analysis the design discharge value comes out to be 3.7 m^3/sec and the available head at the site is 19m. These values were used to calculate the available hydropower potential at the canal i.e. 482 KW. On the basis of design discharge value and the available head, appropriate turbine was selected and economic analysis for the proposed alternatives.

Keywords: Hydropower Potential, Economical Alternatives, Khanpur Dam

1. Introduction

Pakistan does not have large proven reserves of fossil fuels but it is fortunate to have been blessed with significant hydropower potential. The gross hydropower potential of Pakistan is about 55000 MW. The total potential of Indus River is about 3860 MW and its tributaries more than 5726 MW while the power generation is at present hardly 13% of the total potential. There are a number of suitable sites available on Indus and Jhelum which can be developed to meet the growing demands of the country for decades to come. Underutilization of this sector has led to severe energy crisis in our country which has severely hindered our growth and progress.

Hydropower is also significantly cheaper than other forms of energy. Hence its exploitation can prove to be beneficial to country's need of energy resources.

The goal of a developed and prosperous Pakistan cannot be achieved without finding a permanent solution for the prevailing energy crises. The large scale thermal power plants are not the solution to our energy woes as they require very high initial investment and are also at risk in the event of a natural disaster, the recent example being the Fukushima Power plant in Japan. The hydropower in Pakistan is the mean of energy generation that has minimum environmental hazards and significant part of the available power potential is economical.

Luckily our country Pakistan is gifted by enormous hydropower potential which can be utilized to fulfill the ever increasing demand of energy at cheap prices. The long term and the most viable solution to this problem is to make use of the available hydropower potential and generate substantial amount of electricity which will not only cater our needs but the enormous amount of hydropower potential can also be utilized to produce electricity for exporting it to our neighboring countries which will draw in a substantial amount of revenue.

2. Objectives of the Study

1. To calculate the available hydropower potential at canal of Khanpur Dam
2. To propose alternate layouts of the required components for power generation.
3. To analyze economic feasibility of the proposed layouts.

3. Description of Site

Khanpur Dam Project derives its name from the village Khanpur located at 8 miles north of Taxilla on Haripur road and about 25 miles from Islamabad, the Capital of Pakistan. The dam has been constrcuted in a narrow gorge on Haro river downstream of the confluence of Haro and tributary Nilan Kas. It is a multipurpose project. In addition to irrigation supplies (KPK 110 cusecs and Punjab 87 cusecs), it supplies drinking water to Islamabd 33 MGD, Rawalpindi 69.37 MGD and Taxilla Industrial Complexes 28.5 MGD, Tourism and Fisheries are also developed at the Project. The Khanpur village was submerged in reservior and a new Khanpur town has been built downstream of the dam.

After comparative study of a number of sites on Haro river, the final site was selected in 1962 and detailed studies were started. Initially the project was concieved to be 137 ft. high but due to shortage of drinking water in Islamabad and Rawalpindi it was decided to raise the height to 167 ft. it stores 1,06,000 acre feet of water. The project consists of main dam, a check dam, two saddles, a gated spillway, diversion tunnel, putlet works and irrigation canals.

An offtake canal emits from the dam, the water flows through a concrete conduit that is made through the dam embankment. A significant drop is present on the site providing water velocity to flow. The head avaiable on the project site is due to the water stored in the reservoir of the dam. The figure below shows the offtake canal downstream of the dam.

Figure 1: Khanpur Dam

Figure 2: Project Site

3.1. Geological Features

The project area lies in the Hazara Thrust Fault system. The Margala Hill ranges at Khanpur Dam Project are traversed by a system of nearly parallel north-east/south-west tending faults. These faults join the Hamalayan thrust along a syntexial bend towards northeast and to Kirthar, Sulaiman fault zone, towards southwest.

The area around Khanpur Dam is tectonically disturbed and is severely folded, faulted and at places overturned. The axes of the major folds on both the abutments are parallel to sub-parallel and have east-west orientation. The folding and faulting is attributed to the post Eocene major regional tectonic activity. The folds are plunging towards west and their axial planes dip towards south.

The dam is located in an active tectonic area. The recorded earthquake at Tarbela and Rawalpindi are:

Table 1: Seismic Activity and Magnitude

Magnitude	Recurrence in years
4	2
5	12
6	66
7	380
7.5	912

Reference: Eng. Dr. Izhar-Ul-HAq (1983-84).Khanpur Dam Project. *Paper 461*.

From the above data, it is inferred that a maximum expected earthquake at Khanpur Dam Project could be of magnitude 6.2 with a return period of 75 years, while for the Maximum Credible Earthquake (MCE) the magnitude would be of the order of 6.8.

Regional fault passes only 12 km from the site. Seismic studies considering the regional fault and its rupture length of 65 km with the MCE gives 0.28g acceleration at site and 0.2g for maximum expectable earthquake.

Data collection involves the collection of data required for the project to calculate the design discharge, available head, dam design reports etc. Related data was collected from various agencies.

Data was collected from the following government agencies shown in Table 2.

Table 2: Data Duration and Source

DATA	PERIOD	AGENCY
Discharge (Canal Intake)	1991-2013	Khanpur Dam Authorities
Water Level	1991-2013	Khanpur Dam Authorities
Dam Embankment Cross Section		Khanpur Dam Authorities
Dam Design Reports		WAPDA

3.2. Site Visits

Site was visited many times at regular intervals for

1. Collection of data
2. Assessment of Site conditions
3. Discussion with operating agencies

4. Assessment of existing infrastructure
5. Analyze our proposal
6. Prevailing Site conditions

Technical Books, drawings and reports were also studied to help gain insight into the functioning of the hydropower plants.

3.3. Topographic Survey

Topographical survey was done to prepare the topographic map of selected site for our project.

3.4. Preliminary Data Processing

The daily discharge data as received from the authorities of Khanpur Dam for the past 23 years (1991-2013) was entered in spread sheets using Microsoft Excel software to calculate the appropriate value of discharge and for calculation of available hydropower potential at the canal
Topographical Data collected from the site was processed in Microsurvey CAD to generate a topographic map of the site.

4. RESULTS AND DISCUSSIONS

This chapter discusses the results obtained by processing the data. Results include mean annual discharge, flow duration curves, design discharge, available head and selection of turbine, hydropower potential available, proposed layouts and their cost analysis.

4.1 Flow Duration Curves (FDC)

Flow duration curves of years (1991-2013) are shown below:

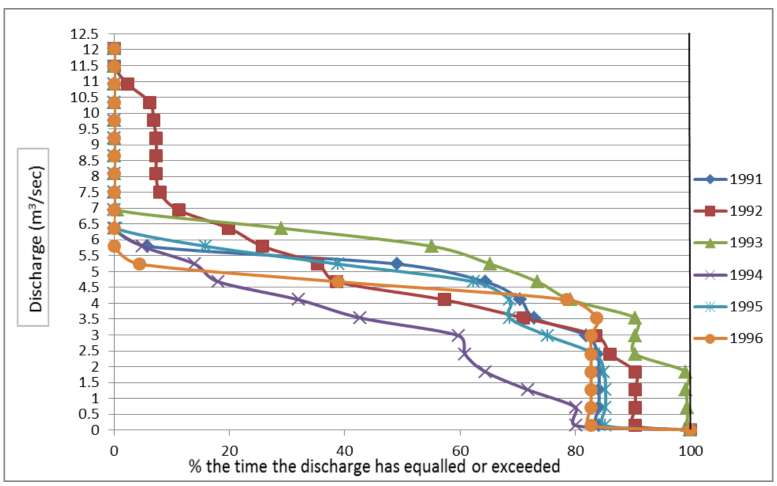


Figure 3: Flow Duration Curves (1991-1996)

For years 1991 and 1993-1996, flow has not exceeded above 7 cumecs throughout the years while the flow ranges between zero to 7 cumecs for these years. for year 1992, the flow reached up to the level of 12 cumecs and ranges between zero to 12 cumecs. The most dominant range of flow throughout these years is 2 cumecs to 4 cumecs.

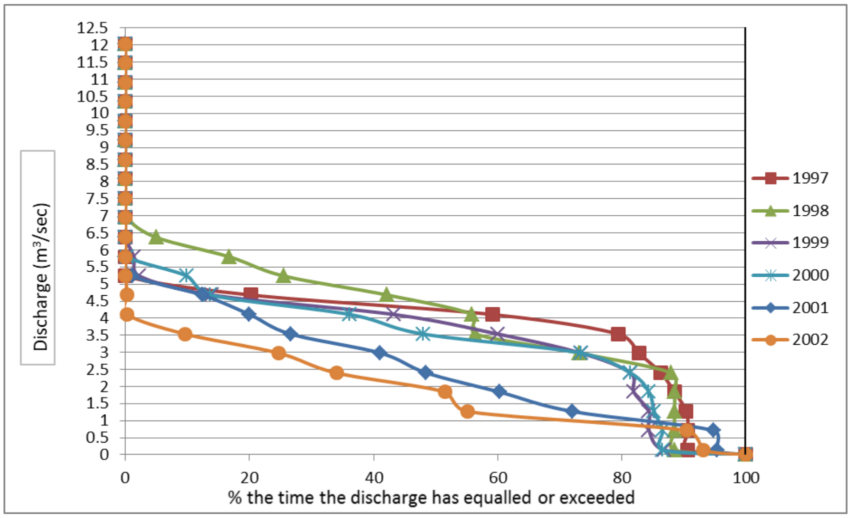
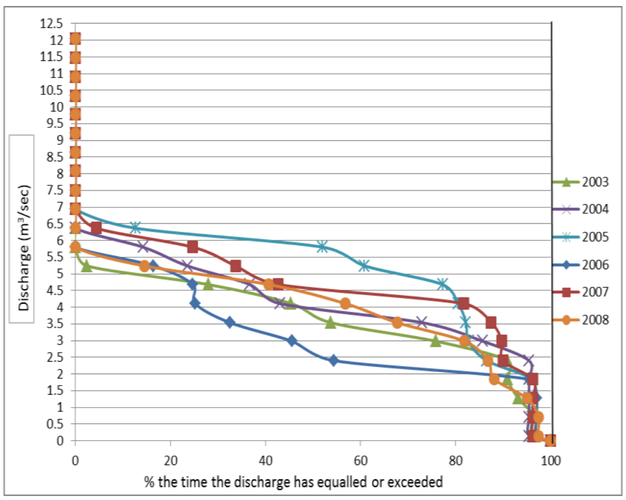


Figure 4: Flow Duration Curves (1997-2002)



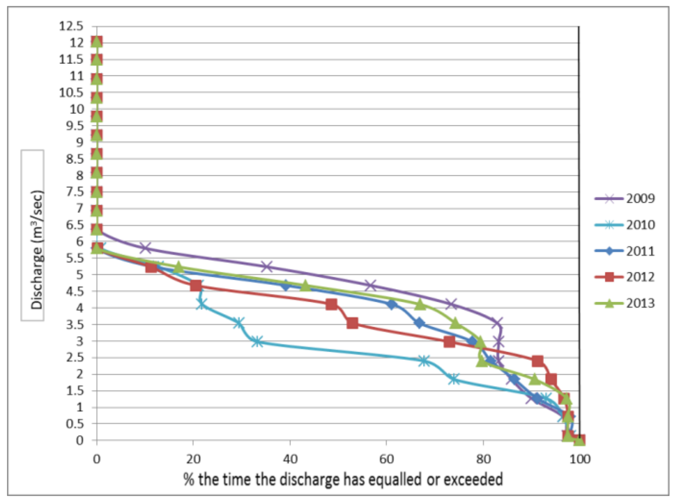


Figure 5: Flow Duration Curves (2002-2008) **Figure 6: Flow Duration Curves (2008-2013)**

The flow duration curves for years 1997-2002 show that flow ranges between zero to a maximum value of 7 cumecs during these years. 2002 was the year in which flow was relatively less than the other years. the most dominant range of flow during these years is 1.5 to 4 cumecs.

Flow duration curves for years 2002-2008 are closely plotted. Minimum value of flow is same for these years i.e, zero. Maximum flow in all of these years reaches up to 7 cumecs. Range of flow which has occurred for most of the time is 2.5cumecs-5 cumecs.
For these recent years maximum flow occurred is 6.5cumecs. Flow duration curves represent that flow between 1.5 cumecs to 4 cumecs is dominant for most of the time during these years. Minimum flow during all these years is zero.

4.2 Mean Annual Discharge

To calculate mean annual discharge of each year from 1991-2013, first monthly discharge value of each month is calculated by taking mean of daily discharge of the whole month and then mean of monthly discharge value of each month will give the mean annual discharge. This is all done in spreadsheets.

4.3 Design Discharge and Available Head

Using flow duration curves of each year, mean annual discharge is calculated for respective years. Reserved flow is zero for each year. Design discharge for each year is obtained by subtracting reserved flow from mean annual discharge. Taking the mean of annual discharge gives the value of design discharge value.

Table 3: Design Discharge Value

Year	Mean Annual Discharge (m^3/s)	Design Discharge Value (m^3/s)	Year	Mean Annual Discharge (m^3/s)	Design Discharge Value (m^3/s)
1991	4.17	4.17	1996	3.85	3.85
1992	4.60	4.60	1997	3.76	3.76
1993	5.29	5.29	1998	3.92	3.92
1994	3.02	3.02	1999	3.32	3.32
1995	4.15	4.15	2000	3.25	3.25
2001	2.04	2.04	2008	3.97	3.97
2002	1.94	1.94	2009	4.35	4.35
2003	5.29	5.29	2010	2.84	2.84
2004	3.66	3.66	2011	3.81	3.81
2005	4.81	4.81	2012	3.61	3.61
2006	2.70	2.70	2013	4.07	4.07
2007	4.17	4.17	Design Discharge (m^3/s)		3.7

Note that reserved flow is zero for all years.
In Table 3, Column 1 shows the year and the column 2 show the mean annual flow of the corresponding years. Column 3 contains the design discharge value for each year which is the difference between mean annual discharge and reserved flow.

Overall design discharge value is 130.41 ft^3/s or 3.7 m^3/s. Refer Table 3.

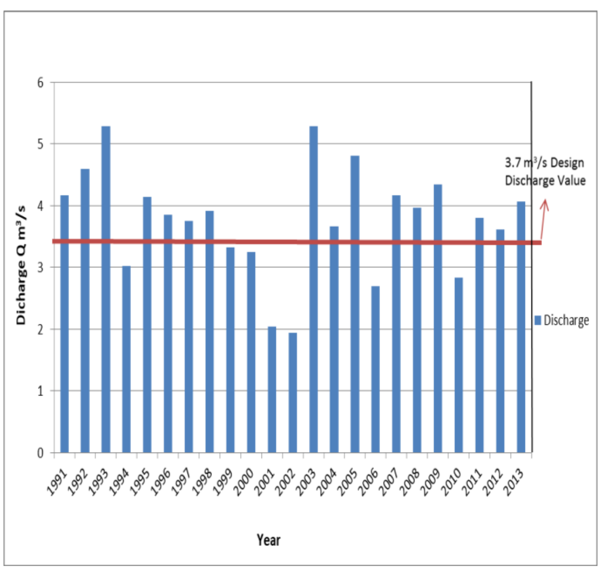


Figure 7: Mean Annual Discharge

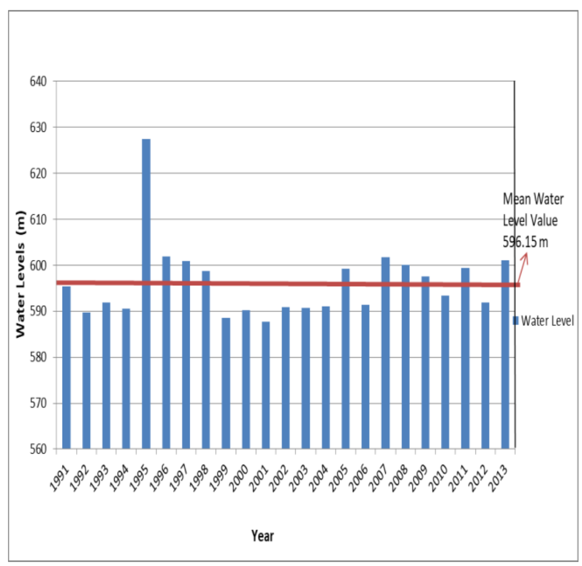


Figure 8: Mean Annual Water Level

The bar chart shows years on the x-axis and discharge in cumecs on the y-axis. The bars show the mean annual flow for each year. The horizontal line shows the design discharge value. To calculate available head mean annual water level is obtained by taking the mean of water levels of the whole year. Mean of annual water level of each year will give the mean water levels.

Above figure shows the mean water level of each year from 1991-2013 and horizontal line shows the mean water level value of all these years that is used to calculate the head.

To calculate the available head take the elevation difference between the mean water level value and reduced mean sea level of the point where the turbine is to be installed.

Mean Sea Level of Point Where Turbine to be Installed= 577 m

Elevation Difference = 596 m – 577 m

The value of Available Head is 19 m.

4.4 Selection of Turbine

Turbine is selected on the basis of design discharge value 3.7m^3/sec and the net head available at the project site i.e, 19m.

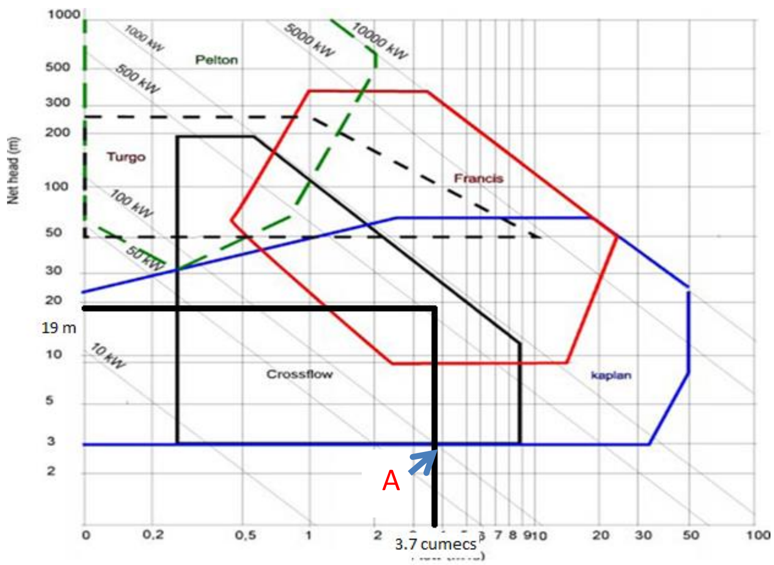


Figure 9: Turbine Selection Graph with Intersection Point Showing the Selected Turbine

Kaplan is a reaction turbine. It is generally used for low heads (under 30m) and for relatively medium to high discharge values. After calculating the design discharge and head values we used the following graph to select this turbine.

Kaplan turbine has many advantages over the other turbines hence it was an ideal choice for our project. Kaplan also has greater efficiency and running capacity and it does not wear easily hence maintenance cost is relatively cheaper than other turbines.

The intersection point A falls under Kaplan, cross flow and Francis turbines. However we selected Kaplan due to the above advantages.

4.5 Layouts
4.51 Layout 1 (using vacuum pump)

In this alternative the mechanism used for transfer of water from reservoir to the turbine include a non-returning valve, vacuum pump and steel pipe.The mechanism of the layout is that the water from the reservoir is suck using a vacuum pump and the discharge is set by using a non-returning valve. The water flows through a pipe, the pipe is then attached to the turbine to run it. Water leaves the turbine from a pipe and ends in the canal. Vacuum pump is installed on top of the embankment. The pipe runs along the surface of the embankment and passes under the road. The pipe is installed in patches and is supported by concrete blocks. Pipe transferring the water is attached to a turbine which is installed on the left side of the canal. Another pipe takes the water leaving the turbine and end it into the canal. Power house is on the left side of the canal.

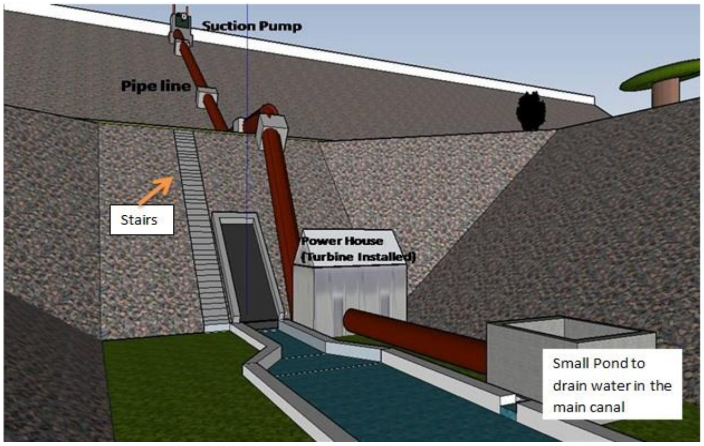


Figure 10: Layout 1 (a)

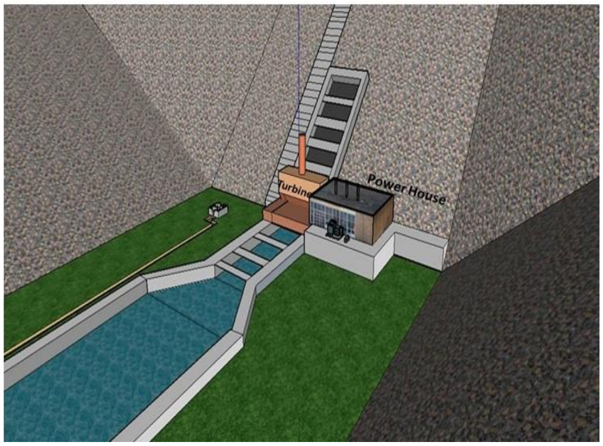


Figure 11: Layout 1 (b)

4.5.2 Layout 2 (without diversion tunnel):

This layout is a simplest than the other layouts. In this layout a turbine is installed at the start of the offtake canal downstream of the dam and at the foot of the embankment. Water passing through the duct provided in the dam embankment runs the turbine. Turbine is attached to the power house which is located at the left side of the turbine. Turbine will be installed during the period in which the canal is closed for cleaning purposes (Bhal Safai).

4.5.3 Alternate 3 (with diversion tunnel):

This layout is the same as layout 2. The difference is that a diversion channel is provided at the start of the off take canal and runs along the main canal and drain into the canal. Diversion channel is provided to use it as a canal during installation of turbine and during maintenance phase. Turbine is installed on the main canal and is attached to a power house located on the left side of the canal. Drawback of this layout is that we have to disturb the dam structure by removing the soil to make space for the diversion channel. Removal of soil may affect the dam safety. Accurate analysis should be done before opting this layout.

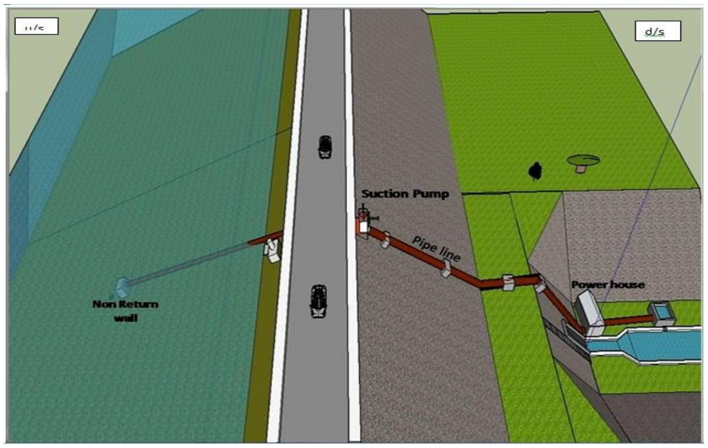


Figure 12: Layout 2 (a)

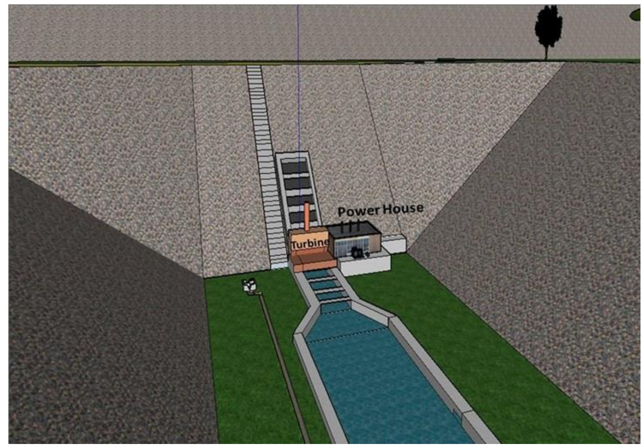


Figure 13: Layout 2 (b)

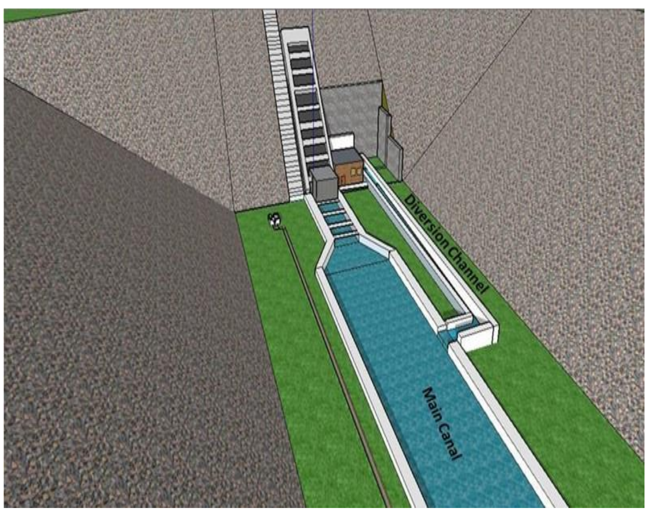


Figure 14: Layout 3 (a)

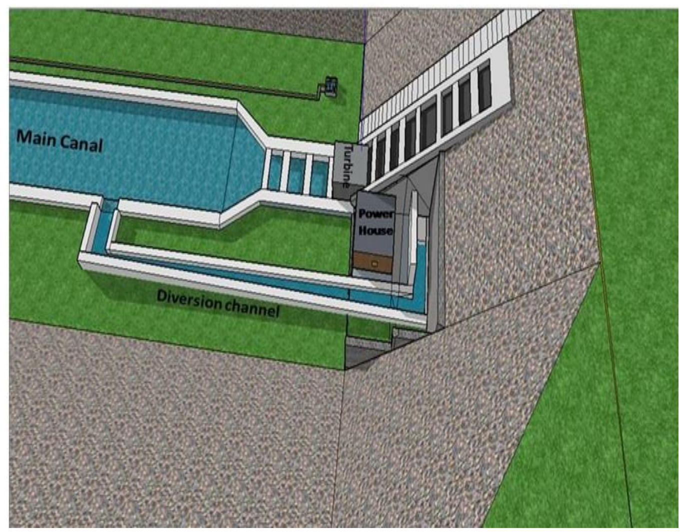


Figure 15: Layout 3 (b)

5. Hydropower Potential Available

For Q=3.7 m^3/s, efficiency of overall system=0.8 and H=19m
P=ρ*g*h*Q*E
P=1000*9.81*19*3.7*0.8= 482 KW
From the design discharge value of 3.7 cumecs and head available at the site 482 KW of electricity can be generated.

6. Hydropower Project Cost
6.1.1. For Layout 1

Pipe Length= 760 ft
Cost/unit length=760*10,000=7.6 Million Rs
Turbine + pump= 100000/KWh
Cost= 482 Kwh*100000=48.2 Million Rs
Powerhouse=20*20=400 ft^2
Cost=1000*400= 0.4 Million Rs

Total cost=52.6 Million Rs
Overhead=10 %= 578.6 Million Rs

6.2.2. For Layout 2

Pipe Length= 25 ft
Cost=10000*25=0.25 Million Rs
Turbine=75000/Kwh
Cost=75000*482=36.1 Million Rs
Dredging +Excavation=1000/ft^3
Cost=1000*25=25000 Rs
Retaining walls=20000 Rs
Powerhouse =20*20= 400ft^2
Cost=400*1000= 0.4 Million Rs
Total cost= 37.025 Million Rs
With 10 % overhead= 40.728 Million Rs

6.2.3. For Layout 3

Diversion Tunnel cost=7*10*100=7000 ft^2
Cost=7000 ft^3*1000Rs/ft^3= 7 Million Rs
Cost of lining= 2400 ft^2*350 Rs/ft^2= 0.84 Million Rs
Total cost= 48.568 Million Rs

After economic analysis, we can deduce that layout 2 is the best possible arrangement to use.
Cost analysis includes the machinery cost, installation cost, masonry work cost and in case of any hazards or delays contingencies cost are included. These costs are calculated for the three proposed layouts and the layout with the lowest cost is selected.

7. Conclusions

1. Project site has the potential for generation of 482 KW of hydropower.
2. On the basis of cost analysis, layout 2 (without diversion) found to be relatively economical and suitable for the project.
3. As a stand-alone system of available hydropower potential, electricity demands for both dam authorities and domestic can be facilitated.

8. Recommendations

1. Detailed design study is needed for the proposed layout 3.
2. Cavitation analysis should be performed to reduce wear and tear of the turbines.
3. Study can be done for sedimentation analysis.

9. References

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Evaluation of blue water footprint for the agriculture sector of Punjab and recommendation of adaptation techniques to meet the water demand of crop

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Abstract

Pakistan, along with some other developing countries, has been ranked as one of the most at-risk because of its vulnerability to climate change and lack of resources to respond. Pakistan's vulnerability factor is more obvious due to its agro-based economy. The Punjab province contributes almost 60 percent to total agricultural production of the country. Water footprint can help in assessing the most efficient adaptation technique to reduce quantity of water. The water footprint has been studied in the agriculture sector for Brazil, Greece, and many other developed countries. The studies on these countries show that Water footprint can be used as an indicator to evaluate agriculture schemes and potentially adaptation measures in highly cultivated regions. Assessment of green water footprints have been studied in many case studies whereas study of blue water footprints is not that common and is applied in the present study for the largest continuous irrigation system of the world. The blue water footprint method described in the Water footprint assessment manual is opted for rice, cotton, and wheat. The current study follows the established blue water footprint method under climate change conditions. The adaptation techniques were identified and employed considering the native socio-economic constrains. The method to evaluate these footprint values requires calculating the return flow, pollutant load, and runoff from the catchment. These factors are difficult to obtain and will yield in variation in the results. Furthermore, the loopholes in irrigation practices can be identified through this study and the sensitivity of these flaws under climate change scenarios will provide better idea of the priority areas that need urgent attention. The study shows the current prevailing flood irrigation method for the irrigation is not sustainable under climate change conditions. Growing demands of food and clothes can be met by improving irrigation techniques. The agriculture of Punjab is going to be adversely affected by the climate change. Technology-based planning and adaptation measure must be taken at the earliest.

1. Introduction

Water scarcity and water-borne diseases are causing serious economic and health crises in Pakistan with accumulated annual national income losses of up to Rs28 billion – roughly 0.6% to 1.44% of GDP. Pakistan, along with some other developing countries, has been ranked as one of the most at-risk because of its vulnerability to climate change and lack of resources to respond. In developing countries, such as Pakistan, climate change poses a serious challenge to social, environmental, and economic development. The effects of global climate change in Pakistan are already evident in the form of growing frequency of droughts and flooding, increasingly erratic weather behavior, changes in agricultural patterns and reduction in freshwater supply. Pakistan's vulnerability factor is more obvious due to its agro-based economy. The Punjab province plays pivotal role in the economy of Pakistan by contributing almost 60% to total agricultural production of the country. This contribution can further be enhanced by identifying the factors that affect agricultural production in the province. Since, Pakistan is an agro-based economy, so the country is dependent on its agricultural production. Climate change is resulting in droughts and floods due to which the crops are being affected. This change in climate stresses on the idea of water footprint which is defined as the quantity of water required to produce the crop. This desired quantity of water can be provided to the crop via adaptation techniques. Adaptation techniques are modifications which helps in adjusting to climate change. These can be composed of different irrigation methodologies, moisture prevention technologies and government policies which can assist in provision of sufficient water for production and maintaining required moisture to ensure proper yielding of crop. Increasing competition for water resources, coupled with climate change may have significant influences on water availability for agricultural production. Water footprint (WF) gives a way to deal with evaluating water assets usage in the rural generation process (Hoekstra & Hung, 2002). Understanding the water footprint (WF) of a nation is highly relevant for developing well-informed national policy (Hoekstra & Mekonnen, 2012).

The water footprint of crop production is defined as the volume of freshwater both consumed and affected by pollution during the crop production process, and it has three components: 1) green water impression (the volume of the precipitation expended in trim creation process); 2) blue water impression (the volume of surface or groundwater devoured in edit creation process); and 3) dim water impression (the volume of freshwater that is required to absorb the heap of contaminations amid the yield creation process) (Chapagain & Hoekstra, 2011). The blue water footprint is an indicator of consumptive use of fresh surface water or groundwater. The term ‘consumptive water use’ refers to one these four cases: 1) Water evaporation, 2) Water incorporation into the product, 3) Water not returning to the same catchment area e.g. it is returned to sea 4) water not returning in the same period i.e. withdrawn in a scarce period and returned in a wet period. The green water footprint is an indicator of the human use of so-called green water. Green water indicates the precipitation on land that does not run off or recharge the groundwater but is stored in the soil or temporarily stays on top of the soil or vegetation. This footprint is the volume of rainwater consumed during the production process. It is mostly used in agriculture and forestry. The grey water footprint refers to the volume of water that is required to assimilate waste, quantified as the volume of water needed to dilute pollutants (Herath, Deurer, Horne, Singh, & Clothier, 2011)

The WF of crop depends on the water consumed by the crop (i.e. both blue and green water consumption). The annual variation of climate factors results in variability of crop evapotranspiration, irrigation water requirements and crop yield (Sun, Wu, Wang, & Zhao, 2013). The climatic change will not only affect crop water consumption but crop yield as well. For instance, rising temperatures will translate into increased crop evapotranspiration, while producing a reduction in crop yield and agricultural productivity. The variation will exert direct influence on crop water consumption process and crop yield per unit area. Consequently, it will have an indirect effect on WF of crop production (Shikun Sun et al., 2013). To adapt agricultural systems to the changing climate, it is important to know how climate change affects agricultural production and water use efficiency. Hence, the assessment of water resources utilization during the agricultural production process under climate change will contribute to improving agricultural water management practices to cope with climate change. Pakistan’s vulnerability factor is more obvious due to its agro-based economy. As the Punjab province contributes almost 60 percent to total agricultural production of the country. Water footprint can help in assessing the most efficient adaptation technique to meet quantity of water required. To safeguard natural resources for future generations and to increase agricultural productivity, adaptations are required of agricultural sector by adopting suitable practices and policies that will safeguard this sector. Various adaptation measures can be used for water management in agriculture sector. Water footprint indicator could help in evaluating applied agricultural schemes with respect to consumption of fresh water and deterioration of water receptors considering climate change scenarios.

Adaptation measures is composed of short and long-term measures. Furthermore, adaptation has two main types i.e. autonomous and planned adaptation. Autonomous adaptation is the reaction of, for example, a farmer to changing precipitation patterns. Planned adaptation measures are conscious policy options or response strategies, often multi sectoral in nature, aimed at altering the adaptive capacity of the agricultural system or facilitating specific adaptations. Short-term adjustments are seen as autonomous in the sense that no other sectors (e.g. policy, research etc.) are needed in their development and implementation. Long-term adaptations are major structural changes to overcome adversity such as changes in land-use to maximize yield under new conditions; application of new technologies; new land management techniques; and water-use efficiency related techniques.

2. Problem Statement

Climate change is becoming a serious threat that has lasting effects on crops and water availability. This change is affecting both the crop yield and the water footprint. Rise of temperature results in greater evaporation rate and decreases crop yield. Both these factors are devastating for crops and result in less productivity. This issue can be addressed with the help of adaptation techniques for water footprint that will help maintaining a sufficient productivity for future demands under increasing water scarcity. Due to the intense land cultivation and groundwater overexploitation, some plain faces serious water related problem e.g. water scarcity, groundwater level depletion, and water quality degradation. Climate change affect the water resources availability, to estimate the potential effects of precipitation and temperature variability on freshwater resources and agricultural crop yield in the region. (Papadopoulou, Charchousi, Tsoukala, Giannakopoulos, & Petrakis, 2016).

3. Literature Review

Climate change as projected for the 21st century may significantly alter crop production (Parry, Rosenzweig, Iglesias, Livermore, & Fischer, 2004) that while global production is likely to remain stable for most of the century, regional differences could grow stronger through time, with only developed countries possibly benefiting from climate change (Shikun Sun et al., 2013). The impacts of climate change upon crop yield and food security are significant worldwide. It is projected that by 2100 around 200 million people are at the risk of hunger (Schmidhuber & Tubiello, 2007). Regional differences in the response of crop productivity to climate change are likely to emerge in Europe. As reported by (Olesen & Bindi, 2002), climate change is expected to have positive impacts only in the Northern countries, and areas of crop suitability may expand northwards (Olesen & Bindi, 2002). Southern areas, on the other hand, will likely have to face decreased crop yield (Bocchiola, Nana, & Soncini, 2013). It is expected that without proper investments to mitigate the impacts of climate change, global irrigation water needs may increase by roughly 20% by 2080 (Bocchiola et al., 2013).

The most crucial impact of climate change on agriculture is related to changes in the water cycle and it needs to be considered within a wide context that includes water demand increase, degradation of water quality, and competitive water use at various levels. The water footprint has been studied in the agriculture sector for Brazil, Greece, and many other developed countries. The study on these countries show that WF can be used as an indicator to evaluate applied agriculture schemes and potentially adaptation measures in highly cultivated regions. These evaluations were focused on climate change to forecast and predict the availability of water for future crops. The study of Greece proposed the adaptation measures such as income/ asset management, government programs and insurance, farm production practices and technological development should be adopted in different scales to assess the impact of climate change on yield especially on water scarce regions. Climate change significantly affects every day human activities such as agriculture and tourism by altering the composition and parameters of global atmosphere over long period of time. In Greece, a substantial part of the national gross domestic product comes from agricultural production, the efficiency of which mainly depends on adequacy and sufficiency of resources such as water and soil fertility (Papadopoulou et al., 2016). The study on Brazil proposed water consumption regulations as an adaptation methodology to stimulate water use efficiency, such as reduction of crop water use, and evapotranspiration, and optimal fertilizer application control.

Different policies are made at the local, national, and international level to cater the issue of water scarcity. To encounter the adverse effect of climate change on crops there is a need of planning at the national level, decision-makers need to define a framework to identify vulnerable regions and then shortlist potential interventions. Evaluation suggests that careful planning gives long-term pay-offs (Dinesh et al., 2016). Adaptation planning at the local level is important to address the granularity of climate change impacts which may vary within national boundaries, and to enable farmers and communities to oversee their own choices and futures under climate change. Multiple approaches to participatory planning have proven effective in matching adaptation investments to local preferences, needs and capacities. Local planning is useful in ensuring that adaptation actions are holistic, rather than focused on only one or two technologies or practices. For example, "climate smart villages" that enhance climate literacy of farmers and local stakeholders are being used in Nepal, Bangladesh, and around 1500 villages in India. In Vietnam, local knowledge has been integrated with scientific knowledge to develop participatory land use plans in three climate-smart villages (Dinesh et al., 2016).

4. Scientific Innovation

The effects of water scarcity and climate change are not studied in Pakistan with water footprint techniques. Furthermore, the study of blue water footprints is rare and is applied here in the largest continuous irrigation system of the world, and possible improvements in the technique will be suggested. The WF of a crop is determined by the crop water consumption (green and blue water) during crop production process and crop yield per unit area (Hoekstra & Mekonnen, 2012). Annual variability of climatic factors would cause the variation of crop water consumption and crop yield per unit area, and it will exert an indirect impact on the WF of crop production. Based on the framework provided by (Hoekstra & Mekonnen, 2012) and (Papadopoulou et al., 2016), the present paper puts forward a modified method of quantifying the blue water footprint of a crop. To reflect the actual water consumption during the crop production process, the loss of irrigation water during the transmission and distribution process from the water sources to field and the return flows are also considered in the evaluation. Furthermore, the adaptation measures are suggested to cope up the issue of adverse climate on crops.

5. Methodology

The method to calculate the blue water footprint as documented by the Water footprint assessment manual is applied for rice, cotton, and wheat production practices in Punjab. The method considers the evaporation, water incorporation and the return flow but in our case the return flow is not much common in Punjab. CROPWAT 8.0 has been used for Water footprint of Wheat, Cotton, and Rice.

The model requires the climatic data of the region (Multan) which was taken from the meteorological department of Pakistan. The data obtained included minimum and maximum temperature, humidity, sun hours, wind speed, radiation, and rain data. The time span for analysis ranged from 2007 to 2017. This dataset was divided on the span of five years. Firstly, the average of the first five years (i.e. 2007-2012) was considered to evaluate the water footprint of crop, cotton, and rice and then the water footprint of the remaining five years was evaluated.

After adding the climate data into the CROPWAT 8.0, the effective rain was calculated. This step was followed by importing crop constants (i.e. rooting depth, critical depletion (fraction), yield response, and crop height). Soil data of Multan was also inserted in the model which included the available soil moisture, maximum rain infiltration, and initial soil moisture depletion. The model evaluated irrigation water required in different stages. The total gross and net irrigation water, actual water used by crops and moist deficit were all calculated. To estimate the irrigation water required per month for each of the three crops, planting date of the crop and percentage area of the particular plant is entered. The results are plotted below:

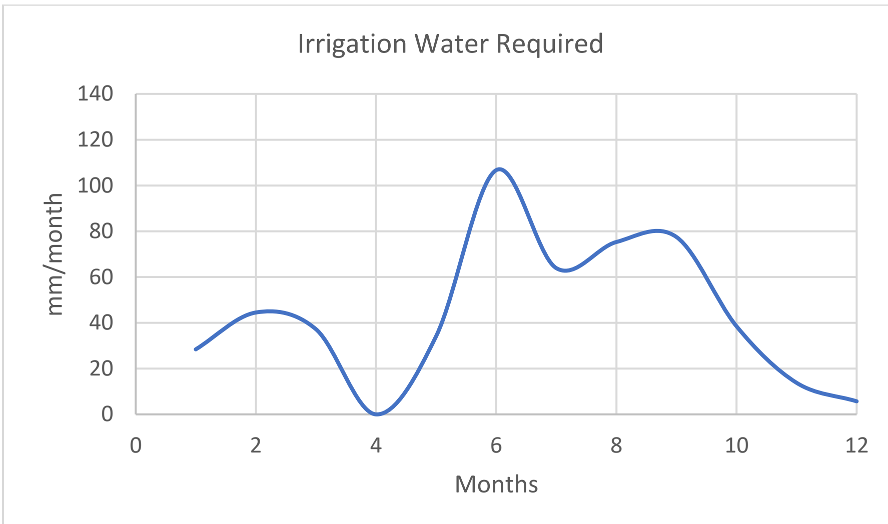

Figure 5.1: Average Irrigation water required per month from 2007-2012

The results show that the need for water varies throughout the year. It was noted that the water requirement is minimal for the 4th month of the year (i.e. April) whereas the highest water was needed in the month of June (i.e. 6th month). The trend illustrates a steep rise in water demand from April to June whereas a huge fall in water need was noticed from October to December. The average of the entire year was found out to be 525.6 mm/year.

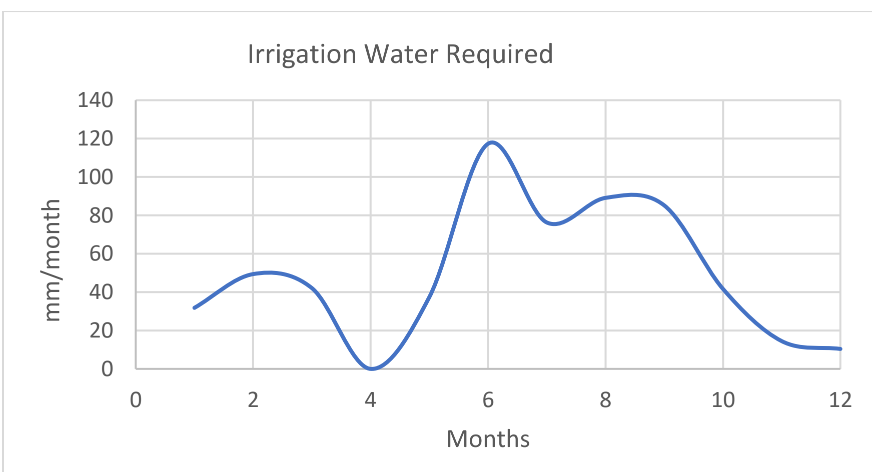

Figure 5.2: Average Irrigation water required per month from 2012-2017

The results of the second trial shows a similar trend but depicts an overall increase in the water requirement per month. The reason for the highest water demand in June is due to the elevation in temperature as a result of summer season. A slight fall in water requirement in July and August is because these months have monsoon throughout Punjab. Furthermore, the fall from October to December shows a fall in water demand due to winters. The avergae value of water is calculated to be 595.2 mm/year from 2012-2017.

6. Result/Output

There was clear difference between the water required from the first trial to the next trial. The irrigation water required for the first trial was 525.6 mm/year and for the second trial it was found to be 595.2 mm/year. There are various reasons for the increase in water demand in the last five years (i.e. 2012-2017). The main reason for this difference is climate change, as the temperature is increasing so the water requirement of the crops is rising due to higher evapotranspiration and evaporation. Furthermore, the rivers are also facing an increase in evaporation due to elevated temperatures and hence, the water scarcity is rising. Pakistan, being an already water stressed country needs adaptation measures to meet the water demands of the crop for future to assure sufficient crop production. If urgent adaptation measures are not adopted than the results will be fatal because the increase in population is already causing an increase in food demand and if the crops are not provided required demand than the future will see a decreased food production which will raise the overall cost of food commodities as the import of food commodities will rise.

7. Discussions

The study shows the current prevailing flood irrigation method for the irrigation is not sustainable under climate change conditions. Growing demands of food and clothes can be met by improving irrigation techniques. Water footprint is an indicator of use of water that measures the volume of water used to produce a product. Water footprint is divided into three parts i.e. blue, green, and grey. Pakistan, along with some other developing countries, has been ranked as one of the most at-risk because of its vulnerability to climate change and lack of resources to respond. The WF of crop production depends on the total water consumption and the crop yield.

Pakistan is an agricultural country. The agricultural dependency of Pakistan is mainly on Punjab as compared to other provinces. Due to climate change, there is less amount of rainfall now as compared to past hence the consumption of blue water for crop production is increasing. Pakistan already has limited water resources and furthermore, we are not using these limited water resources efficiently. Hence, proper management techniques must be adopted in time to fulfill the water requirement of the country. Pakistan is one of top countries in the world that are affected by climate change. The water footprint results have clearly depicted the increase of water demand. It is time that the government and concerned authorities adopt necessary adaptation techniques to save the future generations of Pakistan. There are various adaptation techniques that includes drip irrigation, different farm production practices, government plans and insurances, and asset management techniques. These techniques are currently in practice in different countries of the world to cater the issue of climate change.

8. Conclusion

The change of climate is adversely affecting the agriculture sector of Punjab. The amount of water required for crop production is increasing every five years which is stressing the already stressed water economy of

Pakistan. This rise in water demand needs attention to devise some adaptation methods to improvise crop production with climate change.

9. Recommendation

Current irrigation practices must immediately be replaced by efficient irrigation techniques. Water footprints must be considered for the agriculture policy at the national level. There is a need to devise different policies at national level to adapt to climate change. Changing climate is affecting the rainfall in the region and furthermore, due to global warming the average temperature of the world is increasing. With the increase in average temperature, the evaporation and evapotranspiration rates are increasing. Hence, a national policy to plant more trees is needed to control the rise of temperature in the region.

Water storage capacity of the region must be increased to save more water for multiple use. Different incentives must give to the farmers for the efficient use of water in their fields. There is a need to develop an institute at national level that addresses the issue of water scarcity by educating the masses about the efficient use of water. To predict precisely the future demand of water requirements a detail approach must be made to estimate the losses. Rain harvested areas are affected more by climate change because there source of water for irrigation is only rain, so a detailed study needs to be carried out to control the effect of climate change on such areas.

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