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CONGRESS PROCEEDINGS

8TH INTERNATIONAL CIVIL ENGINEERING CONGRESS

Ensuring Technological Advancements Through Innovation Based Knowledge Corridor

Editors
Dr. Farrukh Arif, Prof. Dr. Abdul Jabbar Sangi, Prof. Dr. Sarosh Hashmat Lodi

Jointly Organised by

The Institution of Engineers Pakistan
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The Asian Civil Engineering Coordinating Council

Federation of Engineering Institutions of Islamic Countries (FEIIC)

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8th INTERNATIONAL CIVIL ENGINEERING CONGRESS

Friday 23rd & Saturday 24th December, 2016

Programme

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Inspection, Evaluation, Maintenance and Preservation of Bridges: How it is Done in the United States

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Abstract
The interstate highway system was funded and built in the mid-1950s by the federal government and subsequently unleashed tremendous growth for the U.S. economy. The state, local and municipal roads were also built during this time period. Today, this highway/roadway network is aging and in serious need for repair or rehabilitation. A large number of the 600,000+ bridges throughout the country are past their 50-year lifecycles and are considered STRUCTURALLY DEFICIENT or FUNCTIONALLY OBSOLETE. Departments of Transportation are grappling with limited funding and escalating construction costs, long design lead-times, and negative economic and social impacts due to reduced bridge widths, causing traffic congestion and resulting in slow rehabilitations/replacements.

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's "Manual for Bridge Evaluation" remains the fundamental guidebook for (1) INSPECTION, (2) EVALUATION, (3) MAINTENANCE and (4) PRESERVATION. 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

Most of the nation's highways (state, local and municipal roads) and highway bridges were built in the 1950s by the federal government and subsequently catalyzing growth in the U.S. economy. Today, this highway/roadway network is aging and state departments of transportation and other bridge owners are faced with significant challenges in addressing the nation's highway bridge repair, rehabilitation/preservation and replacement needs. With more than 600,000 bridges in the United States, many of them closing in on or surpassing their original 50-year design lives, the need to ensure safety is at the forefront of concern in the bridge industry. More than 25 percent of the country's bridges are rated as structurally deficient or functionally obsolete and more than 30 percent of existing bridges have exceeded their 50-year theoretical design life and are in need of various levels of repairs, rehabilitation, or replacement. This issue is exacerbated by increasing travel demands, limited funding, and increasing costs of labor and materials. These circumstances have caused most bridge owners to become more reactive than proactive in their approach to managing and addressing their bridge program needs. Bridge stewards and owners need to become, inevitably, more strategic by adopting and implementing systematic processes, state-of-the-art technology and new innovative methods for inspection, evaluation, maintenance and preservation of bridges that become an integral component of their overall management of bridge assets.

The objective of a good bridge preservation program must be to employ cost effective strategies and actions to maximize the useful life of bridges. Applying the appropriate bridge preservation treatments and activities at the appropriate time can extend bridge useful life at a lower lifetime cost. Preservation activities often cost much less than major reconstruction or replacement activities. Delaying or forgoing warranted preservation treatments will result in worsening condition and can escalate the feasible treatment or activity from preservation to replacement. The latter will result in extensive work and higher cost. A viable alternative is timely inspection, load rating and effective bridge preservation and maintenance of sound bridges to assure their structural integrity and extend their useful life before they require replacement.

A successful bridge program is based on a strategic, systematic, and balanced approach to managing bridge preservation and replacement needs. This paper will shed light on the balanced approach that includes; inspection, evaluation (load rating), maintenance and preservation of bridges, as illustrated in Figure 2. The paper will also highlight the role of new technology in achieving more efficient and definitive assessment of the behavior and safety of bridges.

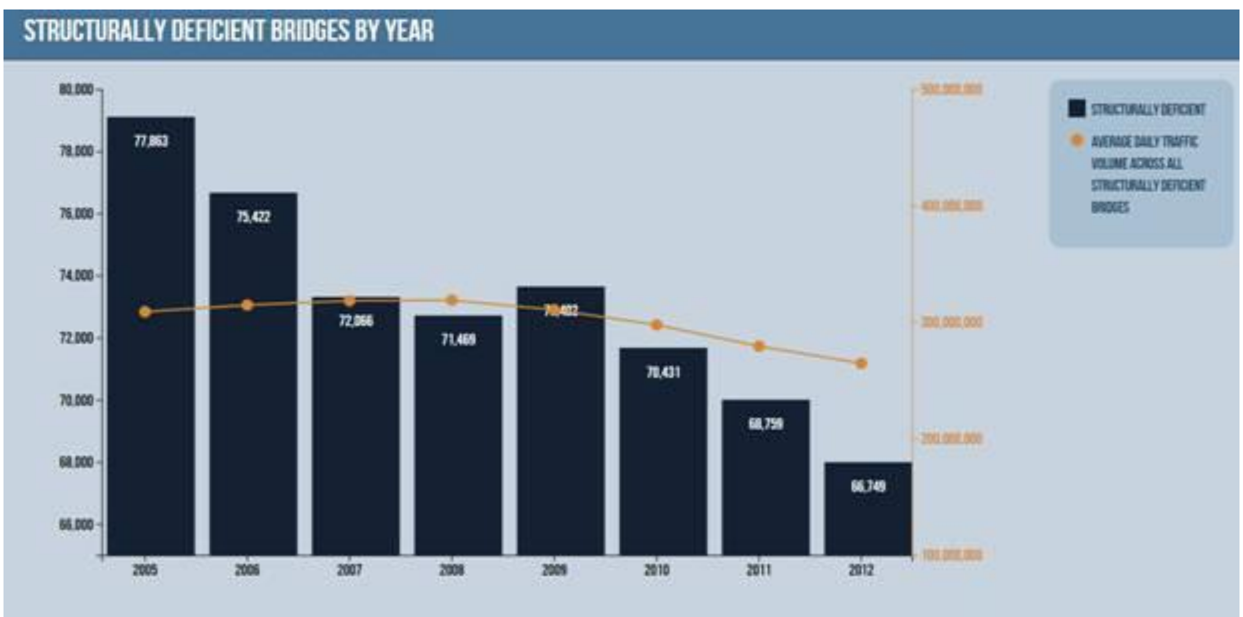

Figure 1: Structurally Deficient Bridges by Year

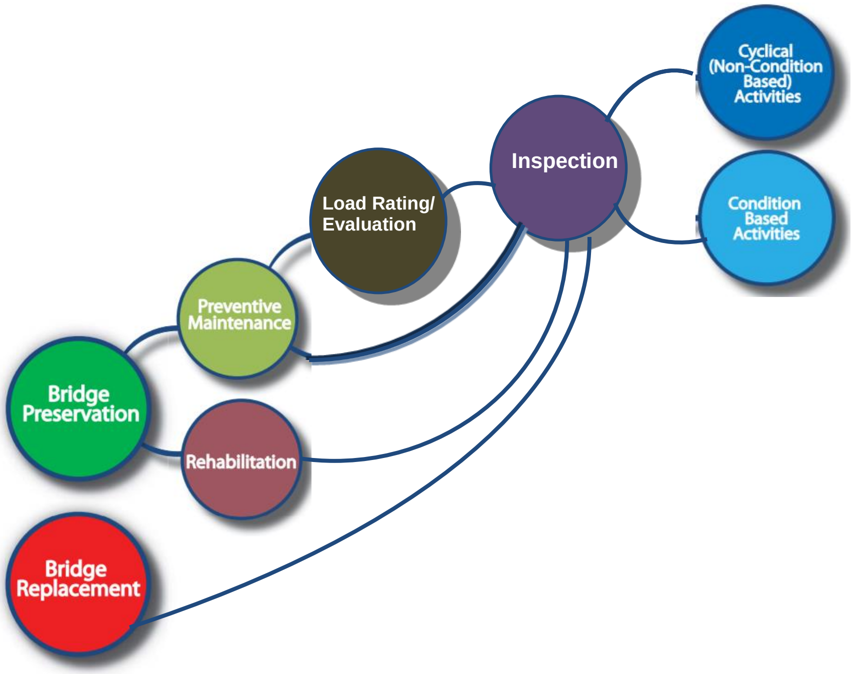

Figure 2: Bridge Action Approach Cycle

2. Bridge Inspection

2.1 Background

During the bridge construction boom of the 1950's and 1960's, little emphasis was placed on safety inspection and maintenance of bridges. In 1967, there was a sudden collapse of the Silver Bridge, a pin-connected link suspension bridge over the Ohio River at Point Pleasant, West Virginia, resulting in the loss of 46 lives. This tragic collapse aroused national interest in the safety inspection and maintenance of bridges. The U.S. Congress was prompted to add a section to the "Federal Highway Act of 1968" that required the Secretary of Transportation to establish a national bridge inspection standard. The Secretary was also required to develop a program to train bridge inspectors. As a result, a 1968 federal act initiated a national bridge inspection program that recognized the need for periodic and consistent bridge inspections. The first National Bridge Inspection Standards (NBIS) were developed in 1971.

Figure 3: Collapse of the Silver Bridge

The NBIS established national policy regarding inspection procedures, frequency of inspections, qualifications of personnel, inspection reports and maintenance of state bridge inventory. Three manuals were subsequently developed which played a vital role to the early success of the NBIS:

1. Federal Highway Administration (FHWA) Bridge Inspector's Training Manual 70 (Manual 70). This manual set the standard for inspector training.
2. American Association of State Highway Officials (AASHO) Manual for Maintenance Inspection of Bridge.
3. FHWA Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges (Coding Guide).

These manuals went through a number of revisions and additions over the years. Currently, The Manual for Bridge Evaluation (MBE), First Edition with 2010 Interim Revisions by AASHTO serves as the fundamental guidebook for inspection, evaluation, maintenance and preservation.

2.2 Inspection Process

Bridge inspection has played, and continues to play, an increasingly important role in providing a safe infrastructure for the United States. As the nation's bridges continue to age and deteriorate, an accurate and thorough on-time assessment of each bridge's condition is critical in maintaining a safe, functional and reliable highway system.

There are seven basic types of inspection:

- Initial Inspection: Performed on new bridges or when a bridge is first recorded.
- Routine Inspections: Those regularly scheduled, usually every two years for most normal bridges.
- Damage Inspections: Those performed as a result of collision, fire, flood, significant environmental changes, loss of support, etc. These inspections are also called Emergency Inspections and are performed on an as-needed basis.
- In-Depth Inspections: Performed usually as a follow-up inspection to better identify deficiencies found in any of the above three types of inspection.
- Underwater Inspections
- Fracture-critical Inspections
- Special Inspections: Performed to monitor a particular deficiency or changing condition. Unusual bridge designs or features such as external, grouted, post-tensioned tendons may require a Special Inspection.

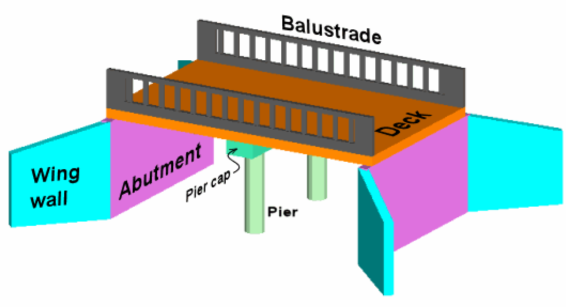

Figure 4: Bridge Elements

Each bridge is divided into four major parts for condition assessment: superstructure, substructure, deck and culverts. Each of the four major parts is rated on a 0-9 scale (see Figure 5) by severity of deterioration. The ratings include the assessment of multiple distress symptoms and are expected to describe the general condition of the bridge. An overall sufficiency rating based on NBI data is used as a performance measure at the federal level for funding allocation, but this measure emphasized large-scale functional and geometric characteristics of bridges, making it hard to use for maintenance decision-making.

Rating	Description
0	Failed condition
1	Imminent failure condition
2	Critical condition
3	Serious condition
4	Poor condition
5	Fair condition
6	Satisfactory condition
7	Good condition
8	Very good condition
9	Excellent condition

Figure 5: Condition & Rating Descriptions

A general inspection team consists of a Team Leader (TL) and an Assistant Team Leader (ATL). There are six basic duties of the bridge inspection team: 1) Planning the inspection, 2) Preparing for the inspection, 3) Performing the inspection in accordance with NBIS and AASHTO MBE, 4) Preparing the report, 5) Identifying items for repairs and maintenance, and 6) Communicating the need for immediate follow-up for critical findings.

It is imperative that the bridge inspection process be able to recognize, document, and alert bridge owners of critical deficiencies. Inspectors must be well-versed in structural behavior and possess an array of assessment techniques to accurately evaluate the structure. An accurate description of the critical deficiency is paramount to determining appropriate response actions, designing repair procedures that eliminate the cause of the deficiency, developing an accurate assessment of risk, and for use as a baseline for suitable monitoring programs. Inspectors should be able to identify the damage and understand the reasons leading to that damage. Accurate diagnosis of the cause of the damage is imperative not only for determining appropriate response actions, but also for the safety of the traveling public. Hence, the requirements by which inspectors are qualified are an important element for effective inspections. The NBIS provides minimum requirements for various inspection personnel, including team leaders and program managers.

2.3 Inspection Techniques

Bridge inspections are performed using a variety of methods to access areas of bridges that are inaccessible from the ground or bridge deck. Different methods work well in different conditions and with different bridge types. Aerial Work Platform (AWP) includes a variety of equipment commonly referred to as under bridge inspection vehicles (UBIU), snoopers, lifts, bucket-trucks and man-lifts. This equipment is the most common method for accessing difficult to reach areas of a bridge. Rope access is another accepted form of access for bridge inspections. This method involves specially trained and certified rope access professionals using ropes and climbing equipment to access portions of the bridge that are inaccessible from the ground or the bridge deck.

2.4 New Technologies

Traditionally, when inspecting bridges, there is a choice between using an aerial work platform (AWP) or under-bridge inspection vehicle, ladders or rope access. This choice is dependent on the type of bridge, the access needed, and the crews' ability to close lanes of traffic. Regardless of the method used to carry out the work, the associated costs and dangers also remain a challenge. But, in cases where access is difficult, the bridge has lane restrictions and is small enough, and might only require ladder or rope access then there felt a need for some new technology that can aid in the current inspection process. In the past few years, remarkable technological advances have been achieved in bridge inspection and testing. A few of the technologies already tested and used for bridge inspection and testing are discussed below.

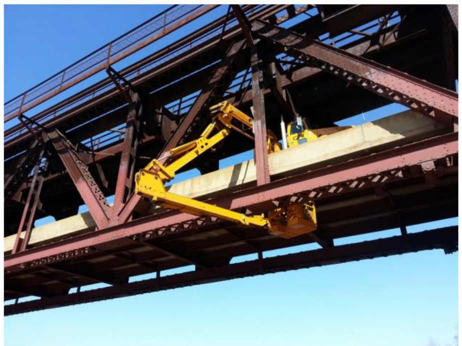

Figure 6: Example of an Aerial Work Platform (UBIU shown)

2.4.1 Unmanned Aerial Systems (UAS)

When it comes to inspecting bridges, UASs, also called drones and UAVs, are often heralded as the next big thing, a truly disruptive technology capable of giving engineers eyes in the hardest-to-reach places, without the need for expensive access vehicles or potentially dangerous rigging and rope access (or even ladders). 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. In order to evaluate the possible use of UAS, the Connecticut Department of Transportation (ConnDOT) and the Minnesota Department of Transportation (MnDOT) recently did a pilot project/study to test and assess the feasibility of using UAS as a tool to assist in the bridge inspection process to provide better data and make inspections safer. During this pilot project, several of the UAS imaging devices were tested, including still images, video and infrared cameras. Then, in the field, they collected various data, such as still images, video, infrared images, site maps and 3D models of bridge elements. The results so far have shown that the inspection detail that UAS provide effectively replicates some of the detail learned through the use of snoopers, without the traffic control requirements, and at significantly lower cost in terms of equipment and traffic control needs.

Figure 7: Bridge Inspection using UAS (Gold Star Memorial Bridge, Connecticut)

Figure 8: 3D rendering using UAS

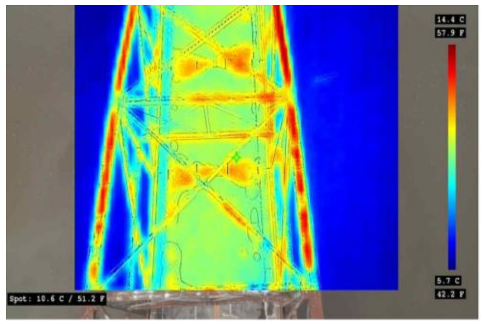

Figure 9: Infrared (point cloud) image taken by UAS in the field

UAS can provide both infrared and 3D modeling details of bridges, effectively identify concrete delamination, gather topographic mapping detail and deck condition surveys with or without the use of zoom camera, and efficiently map riverbank conditions, upstream and downstream, from the bridge site.

For members not requiring a hands-on inspection, a UAV can be used as a tool to assist an inspector in gathering better information than would normally be possible. For example, a UAV would give inspectors the ability to observe the conditions at the bearings or connections that may normally only be observed from some distance much greater than arm's length. The UAS market, although fairly new, is expanding very quickly, and advancements in UAS technology are increasing exponentially in equipment and software for both UAS operating platforms and data collection devises. The goal in the future is to identify bridges in Minnesota that would benefit from UAV inspection.

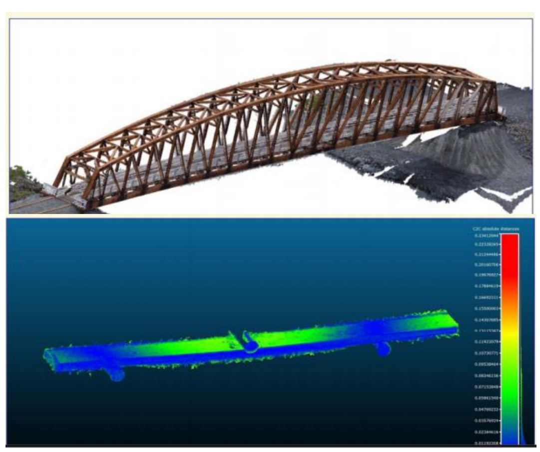

Figure 10: 3D rendering and point cloud (infrared) analysis using UAS

2.4.2 Strain Gauge

The transportation infrastructure is quickly aging. Increases in traffic, in both urban and rural areas, puts more strain on the bridge networks than was originally intended. Bridge engineers need a reliable way to assess structural integrity of bridges to maintain the continuous operation of the road network while ensuring the safety of the public. Visual inspection by experienced professionals has been effective in identifying critical conditions affecting bridge safety.

But, in some cases, material defects and concealed elements do not lend themselves well to visual inspection and may need supplemental methods. The use of the most effective technologies and techniques for bridge condition assessment is an important element in preserving the continued safety of highway bridges. Nondestructive evaluation and bridge condition/health monitoring technologies have been developed in recent years that can assist in the effective condition assessment and monitoring of bridges. A possible solution to these issues is the use of strain gauge.

The wireless strain gauge system has significant sensor accuracy sufficient to identify degradation of bridge conditions, detection of crack size or any other flaws at the earliest possible time, detect flaws as small as 1mil and the ability to detect damage anywhere in a steel member. The system also conducts regression analysis on long-term strain measurements to detect flaws. The major benefit of strain gauge monitoring is that it provides a cost effective and reliable remote sensing capability for long-term bridge monitoring. These systems are capable of measuring a wide spectrum of structural parameters, including strain (stress), vibration, velocity, displacement, inclination, temperature, and humidity, in real time, with a minimum of 10 years of monitoring.

Other attributes of wireless strain gauge systems include:

- Monitoring vibration on piers and detecting collisions (through detection of shocks and high acceleration events).
- Simultaneous and synchronous monitoring of tilt/orientation on piers.
- Confirmation of conditions through camera footage.
- Evaluating steel retrofits by accurately monitoring thermal and live-load induced loading effects on the retrofits.
- Conducting statistical/comparative analysis.
- Monitoring change in curvature (deformation) of the members.
- Conducting comparative analysis on response of the repaired members with identical members.
- For masonry arch bridges, it’s capable of using a few high resolution tilt sensors with a resolution of 0.00016 degrees capable of detecting and tracking movements caused by temperature, settling, or hydraulic damage in scour critical bridges.
- Communication capabilities between the data collection points at a bridge site and the remote monitoring office utilizing cellular communication solutions.

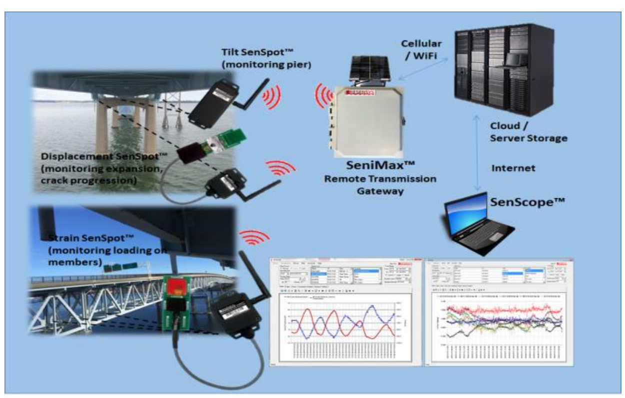

Figure 11: Illustration of strain gauge monitoring system

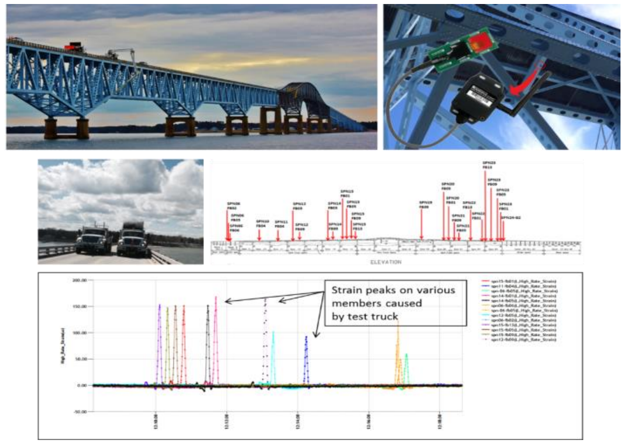

Figure 12: Strain gauge monitoring system at the Robert O. Norris Bridge, Virginia

2.4.3 Infrared (IR) Technology

This technology provides a number of benefits using quality thermography with a thermal imager to show potential structural problems unseen by the naked eye. A single image with a thermal imager can identify defects in bridge elements and concrete structures critical to the structure's safe use. This high-tech equipment can reveal leaking and structural cracks inside bridge elements that are in need of repairs. Such delamination's can be seen with the IR camera, and the beams are clearly visible by showing temperature variations present at the time of inspection. These images could also be dimensionally calibrated to provide repair quantity estimates.

2.5 Load Rating (LR) and Evaluation

The importance of load ratings cannot be overstated. With the ever-increasing demand for transporting goods and services, transportation officials are facing a growing challenge with the safety of in-service bridges under the passage of over-sized and/or over-weight vehicles. A true measure of the system's performance under the impact of irregular loading scenarios requires knowledge of different aspects of the system-level characteristics, including the lateral load distribution behavior. They must be performed to a level of accuracy required for the intended purposes of load posting decisions, repairs, rehabilitation or replacement, issuance of load/overweight permits, and allowing overlays.

Load ratings should be reliable, uniform, and consistent. All the inspection notes, including the deterioration of bridge elements and nondestructive evaluation (NDE) results, other inspection results, or the results of structural monitoring if available, should be incorporated into the rating process. Figure 13 illustrates a load rating process for any bridge evaluation. A new or updated load rating is required for any new bridges, a change in the physical or structural components of the bridge, a change in software preference, a new or different vehicle, a change in the methods of analysis and any new guidance information available.

Overweight load transports require a permit to be issued by the DOT and load rating calculations to ensure any structure on the route is capable of carrying the load. Oftentimes, very large transport vehicles require additional attention when it comes to structural modeling as they have irregular wheel geometries and subject structures to atypical or uncommon loadings. Also, special considerations must be given to the allowable speed they cross bridges with; DOT permits are often issued with a top speed allowance and specific lane guidelines. If these procedures are not followed, the load ratings become invalid and the structures and/or operators are at risk. It is vital that the load raters coordinate with the DOT to make sure that the load rating calculations match with the specifications outlined in the permit.

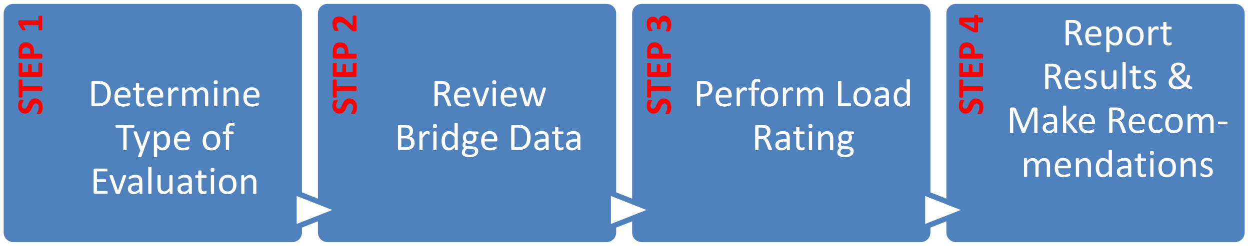

Figure 13: Load rating process flow chart

LR Software	LR Methods	Field Load Testing	Standards
AASHTOWARE-BrR	ALLOWABLE STREE DESIGN (ASD)	STRAIN GAUGE	AASHTO STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES
DESCUS	LOAD RESISTANCE FACTOR (LFR)		AASHTO STANDARD
LARSA	LOAD & RESISTANCE FACTOR RATING (LRFR)		AASHTO LRFD
STAAD			AASHTO MANUAL FOR BRIDGE EVALUATION
CANDE FOR CULVERTS			STATE LOAD RATING GUIDANCE
MIDAS			
MATHCAD			

Figure 14: Load Rating Tools and Standards

2.6 Preventative Maintenance (PM)

Preventive maintenance is a planned strategy of cost-effective treatments to an existing roadway system and its appurtenances that preserves the system, retards future deterioration, and maintains or improves the functional condition of the system (without substantially increasing structural capacity). Bridge inspection, along with load rating evaluation and PM, go hand-in-hand. The outcome of the evaluation determines the need for PM on any bridge.

Bridge owners typically apply systemic PM to elements or components of structures with significant remaining useful life. As a major part of bridge preservation, PM is a strategy for extending useful life by applying cost effective treatments to sound bridges (good or fair condition). The concept of preventive bridge maintenance suggests that a planned strategy of cost-effective treatments should be performed to keep bridges in good condition, retard future deterioration, and avoid large expenses in bridge reconstruction or replacements. Preventive maintenance includes cyclical (non-condition based) and condition-based activities as illustrated in Figure 2.

2.7 Bridge Preservation

Bridge preservation is actions or strategies that prevent, delay or reduce deterioration of bridges or bridge elements, restore the function of existing bridges, keep bridges in good condition and extend their life. Preservation actions may be preventive or condition-driven. Effective bridge preservation actions are intended to delay the need for costly reconstruction or replacement actions by applying preservation strategies and actions on bridges while they are still in good or fair condition and before the onset of serious deterioration. Bridge preservation encompasses preventive maintenance and rehabilitation activities (refer to Figure 2). An effective bridge preservation program: 1) Employs long-term strategies and practices at the network level to preserve the condition of bridges and to extend their useful life, 2)

Has sustained and adequate resources and funding sources, and 3) Has adequate tools and processes to ensure that the appropriate cost effective treatments are applied at the appropriate time.

2.8 Rehabilitation

Rehabilitation involves major work required to restore the structural integrity of a bridge, as well as work necessary to correct major safety defects based on the findings of bridge inspection reports and load rating analysis. As shown in Figure 2, bridge rehabilitation activities are considered bridge preservation. However, functional improvements such as adding a travel lane or raising vertical underclearance, while often considered rehabilitation are not considered preservation. These projects typically require significant engineering resources for design, a lengthy completion schedule, and considerable costs.

2.9 Replacement

Total replacement of a structurally deficient or functionally obsolete bridge with a new facility constructed in the same general traffic corridor. Similar to bridge rehabilitation, bridge replacement projects require engineering resources for design, a substantial and complex completion schedule, and considerable costs. Life cycle costs and other economic factors are usually considered when weighing rehabilitation versus replacement costs.

3. Conclusion

The methods of bridge inspection and non-destructive evaluation (NDE) are evolving, thus helping the bridge owners in making decisions for preventive maintenance, preservation and rehabilitation processes in a timely fashion. Looking forward, there are a number of research and developments still needed that could improve the tools and methodologies available to help ensure the long-term health and safety of bridges.

Funding is a major challenge for the aging infrastructure throughout the United States. While a new commitment to repair, rehabilitate and replace bridges in 2016 has finally captured the attention of elected representatives, including the President-Elect, there will still be a major back-log of structures that need to maintain their safety and functionality. Utilizing the methods discussed in this paper will help achieve the goal of an updated, start-of-the-art infrastructure.

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Mechanical Properties of Mechanically Anchored Blocks for Masonry Construction

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Abstract

Mechanical properties of a newly proposed solid concrete mechanically anchored block (masonry unit) were investigated in comparison with the ordinary masonry units used in the masonry wall constructions. These masonry units were investigated individually and in masonry assemblages to ascertain their mechanical characteristics. The investigation involved testing the masonry constituents (i.e. masonry units, and mortar / grout) for their characteristic compressive strength, followed by the testing of stack bonded masonry prisms (of both cases) for their characteristic compressive strengths. Masonry triplets were tested in unconfined direct shear test to determine the joint shear strength. The flexural bond strength of the masonry bond was tested using the z-shaped specimen test. Diagonal wall panels (of dimensions: 1219 mm × 1219 mm) were tested to ascertain the masonry shear strength in case of the mechanically anchored blocks. The experimental investigations concluded that although the mechanically anchored blocks exhibited a relatively lower compressive strength individually, the blocks perfomed better than the ordinary blocks in masonry form.

Keywords

Compressive Strength, Diagonal Tension Test, Masonry, Shear resistance, Walls.

1. Introduction

Walls; whether partition or load bearing, form a major portion of a Building Structure that may experience various lateral loading condition during its lifetime. It is therefore of great importance, that these walls must effectively resist the lateral loads without collapsing / falling down. Walls are primarily expected to sustain axial loads; however, due to the complexity of load transfer mechanisms these may be subjected to a combination of loading conditions, thereby making them predominantly prone to lateral excitations (i.e. earthquakes, wind, etc.). These lateral excitations can be either in out-of-plane or in-plane directions. Masonry construction technique is the primary method of constructing walls, and is used for its economy and ease of construction. Masonry walls are nonhomogeneous and anisotropic composite structures; made or built using masonry units, mortar, and steel reinforcement (if any). However, the use of unreinforced masonry in seismic zones is objectionable due to the experience of catastrophic failures (Meli, 1974).

Masonry walls perform very poorly if and when subjected to out-of-plane loading conditions, and as a result, fail in flexure. Whilst, when subjected to in-plane loading conditions the walls perform better due to increased stiffness by virtue of in-plane geometry, but will experience a brittle failure due to shear overstressing. Since, masonry having a relatively high compressive strength, is very weak in resisting bending and shear. Other failures i.e. overturning, inward-outward fall, and curtain-fall collapse are also observed in masonry walls.

Several methods i.e. confined masonry, reinforced masonry walls, and retrofitting have been proposed to increase the dynamic – lateral load resistance of masonry wall structures, however, these methods are not very popular either due to being uneconomical or due to lack of required masonry skills, especially in the underdeveloped and developing countries. The current study proposes an economical yet effective method of lateral and accidental load resistance by providing shear keys in the form of mechanical anchors in to the individual masonry units that in turn improve the joint shear strength so as to increase the lateral load resistance of the masonry walls. The mechanical properties of the masonry units and assemblages were determined in prepartion for the shake table test of the MAB walls, and the numerical modelling of the MAB walls.

2. Mechanical Properties – Experimental Investigations

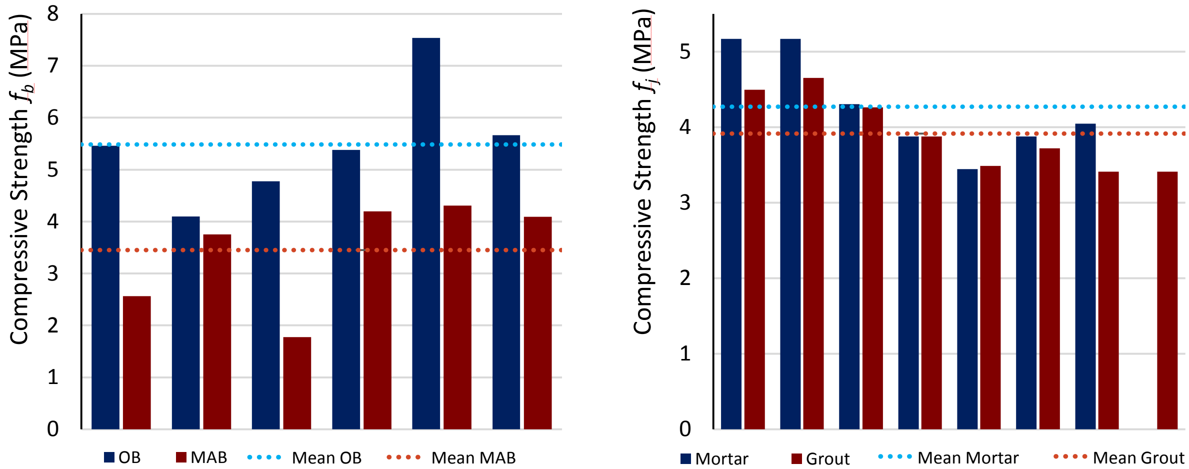
Two types of solid concrete masonry units (i.e. ordinary blocks, and mechanically anchored blocks) of dimensions 150mm × 150mm × 300mm; were manufactured (scale 1:1) for the purpose of this research. The mechanically anchored blocks were manufactured using a purpose-built two-part MS mould (See Figure 1) with manual compaction. The cones / anchors in the mechanically anchored blocks were of square-trapezoid shape of dimensions 50mm (base sides), 25mm (top sides) and 50 mm height. The masonry units were all made using CS (1:6) – based on standard practice, and were cured for 28 days before testing. The average mass of each type of masonry unit was around 14 kg. The experimental investigations were conducted in two regimes; wherein the first testing regime involved testing of masonry constituents (i.e. masonry units, and mortar), whilst, the masonry assemblages were tested for their mechanical properties in the second regime.

(a) (b)

Figure 1. Manufacturing of Mechanically Anchored Blocks
(a) Mould used in the manufacturing of the MAB. (b) Mechanically Anchored Block (MAB)

2.1. Compressive Strength of Masonry Constituents

Six (06) specimen for each type of masonry units (blocks) were tested for their characteristic compressive strength (f_b) in accordance with (ASTM:C140/C140M, 2013) by loading the samples on 150mm × 300mm face; wherein the average compressive strength of the blocks was found to be 5.486 MPa for ordinary blocks (OB) with a standard deviation of ±1.155 MPa, and 3.448 MPa with a standard deviation of ±1.03 MPa for mechanically anchored blocks (MAB). However, the average compressive strength (f_c') of the block forming material was determined as 6.736 MPa with a standard deviation of ±0.876 MPa, thereby indicating the strengthwise similarity between the OB samples and the block forming material.



(a) (b)

Figure 2. Characteristic Compressive Strength Comparison
(a) Ordinary Blocks vs. Mechanically Anchored Blocks (b) Mortar vs. Grout

For joint material, CSM (1:6) and lean cement based grout (w/c 0.7) were used for the OB and MAB masonry assemblages respectively. Mortar (07 specimen) and grout (08 specimen) cubes were tested for their compressive strength (f_j) in accordance with (ASTM:C109/C109M, 2016) at 14 days. The average compressive strength of the mortar specimen was determined as 4.27 MPa with a standard deviation of ±0.665 MPa, and 3.914 MPa with a standard deviation of ±497 MPa for the grout specimen.

2.2. Compressive Strength of Masonry Prisms

Eight (08) stack bonded type masonry prisms (consisting of four masonry units) with height to thickness ratio of 4.166 (correction factor: 1.159) were tested in accordance with (ASTM:C1314, 2003) for each case (i.e. OB, and MAB) at 14 days. The mortar joints' thickness in OB masonry prisms varied between 8-12 mm, while the thickness for the grout based joints in MAB prisms was not more than 7 mm. The prisms were capped using sulfur based capping.

(a) (b) (c)

Figure 3. Failure Modes in Masonry Prisms
(a) Face Shell Separation (b) Shear Break (c) Cone & Shear

The average characteristic compressive strength (f_m') was found to be 1.98 and 2.384 MPa for OB and MAB masonry prisms respectively, with standard deviations of ±0713 MPa and ±0737 MPa respectively. The average masonry efficiency (μ) was found to be 35.7% and 59.8% for the OB and MAB masonry prisms respectively. The modulus of elasticity (E_m) was determined using the method described in (Masonry Standards Joint Committee, 2011); and was found to be 436.5 MPa and 663.424 MPa for OB and MAB cases respectively.

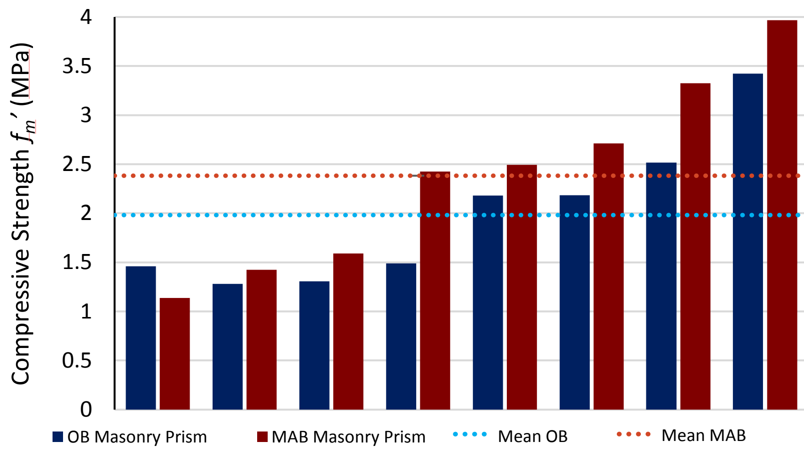


Figure 4. Characteristic Compressive Strength Comparison:
OB Masonry Prism vs. MAB Masonry Prism

Eurocode (CEN, 1996) provides an empirical relationship for predicting the masonry characteristic compressive strength by corelating it with the compressive strength of mortar (f_j) and masonry unit (f_b). $f_m' = K f_b^{\alpha} f_j^{\beta}$ where, (K, α and β) are coefficients proposed by various researchers and the codified provisions. A comparison of these empirical models has been given in Table 1.

Table 1. Comparison of Experimental Results with Analytical Predictions

Model	Experimental values (MPa)			Predicted f_m' values (MPa)										Remarks
	f_b	f_j	f_m'	Eurocode 6[d]		MSJC[e]		Dayaratnam[f]		Bennett[g]		Kaushik[b]		
Sarangapani[a]	8.2	3.1	2.3	2.76	{16.5}	2.78	{17.1}	1.39	{65.9}	2.46	{6.5}	2.54	{9.3}	
Sarangapani[a]	10.7	4.1	2.9	3.61	{19.7}	2.78	{4.3}	1.82	{59.2}	3.21	{9.7}	3.16	{8.3}	
Kaushik[b]	20.8	3.1	4.1	5.29	{22.5}	2.80	{46.3}	2.21	{85.7}	6.24	{34.3}	4.00	{2.4}	
Kaushik[b]	20.8	15.2	6.6	8.52	{22.5}	2.80	{135}	4.89	{35.0}	6.24	{5.8}	6.66	{0.9}	
Kumar[c]	3.29	5.06	0.84	1.68	{50.1}	2.77	{69.6}	1.12	{25.1}	0.99	{14.9}	1.90	{55.7}	Unplastered at 7 days
Present Study	5.48	4.97	1.98	2.39	{17.3}	2.77	{28.5}	1.44	{38.0}	1.64	{20.4}	2.42	{18.2}	OB Prism at 14 days
Present Study	3.45	3.91	2.38	1.61	{47.7}	2.77	{13.9}	1.01	{135}	1.04	{130}	1.79	{33.1}	MAB Prism at 14 days

{} shows percentage error.
a. (Sarangapani et al., 2005); b. (Kaushik et al., 2007); c. (Kumar et al., 2016); d. (CEN, 1996); e. (MSJC, 2011); f. (Dayaratnam, 1987); g. (Bennett et al., 1997)

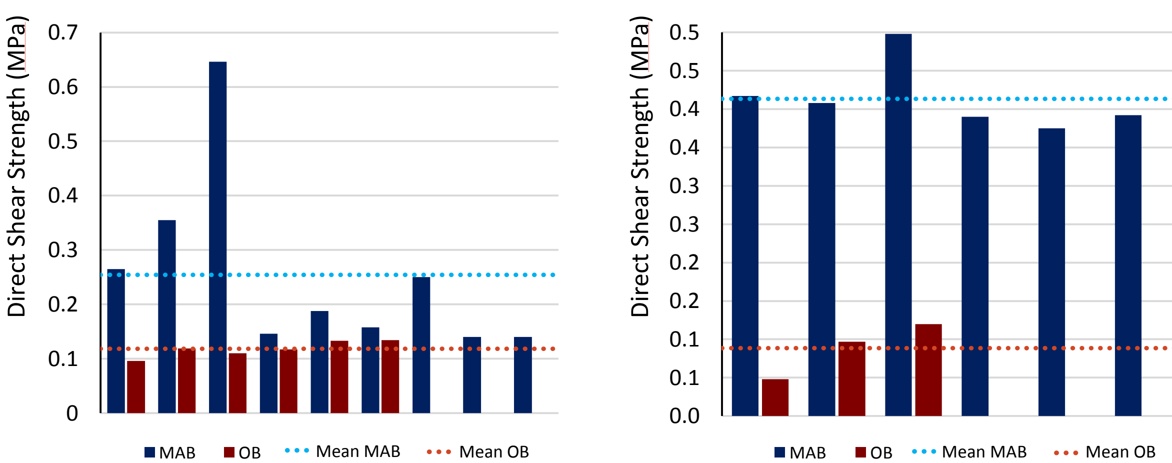
2.3. Direct Shear Test on Masonry Triplets

Unconfined direct shear test on OB and MAB masonry triplets was performed to evaluate the joint shear strength. The OB masonry triplets were bonded together with CSM (1:6), while lean grout was used as bond in MAB masonry triplets. Two set of tests were performed on the samples aged 14 days; wherein the load was applied on the 150 mm × 300 mm (header face parallel to line of action – shiner orientation), and 150 mm × 150 mm face (stretcher face parallel to the line of action – sailor orientation) of the middle block in the first and second test sets respectively.

(a) (b) (c)

Figure 5. (a) MAB First Set - Load applied on the 150 mm × 300 mm face.
(b) MAB Second Set - Load applied on the 150 mm × 150 mm face.
(c) OB Second Set - Load applied on the 150 mm × 150 mm face.

In the first set; the average shear strength of the OB masonry triplets was 0.118 MPa with a sample standard deviation of ±0.014 MPa, and 0.254 MPa with a sample standard deviation of ±0.164 MPa for the MAB masonry triplets. In the second set; the average shear strength of the OB masonry triplets was 0.088 MPa with a sample standard deviation of ±0.037 MPa, and 0.413 MPa with a sample standard deviation of ±0.044 MPa for the MAB masonry triplets.



(a) (b)

Figure 6. Unconfined Direct Shear Test Strength Comparison
(a) First Set – Load applied on the 150 mm × 300 mm face
(b) Second Set – Load applied on the 150 mm × 150 mm face

2.4. Flexure Bond Strength Test

The masonry bond strength is of prime importance in masonry walls subjected to forces applied normal to the face of wall. (i.e. wind, eccentric gravity loads etc.) Various test procedures have been proposed to determine the tensile bond strength i.e. bond wrench test, direct tension test, and crossed couplet test etc. A new z-shaped specimen (tensile strength of masonry bond test) was proposed by (Khalaf, 2005); wherein two masonry units staggerred together in z-shape with a bonding agent. The test provides a simple way to evaluate the flexural bond strength by bending.

Figure 7. Z-Shaped Specimen (Flexure Bond Strength)

Three (03) specimens were constructed with two ordinary blocks bonded together by a 10 mm to 14 mm mortar in staggered arrangement and cured for 14 days. The specimens were loaded to failure by applying the load at a displacement-controlled rate of 1 mm/min. The flexural bond strength f_{fb} was determined using linear distribution of the bond stress at the block-mortar interface. The average flexural bond strength was determined as 0.347 MPa (50.42 psi) whilst the flexural bond failure load was 3.7 kN (0.832 kips).

2.5. Diagonal Tension Test

Diagonal wall panels (1219 mm × 1219 mm) of mechanically anchored blocks were made; wherein the masonry units were held together with 4-7 mm lean grout were tested in accordance with (ASTM E 519-02, 2002). The age of samples when tested was 7 days. Capping was done using cement paste and plaster of Paris before placing the loading shoe. Strain displacement was recorded using four (4) string potentiometers attached near the four corners. Readings for the load were taken from a calibrated Load Cell.

(a) (b) (c)

Figure 8. **(a) Sample 1: Shear Failure accompanied by slip of the first masonry course; (b) Sample 2: Shear Failure accompanied with tension cracking at the free edges; (c) Sample 3: Failure due to tension.**

Dimensional variation for all three samples was less than 0.2%. The panels performed similar to the plastered wall panels as tested by (Kumar et al., 2016) having an average shear strength of 0.3136 MPa. Whereas, the failure shear strains were found to be in the range of 1.53×10^{-3} and 3.675×10^{-2}, with an average shear strain of 1.35×10^{-2}. The average shear modulus (G) was determined as 167.0363 MPa.

Focardi & Manzini (1984) have reported four types of failure modes in diagonal wall panels. The wall panels tested for this research showed two different modes of failure. DT-S2 failed when a tension crack along the loading diagonal was formed. The samples DT-S1 and DT-S3 failed due to sliding of the first masonry course along the masonry joint.

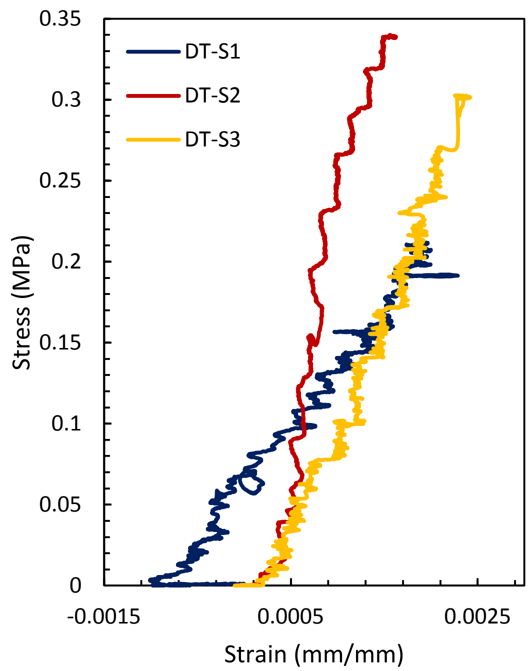


Figure 9. Stress-Strain Curve from Diagonal Tension Test

3. Conclusion

The mechanical properties / characteristics of the masonry constituents and the masonry assemblages has been presented following the experimental investigation.

The average compressive strength of OB units was higher than that of the MAB units. The mechanically anchored blocks were more susceptible to damage at or near the female cone area. The ordinary blocks by virtue of their uniform shape and rigid build were relatively stronger and facilitated better stress distribution.

However, the average characteristic compressive strength of the MAB masonry prisms was found to be significantly higher than the OB masonry prisms. Therefore, higher masonry efficiency for the MAB case was observed. The stress-strain behaviour in compression for the MAB masonry prisms was steeper compared to the OB masonry prism, resulting in higher modulus of elasticity of the MAB masonry prisms.

The tensile (flexural) bond strength of the OB masonry form was determined as 0.347 MPa.

The joint shear strength of the masonry assemblages was determined by unconfined direct shear test of masonry triplets, which yielded better results for the MAB case in both set of tests; as the resistance to direct shear in or near masonry joints was significantly higher or greater for the MAB masonry triplets due to the mechanical lock-fix mechanism, whilst ordinary blocks performed poorly in the test.
The shear strength of MAB masonry wallets as determined by the diagonal wall tension test was in the range of 0.29 – 0.34 MPa.

4. Acknowledgments

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Shear Failure Mechanism and Slip Behaviour of Inclined Headed Shear stud connectors in the Steel-Concrete Composite Beam

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Abstract

This paper describes the local behavior of shear failure due to the application of inclined shear stud connectors on steel-concrete composite beam instead of using vertical shear stud connectors and the shear failure mechanism of inclined shear stud connectors influences on the ultimate limit state of composite beam. In order to study the hypothesis of the shear failure mechanism of inclined shear stud connectors, Finite Element (FE) models were used such that FE models of steel-concrete composite beam with vertical shear stud connectors were developed in the categories of full and partial shear connections and validated using existing experimental model. The validated model was then used to develop the composite beam with inclined shear stud connectors and studied the shear failure behavior. Moreover, the analyses were done based on the stresses developed on the shear stud connectors such that fractures of the concrete and shear stud connectors were determined in order to analyse the shear failure mechanism. In addition, the distribution of the stresses induced that are local to the dowel, was monitored throughout the analysis by nodal stresses while monitoring the deformation of the shear stud connectors. In comparison of the composite beam with inclined and vertical stud connectors, it was found that there is an advantage in the developed shear failure mechanism where the shear stud connectors loaded as inclined positions.

Keywords

Inclined shear stud connectors, Composite beam, Shear failure mechanism, Ultimate limit state, FE modelling.

1. Introduction

The mechanical shear stud connectors of the steel-concrete composite beam, which are a small device that transmitted high shear forces from concrete slab to steel beam, are key components in limiting ultimate limit state of the composite beam. The function of the shear stud connectors in between concrete slab and steel beam is extremely complex and the ultimate limit state is influenced by the shear flow forces developed in the shear stud connectors, which have to be resisted by the bond in between shear stud connectors and concrete slab. Bavan et al. (2016)[a, b, c] presented that the ultimate limit state of the composite beam is improved by the application of shear stud connectors as inclined positions and they have studied with full and partial connections including various parameters such as slab thickness and dimensions of connectors. Even most research studies to date have examined the stud connectors, there is no research study existed in the local behavior of the shear stud connectors connected as inclined position such that this paper studies the local behavior of the inclined shear stud connectors.

2. REVIEW OF LETERATURE ON STEEL-CONCRETE COMPOSITE BEAM WITH FULL AND PARTIAL CONNECTION

The test setup and analysis were adopted from the experimental program done by Tan et al. (2009), which is reviewed in this research study. It should be noted that the study was the composite beam subjected to combined flexure and torsion, and the parameters of full and partial shear interaction also were analysed. But, the composite beams subjected to flexure only, which were two specimens labelled as CBF1 and CBP1 as controlled samples, were used in this research study. Both specimens were designed with similar structural components with the exception of numbers of shear stud connectors and shear stud connectors were included based on 100 % and 50 % of shear interaction as indicated in the Figure 1(a) and Figure 1(b), respectively. Each specimen consisted of two structural components lengthen 4600 mm of a steel section classified as 200UB29.8 and a 120 mm thick and 500 wide concrete slab reinforced by 12 mm and 10 mm steel bars in longitudinal and transversal directions, respectively. The dimensions of shear shear stud connectors used in the experiment were with a nominal diameter of 19 mm and with a pre welded length of 95 mm. Since the locations of load application were varied because of the prediction of results in terms of parameters of the research, the experiments labeled as CBF1 and CBP1 were control samples, which of the vertical loads were applied on the top of concrete slab of both specimens at two locations such as 1/3 and 2/3 of beam from one end of the beam. In these specific specimens, the supports were applied on the bottom of the steel beam as roller system at one end and as pin system at another end. The ultimate limit states for both specimens of CBF1 and CBP1 at the given flexural moment against deflection at mid-span of the beam were presented as 220 kNm and 188 kNm, respectively.

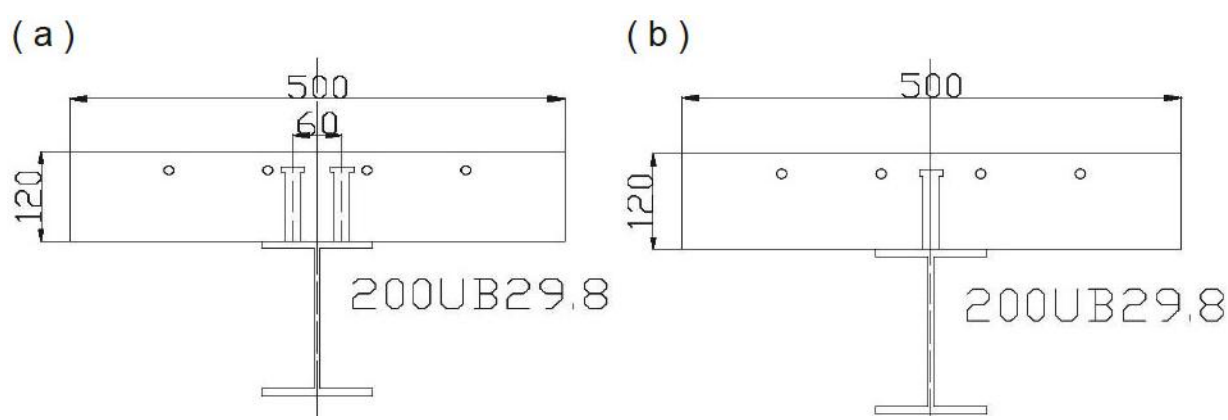

Figure 1: Cross section view of test specimen (a) Full shear connection (b) partial shear connection

3. NUMERICAL SIMULATION OF THE EXPERIMENT AND VALIDATION

Based on the experimental programs done by Tan et al. (2009), the material geometries were developed in the commercial software ABAQUS by considering the differences of full and partial shear connections such that there were two categorized FE models in terms of numbers of shear stud connectors. Both FE models were analysed individually by Abaqus explicit solver with the prediction of quasi static solution.

The solid components such as concrete slab, steel beam and shear shear stud connectors were discretized using typical 3D elements of continuum, 3D, 8-node reduced integration element (C3D8R) with adapting suitable meshes for 3D elements. The effects of reinforcing bars were not important in this study and so, after a preliminary study, it was predicted to discretize the truss, 3D, 2 node elements T3D2 element, which were economical with slight differences in results corresponding with solid elements. The boundary effects of all material geometries on the response during loading were imposed by developing interaction algorithms. The bottom surface of the concrete slab with the top surface of steel beam was adopted the boundary effect such that the surface-surface contact algorithm was applied and simultaneously, sides of shank and top, sides and bottom of head of shear shear stud connectors with concrete slab were in boundary effects during deformation of shear shear stud connectors while load application such that the surface-surface contact algorithm was adopted. In addition, the material geometries steel beam and shear shear stud connectors were formed as one part with the consideration of its welded connections. Taking into account the surface-surface contact algorithm leads to determine the nodal sliding, penetration and separation such that the nodal sliding in between the material geometries was determined by tangential behaviour, and separation and penetration of nodes were determined by normal behaviour. Taking into account the wires created for all reinforcing bars leads in linking of reinforcing bars inside the concrete slab by embedding constraint algorithm. In order to consider the supports in the FE model similar to the experiments, the base of the mesh in the specific two locations at a spacing distance of 4000 mm were as fixed constrained at one end and as a roller at the next end. The test was simulated by applying a load at two locations symmetrically using a steel beam from the center in the category of patch load.

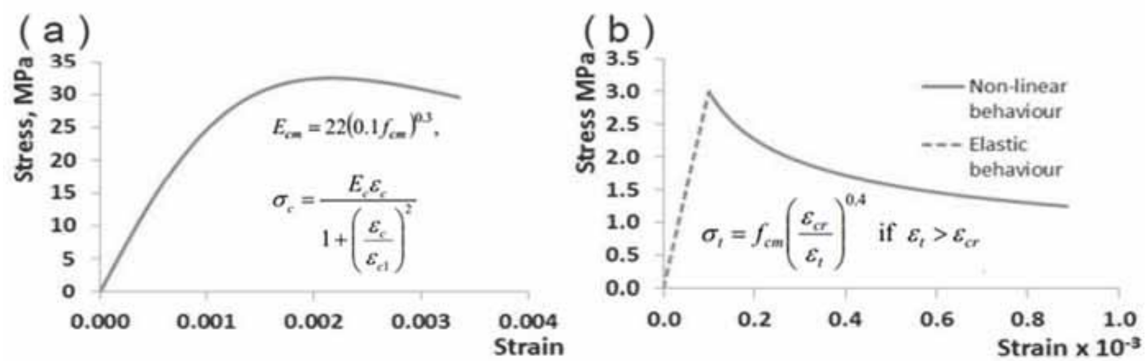

Figure 2: Concrete material property (a) Compressive behaviour (b) Tensile behavior

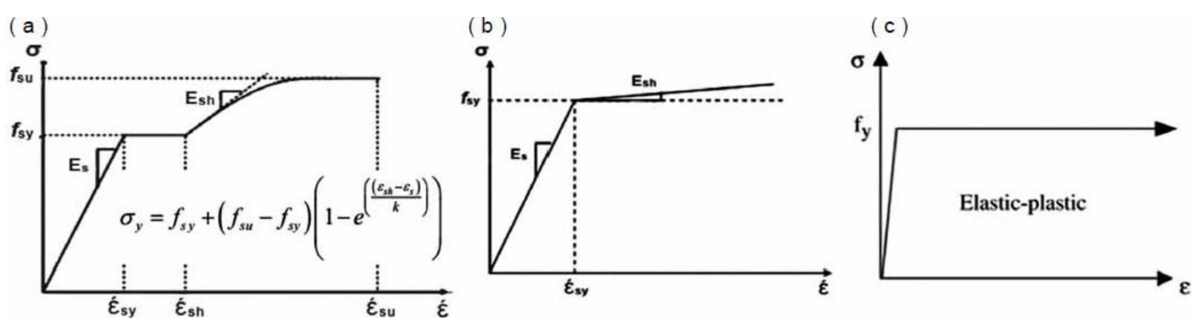

Figure 3: Material property (a) Steel beam (b) Reinforcing bars (c) Shear connectors

The involved material components in the composite beam are steel and concrete, specifically, the steel are with various parameters in the components of steel beam, shear stud connectors and reinforcing bars. The parameters of concrete and various steel components were derived from the available results of the tests done by Tan et al. (2009). The plastic damage model proposed by Lubliner et al. (1989) was included in order to determine the biaxial parameters of concrete with considering the plasticity constitutive parameters, which were introduced by Kmiecik and Kaminski (2011). In particular, the uniaxial parameters of concrete are in two different criteria such as compressive and tensile strengths and the compressive strength is very much higher than tensile strength and those were obtained by cylinder test and indirect tensile splitting test on the same day of the experimental analysis. In view of these considerations, the parameters were implemented in the material model of concrete as a set of twelve numbers of parameters in uniaxial stress, strain correlations under compression and tension separately using proposed derives by Desay & Krishnan (1964) and Eurocode (1994) as shown in the Figure 2(a) and Figure 2(b), respectively. The material geometries of steel were included typical coupon test to determine parameters of yield stress, ultimate stress and the percentage of elongation of flange and web of the steel beam and reinforcing bars. The elemental formulation proposed by Gattesco (1999), which gives better non-linear behaviour to the structural steel beam, was developed as indicated in the Figure 3(a). The reinforcing bars consist of steel bars the behavior of which was assumed to have a bilinear stress, strain characteristic behavior as shown in the Figure 3(b). In addition, the parameter of ultimate strength of the shear shear stud connectors was determined by push test. Moreover, the shear shear stud connectors consist high strength steel the behavior which of assumed to be elastic perfectly plastic for both tensile and compressive behaviours as indicated in the Figure 3(c).

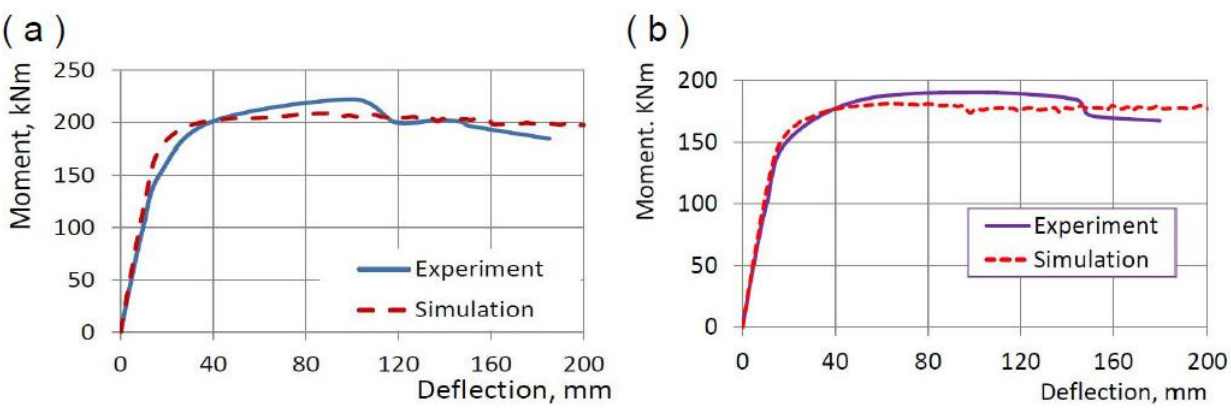

Figure 4: Comparison of moment-deflection relationship of composite beam (a) Full shear connection (b) Partial shear connection

The developed model were included the convergence studies in both categories, such as composite beam with full and partial connections. In the view convergence study, different mesh configurations which of each material component were included with the course, medium and fine meshes, were adopted to study and compare the ultimate moment-deflection relationship and material fractures. The FE model was finalized where the very close results were monitored by saving computational time. The stiffness of the composite beam in the finite element model showed the great agreement throughout the analysis with the model of experimental analysis. As predicted, the analysis of the results presented in the Figure 4 (a) shows close agreement in the moment-deflection relationship throughout the analysis up to the end for a composite beam with full shear connection. In addition, Figure 4(b) shows that, obviously, very close in the elastic state until the shear connection failure in between FE model and experiment and continuously, as ultimate limit state, the experiment and FE model were given very close agreement for a composite beam with partial shear connection. Based on the figures shown, as the results obtained by the FE solution have a good agreement with experimental results, it could be concluded that the FE models developed can be used for parametric studies.

4. PARAMETRIC STUDY

The validated FE models were then replaced by inclined shear shear stud connectors as shown in the Figure 5 and a research study was carried out in the determination of effects due to the inclined shear stud connectors of the composite beam with full and partial shear connection subjected to combined axial loads. While developing the concrete slab and steel elements similarly, the shear stud connectors were developed in an inclined form embedded in the concrete slab. The number of shear stud connectors in the FE model of the composite beam with full and partial shear connection were applied as same with experimental programme done by Tan et al. (2009) which of the numbers of shear stud connectors were satisfied the design models established in Eoro Code in terms of full and partial shear interaction. It was concluded in the previous research studies done by Bavan et al (2016) that an increase of 12.1% in the ultimate moment capacity of composite beam with partial shear connection subjected to inclined shear stud connectors was predicted by the comparison of ultimate moment capacity of composite beam with partial shear connection subjected to vertical stud connectors. In addition, there was not any reduction observed in the ultimate limit capacity of the composite beam with full shear connection in the subjection of inclined stud connectors.

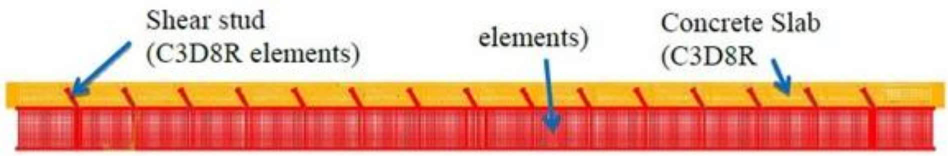

Figure 5: FE model of composite beam with inclined shear stud connectors

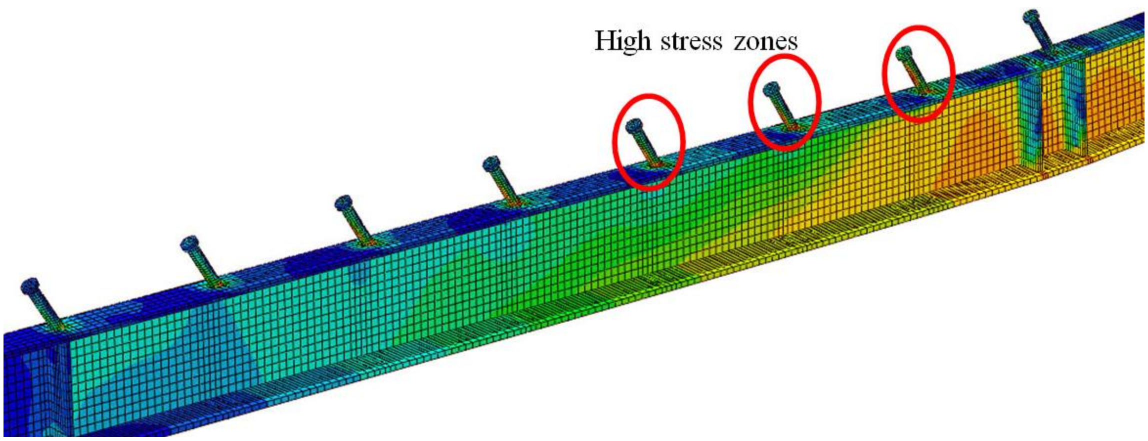

Figure 6: High stress zones developed in the inclined shear shear stud connectors of the composite beam with full shear connection.

Subsequently, the local behavior of shear stresses developed in the material components and those influences on ultimate limit state were studied and extensively discussed in this paper. The concrete slab and shear stud connectors near the loading point where the composite beam with full shear connection were predicted with induced high stress zones, while in the composite beam with partial shear connection, high stress zones developed in the shear stud connectors near the supports. It could be understood that these stresses developed due to the slip of the shear stud connectors and surrounding concrete of the particular connectors also was predicted with high stresses, which could be determined cracking and crushing failure of the concrete. In order to simply this study, the failure zones were determined accordingly such that the concrete slab and shear stud connectors near the loading point in the case of full shear connection as shown in the Figure 6 and concrete and shear stud connectors near support in the case of partial shear connection as shown in the Figure 7 were monitored throughout the analysis and the behaviours were compared in between vertical and inclined stud connections. While similar behaviours of moment at mid span against deflection in the elastic state were predicted in the comparisons in between vertical and inclined shear stud connectors in the applications of full and partial shear connections, the slip was induced in the application of the partial shear connection in the plastic state and the slip was observed to be postponed in the composite beam with inclined stud connectors. The gap induced by slip, which was in between the undeformed and deformed of the shank of the shear stud connectors, was monitored and early deformation was observed in the composite beam with vertical shear stud connectors in both applications of full and partial shear connections such that the shear flow forces can be resisted well by the inclined shear stud connectors than vertical shear stud connectors and accordingly, it is beneficial in the application of shear stud connectors as inclined position.

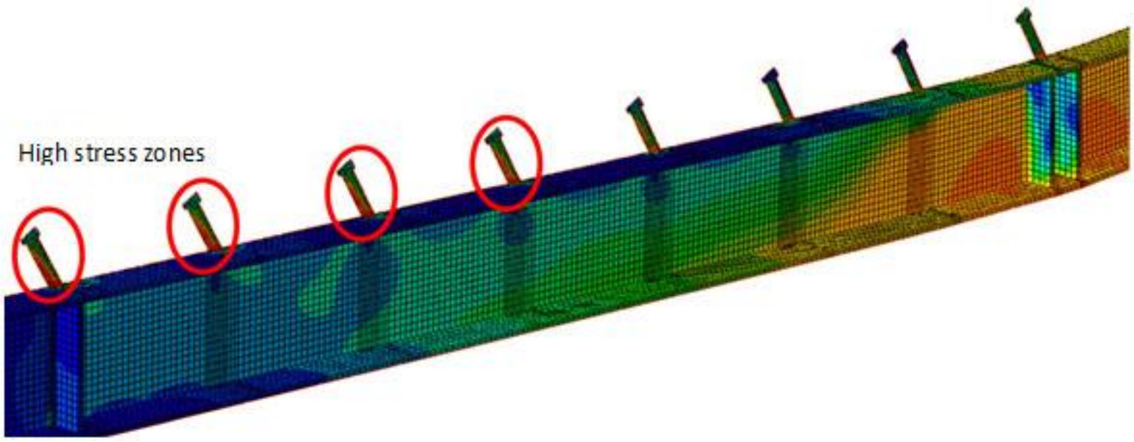

Figure 7: High stress zones developed in the inclined shear stud connectors of the composite beam with partial shear connection.

In addition, the nodal displacements of the high stress zones of shear stud connectors in the plastic state were monitored throughout the analysis against deflection and reduction in slip was observed in the application of inclined shear stud connectors such that it could be moreover concluded that the slip behavior was improved with the application of shear stud connectors as inclined position. The strain at surrounding concrete and shear stud connectors were monitored against applied load and based on the material properties, the failure of the surrounding concrete and specific shear stud connectors in that location were determined in terms of fracture of the elements. In terms of the stress strain behavior of the material components, the inclined shear stud connectors and surrounding concrete were reached those maximum strains later than the vertical shear stud connectors and those surrounding concrete such that the inclined shear stud connectors in the bearing zones have to be restrained highly to resist the stresses developed than vertical shear stud connectors. As fracture is occurred prematurely by shear stud connectors in the composite beam with partial shear connections, the ultimate moment of the composite beam increased with the application of inclined shear stud connectors such that it could be determined that the ultimate state of the composite beam can be influenced by the actual slip capacity of the composite beam.

5. CONCLUSIONS

In the full scale composite beam, as the shear stud connectors will be loaded indirectly, a full scale experimental analysis was included in this research study to determine the local behaviors of the inclined shear stud connectors. This research study brings that the application of the shear shear stud connectors as inclined position were improved the resisting behavior of shear stud connectors. The local parameters maximum slip, increase in slip with deflection and reduction in slip due to deformation behavior were monitored in between the application of shear stud connectors as inclined and vertical position and it could be concluded ultimately that the composite action is improved by the application of inclined stud connectors.

5. Acknowledgements

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STRENGTHENING OF REINFORCED CONCRETE BEAMS WITH TEXTILE REINFORCED MORTAR (TRM) IN FLEXURE

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ABSTRACT

In recent years, significant research related to strengthening of Reinforced Concrete (RC) structures has focused on usage of fibres in flexure and shear. For flexural strengthening the fibres applied on the tension face contribute effectively to the load carrying capacity and serviceability. Use of Fibre Reinforced Polymers (FRPs) for strengthening of RC often resulted in the undesirable failure mode governed by premature debonding of FRP strips. In recent past, researchers have been testing alternate ways to bond the fibres using cementitious mortars instead of epoxies as used for FRPs, known as Textile Reinforced Mortars (TRMs). The use of TRMs surpasses the epoxies in terms of bond behaviour. This paper investigates the effect of length variation of TRM strips on the Load carrying capacity, failure mode and effect of anchorage of RC beams strengthened in flexure using TRM. Six RC beams are used to investigate the above-mentioned effects, out of which one serves as control beam while the rest have varying strip length. One of the latter was provided with anchorage to investigate the contribution of anchorage. The results showed encouraging results in terms of bond behaviour and ductile failure mode of the beams.

KEYWORDS:

Textile Reinforced Mortar, Beams strengthening, Bond Length, Load Deflection, Fibre Composites.

1. INTRODUCTION

Strength upgradation of reinforced concrete structures has been one of the major focus areas of researches pertaining to structural engineering. The need of upgradation or strengthening may arise as a result of drop of strength in the actually constructed structure due to aging of concrete, environmental effects, application of external forces like earthquakes etc., rusting of rebars, or due to introduction of new code requirements. Usage of fibres like FRP and TRM have been a popular solution in this regard because of advantages like light self-weight, ease of application, (Teng et al. 2002), FRPs has shown shortcomings like poor response at high temperature, expensive epoxies which are hazardous for manual workers due to the types of solvents used, troublesome application on wet surfaces and lower temperatures, substrate incompatibility with epoxy resins (Olivito et al. 2013).

Further investigations with respect to fibres has shown that cementitious composites are good in overcoming the aforementioned issues with the usage of FRP (Garcia et al. 2010). The Matrix-Fibre interaction in case of usage of cementitious matrix can be improved by replacing fibre sheets with two directional fibre grids. The mechanical attributes of textile are easily altered through the roving spacing which could be adjusted as per the required degree of penetration of mortar needed via openings. The mortar to be used should have strong interface bonding properties, less shrinkage, high workability and considerable tensile and shear strength to avoid debonding failure of substrate and the textile grids. The application of textile grids for reinforcement of cementitious based materials has been done since back in 1980's (Gardiner and Currie 1983), but recently the focus has started to shift from epoxy based binders to cementitious based binders due to aforementioned reasons. When textile fibre grids are used in conjunction with cement based binders, they are termed as Textile Reinforced Mortar (Triantafillou and Papanicolaou 2006). The modes of failures that such systems may come across include the mesh being pulled out from the matrix, debonding of fibres at the fibre-matrix junction, composite debonding at the concrete interface, and rupture of fibres outside the bond length (Carloni et al. 2013, D'Antino et al. 2014, 2015). TRM has shown encouraging results for flexural (Pareek et. al. 2007, D'Ambrisi and Focacci. 2011) and shear strengthening (Blanksvärd et al 2009, Khan and Masood 2016) of RC members and needs to be further tested and investigated. TRM may help in reduction of concrete cover required for protection of rusting of steel, low self-weight, and is effective in improving both failure loads and deflection behaviour of beams in both shear and flexure (Brückner et al. 2006, Triantafillou and Papanicolaou 2013). The number of layers and the arrangement of TRM (strip, U-wrap) also influence the effectiveness of TRM systems (Aljazaeri and Myers 2015).

This study explores the influence of change of length of TRM strips on the failure loads, mode of failure and effect of anchorage of RC beams detailed as per ACI 318-08, strengthened in flexure using TRM. This is explored in the paper using experimental results of TRM strengthened beams in flexure with varying TRM lengths and presence or absence of end anchorage as per the experimental program discusses in the following sections.

2. EXPERIMENTAL PROGRAM

Total of six (06) RC beams were cast for the study. The length of the beams was 1.82 m each, with a section of 152 mm × 203 mm. All the beams were designed in accordance to the ACI 318-08 code, using steel stirrups as the shear reinforcement provided 10 mm bars @ 7mm c/c and longitudinal reinforcement was were 2-12 mm bars.

Beam ACI-1was taken as control beam with no application of TRM, while remaining five (TRM-A series and TRM-B1) were strengthened with TRM in Flexure with varying lengths as discussed in Table 1. eam TRM-B1 was also provided with anchorage to observe its impact in load carrying capacity. All the beams were tested with shear span to depth (a/d) ratio of 4.27. Uniaxial compressive of concrete used for the beams was 20.68 MPa (3 ksi) while yield strength of steel was found to be 413.68 MPa (60 ksi). Two longitudinal 12 mm bars were provided in the tension zone and the same at the top for hanging the 10 mm stirrups which were provided at 82 mm c/c as shown in Fig. 1. The nomenclature of beams is given in Table 1.

Table 1. Nomenclature of beams

No.	Beam ID	Beam Description	TRM Anchorage at support (mm)
1	ACI-1	Control Beam with reinforcements according to ACI Code (No TRM)	-
2	TRM-A1	Beam with applied TRM length=800 mm at midspan	No Anchorage
3	TRM-A2	Beam with applied TRM length=1100 mm at midspan	No Anchorage
4	TRM-A3	Beam with applied TRM length=1400 mm at midspan	No Anchorage
5	TRM-A4	Beam with applied TRM length=1700 mm at midspan	No Anchorage
6	TRM-B1	Beam with applied TRM length=1700 mm at mid-span end anchorages on each side	150

For TRM application, high strength mortar was used, with a 28 days strength of 26.2 MPa (3.8 ksi). The length of longitudinal strips were varied from 800 mm to 1700 mm with an increment of 300 mm per beam type. TRM-B1 was provided with 150 mm width U-shaped anchorages. The thickness of TRM strips and wraps was taken as 5 mm as shown in Fig. 2 and 3. The beams were tested in four point bending at a loading rate of 0.5 kN/min. Deflections and crack progression was noted up to the failure of the beams.

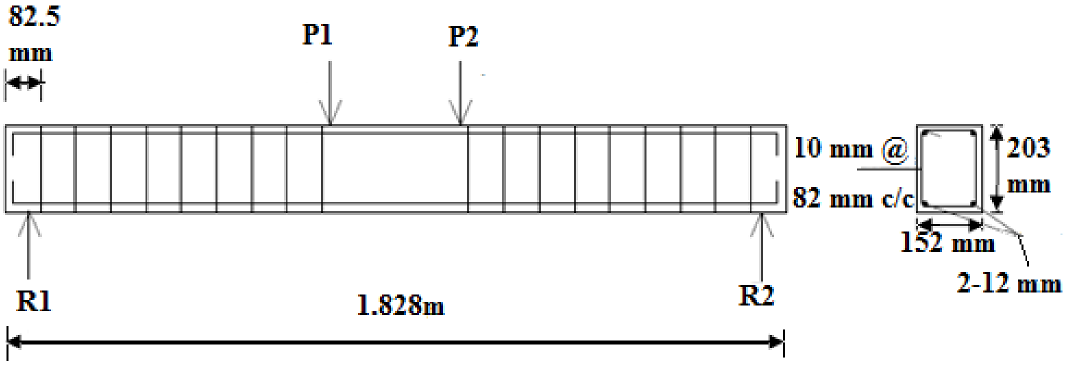

Fig. 1. R/F details for ACI-1 beam with a/d ratio 4.27

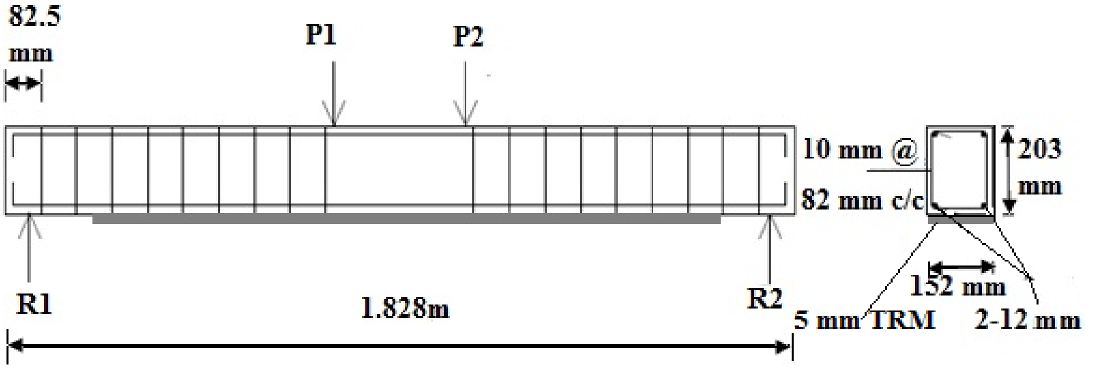

Fig. 2. R/F details for TRM-A beams without anchorage a/d 4.27

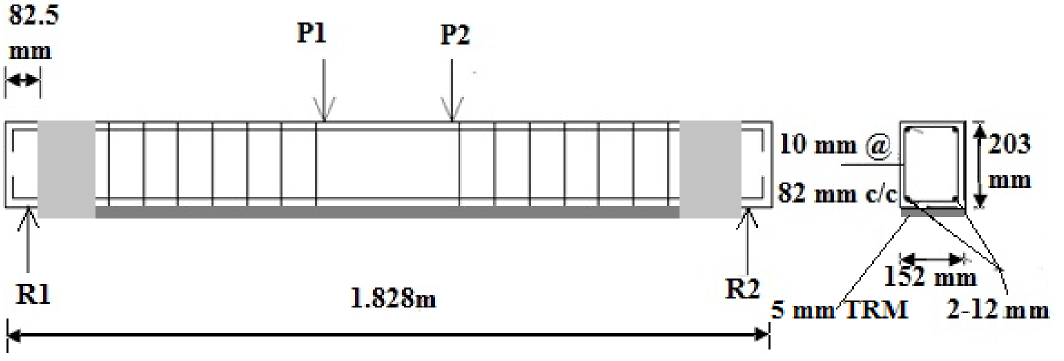

Fig. 3. R/F details for TRM-B1 beam with anchorage a/d ratio 4.27

3. RESULTS AND DISCUSSIONS

Results of control and TRM strengthened RC beams tested under four-point bending are discussed in terms of their failure modes, ultimate load carrying capacities and load-deflection curves in following subsections.

3.1 FAILURE MODES AND LOAD CARRYING CAPACITIES

All the beams, control and strengthened (ACI-1, TRM-A 1-5, TRM-B1), failed in flexure. No significant increase in load carrying capacities was noted in the beams TRM-A1 and TRM-A. Other strengthened beams depicted significant increase in load carrying capacity. The control beam and all the TRM strengthened beams failed in a ductile manner in flexure, with flexural cracks appearing at the mid span, which progressed towards the top until the crushing of concrete. For beams with TRM, as the cracks in mortar widened, the basalt fibres were exposed but no rupture or fibres or debonding of TRM took place.

The control beam had wider cracks (widest crack 3 mm at failure) which were spread from the mid span to up to 150 mm before the supports. For strengthened beams the major flexural crack was similar but lesser in width (between 1 to 2 mm). For beam TRM-A1, the portion with TRM had closely spaced narrower cracks. The crack spacing was increased in the portion without TRM and the cracks in the region without TRM were similar to those in the control beam. The beam TRM-A1 also had wider cracks similar to the control beam. For the rest of the strengthened beams more evenly distributed closely spaced cracks appeared which were lesser in width to those as compared to the control and TRM-A1 beam. For strengthened beams other than TRM-A1, the cracks didn't extend beyond midway between the mid span and the support, eliminating the shear-flexure zone cracks as the TRM strip length was increased. No significant difference in the crack patterns of beams TRM-A4 and TRM-B1 was observed as shown in Fig. 4.

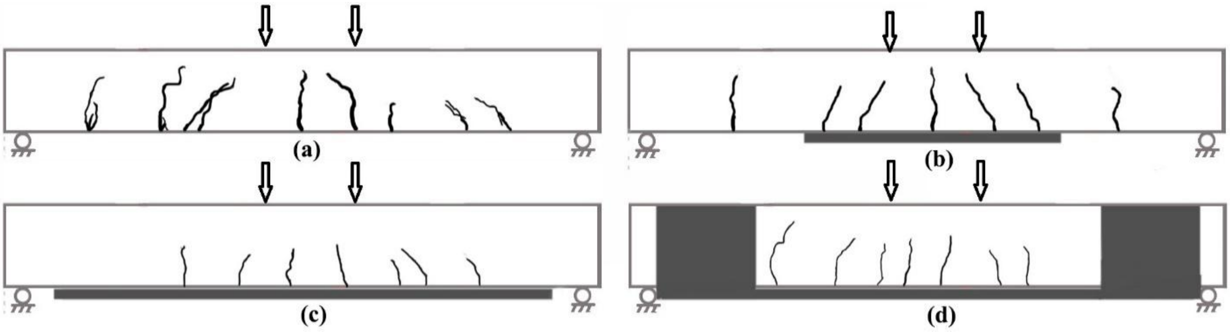

Fig. 4. Crack Patterns for (a) Control (b) TRM-A1 (c) TRM-A4 (d) TRM-B1 beams

The beams TRM-A3, TRM-A4 and TRM-B1 showed 22.73%, 12.63% and 11.05 % increase in failure loads respectively as compared to the control beam. While the results of TRM-A4 and TRM B-1 showed similar effect in failure loads, the higher percentage increase for TRM-A3 may correspond to the higher yielding load of the same beam. The beam TRM-B1 did not show any impact on the failure load although the anchorage added to the ductility of the beam. Failure modes and load carrying capacities are summarized in Table 2.

Table 2. Results comparison of beams

S. No	Beam ID	TRM Strip Length (mm)	Max. Load at Yielding (kN)	Failure Load (kN)	Increase in Load Carrying Capacity
1	ACI-1	-	48	51.23	-
2	TRM-A1	800	48	51.13	0%
3	TRM-A2	1100	49	51.86	1.23%
4	TRM-A3	1400	60	62.875	22.73%
5	TRM-A4	1700	52	57.7	12.63%
6	TRM-B1	1700	50	56.89	11.05%

3.2 LOAD DEFLECTION CURVES

Load-Deflection curves for control and strengthened RC beams are shown in Figure 5 which reflects that the load-deflection curves of control and strengthened RC beams are very much similar with strengthened beams showing higher load carrying capacities and lesser deformation. All the beams showed ductility and central deflection at failure has improved for TRM strip lengths 1400 mm and beyond is more or less the same. Load-deflection curves for beams TRM-A1 is almost the same as beam ACI-1 but rest of the beams strengthened with TRM are stiffer than both the control beam ACI-1 and TRM-A1. The higher yield points for TRM-A3, TRM-A4 and TRM-B1 may be a result of better crack control due to the usage of TRM over almost the entire span of the beam causing lesser stresses in steel at higher loads. The crack control may also have played its part to give stiffer response for these beams.

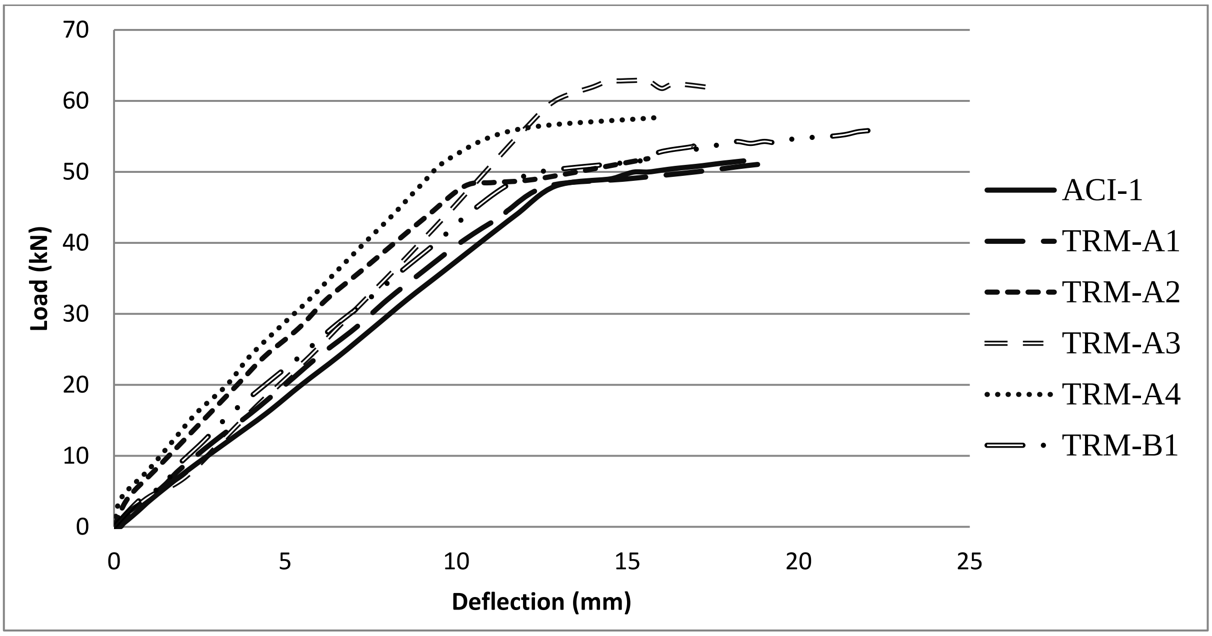

Fig. 5. Load-Deflection curves of Beams ACI-1, TRM-A1-4 and TRM-B1 at a/d 4.27

The load carrying capacity increases as the TRM strip length is increased especially after TRM strip length 1400 mm and beyond have shown significant increase. The increase in difference of loads at yielding and failure has also increased with TRM strip lengths 1400 mm and beyond depicting the impact of TRM. By comparing results of beam TRM-A4 and TRM-B1, which have the same TRM strip length, it is observed that no significant difference in terms of yielding and failure loads was observed.

4. CONCLUSIONS

Following conclusions are drawn from the study:

- Test results indicate that TRM that uses Basalt Fibres is effective in flexural strengthening of beams and also enhances in the performance of the beam with respect to serviceability.
- Although no debonding failure was observed in any of the TRM strengthened beams, the tests showed that beams past 1400 mm reflected better performance in terms of crack and deflection control, thus the strips should be applied to the entire length of the beams to ensure the same.
- The strip lengths past 1400 mm showed significant increase in load carrying capacity of the beams. Further investigations may be done with respect to number of layers of TRM used and optimum strip lengths of TRM.
- Usage of anchorage has not affected the failure load but enhanced the ductility of the beam.

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Causes of Delay in Highway Projects in Pakistan

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Abstract
Construction industry of Pakistan is playing main role in the economy as well as in employment. The construction industry is facing problem of delays in construction project especially in highways projects. Delay is the most critical problem in highway projects which give negative impact to the stakeholders because it exceeds not only time but also budget of the projects. Hence, the objective of this research is to identify the critical factors which cause the delays in highway projects of Pakistan. From in depth literature reviews identified a total of 70 factors were identified which causes of delays. A Questionnaire was developed and distributed to the 80 selected respondents from clients, consultant and contractors to rank the factors of delays in highway projects. By using average index method, critical factors which cause delay were found, critical factors of delays in highway projects in Pakistan are financial difficulties faced by contractor, poor site management, frequent design changes, land acquisition and political influence. This research can help to overcome delay factors and causes of delay and some suggestion for reducing these delays in highway projects in Pakistan.

Keywords
Causes of Delay, Highway Projects, Pakistan

1. Introduction

A project is said to be successful when it is completed within time frame and with standard quality (Frimpong et al, 2003). In project management a successful project can be defined as project is completed within approved time, cost and with standard quality (Trost & Oberlender, 2003). According to (Aziz et al.,2016) highway project which is completed within the time frame or approved time is known as successful project.

In Pakistan, transportation sector is knows as fourth largest sector which contributes more than 10% to (GDP) gross domestic production and over 17% to the (GCF) gross capital foundation (Javied and Hyder,2009). The government of Pakistan is trying to improve quality standards of highway network (ESP 2014-2015). Construction industry has always problem from fragmentation owing to the project execution and different financial, managerial and technical specialists that are in a project. So all projects should be completed on time and budget (Sullivan & Harris,1986).

A number of researches have been found to find factors which cause delay in construction industry. However few researches have been carried out to find factors which cause delay in highway/road projects. In Pakistan, number of researches has been found to identify factors which causes delay in construction projects, only few studies have been found to identify critical factors which causes delay in highway projects.

2. Previous Studies

A study conducted by Mezher et al., (2006) those factors which causes delay in highway projects at Lebanon from client, designer and contractors firms. It was resulted that client has concern with financial matters and contractor contractual relationship between parties where the designer consider that project management as most important causes of delay in highway projects. According to Mahamid et al., (2012) conducted questionnaire survey and investigated that influence of government in project, project awarded on lowest bid rate, delay in cash flow by client and lack of competent consultant staff and errors and incomplete drawing and design are most significant factors of delay.

Ejaz et al., (2013) investigated 26 factors of delay in highway projects of Pakistan. Questionnaire was designed and distributed to client, consultant and contractors to select significant factors which cause delay. Result of conducted survey was (i) fluctuation of items and material and (ii) deficiency in cash flow by client. However, Haseeb et al., (2011) identified critical factors which causes delay factors in Pakistan were shortage of finds, change in orders, lack of equipment and materials, poor planning and improper site management.

Choudhry et al., (2012) found factors of delay in highway projects of Pakistan, factors were delay in cash flow, weather impact, financial problems, purchasing of land, change in scope and incompetent management and no any planning by contactor. Another research conducted by (Santoso and soeng, 2016) through questionnaire survey and point out critical factors of delay were poor arrangement at site, delay in payment, productivity labour, award of contract on low bid and weather impact.

3. Objectives of Research

The objectives of the research are:

(i) to identify the factors which causes delay in highway projects of Pakistan

(ii) to rank factors which causes delay by average index method

4. Research Methodology

In this research quantitative approach is used to understand the perception of stakeholders towards factors contributing delay in highway projects of Pakistan. In this regard, investigation was made in two stages. First stage includes literature review and conducting interviews. From this stage 70 factors were identified which causes delay in construction projects. In second stage, a questionnaire was designed which comprises of two parts part A and part B. Part A contains demography (bio data, experience, qualification) of respondents was asked. Part B contained factors of delay in construction projects from which respondents were asked to score and rank each factor.

5. Data Analysis

Collected data was analyzed by using formula of average index (AI)

$$AI = \frac{\Sigma(1X1+2X2+X3+4X4+5X5)}{\Sigma\ X1+X2+X3+X4+X5} \quad \ldots\ldots\ldots(1)$$

Where,
N= Total number of respondents
X1=Number of respondents for “Not Significant”
X2= Number of respondents for slightly significant
X3= number of respondents for moderately significant
X4= number of respondents for Significant
X5= Number of respondents for highly significant.

In this research, evaluation values to assess significant level is used as follow:

AI value from 1.00 to 1.50	Extremely Significant
AI value from 1.50 to 2.50	Very Significant
AI value from 2.50 to 3.50	Moderately Significant
AI value from 3.50 to 4.50	Slightly Significant
AI value from 4.50 to 5.0	Not Significant

All the respondents were asked to rank factors which cause delay in highway projects.
Before data collection a preliminary study was carried out by conducting interviews from 7 experienced persons who are involved in highway projects. This study was done to validate the questionnaire relevancy in highway projects of Pakistan. Table 1 shows the profile and experience of respondents interviewed.

Table 1: Profile of respondents interviewed for questionnaire validity

No.	Organization	Position	Experience in highway projects
01	Consultant	Resident Engineer	28 years
02	Consultant	Resident Engineer	27 years
03	Client	General Manager	27 years
04	Client	Director	24 years
05	Client	Project Director	20 years
06	Contractor	Project Manager	18 years
07	Contractor	Project Manager	17 years

Table 1: shows that respondents have more experience in highway projects ranging from 28 to 17 years. Total experience of the 07 respondents is 161 years (average experience of 23 years). Interviewed respondents were senior in their organizations and were holding good positions on each organization. Hence, data was collected by developed questionnaire.

Reliability test was also conducted to measure the consistency and stability. In this test the value of Cronbach's alpha is determined. The value of Cronbach's alpha is between 0 to 1. Value of Cronbach's more than 0.7 is known as reliable data (Sekaran & Bougu, 2010). So in this test value of alpha was 0.87.

6. Results and Discussions

Questionnaire survey was conducted by distributing a total number of 80 questionnaire sets. Questionnaire were distributed randomly to30 contractor's, 30 consultant and 20 client personnel. From 80 questionnaires, 69 questionnaires were received from respondents. From 69 few questionnaires cannot be further analyzed due to incomplete data. Details of survey are as in table 2.

Table 2: Details of conducted surveys

Parameter	values
Total no: of questionnaire distributed	80
Total no: of questionnaire received	69
Total no of incomplete information	09
Total no of valid questionnaire	60
% of questionnaire received	86.2
% of questionnaire valid	75

Data collection was analyzed by using average index method and factors whose value is between 1.00 to 2.00 were selected as critical factors which causes delay in highway projects. Results of identified factors which causes delay are presented in table 3

Table 3: Critical Factors which Causes Delay in Highway Projects

Factor ID	Factor	average Index	Rank
13	Financial difficulties faced by contractor	1.59	1
26	Poor site Management	1.62	2
32	Frequent design changes	1.67	3
18	Land acquisition	1.75	4
21	Political Influence	1.81	5
51	Mistakes during Construction	1.86	6
41	Late delivery of materials	1.89	7
29	Financial difficulties by owner	1.92	8

05	Inadequate planning and scheduling	1.99	9

Table 3: shows that financial difficulties faced by contractor, poor site management, frequent design changes, land acquisition, political influence, mistakes during construction, late delivery of materials, financial difficulties by owner and inadequate planning and scheduling with their average index value 1.59,1.62,1.67,1.75,1.81,1.86,1.89,1.92 and 1.99 respectively are the critical factors which causes delay in highway projects of Pakistan.

7. Conclusions

This research explores factors which causes delay in highway projects. 70 factors were identified from literature review which causes delay in construction industry. A pilot study was conducted to validate the factors of delay then questionnaire was designed and distributed to stakeholders of highway projects to rank each factor. After statistical analysis of each factor by average index 9 factors were identified as critical factors which cause delay in highway projects. The critical factor which causes delay are financial difficulties faced by contractor, poor site management, frequent design changes, land acquisition, political influence, mistakes during construction, late delivery of materials, financial difficulties by owner and inadequate planning and scheduling.

The finding of this research is not only beneficial to control factors which causes delay of highway projects but it also help to reduce the cost of highway projects.

8. Future Research

This research has identified critical factors which causes delay in highway projects of Pakistan but mitigation measure of critical factors which causes delay in highway projects can be exercised. And similar kind of study for different type of project is also suggested.

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Risks In EPC Hydropower Projects: A Case Of Pakistan

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Abstract
EPC contracting method involves designing, procurement and physical construction. Owing to its specialized nature, this delivery method is usually employed for large scale infrastructure development such as hydropower, industrial, healthcare and transportation projects. EPC hydropower projects present various risks and complexities. The current study analyzes risks inherent to EPC hydropower projects in Pakistan. Based on pertinent literature, 50 risks are identified and characterized. Representatives of client, contractor and consultant are engaged for data collection. Based on gathered data, identified risks are ranked as per their significance. The results show that most severe risk is that of subcontractor performance followed by local population resistance. The group of construction risks houses the most severe risks followed by that of geological & hydrological risks. Further, the risks are allocated to stakeholders in order to help the policy makers in allocation of EPC hydropower risks. The study highlights key risks in EPC hydropower projects of Pakistan, management of which can help the developing countries in formulating a better risk management strategy based on the experience of Pakistan.

Keywords
EPC, Risk, Hydropower Projects, Risk Management, Risk Responsibility

1. Introduction

Engineering Procurement Construction (EPC) is a project delivery model involving a turnkey entity responsible for handling the design, engineering and procurement aspects of construction. It is followed by project execution, usually at a firm price, with fixed completion date and guaranteed performance (Kelley, 2012). All these responsibilities can be undertaken by contractor's in-house team or may be subcontracted (Loots and Henchie, 2007). Hydroelectric power plants, being a composite structure, encompass uncertain factors with high risks. Large amount of capital investments, complex structures and extensive construction periods add into uncertainty (Sudirman and Hardjomuljadi, 2011). Financing hydropower projects on large scale possesses enormous risks for investors, lenders and sponsors due to reasons like political, economic, construction complexities and heavy capital investment.

Risks are inherent to any construction project. Project risk management aims at increasing the likelihood and impact of positive events while decreasing the chances and effects of adverse events (Rose, 2013).

The ultimate objective is to change unmanageable and uncertain events into comparatively controllable events (Wang et al., 2013). Wei et al. (2012) argue that due to uncertain factors, efficient management and control, hydropower project is complex and challenging task for EPC entity. Therefore, for effective project governance, it is important that EPC contractors define and identify risks thoroughly, quantify them and evaluate their effect on project objectives.

Pakistan lacks research towards risk management in construction field. A study conducted by Masood and Chouldhry (2010) focused on contractor's perception about risk factors. No concrete effort has been made so far for developing guidelines for international EPC contractors that incorporate risk management in hydropower projects. The aim of this research is to communicate the significance of risks in hydropower projects to public, private and foreign stakeholders. Such assessment will facilitate EPC contractors in better management of financial, legal, reputational and administrative issues by increasing their awareness of sensitivity of uncertain events in new environment.

2. Literature Review

EPC is a contracting method in which a contractor is employed for project delivery from design phase to procurement, and physical construction (Wai et al., 2012). The basic premise of EPC concept assumes that the project promoter could define the desired outcome and carry out a bidding process. EPC may encompass delivery of sophisticated exclusive product, composed of several interrelated components and subsystems. Extensive financial commitments are required to successfully complete such projects (Yeo and Ning, 2002). EPC delivery method is often used as contractual agreement for industrial facilities such as refineries and plants. These projects are usually risky in nature because of their overlapping phases, and interdependent activities, and involve specialized equipment and machinery making delivery time vital for cost saving, avoiding delays and overall project progress. The teams of engineers, suppliers and contractors under a single point responsibility of EPC entity would develop a fixed cost date, ascertain contract and accept liquidated damages for performance and delays. All parties want to maximize their own advantage, but they also know that persistent quest of benefit may damage the very goal they are seeking (Ping and Tao, 2013). Contracts are comprehensively negotiated as EPC entity has to design the facility to optimize performance of equipment. Comprehensive acceptance/completion protocols are mentioned in the contract, whereas liquidated damages and damages for failure in achieving agreed upon performance are also mutually agreed (Kelley, 2012).

In recent years, several studies investigated different characteristics of risk management in international hydropower projects revealing risks and barriers in execution of these EPC projects. These ventures have enormous construction scale and involve substantial capital investments, growing construction complexities, vulnerabilities and uncertainties (Wei et al., 2012). Also, clients are becoming more demanding and critical, having a tendency to designate most of the risks to the EPC entity (Ping and Tao, 2013). Ling and Hoi (2006) categorized risks faced by international Architect-Engineer-Construct (AEC) contractors working in foreign lands into political, contractual, economic and financial, procurement, design, construction related and cultural risks. It was suggested that foreign firms should entrust locals for operations and detailed construction, while competent foreign managers should take care of strategic management. Wei et al. (2012) ranked 23 risk factors from international EPC hydropower projects from Chinese perspective. The 10 most critical risk factors pertain to contract, cost, owner, local politics, design, subcontractor, finance, consultancy surveillance, market, and social environment.

Similarly, Wu et al. (2010) highlighted four principal risks related to project, contractor management, social and others. Risks are never fully eradicated in construction projects. Their response strategies include contractually transferring risk to other party, sharing between both parties or by mitigation measures to limit negative consequences. Risks should be reasonably allocated to parties to the contract. Transferring it solely to one party is unjustifiable (Peckiene et al., 2013). Optimal risk allocation mechanism allocates risk responsibility to the best suited party, capable of controlling and managing it at least cost. Final responsibility lies on EPC contractors to evaluate all these investigations and present their offer. Wang and Tiong (2000) argued that allocating indefinite geological and hydrological risks to contractors is not a practical approach, as EPC contractors may add unrealistic risk surcharge making the project unfeasible. Wei et al. (2012) expressed their views on responsibility of these critical risk factors and stated that owner shall bear most of the risks of social environment. Political, market and financial risks shall be shared by owners and contractors jointly whereas contract, schedule and design risks shall be borne by the contractor. As a result of detailed literature review in current study, nine risk groups and fifty risk factors are identified which cover the broad spectrum of hydropower projects focusing on EPC delivery method.

3. Methodology

To identify risks associated with EPC hydropower projects in Pakistan, 16 relevant papers were consulted from which 50 risks were identified and classified into nine groups. Questionnaire survey was selected as principal data collection tool for ranking the key risks. The basis of ranking was the probability of occurrence and impact on project objectives. The questionnaire contained three sections: general information of respondents, probability and impact of each risk on Likert scale of 1 to 5 and assigning risk responsibility. Based on the scores, probability impact matrix is generated, where risk score is calculated using Equation 1.

$$R = \rho * I \qquad (1)$$

Probability (ρ) and Impact (I) are average values of all responses calculated using Equations 2 and 3 in which 'x' represents individual score and 'n' is total number of respondents (53).

$$P = \frac{\sum x}{n} \qquad (2)$$

$$I = \frac{\sum x}{n} \qquad (3)$$

Risk groups are compared with each other by calculating mean value of risk score of individual risks, using equation 4, where 'R' represents individual risk score and 'N' is the total number of risks present in the group.

$$Mean\ Group\ Risk\ Score = \frac{\sum R1, R2, R3 \ldots\ldots Rn}{N} \qquad (4)$$

The questionnaire was targeted at engineers, managers, contracting and procurement professionals, designers from contractors, consultants and clients related to EPC hydropower projects. Risk responsibilities of the parties were also determined. Questionnaires were distributed through email and by personal interaction. Fifty-three valid responses were received from participants of these projects. To obtain reliable results it is very important that respondents have understanding of risks and are familiar with the used taxonomy.

4. Results

4.1 Respondent Demographics

Twenty-eight (53%) respondents were having Master degree, twenty-four (45%) Bachelors (majority having civil engineering degree) and one of them was having other qualification. Moreover, twenty-five (47%) respondents were having experience less than 4 years, seventeen (32%) having experience ranging from 5 to 8 years and remaining 11 (21%) had experience of more than 8 years. Overall 27 of them were designated as engineers, 20 as managers and remaining 6 were academic professionals. Further, majority of respondents (42%) represented EPC contractors, followed by consultants (26%) and clients (21%).

4.2 Risk Ranking

Table 1 highlights top 15 risks in terms of severity towards EPC hydropower projects that are ranked both on group and individual levels. Construction risks lead the categories with six risks out of top 15 most severe risks. Severity of the risk group can be assessed by the fact that top two risks belong to this category. According to survey results, poor performance of subcontractor is the most severe risk. The root cause of this problem lies in inappropriate vetting of subcontractor by the main EPC contractor. EPC companies are generally inclined to maximize their profit and consequently minimize the cost. Therefore, sometimes EPC firms subcontract the project without proper screening. Local population resistance comes out to be second most severe risk with a score of 15.84. Local residents oppose and resist work progress due to various reasons. Mostly the cause of opposition is to get maximum benefits from the project in form of supply contracts, jobs, construction contracts, etc. The EPC contractor is usually bound by contract to provide opportunities to local community and they try to fulfil their obligations but with some limitations. Similarly, communication plays big role in creating complications in the project matters (Sudirman and Hardjomuljadi, 2011). Mostly EPC contractors' staff are from non-English native countries which in turn suggests that their senior staff cannot communicate effectively in English (contractual language). In case of geological and hydrological groups, there are three high ranked risks in this group. Table 1 shows that slope stability risk is ranked 3rd, differing subsurface soil conditions occupies 5th spot followed by floods during construction at 7th. Overall risk score of this group is 11.45 which is relatively lower due to the last ranked risk i.e. severe drought. The hydropower resources in Pakistan are mainly located in mountainous areas in northern region of the country (PPIB, 2012).

Risk of differing sub surface soil conditions for other structures like powerhouse, weirs and sand trap are faced by the contractor (Sudirman and Hardjomuljadi, 2011). Pakistan faced unprecedented floods in the past, with two of them in past 5 years. Apart from heavy causalities and property damage, hydropower projects also suffered. In case of planning risks, two are present in most severe EPC risks list. Inability to schedule project work accurately is at 11th place and in-depth geotechnical assessment at 14th. Engineering designs are subjected to changes and modifications due to varying reasons, making it difficult for EPC contractors to schedule and execute the project precisely. Government agencies and clients carry out geotechnical investigations during pre-feasibility and feasibility stages.

EPC contractors are responsible for conducting in-depth geotechnical investigations during planning phase, negligence may have disastrous effects. The contract risk group is ranked 4th with mean risk score of 10.38, none of the individual risk is in top 15 most severe risk list. Delays and failure in invoice payments is ranked 18th in overall list. Private sector projects are usually not affected by this risk, as project company usually secures finances through its equity, sponsors and lenders. In procurement risks category, delays in equipment delivery to site is ranked 15th. Most of the sophisticated and advanced technological equipment is imported in Pakistan. There are always risks of delays in equipment delivery due to miscommunication between parties, changes and revisions in construction drawings and lack of project finances (especially government sector). Imperfect data transmission to vendors and international relations is low in the list. Pakistan generally has good trade relations with other nations, so this risk lies low in the list at 49th position.

Table 1: Score, Grouping and Ranking of Risks

Group	Risks	Risk Score	Rank	Mean Group Score	Group Rank
Contract Risk	Incomplete and unbalance contract clauses	11.74		10.38	4th
	Owner default risk	8.11			
	Delays or failure in invoice payment	11.47			
	Lack of attention to contract requirements	11.06			
	Subcontractor or supplier break contract risk	9.52			
Planning Risks	Inaccurate Survey data	8.38		11.14	3rd
	In-depth geotechnical assessment not conducted	12.72	14th		
	Work scope not clearly defined	10.08			
	Inability to schedule project work accurately	13.40	11th		
	Inaccessibility of foreign designer	10.15			
Engineering Risks	Unfamiliarity with design, codes & standards	10.67		9.78	6th
	Design failures	8.49			
	Failure in converting basic design to detail design	8.03			
	Inadequate design quality	11.52			
Procurement Risks	International relations	6.44		9.87	5th
	Ambiguity in project cash injections	11.10			
	Imperfect data transmission to vendor	9.97			
	Delays in equipment delivery to site	11.98	15th		
Construction Risks	Deficiencies in quality inspections and audits	11.14		12.66	1st
	Health & Safety Risks	13.54	10th		
	Machinery & equipment installation risks	14.34	6th		
	Subcontractor performance issues	16.08	1st		
	Changes in owner requirements	10.24			
	Technological complexities	9.74			
	Lack of experienced personnel	12.89	13th		
	Poor Coordination & Communication between parties	14.07	8th		
	Local population resistance	15.84	2nd		
	Project Management disabilities by main contractor	11.25			
	Pre commissioning requirements not fulfilled	10.19			
Social & Environmental Risks	Land acquisition & compensation	13.32	12th	9.06	9th
	Social acceptability risk	8.52			
	Adverse effect on ecosystem	10.44			
	Water quality deteriorated	7.54			
	Cultural differences	6.48			
	Site contamination	8.06			
Geological & Hydrological Risks	Differing Subsurface soil condition	14.67	5th	11.45	2nd
	Slope stability	15.18	3rd		
	Ground water treatment risk	8.88			
	Severe droughts	6.18			
	Floods during construction	14.09	7th		
	Over estimating production	9.67			

Financial & Economical Risks	Unpredicted Inflation	8.07		9.58	7th
	Adverse changes in interest rate	7.26			
	Fluctuating exchange rate	10.78			
	Taxation rate increases	8.09			
	Financial uncertainty of contracting parties	13.70	9th		
Regulatory & Legal Risks	Change in laws & regulations	7.73		9.55	8th
	Withdrawal of political support	8.07			
	Drastic changes in national policy	7.73			
	Poor law & order condition	14.70	4th		

In engineering risks category, inadequate design quality is the most severe risk, and is overall 17th in list. There are concerns raised from client and consultant engineers about poor design quality and lack of control. Although they perform design vetting, it is not synonymous to self-designing. Government sector EPC projects suffer in design quality because clients are not fully familiar with design, codes and standards. They are unable to clearly mention the required specifications according to international standards. Most of the financial and economic risks are covered under the umbrella of Purchase Power Agreement. Ansar et al. (2014) concluded that higher the long term inflation rate of host country, higher is the cost overrun of hydropower project. The risk which makes into most severe risk list is financial uncertainty of contracting parties. It is not an issue in privately financed projects but EPC contractors in government projects usually suffer due to this risk. In case of legal and regulatory risks, poor law & order conditions in the country is crucial risk having catastrophic effects. It is at 4th position in top most severe risk list. Pakistan is in war on terror for more than one decade. It has costed the growth of infrastructure and development projects in the country. Although things have moved in the positive direction since past few years, still a lot has to be put in order. In social and environmental category, risks are ranked low except land acquisition risk, which is at 12th position. Land acquisition risk is common for most of infrastructure projects (Sudirman and Hardjomuljadi, 2011). It is more challenging issue for private sector projects as government sector clients have advantage to acquire land through increased compensation.

4.3 Risk Allocation

Last portion of questionnaire required respondents to allocate individual or shared responsibility to contracting parties. Benchmark of 50% is considered for allocation of responsibility to a party in accordance with Iqbal et al. (2015). Percentage is calculated by using Equation 5 in which 'R' is the number of respondents electing the specific category and N is the total number of respondents (53). Four categories are created as a result of survey: client's responsibility, contractor's responsibility, shared responsibility and indecisive group where none of the party has 50% response rate.

$$\text{Percentage responsibility} = \frac{R}{N} * 100 \qquad (5)$$

Figure 1 shows the percentage of questionnaire respondents indicating the responsibility of clients. Most of the finance group risks are allocated to clients with 60% response rate. Client has to bear the risks in construction process like their own breach of contract, change of expected results of project and requirements, change in law as well as the risk of force majeure (Ping and Tao, 2013). Figure 1 also shows that owner default risk, delays in invoice payment and work scope definition are all allocated to client.

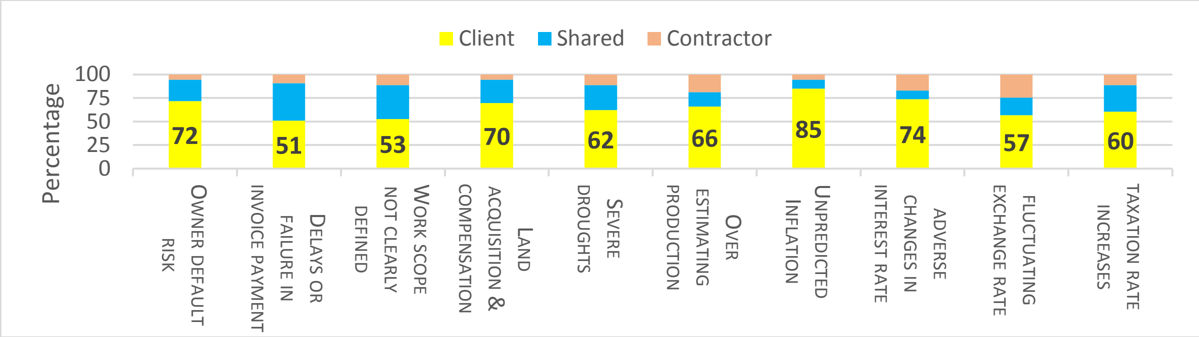

Figure 1: Client Responsibility

Around 75% respondents indicated that land acquisition is responsibility of client who sometimes delegate it to EPC contractors. Among the list of most severe risks, only land acquisition is allocated purely to client. Although most of the risks in EPC are accepted by contractor, survey respondents think that most of legal and regulatory risks should be jointly shared by client and contractor as shown in Figure 2. EPC contractors cannot effectively mitigate the risk of local population resistance alone, they need support from client and government authorities. Some risks could not be allocated to any contracting party due to inconclusive results. Three risks in this group scored less than 50%. International relations of country are not only important for trade and commerce, but also for appointing international staff. Other risks in this category are from social & environmental risk group. Survey results show the tendency of respondents to allot responsibility of social acceptability risks to clients.

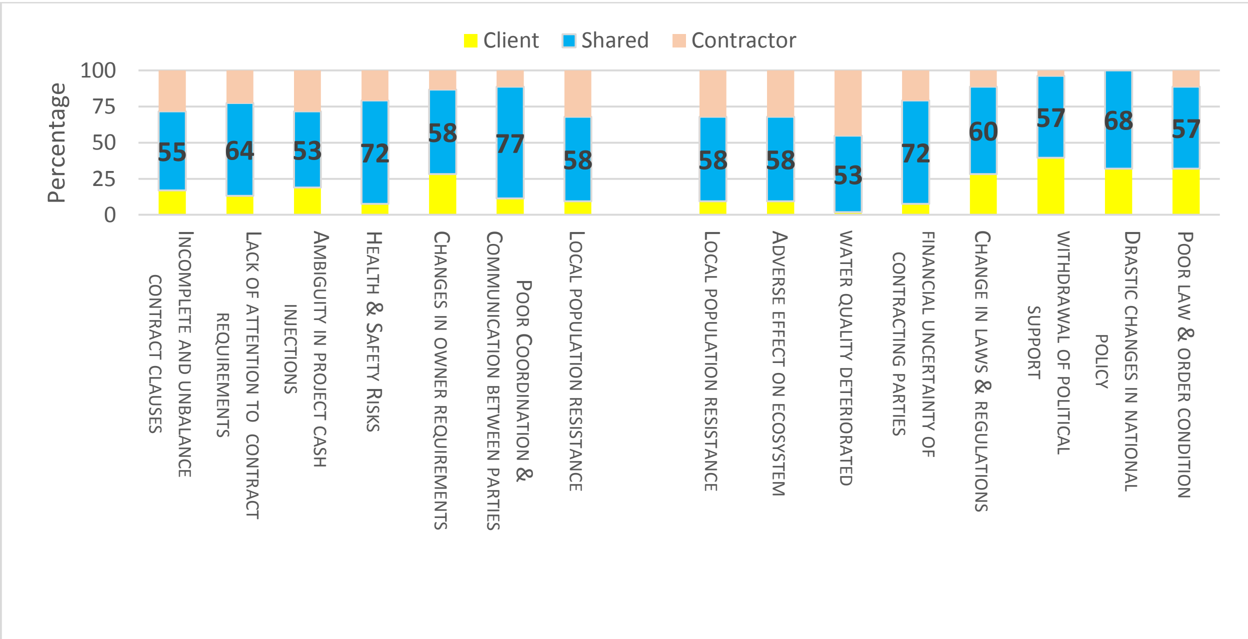

Figure 2: Shared Responsibility Group

5. Conclusions And Recommendations

Using extensive literature review and input from practitioners, this study identified 50 risks in EPC hydropower projects in Pakistan which are further grouped and ranked. Using survey results, top 15 severe risks are highlighted in which ‘construction risks’ group is most widely represented with 6 risks, followed by ‘geological & hydrological risks’ group. Subcontractor performance issues is the most severe risk. Responsibility of 23 risks was allocated to EPC contractors, and 10 to clients. A total of 14 risks need to be jointly shared between contractors and clients, while responsibility for 3 risks was undecided. It is also indicated that Pakistani construction industry does not fully acknowledge the importance of risk management in projects due to lack of awareness.

A limitation of current study is the availability of data and lack of risk management practices. Investment in risk management system of organizations, especially EPC contractors, is required for improvement. Client should include risk manager as a ‘key person’ and demand proper application of risk management tools and techniques. Risk management plan should be given same importance as other knowledge areas of project management. For future research directions, it is suggested to conduct a study with focus on establishment of risk management framework and policy guidelines for EPC hydropower projects in Pakistan. This will help the policy and decision makers in developing countries to learn from experiences of Pakistan.

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Assessment of Critical Factors for Prequalification of Contractors in Construction Industry of Pakistan

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Abstract

The success of project depends upon the right selection of good construction team. This selection of technical team is itself a complicated process. All construction works in Pakistan Public sector are performed through a tendering process, which involves the prequalification of contractors before going into bidding stage. Almost all clients prefer their own factors for prequalification hence there is no single set of factors for the selection. This study is aimed to provide a common decision making approach that best suits the contractors' prequalification in Pakistan construction industry. For the assessment of factors the data is collected through literature review, unstructured interviews, and by floating questionnaires among the practitioners. The factors are categorized in different main criterion and then results are discussed with the experts of field in the context of construction industry of Pakistan for their suitability in the prequalification process. The results of this study will be helpful for all Public sector clients for effective prequalification of the contractors.

Keywords:

Contractors' Prequalification, Evaluation Criteria, Contractors, CI of Pakistan, Construction Clients.

1. INTRODUCTION:

Construction players have key role in the construction industry. The performance of work is extremely affected by its resources. Clients choose their favorite contractor to carry out their work in terms of cost, time, quality, safety, scope, satisfaction etc. But to have their favorite contractor in the construction team is not an easy task as construction industry is full of risks, here every next project is change from every previously carried out work. Because construction projects are very massive and complex in their nature, they involve serious issue of cost, time, litigations, safety, claims and quality (Singh et al -2005).

By looking these potential risks, clients are always worried about the selection of best contractors among the available list of bidders. To select the one contractor among the list is again a difficult task, as clients have to look various parameters beside the minimum bid. The construction clients always select the one who can deliver the work within stipulated time and within estimated budget with better quality.

Public sector clients works through tendering process which is actually a two phase system, before bidding a prequalification process is there, and then a bidding process in which contractors are required to fill the amount of the work to be carried out. Hence public sector projects are commonly carried out by awarding the contracts through a bidding process which is itself a very crucial task for construction clients. As construction projects are too complex, then the best suited constructors must have to be selected before directly giving them bid documents. Hence it is has become obligatory to pre-qualify the contractor before going directly to second phase of tendering i.e. bidding. Due to high complexity and dynamic nature of construction projects, contractor selection process has become more tedious and it has added further difficulty in selection process for contractors (Abbasy et al -2013).

Different methods for evaluating tender proposals are practiced by owners of private construction sector. These clients in most cases develop their specific procedure to award the work to most suitable contractor (Singh et al -2005). On the contrary the major criterion for contractor final selection is based upon the lowest price quoted in the tender (Barrie et al-2008).

The most useful method of contract award in Pakistan is minimum bidding. This method has inherent flaws of low performance and high market competition (Tariq Hussain Khan -2015). On the contrary, various inherent flaws occur when projects awarding is based on least contract price. Due to high variation, increment in final cost of the project, delays in contract duration, more trend of compromising on quality, and confrontational relationship among two parties are the serious drawbacks related minimum tenderer award system.

New emerged techniques for tender evaluation and prequalification of contractors are confirming that this prequalification area is subject of concern so it still needs more investigation (Abidali et al -1995 & Tam et al -1996). Due to emergences of more recent approaches in last few years, there is now deviation of trend from a lowest bid win system to multicriteria selection approach has been established (Wong et al -2000).

According to a recent research in Pakistan, minimum bidder bid awarding system is mostly used method for awarding the contracts in public sector construction industry of Pakistan in which almost 83% of public projects are given to minimum bidder Tariq Hussain Khan -2015).
Continuing problems of cheap quality of constructed works, more cost and schedule overruns, high occurrences of litigations and submitting claims, have become the major issues of contracts in construction industry of Pakistan (Tariq Hussain Khan -2015).

Same study of (Tariq Hussain Khan - 2015) concluded that when contracts are awarded to lowest bidder 60% of contractors have gone to serious issues related to quality in the construction and the built facility possess defects within the defective liability period. Also the study concludes that those completed projects have time overrun issues due to unsatisfactory planning.

The above literature provides clear picture that, the construction industry of Pakistan clients mostly accept the lowest bidder until and unless they have some strong justification to reject that contractor.
When the minimum bidder is mostly selected for the work, so why not all bidders must be well qualified through a well established prequalification process. Hence this research is aim to assess the most critical factors that require by the clients to qualify the contractors for bidding stage. The research concludes at the end by suitable factors that must be kept in the prequalification process. This work will help not only Public sector clients but the private sector clients too for the selection of contractor on other than least responsive bid system.

2. RELATED PREVIOUS STUDIES:

In Public construction projects the work is mostly carried through a tendering process. The project is then awarded to any one contractor from the number of competitors. Mostly the contract is awarded to prequalified contractor. Contractors are usually selected on some initial evaluating criteria's, like based on their previous experience, financial and technical capabilities, possessing related equipment to complete works etc. In prequalification process various factors are considered to evaluate the contractor .Hence the factors that affect the contractor selection is essential to identify.

A study in Greece (K.P. Anagnostopoulos and A.P. Vavatsikos -2006) determined various important criteria and sub- criterion and they have developed a decision model based on AHP technique. The authors considered following Criteria and Sub- criteria in their model; Financial performance (Sub Criteria; Net worth ratio (earnings before interests and taxes/owner's equity), Current ratio (current assets/current liabilities) , Credit ratio (owner's equity/total assets), Asset turnover ratio (sales/total assets), Firms growth (total turnover during the last three years), Ratio of fixed assets/long term liabilities, Technical performance (Sub criteria ; Equipment owed by the contractor, Employed engineers by each candidate (number), funding by the tender Contractor's years in business (years), Contractor's activity during the last three years Candidates experience in similar projects), Training programs for the personnel, Health and safety policy (Sub- criteria; Indemnities paid for labor accidents during the last five years, Investment in health and safety), Past performances in public works (Sub Criteria; Cost overruns at executed contracts (bid price/final cost), Attitude towards to claims), Schedule overruns at executed contracts (bid duration/final duration).

(Tan and Ghazali - 2011) determined forty (40) critical factors for contractors and divided them in seven main categories: (i) project management factors; (ii) client related factors; (iii) design team related factors; (iv) project manager related factors; (v) contractor related factors; (vi) procurement related factors; and (vii) business and work environment-related factors.

A study conducted in Spain (Ana Nieto-Morote & Francisco Ruz-Vila -2012), developed a decision model based on fuzzy set theory. The authors identified set of criteria and sub criteria for the development of model. They use eight major major criteria namely; Technical capacity, Financial stability, Experience, Past relationship, Management capability, Reputation, Past performance and Occupational health and safety. Further they divided into sub criterion as; A. Technical capacity 1. Experience of staff 2. Qualification of staff 3. Innovate method 4. Labor and equipment B. Financial stability 1. Financial soundness 2. Liquidity 3. Credit ranting C. Experience 1. Type of past project completed 2. Size of past project completed 3. Experience in local area 4. Number of projects completed D. Past relationships 1. Relationship with subcontractors 2. Relationship with supplier 3. Client satisfaction E. Management ability 1. Organisational culture 2. Quality management system 3. Management knowledge, F. Reputation 1. Past failures in completed projects 2. Number of years in construction 3. Claims and litigation in past G. Past performance 1. Projects completed on budget 2. Quality level of projects performance 3. Projects completed on time, H. Occupational health and safety 1. Management safety accountability 2. Safety performance.

(E-Abbasy et al -2013) conducted a research for the selection of contractors for highway projects in 2013, the authors selected four major categories i.e. A. Project's main requirements (Sub criteria includes; Project bid price, Project duration, Risk sharing with the owner B. Financial capability (Sub criteria includes; Financial stability, working capital) , C. Past performance (Sub criteria includes; % of previous work completed on time, Past relationship with the owner, Response to claims, Health and safety records) and D. Experience (Sub criteria includes; Contractor's staff Experience , Experience with similar nature projects, Equipment availability).

(Neringa GUDIENE -2014) established critical success factors for Lithuania construction industry. According to the research based on AHP technique, these factors were identified as most critical major criteria ; (1) clear and realistic project goals, (2) project planning, (3) project manager's competence, (4) relevant past experience of the project management/team, (5) the competence of the project management/team, (6) clear and precise goals/ objectives of the client, (7) the value of the project (8) the complexity and uniqueness of the project, (9) the project manager's experience, and (10) the client's ability to make timely decisions.

(Mahdi Safa et al -2015) identified five criteria are for contractor selection: (1) cost, (2) time, (3) field service and engineering rates, (4) experience, and (5) the financial stability of the contractor.

A recent research in Malaysia (Pooria Rashvand et al -2015) only highlights the contractor management capabilities and practices in the evaluation model. The author determined the weightage of these factors to use in the models; Monitoring and controlling, Problem Solving skills, Team development skill, Management knowledge, Resource management.

A recent case study in China (Bingsheng Liu-2015) linked the project success factors with the contractor delivery system in contracts. The authors have identified four top most important factors related to contractor for project delivery system (PDS) , these are; contractor's coordination and communications, contractor's ability in financial management, contractor's design capability and contractor's experience with similar types of projects.

In literature different authors have considered different criteria and sub criteria. This research focuses on factors from literature also available guidelines on national and international level. The study mainly focused on current prequalification practices in different departments of Pakistan.

3. RESEARCH METHODOLOGY

The study is aimed to determine the critical success factors for contractor prequalification. Rockart (1982) defines the critical factors as, those limited important areas in which favorable results are absolutely necessary for a particular manager to achieve his or her goal.

The author initially review the literature and various national and international procurement guidelines including World Bank, Asian Bank Development (ADB), Public Procurement Regulatory Authority (PPRA) and Pakistan Engineering Council (PEC) guidelines. From the literature and procurement guideline various critical factors related to prequalification of contractors were identified. The factors were then discussed with various highly profiled field experienced person from Public sector department of Pakistan who were involved in prequalification process.

To access the factors related to CI Pakistan, Public and Semi Public Departments were selected to review their current procurement procedure. The author review the prequalification process of Works and Services Department Sindh, Irrigation and Power Department Sindh, Hyderabad Electric Supply Company (HESCO), Mehran University of Engineering & Technology (MUET) Jamshoro and NESPAK consultant Pakistan.

After reviewing the literature, standard procurement guidelines and current practices of public construction industry of Pakistan, final questionnaire is designed. The factors are divided into eight major criteria i.e. 1. Financial Soundness 2. Past Experience 3. Technical Capabilities 4. Equipment Capability 5. Managerial Capabilities 6. Health & Safety 7. Past Relationships 8. Geographic Location of Contractor.

4. DATA COLLECTION AND ANALYSIS

The data is collected using a structured questionnaire survey. The targeted respondents were the Chief Executive Officers (CEO), Project Managers, Procurement Officers and Contract Engineers from Public and private Construction industry of Pakistan. So the author has mostly focused on the experts which remained closer to the prequalification process.

A total of 80 questionnaires were distributed randomly in different practitioners involved in construction industry of Pakistan. Out of 80 questionnaire 75 were completely filled and valid for analysis. Table 1 shows the summary of data collection.

A Likert scale was adopted to analysis the data. Level of significance were assessed with Statistical Package for Social Sciences (SPSS) using Average Index technique.

Table 1: Summary of data collection

No. of questionnaires were distributed to respondents	80
No. of questionnaires were received from respondents	75
No. of incomplete filled questionnaires from respondents	5

5. RESULTS & DISCUSSIONS

The collected data from questionnaire is analysed using SPSS Software. The factors are divided into eight major groups. The ranking of factors are determined from the mean value given to each factor by the respondents using Average Index (A.I) technique. The table 2 shows the results of data analysis.

Table 2: Average Index and Rank of Each Factor

No.	Factors of Prequalification	A.I	Rank
1. FINANCIAL SOUNDNESS			
1	Net worth of firm	4.20	1
2	Available credit line from bank	4.16	2
3	Average Annual Turnover for last 05 to 10 years	4.12	3
4	Bid Capacity of Contractor	4.08	4
5	Bank Statements showing the finance record of contractor	3.88	5
6	Audited Financial reports	3.76	6
7	Letter of references from reputable bank	3.40	7
2. PAST EXPERIENCE			
8	Contractor experience in similar projects during last 10 years	4.84	1
9	Contractors General Experience	4.56	2
10	Compliance with specifications and quality standards	3.72	3
11	Frequency of claims that has gone to arbitration/litigation	3.64	4
12	Relationship with material suppliers	3.60	5
13	Local Area experience	3.52	6
14	Reference from previous clients	3.44	7
15	Enlistment with Government & other agencies	3.44	7
3. TECHNICAL CAPABILITY			
16	Experience of Engineers	4.64	1
17	No. of available Registered Engineers	4.60	2
18	Availability of skilled staff for the particular project	3.80	3
19	Experience of Associate Engineers	3.60	4
20	No. of Available Associate Engineers (DAE)	3.12	5
4. EQUIPMENT CAPABILITY			
21	Adequacy of contractor plants and equipment	4.08	1
22	Suitability of the equipment for particular work	4.00	2

23	Equipment own by the contractor	3.44	3
24	Equipment operator experience	3.40	4
5. MANAGERIAL CAPABILITIES			
25	Frequency of manager projects completed within Budget and Time	3.96	1
26	Qualification and Experience of administrative staff	3.68	2
27	Quality Management system of Firm	3.44	3
28	Organisational hierarchy system of Firm	3.36	4
6. HEALTH & SAFETY			
29	Health & Safety Accountability	3.40	1
30	Accident Records of previous projects	3.36	2
31	Health & safety Policy of firm	3.28	3
32	Insurance of all working Staff	3.04	4
7. PAST RELATIONSHIPS			
33	Client satisfaction with Contractor in Past	3.96	1
34	Contractor working relationship with the client	3.88	2
35	Contractor working relationship with the consultants	3.76	3
36	Contractor relationship with Supplier	3.16	4
37	Contractor relationship with subcontractors	3.00	5
8. GEOGRAPHIC LOCATION OF CONTRACTOR			
38	Contractor experience in geographic location of project	3.64	1
39	Contractor familiarity with local labors	3.60	2
40	Contractor familiarity with local suppliers	3.16	3
41	Contractor experience with local Authority	3.16	3
42	Contractor familiarity with local weather	3.00	4

Financial soundness of the firm or contractor is its financial resources to carry out the work. The research highlights top two factors of financial soundness as “Net worth of firm” and “Available credit line from bank”. Net worth of firm indicates the capacity of contractor for the current work while the second factor indicates the contractor is sound in terms of its bank account. It means whenever contractor require amount for work the banks are willing to give the loans to carry out the work hence client will have no interruption of progress in the work.

Past experience is basically divided into two parts, the general experience and experience particular to same work done in the past. The top two factors are “Contractor experience in similar projects during last 10 years”, “Contractors General Experience”. Similar nature experience is more important because the contractor which is willing for work must have knowledge related to the work.

Technical capability is concern with the available technical team and their experience. Top two factors of the research are, “Experience of Engineers” and “No. of available Registered Engineers”. Engineers are the main human resources in terms of technical ability of contractor because the more the quantity and experience of engineer the more sound is contractor.

Equipment capability is related with available plants and equipment by the contractor and their quantity and suitability for the concern work. Critical factor in this criteria are, “Adequacy of contractor plants and equipment” and “Suitability of the equipment for particular work”. Because due to complex procedures of design work, a client choose the contractor who is sound enough in terms of available equipment.

Managerial Capability includes the qualification and experience of key managerial staff to manage the work with available resources. Most of the time the contractor fails to provide the service just because of this criteria. Top most identified factors includes, “Frequency of manager projects completed within

Budget and time” and “Qualification and Experience of administrative staff”. Mostly this criteria is ignored in the past but now a days various clients are including in their prequalification criteria. Sindh Procurement Regulatory Authority (SPRA) also includes this criteria in its procurement guidelines.
Health and safety is also an important criteria but in Pakistan it is not usually of paramount importance unlike in developed countries. But recently different clients are adding this criteria including USAID in its Sindh Basic Education Program (SBEP) packages for schools construction in Sindh. The identified critical factors are, “Health & Safety Accountability” and “Accident Records of previous projects”.
In past relationship of contractor “Client satisfaction with Contractor in Past” and “Contractor working relationship with the client” are critical factors. The client ask to client to provide the certificate of appreciation from previous clients to check the ability of contractor. The client satisfaction in past is again an important factor to be considered.
Geographical Location of contractor is again an important criteria. Because the local contractor will have good information about the area and also can tackle various difficult situation with ease. The top ranked factor of includes, “Contractor experience in geographic location of project” and “Contractor familiarity with local labors”.

6. CONCLUSION & FUTURE RECOMMENDATION

The final selection of prequalified contractor in Public sector of Pakistan is mostly based on the minimum bid criteria. However this prequalification of contractors is itself a debated issue over the past few decades. Different set of criteria has been set by different Public and private sector organizations. This paper describes important criteria and sub factors for prequalification of contractors in Public sector of Pakistan, but the study has only focused the significance level of sub criteria that are already divided in major eight categories based upon literature and unstructured interview with the experts. This research includes the decision criteria by reviewing the current practices of prequalification process in various Public sectors of Pakistan. It also includes the factors from standard guidelines of Asian Development bank (ADB), Public Procurement Regulatory Authority (PPRA) and Pakistan Engineering Council (PEC). Different criteria like, Management Capabilities, Health & Safety, Past relationships of contractors and Geographical location of contractors, has also been included which is ignored in current practices of prequalification process in Pakistan and which has significant role in selection of contractors. The research is helpful for Public and also for Private sector clients to prequalify their desired contractor. However the results are purely based on the expert’s views about the significance level of each factor as per Construction Industry of Pakistan.

The author suggest to use any suitable multi criteria decision making technique to give proper weightage initially to main criteria and then to the sub factors as global and local weights to finally developed a decision support model. This developed model could be used by public and private sector clients for prequalification of contractor. The model would give score or weightage to all contractors and client can select the top ranked contractor as per their desired criteria.

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USE OF BUILDING INFORMATION MODELING FOR CONTROLLING AND IMPROVING LABOR PRODUCTIVITY IN CONSTRUCTION PROJECTS

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Abstract

Construction industry faces low labor productivity, and one reason being due to lack in planning and poor communication. The process of Building Information Modeling (BIM) is implemented to resolve these problems by providing a much clear image of design conflicts or constructability issues. BIM is used as an effective tool in a planning and communicating process which may results in amplified productivity rates. Productivity increases as BIM practices are employed which results in decreasing the rework and idle time for laborers. In this study, issues such as; RFI, CO and material shortage that affect labor productivity due to managerial process' bottlenecks will be highlighted and their probable elimination or minimization through BIM will be the focus. Impact of BIM use in-terms of degree of effectiveness and reduction in degree of occurrence will be assessed from different published papers, best practices and projects and a theoretical frame work will be developed for enhancing the overall Labor productivity using BIM. This study will pave the way for further research in this area.

Keywords

Labor productivity, Poor planning, Communication, RFI, CO

1. Introduction

Construction industry faces low productivity as that of other industries, mainly due to poor planning and communication. The process of Building Information Modeling (BIM) is implemented to resolve these problems by pretending physical work-space and communicating design concentrating graphically, providing much clarity of the conflicts of design or constructability issues so that they are set on prior to construction starts. The design which follows standard declaration tools are specifications and 2D Drawings, but there are some uncertainties associated with these things. When the plans and specifications fail to communicate design clearly, builders have to waste the time requesting for clarification of design, changes in plans, and sometimes the components which needs re-working according to the builder's clarification of the documents, but in fact not in compliance with the owner's needs. These insufficiencies increase the time and resources necessary to complete the product, that's the reason productivity rates, are lower and costs are expected to be more higher.

2. Factors of Poor Field Productivity

One of the major reasons of poor filed productivity rate is the lacking of planning and control over the construction activities. Construction productivity rate normally lie in range of 40% to 60% liable to skill (Eastman, 2008). Half of the time of worker's work is utilized in performing final product and half of that time is spent useless. Adrian (1987) proposed that 35% of worker's time is wasted due to waiting for the response in terms of instruction, material or other workers. It reflects the lacking in the role of single person that will collaborate, instruct and control the activities. BIM is famous for its collaboration and well communicative technology among all of its features.

3. BIM and Productivity

Use of BIM may result in efficient productivity rates rise in all the methods which rise rate in construction. It involves all the causes concerned with design and coordination. It results in low material wastages, decrease process and allow prefabrication. One of the major advantages of using BIM is to have earlier better design which results in high productivity through compression of overall project schedule. On one of the project of a large firm which has unreliable results which show that two to three times reimbursement is gained for the contractor for BIM expenses (Post, 2008).

3.1 RFI (Request for Information) and BIM

On any construction project the numbers of RFI represents clearness and entirety of plans and specifications of the project. When BIM is implemented on any of the project it results in few RFI as compare to the normal type of projects in which numbers of RFI are much larger. This type of reduction in RFI or in other words the reduction in the confusion of the design occurs on BIM projects as per the existing literature on BIM.

3.2 Change Order and BIM

Owner rewards extra cost to the contractor in terms of change order when changes in projects are not the part of actual contract document. The number of change order on any project represents the amount of misperception and cost that is incorporated due to delays which occur due to lacking in the information, changes in work or re-work when a problem is identified or resolved. BIM will result in significant reduction of change orders as the required changes can easily be predicted and if any change occurs it can easily be implemented and that will compensate all the changes in each of the component of design.

3.3 Material Schedule Compliance and BIM

Based on the expected competency of crews, construction schedulers should be able to forecast work duration to meet productivity plans. Scheduling normally emphasizes that the work should be complete sooner so that diverse tasks can be coordinated diligently. When contractor realizes that schedule is on the track, they are more likely to start the work as planned to initiate.

3.4 Productivity Improvement through Automatically generated shop Drawings

Less time is consumed in shop drawing production due to the automatic drawing production from standardized parametric components.

4. Factors affecting construction productivity around the globe

4.1 Thailand:

A study was conducted in Thailand to find out the critical factors which affect construction productivity and their potential for productivity improvement. 34 Project managers which were working in the construction industry of Thailand were asked to fill up a structured questionnaire. Those factors were ranked using RII concept according to the perception and experience of the project managers. Among the all, the top ten factors are:

1. Lack of material.
2. Incomplete Drawings.
3. Inspection Delay / RFI
4. Incompetent supervisor.
5. Instruction time.
6. Lack of tools and equipment.
7. Poor communication.
8. Poor site condition.
9. Change orders.
10. Poor Site layout.

Table below shows the RII for each factor with respect to degree of effectiveness and with respect to Potential for productivity improvement.

Table-01 Factors which affect Productivity and potential for productivity

Factors	Effect on Productivity			Potential for productivity improvement		
	Total Score	RII	Rank Order	Total Score	RII	Rank Order
Lack of material	131	0.642	1	91	0.535	11
Incomplete Drawings	121	0.593	2	93	0.547	4
Inspection Delay	114	0.559	3	88	0.518	15
Incompetent supervisor	113	0.554	4	94	0.553	2
Instruction time	111	0.539	5	95	0.559	1
Lack of tools and equipment	110	0.525	6	93	0.547	4
Poor communication	107	0.515	7	92	0.541	7
Poor site condition	105	0.510	8	66	0.388	22
Change orders	104	0.505	9	82	0.482	19
Poor site layout	103	0.490	10	93	0.547	4
Rework	100	0.475	11	94	0.541	2
Abseentism	97	0.466	12	91	0.541	11
Occasional working over time	95	0.461	13	92	0.506	7
Tools/ equipment breakdown	94	0.456	14	92	0.418	7
Interference from other trade or crew members	93	0.441	15	86	0.512	18
Over crowding	90	0.431	16	71	0.541	21
Workers turn over and changes in crew members	88	0.422	17	87	0.524	17
Specification and standardization	86	0.397	18	92	0.276	7
Scheduled working over time	81	0.392	19	89	0.518	14
Weather	80	0.382	20	47	0.535	23
Changing of foremen	78	0.373	21	88	0.544	15
Safety accidents	76	0.375	22	91	0.536	11
Shiftwork	69	0.338	23	73	0.429	20

(Makulsawatudom, A and Emsley, M (2001) Factors affecting the productivity of the construction industry in Thailand: the project managers' perception. *In:* Akintoye, A (Ed.), *17th Annual ARCOM Conference,* 5-7 September 2001, University of Salford. Association of Researchers in Construction Management, Vol. 1, 281-90.)

4.2 Indonesia, Nigeria, Uk and USA

The main Craft's man productivity problem in the Indonesia was due to Lack of material due to re-work, absenteeism, interference, lack of tools and equipment failure. The main causes of material shortage were 'In proper material storage', 'excessive paper work request', 'on-site transportation' and 'inadequate planning' but the main causes of re-work were changes in design and improper instruction. (Kaming et al.1997).A comparison table which was developed by **Kaming et al, (1997)** is given below which compare productivity problems in Indonesian with other countries like Nigeria, UK and USA.

Table-02 Ranks of factors affecting productivity

Productivity Problems	Indonesia	Nigeria	UK	USA
	Rank	Rank	Rank	Rank
Lack of Material	1	1	1	1
Lack of equipment	5	3	5	2
Interference	3	6	2	5
Abseentism	4	5	6	6
Supervision delays	6	4	4	4
Rework	2	2	3	3

(Proceedings of the Eastern Asia Society for Transportation Studies, Vol.4, October, 2003)

4.3 Trinidad and Tobago

A study was conducted in Trinidad and Tobago to find out the factors affecting construction productivity. Data was collected from the members of Trinidad and Tobago Contractor association.

Among 42 factors the top ten factors which affect construction labor productivity in Trinidad and Tobago are:

1. The lack of labor supervision.
2. Un realistic scheduling and expectation of labor performance.
3. Shortage of experienced labor.
4. Construction manager's lack of leadership skills.
5. Skill set of laborers.
6. Delay in responding to Request for information (RFI).
7. Payment delay.
8. Communication Problem between Construction manager and labor.
9. Rain and late arrival.
10. Early quitting and frequent scheduled breaks.

4.4 USA web survey

A web survey was conducted in September 2012 in USA in which different factors which affect the construction productivity were highlighted.

Table-03 Factors affecting productivity in USA

Factors	RII	Rank
Super vision delays	488.78	1
Variation in the drawings	488.75	2
Incomplete Drawings	483.0	3
Rework	471.5	4
Design changes	465.75	5
Inspection delays from the authorities	448.50	6
Payment Delays	442.75	7
Complex Design in the provided drawings	437.0	8
Implementation of govt. laws	419.75	9
Training session	414.00	10

(Study of factors affecting labor productivity at a building Construction project in the usa: web survey a paper Submitted to the graduate faculty Of the North dakota state university Of agriculture and applied science by Mahesh madan gundecha)

Table-03 shows that supervision delays rank first among all the factors with high RII variation in drawings are part of CO which affect construction productivity inspection delay is also a part of these factors which are normally caused due to late response of RFI's

4.5 Comparison with the previous studies

Table-04 Comparison of productivity factors

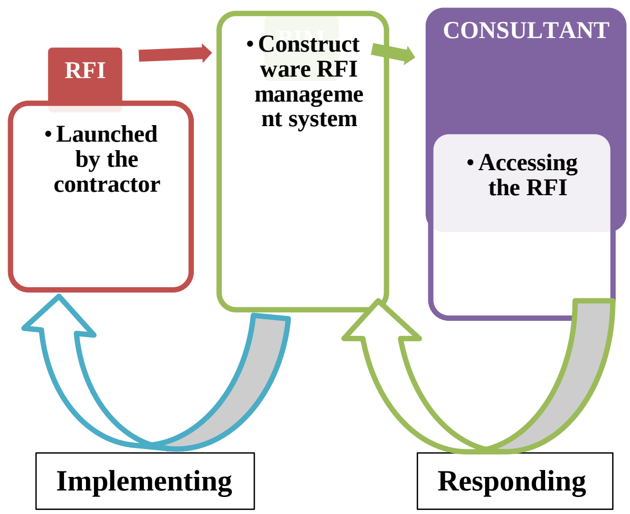
Rank	USA(Present study)	Nigeria(Olomolaiye et.al 1987))	Egypt(Enshaasi et.al 2006)	Malaysia (Abdul Qadir et.al 2005)	Singapore (Lim and Alam 1995)
1	Lack of required construction material	Inadequate or poor planning	Material shortage	Material shortage at project site	Difficulties recruiting supervisors
2	Shortage of Power and /or power supply	Mismanagement of funds	Lack of labor experience	Stopping of material delivery due to financial problem	Difficulties recruiting workers
3	Accidents during construction	Delay making decision and approval by the owners	Lack of labor Surveillance	Change order by CO causing project delay	High rate of labor turnover
4	Lack of required construction tools/ equipment	Affection for the use of low-quality material	Misunderstanding between laborers and superintendents	Non timely issuance of drawings by consultant	Labor abseentism at the work site
5	Insufficient lighting	Poor condition and communication	Drawings and specification changes during execution	Not able to organize site activities	Communication problem with foreign workers
6	Poor site condition	Late deliveries	Payment delays	Late issuance of payment by client	Inclement weather
7	Weather condition	Contractor's lack of experience	Labor disloyalty	Late supply of materials	Health issues
8	Differing site condition from plan	Discrepancies among architectural, structural, mechanical etc.	Inspection delays	Non-availability of labor for construction task	Material storage
9	Material storage location	In adequate and un clear drawings	Working saving days without holidays	Coordination problem with sub-contractor	Alcoholism and similar problem among work force
10	Working overtime	Bad weather condition	Tool and equipment shortages	Equipment shortage	Disruption of Power/ water supply

(Study of factors affecting labor productivity at a building Construction project in the usa: web survey a paper Submitted to the graduate faculty Of the North dakota state university Of agriculture and applied science by Mahesh madan gundecha)

Table-04 shows the comparison of current study with the studies which were conducted in the past related to the factors which affect construction productivity and among all the studies material shortages and changes in drawings are two main factors which were highlighted in these studies.

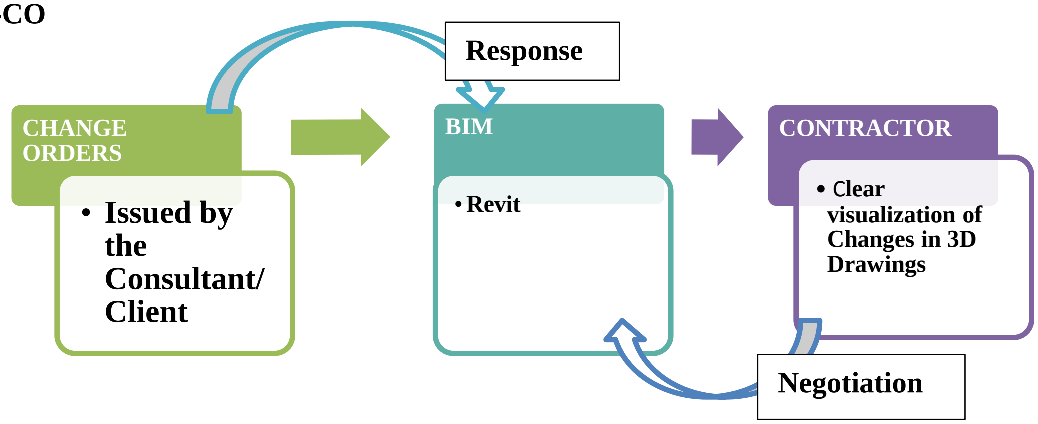
5. Conceptual frame work for productivity enhancement using bim

5.1 BIM-RFI:



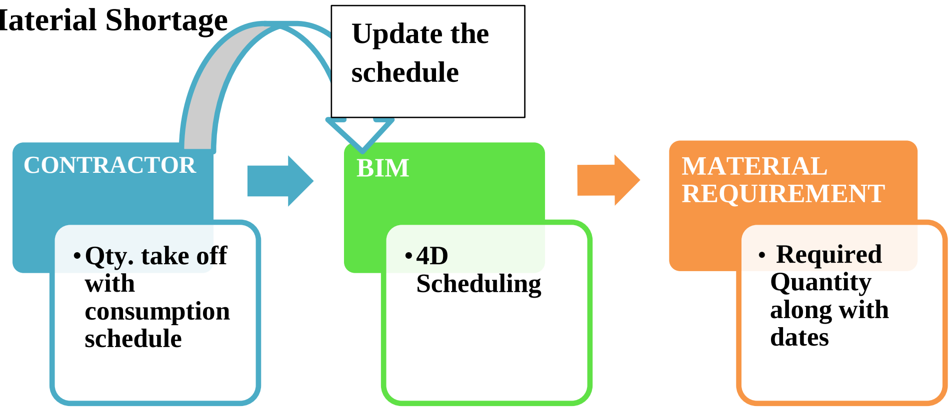
Since Request for information is one of the leading factor which affect the productivity by implementing BIM a significant decrease in timing of responding to the RFI may takes place. When contractor launches a RFI through construct ware application of BIM the consultant receives a notification and it is therefore much easy for both the parties to not only manage the RFI records through BIM but also the accessing and responding of RFI becomes much faster thus, results in higher productivity by saving waiting time for the further work by the contractor and if still something is not clear then contractor easily launches another RFI quickly for the further clarification.

5.2 BIM-CO



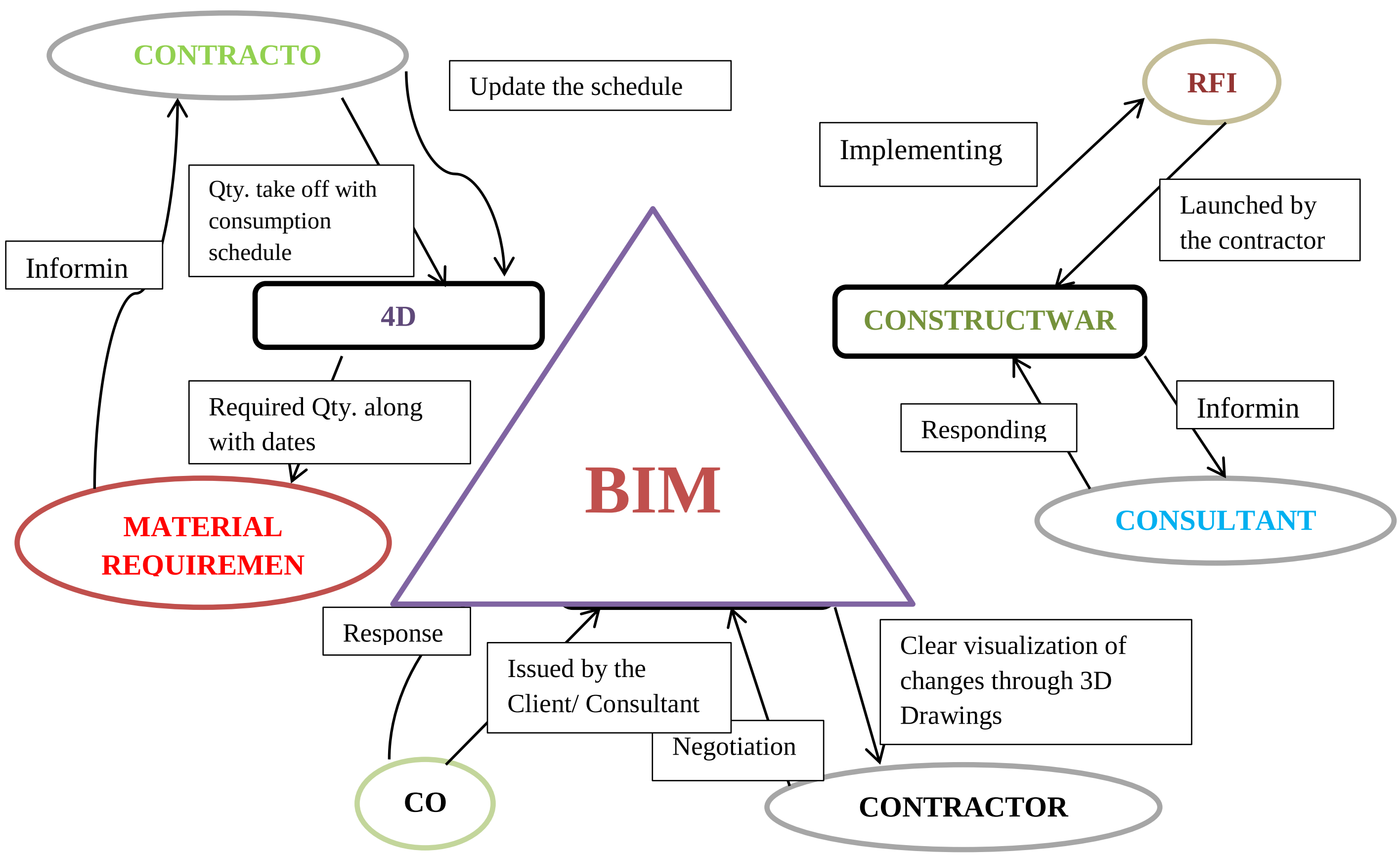
Change order is one of the factors which almost occur more rapidly on the construction sites and as discussed above it is the factor with RII of top five and top ten in the construction industry of different countries with respect to its degree of effectiveness to the labor productivity. When there is a change in the design by the Consultant, it happens many times that the changes are not clearly visualize in the 2D drawings by the contractor so sometimes it leads to the major re-work as well. When BIM is used to convey the changes using 3D drawings of Revit it is clearly visible and easy to visualize by the contractor and if contractor need any negotiation it will be easy for consultant to respond quickly using the platform of BIM.

5.3 BIM-Material Shortage



Shortage of material on the construction site is the factor with highest RII among all the factors which affect labor productivity in most of the countries as discussed above. In proper planning with respect to material delivery, storage and its consumption is the main reason behind the material shortage on the construction sites. When BIM is implemented and contractor enter the schedule of its material with respect to the date and consumption BIM-4D scheduling notify the material shortage before its fully consumption so that the construction manager order the quantity of material accordingly and when another shift of material is arrived then contractor updated the schedule for further in coming shift of material.

5.4 Complete Frame Work



When BIM is implemented on the construction site the major factors like Material shortage, RFI and CO which affects the labor productivity significantly may be controlled and it may lead to the overall productivity enhancement. Using Construct ware application of BIM we can manage the RFI's in an efficient manner. When contractor launches RFI construct ware notifies the consultant to respond it in a quickly manner so it saves the waiting time for the contractor, by using Revit to draft your drawings on 3D view any changes in design can be easily conveyed to the contractor without leading to any un necessary re-work which saves the chances of conflicts and also the time and cost associated with those un necessary re-work. Proper planning with respect to the material availability on the construction site and their consumption and re-order of material and the time at which material arrives at the construction site is one of the main feature that we may control using 5D scheduling technique of BIM. When these major factors like RFI, CO and material shortage are controlled labor productivity may be controlled in a significant manner by using BIM.

6. Conclusion and Recommendation

Using BIM on construction projects labor productivity can be enhanced in a great manner as the factors which affect productivity most can be controlled which ultimately results in higher Productivity. Shortage of material, RFI and CO is the most common factors which affect the construction labor productivity and they can be controlled using the developed frame work by Implementing BIM.

Since BIM is a revolutionary technology and it may help in controlling the main factors which affect labor productivity on construction site like RFI, CO and material shortages. A detailed study must be carries out in which the factors which affect construction labor productivity must be ranked using RII and then a detailed model using BIM should be developed to study the effect of BIM on those factors and the results should be presented in comparison of the productivity when no BIM is implemented.

7. Future Out look

A study will be made in the future to find out the current status of productivity in the construction industry of Karachi to evaluate the factors the factors which affect construction productivity more and then a model using BIM will be developed to access different case study with respect to productivity.

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Experimental Evaluation of Mechanical Properties of Locally Available Deformed Steel Bars

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Abstract:
The deformed steel bars are the significant material for modern reinforced concrete (RC) structures. The size, strength and grade of reinforcing steel bars are three basic parameters for the safe, durable and reliable concrete structures. Therefore, experimental investigations were carried out to investigate the mechanical properties i.e tensile and bending strength of locally available deformed steel bars. Five brands of deformed bars A, B, C, D & E of Grade 60 and 40 were selected for the experimental work. The results were checked according with the ASTM standards. It was investigated that the tensile strength of all five brands has meet the ASTM standards but brand A and E fails in bending. Furthermore, three brands A of G-60, D of G-40 & E of G-40 fails in elongation test. Hence, the purpose of this study to increase awareness among the professionals about quality of steel bars available in local market.

Keywords:
Deformed steel bars, tensile strength, bending strength, elongation.

Introduction:

Steel is the important material for the construction industry and it plays vital role in the sustainable development of the society. Steel is an iron-based material containing low amounts of carbon and alloying elements that can be made into thousands of compositions with exacting properties to meet a wide range of needs, about 26 diverse elements are used in various proportions and combinations in the manufacture of both carbon and low alloy structural steels [1] and the deformed steel bars are the significant material for modern reinforced concrete (RC) structures. The size, strength and grade of reinforcing steel bars are three basic parameters for the safe, durable and reliable concrete structures, actually the reinforcing steel helps to resist tensile stresses in reinforced concrete members and their use in construction works is specified by relevant codes such as BS 4449:1997. Reinforcing steel for construction works are specified in terms of their yield strength, ultimate tensile strength, percentage elongation. By assessing these properties, it is ensured that their use in construction works meet relevant code specifications [2] Therefore, an experimental investigations were carried out to investigate the mechanical properties i.e tensile and bending strength of locally available deformed steel bars and results were compared with the ASTM standards.

Material and Methodology:

The Five brands of hot rolled deformed bars A, B, C, D & E of Grade 60 and 40 were collected for the experimental work. For each company, three bars (10mm, 16mm & 25mm) were randomly chosen. Total 30 samples were prepared, fifteen samples for Tensile strength test and fifteen for bending strength test. Each bar diameter measured in three places with the help of vernier-caliper and average diameter were obtained as the nominal diameter for the bar. Then each specimen was subjected to tensile strength in accordance with ASTM (American Standard for Testing and Materials) specifications, and after fracture, the yield and ultimate strength, characteristic strength and percentage elongation were calculated.

Table 1: ASTM Standard

ASTM Code	Grade of Steel	Yield strength (MPa)	Ultimate Strength (MPa)	% Elongation	Modulus of Elasticity (GPa)
A615/A615M*	G-40	280	500	11 %	200
	G-60	420	620	9 %	220

**ASTM A615/A615M: Deformed and plain carbon-steel bars for concrete reinforcement*

Result and Discussion:

The stress that causes a material to lose its elastic behavior is called yield strength and the tensile strength or ultimate strength is the maximum stress that a material can handle before breaking. To analysis the mechanical properties of locally available deformed steel bars, the laboratory results are demonstrated in figures 1 to 4 and in Table 2 & 3.

Table 2: Experimental investigation of Mechanical Properties of deformed steel bars

S. #	Brand & Grade	Nominal Diameter (mm)	Cross-sectional Area (mm^2)	Measured Diameter (mm)	Cross-sectional Area (mm^2)	Yield Strength (Mpa)	Ultimate Strength (Mpa)	Modulus of Elasticity (Gpa)	% Elong ation
1	A (G-60)	9.5	70.88	9.38	69.07	655	751	208	5.47
2		16	201.06	15.93	199.21	626	734	216	12.5
3		25	490.86	24.99	490.63	486	737	447	15.63
4	B (G-60)	9.5	70.88	15.67	192.80	584	698	117.59	12.5
5		16	201.06	15.90	198.50	474	734	209	12.5
6		25	490.86	24.85	485.05	653	737	238	15.6
7	C (G-40)	9.5	70.88	9.37	68.92	408	532	1136	17.5
8		16	201.06	15.13	179.90	360	557	234	21.88
9		25	490.86	24.37	466.60	416	546	310	23.44
10	D (G-40)	9.5	70.88	10.03	78.97	593	722	331	7.82
11		16	201.06	15.63	191.80	560	747	209	15.63
12		25	490.86	25.18	498.10	283	415	301	23.44
13	E (G-40)	9.5	70.88	9.38	69.07	478	599	377	6.25
14		16	201.06	15.67	192.80	461	514	384	12.5
15		25	490.86	24.91	487.50	392	586	392	10.94

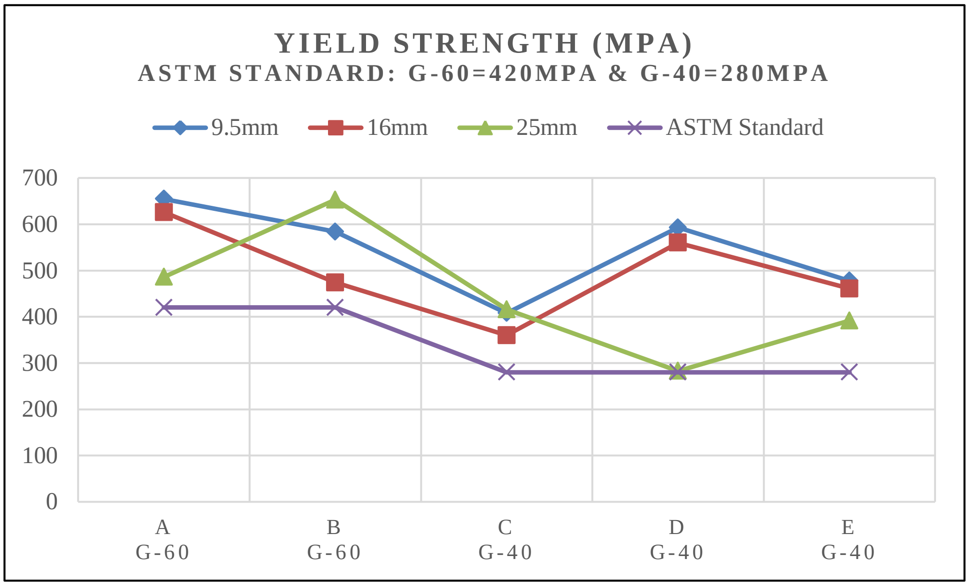

Figure 1: Yield strength values of different brands

From the above results, the yield strength values in all five the brands were found greater than the requirement of ASTM standards.

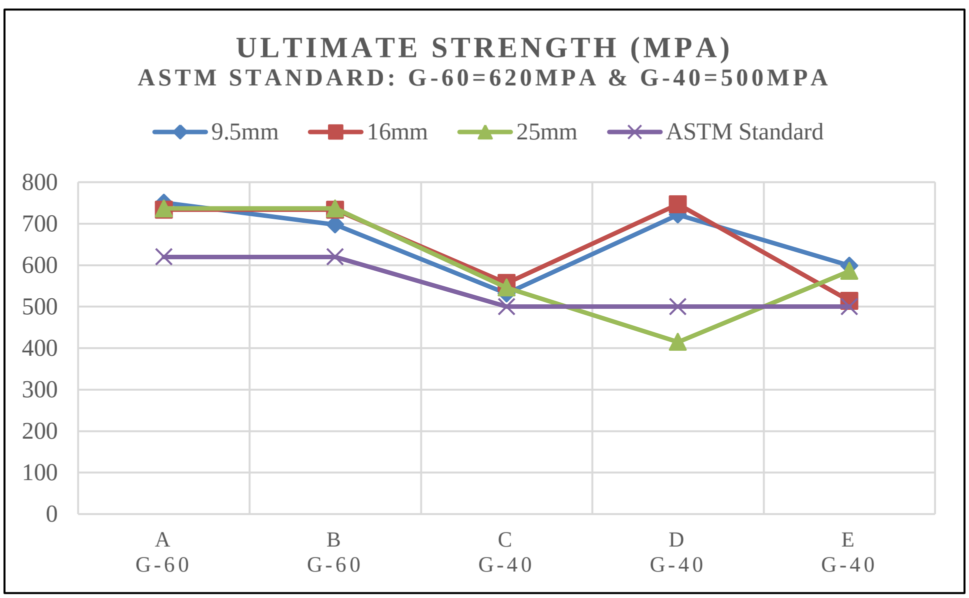

Figure 2: Ultimate strength values of different brands

From the above results, the ultimate strength values were also found greater than the requirement of ASTM standards except one brand D of Grade-40.

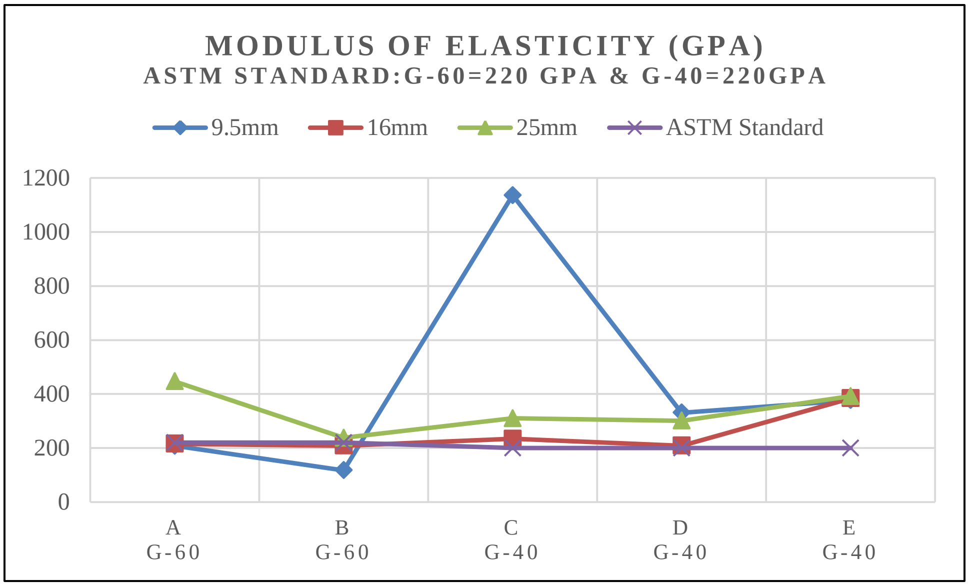

Figure 3: Modulus of Elasticity Values of different brands

From the above results, the modulus of elasticity values were also found greater than the requirement of ASTM standards except one brand B of Grade-60.

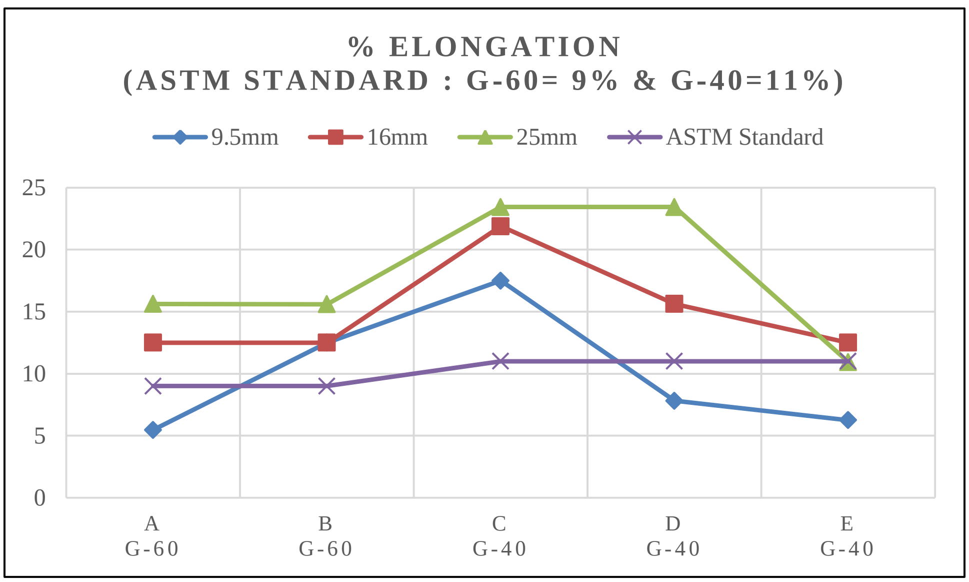

Figure 4: % Elongation values of different brands

Above result indicated that the percentage elongation values were found good in two brands (B & C) only and in other three brands (A, D & E) fails according to the ASTM standards.

Table 3: Experimental Analysis of Bending of deformed steel bars

S.No.	Brand & Grade	Measured Diameter (mm)	Cross-sectional Area (mm^2)	Length (m)	Weight (Kg)	Remarks
1	A (G-60)	9.38	69.07	0.568	0.54	Not Satisfactory
2		15.93	199.21	0.557	1.56	Satisfactory
3		24.99	490.63	0.572	3.85	Satisfactory
4	B (G-60)	15.67	192.80	0.568	1.51	Satisfactory
5		15.90	198.50	0.565	1.56	Satisfactory
6		24.85	485.05	0.573	3.81	Satisfactory
7	C (G-40)	9.37	68.92	0.562	0.54	Satisfactory
8		15.13	179.90	0.562	1.41	Satisfactory
9		24.37	466.60	0.573	3.66	Satisfactory
10	D (G-40)	10.03	78.97	0.568	0.62	Satisfactory
11		15.63	191.80	0.574	1.51	Satisfactory
12		25.18	498.10	0.562	3.91	Satisfactory
13	E (G-40)	9.38	69.07	0.568	0.54	Satisfactory
14		15.67	192.80	0.568	1.51	Satisfactory
15		24.91	487.50	0.565	3.83	Not Satisfactory

Although the weight of deformed has been calculated as fallows,
Weight per meter of deformed bar in Kg = D^2 / 162.2, Noted that diameter should be in "mm"

The randomly selected bars has been checked for the bending and from the laboratory investigations it was found that the brand A of G-60 and E of G-40 fails in bending.

Conclusion:

From the experimental investigation it was observed that the size, strength and grade of reinforcing steel bars are three basic parameters for the safe, durable and reliable concrete structures. Therefore, mechanical properties i.e tensile and bending strength of locally available deformed steel bars were checked. Five locally available brands of deformed bars A, B, C, D & E of Grade 60 and 40 were selected for the experimental work. The results were checked according with the ASTM standards. It was investigated that the tensile strength of all five brands has meet the ASTM standards but brand A and E fails in bending and brand D was found lower ultimate strength as per ASTM requirement. In addition to that three brands A of G-60, D of G-40 & E of G-40 fails in elongation test.
Hence, it is recommend that before the selection of the steel bars for the RC structures, the laboratory based investigations are required to check the mechanical properties of steel bars, by adopting this technic professionals should be confident enough about the quality of deformed steel bars used in the civil structures.

Acknowledgment:

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Evaluation Of Response Reduction Factor For RC Frames

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Abstract
Reinforced concrete frame structures with infill masonry are a common indigenous building type. In past, the influence of infill masonry in RC structures has been investigated widely and it constitutes a critical role in the lateral load response of RC structures. The behavior of masonry is highly non-linear therefore it is necessary to incorporate the homogenous properties and failure modes of infill through a rational computational model approach using macro-modeling technique which gives a realistic global response of structure efficiently. In general the response of a structure is assessed through a response reduction factor '*R'* which allows the practitioner to use a linear elastic force-based design while accounting for non-linear behavior and deformation limits. This study focuses on the estimating the rational value of '*R'* for RC buildings present in the locality of Karachi and check its adequacy with code recommended '*R'* value. To this end, a 2D-frame has been assessed by developing its computational models for bare frames using computer code SAP2000 and results are presented in terms of pushover curves for different limit states.

Keywords:
Seismic Assessment, Pushover Analyses, Macro Model, Response Reduction Factor.

1. Introduction

Karachi, the city of lights and a major urban centre of pakistan is the densly populated with an estimate of 14 million people. It lies close to a plate boundary and is in the grasp of tectonically active regions surrounding the city.It is approximately 150 km east of triple junction between Arabian, Indian and Asian Plate.There is a very limited historical account available for any earthquakes experienced by this megacity but the city has felt a damaging shake in past 200 years due to active structural zones (Bilham.R *et al*,2007). Karachi lies in Zone 2B of according to the latest seismic zoning map of pakistan is close to experience a devastating earthquake in the coming years although no document proves that it had been hit by an seismic activity in the past but that maybe because of unrelaible historcal data.

Reinforced concrete frames with masonry infill are a common building type in Karachi. These buildings are both engineered and non-engineered based on the construction and design practice followed.The seismic code documented in Pakistan is adopted by Uniform Building Code which gives a false representation for most of the structures designed in Pakistan since the construction and design practices are not done according to UBC and leads to an inadequate and incompetent building resulting in loss of life if a calamity strikes .The philosophy of an earthquake resistant design is that a building should resist a ground motion without collapse but some damage is acceptable. So the seismic loads imposed on the structure are much greater than it is designed for. A response reduction factor ‘R’ is used to reduce the design forces; it is an essential tool which is used to describe the level of inelasticity expected in a lateral structure system during an earthquake. Different codes proposed different values for the “R” depending on the condition of the lateral force resisting system. Since Pakistan follows Uniform Building Code (UBC-97) the value of R is usually taken as 3.5 for OMRF, 5.5 for IMRF and 8.5 for SMRF. These values recommended by UBC are overestimated so are unrealistic for the indigenous structural practices followed in Karachi. Thus this study focuses on estimating a rational value of “R” for RC frames in Karachi and compares it with the different seismic codes recommended values. To this end a six storey, 3 bay frame is considered for non-linear pushover analyses as bare frames for conditions of Ordinary (OMRF), Intermediate (IMRF) and Special moment resisting frame (SMRF).

2. Case Study

2.1 Prototype Reinforce Concrete Moment Resisting Frame

A prototype MRF was selected which was a 3 Bay, 6 Storey frame. This frame was altered for OMRF, IMRF and SMRF by changing important sectional parameters giving a range of common existent RC building parameters in Karachi. The material properties are identical for all frame types and are assumed to be (a)reinforcing yield strength *fy*= 420 MPa (60 Ksi) , (b)concrete compressive strength *f'c*=30 MPa (4.35 Ksi), (c) live load (25% for earthquake) is 2.5 KPa, (d) superimposed dead load is 3.0 KPa . The storey height is 5 and 4 meters respectively. The size of column and beams were 750x750 mm and 500x750mm respectively. The member cross section and detailing for IMRF frame is shown in Figure 4. The cross section area for all the frames will be same only the degree of confinement was changed to make the frame an OMRF and SMRF. For OMRF the confinement was changed from 150 mm to 200mm and for SMRF it was changed to 100mm.

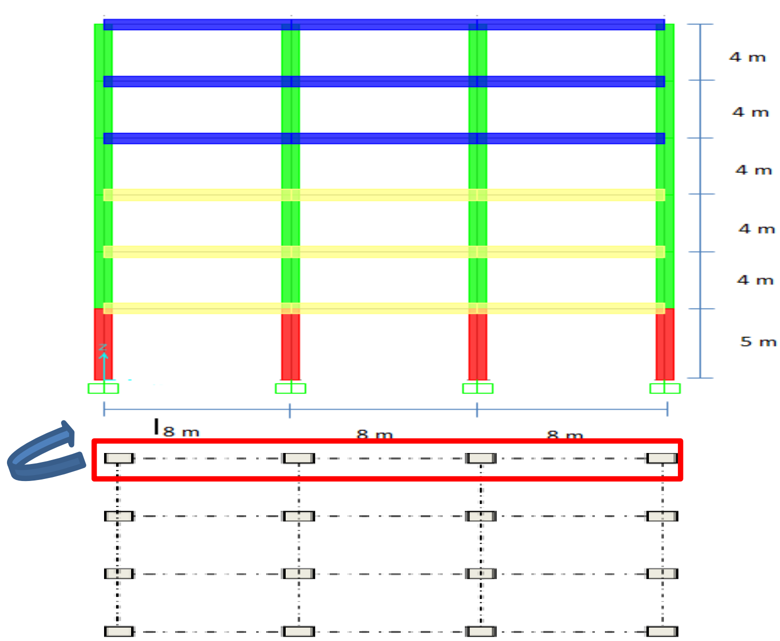


Figure 1: Plan and Elevation of Prototype Structure

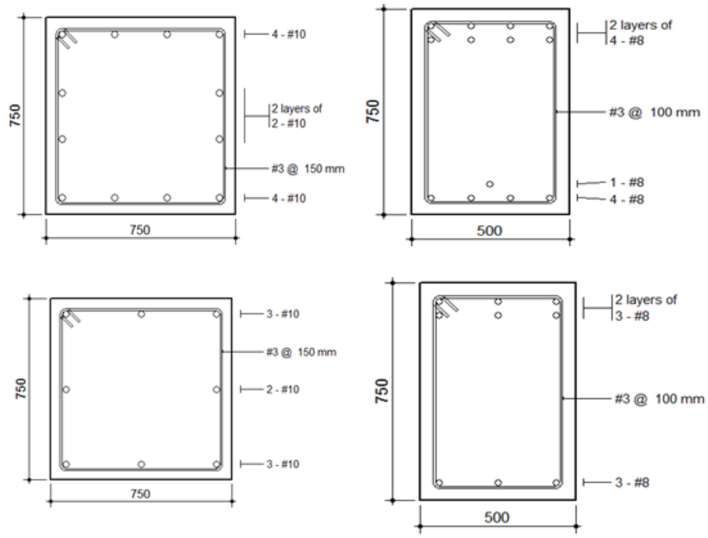


Figure 2: Member Cross Section and Detailing (a) Column at Ground Floor (b) Column for all the remaining floors (c) Beam for 1st to 3rd Floor (d) Beam for 4th to 6th floor

2.2 Type of Building Structures in Karachi

Reinforced concrete structures in Pakistan are mainly found in the urban region namely Karachi, Islamabad and Lahore; it constitutes about 7.64% of the total building stock in Pakistan. In Karachi, the percentage of RC structure is higher i.e. 15 to 20% of the total building stock. Usually RC buildings in Karachi are from 15 to 20 stories tall with a story height of 3 to 3.35 meters (WHE Report 159).

RC buildings are classified into basic load bearing structures and moment-resisting frames with infill for floor panes and walls. The material of infill for Karachi comprises mainly of concrete blocks since sand, gravel and aggregate are easily available.

The type of construction usually practiced in Karachi ranges from one to around six storey for bungalows and about eight storey for apartment complexes. RC Buildings are more common in urban areas because it has many advantages over other mode of structures such as ease in availability of material, construction tools and expertise (WHE Report 167).

The construction practices in Pakistan vary in nature; the buildings are divided as engineered and non-engineered. Engineered buildings are designed and built by engineers or contractors having awareness of earthquake engineering. So the RC Buildings which are designed according to seismic provisions show considerable seismic resistance when compared to other construction practices. Non-Engineered buildings are not designed or supervised by professional expertise. The research is ongoing and will include non-engineered buildings since this is also a major construction practice in Karachi.

2.3 Seismic Design Code of Pakistan

Pakistan is a seismically vulnerable country and many parts of it lie in risk zones. It has experienced about 18 devastating earthquakes and a total of 61000 fatalities (Lodi.S.H, 2015).

After the devastating Kashmir Earthquake of 2005 a need for seismic code for Pakistan was greatly felt and Earthquake Reconstruction and Rehabilitation Authority (ERRA) decided to adopt Uniform Building Code (UBC 97) and named it Building Code of Pakistan (BCP 2007) (Zafar.A ,2009) .The value of R in BCP 2007 are based on ductility and over strength factors designed for buildings in USA since the structures in Pakistan are not as ductile when compared with USA giving a false representation leading to an inadequate structure which may cause failure and loss of life in a seismic event.

Table 1: Value of R factor from BCP 07

Basic Structural System[2]	Lateral-Force-Resisting System Description	R
3. Moment-resisting frame system	1. Special moment-resisting frame (SMRF)	
	a. Steel	8.5
	b. Concrete[4]	8.5
	2. Masonry moment-resisting wall frame (MMRWF)	6.5
	3. Concrete intermediate moment-resisting frame (IMRF)[5]	5.5
	4. Ordinary moment-resisting frame (OMRF)	
	a. Steel[6]	4.5
	b. Concrete[7]	3.5
	5. Special truss moment frames of steel (STMF)	6.5

3. Evaluation of Response Reduction Factor

In earthquake engineering the concept of controlled damage is kept in mind when designing a structure because if concept of elastic deformation is applied for earthquake loads than it would result in the form of very uneconomical structures. So many seismic design codes use the concept of response modification factor or behavior factor, it is named differently in for different seismic codes. The response reduction factor reflects the capacity of structure to dissipate energy through inelastic behavior. It is a combined effect of over strength and ductility (ATC 95) which are incorporated in earthquake resistant design through response modification factor.

$$R = R\mu \, x \, \Omega \tag{1}$$

The response reduction factor basically scales down the elastic response of structure (Brozi and Elnashai 2000). It represents as a ratio of maximum elastic force for a specific ground motion V_e to a lateral force V_d which is the structure is designed for to withstand the earthquake.

$$R = \frac{Ve}{Vd} \tag{2}$$

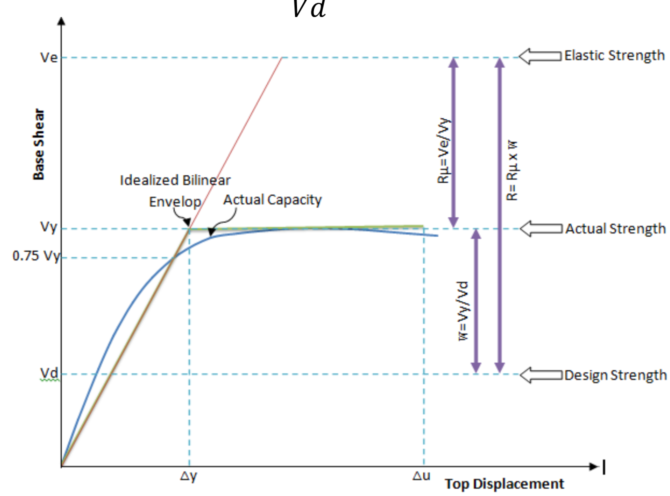


Figure 3: The concept of Response Modification Factor and its relationship between over strength and ductility factor.

3.1 Ductility Factor Rμ

Ductility is a property which allows the structures to dissipate energy due to which the elastic design force can be reduced to idealized yield strength level.

$$R\mu = \frac{Ve}{Vy} \tag{3}$$

For a single degree of freedom the relation between μ and Rμ is well established by researchers Newmark and Hall in 1982 (Uang 1991).

3.2 Over strength Factor Ω

It is the reserve strength of the structure which plays a part in collapse prevention of the building. It is described as the ratio of actual yield strength to the lateral design strength.

$$\Omega = \frac{Vy}{Vd} \tag{4}$$

It is a result of many factors such as internal force redistribution, strain hardening, high material strength, member oversize, nonstructural elements and so on (1991).

3.3 The Value of R factor in other codes

Many countries prone to earthquake events have developed their own seismic codes which include the value of response modification factor. In different codes the formulation and values of R varies depending on the lateral force resisting system and ductility class of structure. The R factor is named as behavior factor in Eurocode 8 and response modification factor in ASCE-7 (2005).

Table 2

Structural System	R	
	Eurocode 8	ASCE-7
OMRF	3	3
IMRF	-	5
SMRF	5	8

4. Non-Linear Static Pushover Analysis

There are two types of non-linear analysis proposed in ASCE-41 (a) Nonlinear Dynamic Analysis and (b) Nonlinear Static Pushover Analysis. The later approach is adopted for presenting the work in this paper. Non-Linear static approach is reliable, simpler and less time consuming than the dynamic approach.

Numerical analysis has been performed on a bare frame building and modeling is carried out on sap2000. Although the scope of study includes infill material but due to time constraint this model is presented in the paper since the research is still proceeding.

In sap2000 the inelastic member elements are modeled as elastic members with user defined plastic hinges using FEMA 356 and section analysis.

4.1 Computational Model of Bare Frame With Variant Ductility

In Sap2000 a numerical model for bare frame was developed and ductility of the member sections was altered for changing the frame from IMRF to OMRF and SMRF. There are many ways to improve ductility of a structure the method adopted in this research is by improving section ductility by refining the degree of confinement.

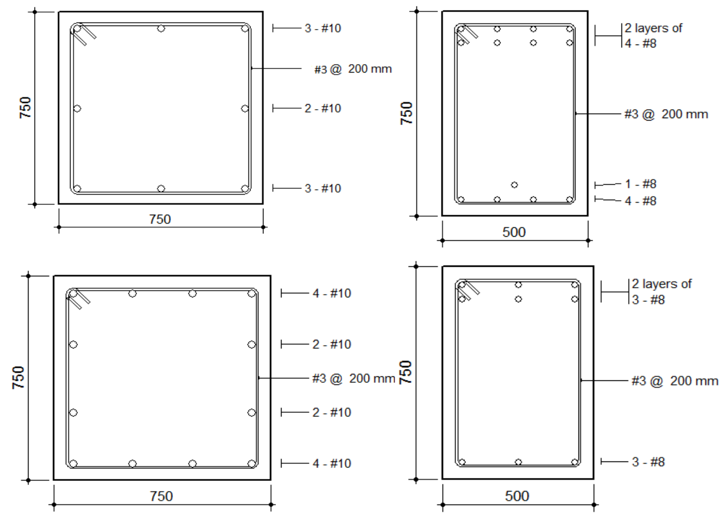


Figure 4: OMRF cross section with degree of confinement reduced

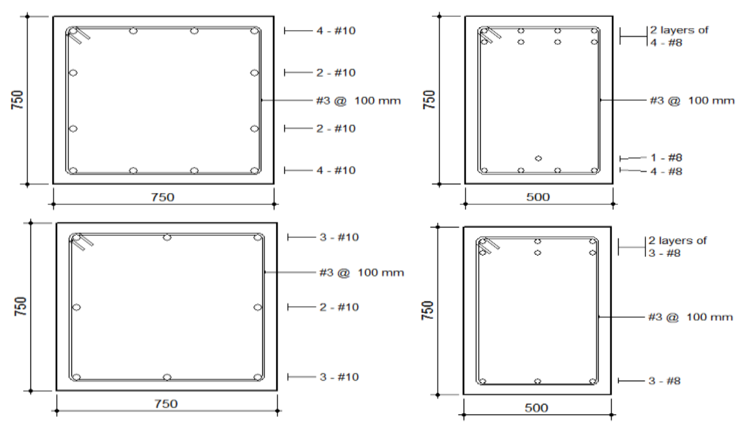


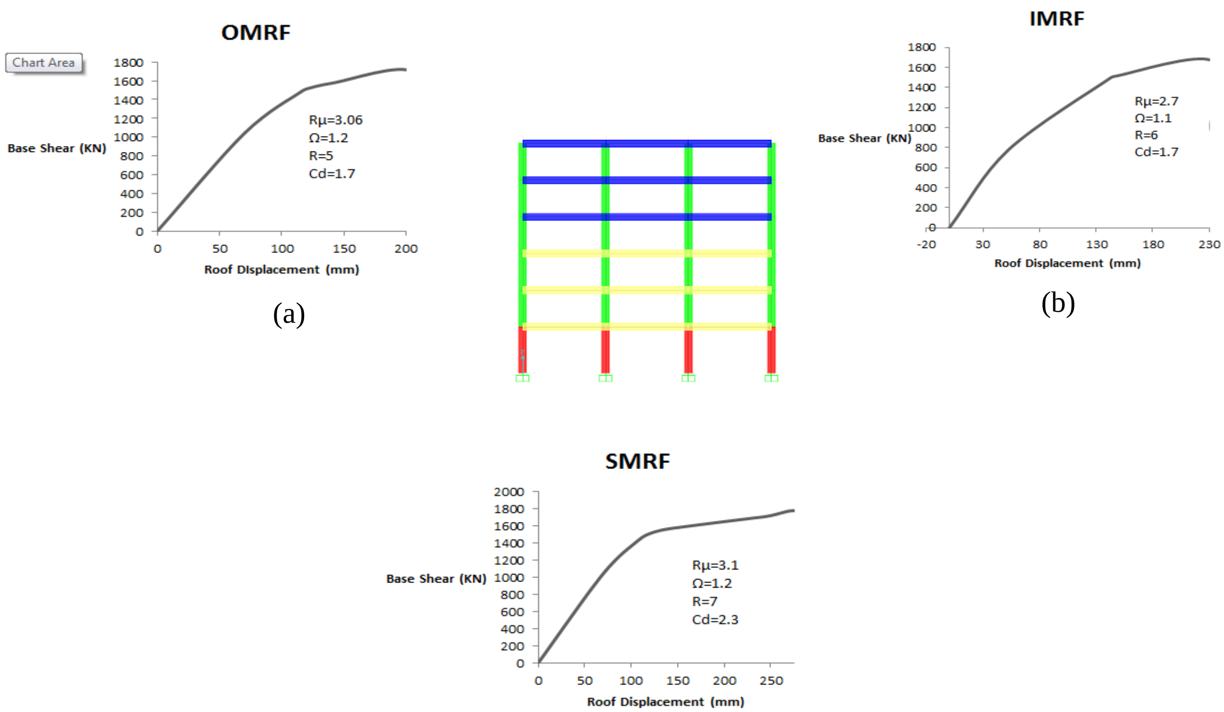
Figure 5: SMRF cross sections with improved ductility.

5. Interpretation of Results

In this section the response reduction factor calculated for OMRF, IMRF and SMRF is discussed and compared. The results are shown in the form of pushover curves obtained from the numerical analysis.

5.1 Response Reduction Factor

As mentioned above the R factor reduces the design forces taking advantage of the fact that the structure has reserved strength and capacity to dissipate energy. The results from case studies are presented in this section. The value of R is compared with UBC 97 and calculated values of analysis are listed in figure 6 (a) for OMRF the value of R is 5 which is higher when compared with the code recommended value 3.5,(b) for IMRF the value of R is 6 which is slightly higher when compared with the code which is 5.5, (c) for SMRF the value is 7 which is also lower when compared with the code recommended value 8.5.



(a) (b) (c)

Figure 6: Performance of (a) OMRF (b) IMRF (c) SMRF

6. Conclusion

This research explores analytically the response reduction factor R for indigenous building type in Karachi and how it varies from the code recommended value.

A prototype structure was selected which showed a similar range of geometric and material properties practiced locally. Through this model it was found that the R factor suggested in seismic codes is quite comparable with code recommended value for OMRF and IMRF but for SMRF the value gives a false representation.

The research is still progressing and will include infill frame structures. The structures will be present physically in the locality and will be analyzed using a more refined computer code. Parametric analyses will then be carried out using these frames to explore the effect of material and geometric characteristics on the value of R so that more precise predictions for the value of R can be adopted.

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Separation and Gap Induced by Slip Behaviour of Inclined Headed Stud Connectors in the Steel-Concrete Composite Beam

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Abstract
This paper addresses the advantage of the inclined headed shear stud connector on the ultimate moment capacity of the composite beam in terms of separation and gap induced by slip behaviours. The FE models for a composite beam with full and partial connections with acceptable validation by means of available experimental models, were used to study. Nodal displacements and stressed developed into the FE models were extensively analysed throughout the analysis in order to determine the behavior of the inclined shear stud connectors . Finally, it was concluded that presented in the form of inclined headed shear stud connector, this solution is very convenient to use, and sufficiently accurate function of the headed shear stud connector is predicted, which will be useful in the structural applications.

Keywords
Inclined shear stud connectors , Composite beam, Seperation, Gap induced by slip, Ultimate limit state, FE modelling.

1. Introduction

Engineering understanding of the mechanical shear stud connectors of the steel-concrete composite beam has mainly based on vertical application and various forms of application in such small device instead of headed shear stud connectors have been updated to incorporate in research and design applications. The ultimate limit state of the composite beam in the application of inclined stud connectors was studied by Bavan et al. (2016)[a, b, c] and they found that the global behavior of the composite beam is improved by the application of shear stud connectors as inclined form. This paper studies the local behavior of the composite beam such that the separation and gap induced by slip behaviors have been postponed in the application of the shear stud connectors as inclined form, which will be more reflective of the actual use of composite beams.

2. Leterature Review of Experimental Programme

A series of the steel concrete composite beam was tested by Tan et al. (2009) in order to determine the behavior of the composite beam subjected to combined flexure and torsion, which is reviewed in this research study. The test setup of full scale composite beam and loading setup adopted from the experimental program done by Tan et al. (2009) are shown in the Figure 1(a) and Figure 1(b), respectively. The steel concrete composite beam consisted of concrete slab, steel beam and shear connectors. The shear connectors were considered as welded connection along the flange of the steel beam with a spacing of 285 mm in order to provide interconnection in between steel beam and concrete slab and based on the degree of shear interaction, the numbers of the shear stud connectors were maintained such that the shear stud connectors were welded in a double line at 30 mm distance from centre line in order to get full shear interactions and single lines at the centre of the beam in order to get partial interaction. The test arrangement of specimens with interconnected structural components, and load and support applications are indicated in the Figure 2. There were two series of the composite beam included in the tests such that composite beam with full shear connection altogether six beams, were labelled from CBF1 to CBF6 and composite beam with partial shear connection altogether six beams, were labelled from CBP1 to CBP6. However, the composite beams labelled as CBF1 and CBP1 were only used in this research study.

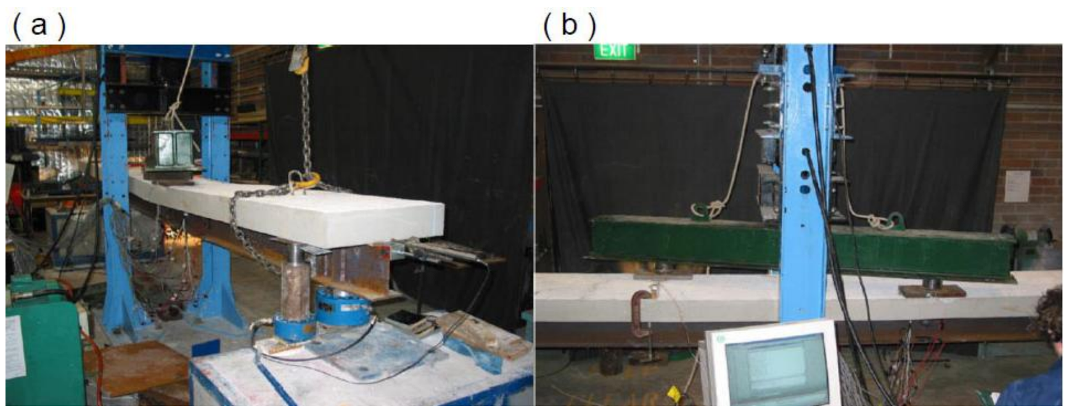

Figure 1: Experimental programme (a) Full scale specimen of steel concrete composite beam (b) Loading setup; Source: Tan et al. (2009)

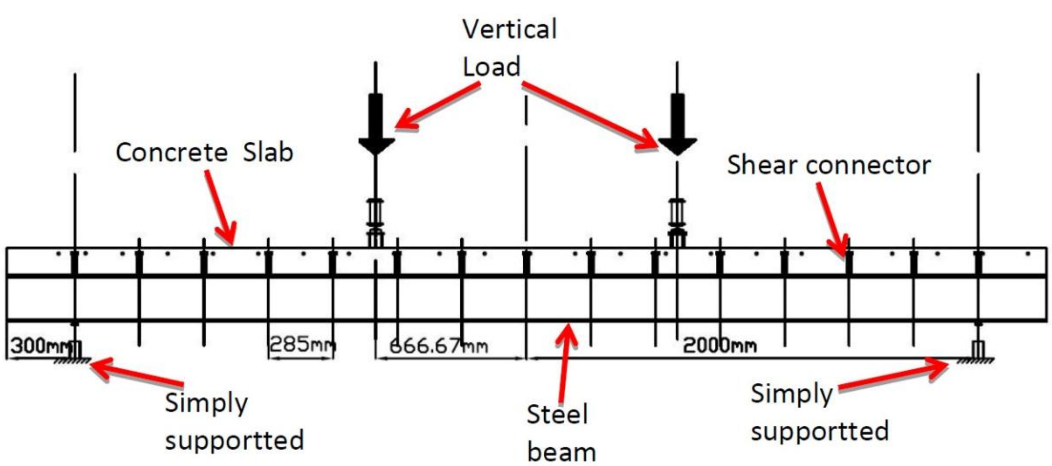

Figure 2: Application of load and supports

3. Development of Fe Models and Validation

The FE models for composite beams labelled as CBF1 and CBP1 were developed by Abaqus and the geometries, load and support applications, and elements used in the FE models of the composite beam with full and partial connections are shown in the Figure 3. In addition, material models also were developed for the geometries connected with the composite beam such that concrete, steel beam, shear stud connectors and reinforcing bars were developed separately and applied to the developed FE models. The material tests for concrete and steel done by Tan et al. (2009) were used to develop the material models. While considering the plastic damaged model proposed by Lubliner et al. (1989), the compressive behavior and tensile behavior of concrete material models were developed separately using the derivations of Desay & Krishnan (1964) and Eurocode (1994) as shown in the Figure 4(a) and Figure 4(b), respectively. Moreover, the steel components were considered by coupon test to all specific steel material models in order to determine parameters of yield stress, ultimate stress and the percentage of elongation of flange and web of the steel beam and reinforcing bars. The material model of the structural steel beam was developed using the derivation of Gattesco (1999) as shown in Figure 5(a) in order to develop better non-linear behaviour to the structural steel beam. Meanwhile, the reinforcing bars consist of steel bars the behavior of which was assumed to have a bilinear stress strain characteristic behavior as specified in Figure 5(b). The material model of shear stud connectors was developed as elastic plastic model as shown in Figure 5(c).

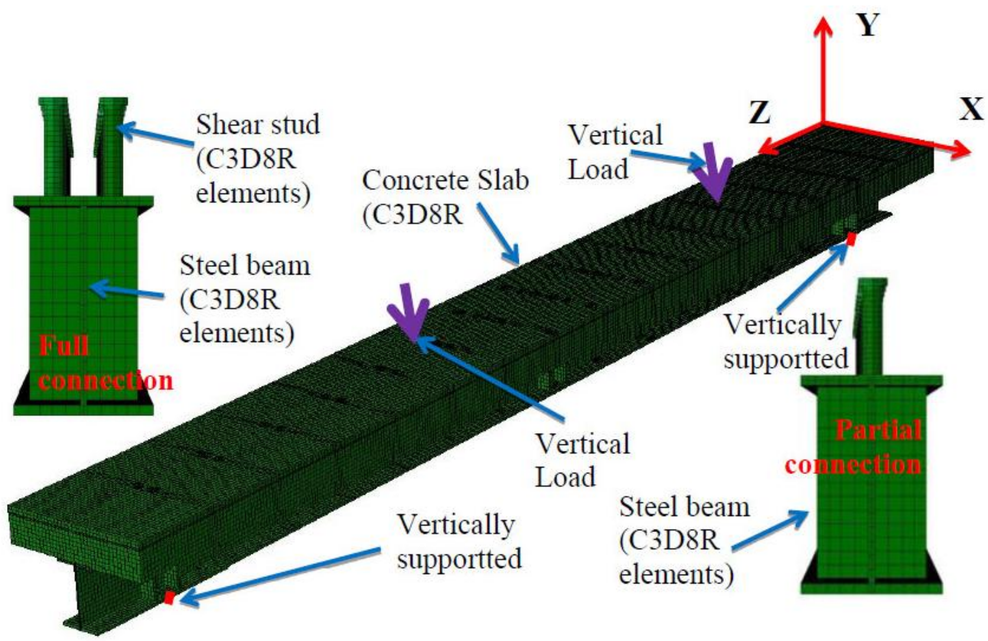

Figure 3: Finite Element model of the specimen

A more comprehensive approach was included in the prediction of results in terms of fractures on material geometries, in which a numerical solution based on Abaqus explicit solver with quasi static solution was used to validate both of the FE models. In this approach, all simulations were studied throughout the analysis using slow nodal displacement accounted by analyzing the concepts which allow the concepts of internal-kinetic energy concept and applied-support loads equilibrium. The concrete behaves with crushing and cracking failures and the sudden drop due to the fractures may encounter excess kinetic energy in explicit solver, which eliminates the static state. In order to simulate this occurrence, the uniform slow nodal-displacement control was selected in load application with studying various deformation rates in which the kinetic energy was maintained lesser than 5 % of internal energy throughout the analysis. Besides, the static state was evaluated by the applied load-support reaction relationship in which both have to be very close numeral to each other. The FE model developed for the specimen of CBF1 in which the specimen was fabricated with full shear interaction gave the similar failure criteria at ultimate limit state such that the concrete slab near the loading point was shown the crushing failure as shown in Figure 6. For moment, the maximum moment in the experiment and FE model was 221.7 kNm and 208.2, respectively, with a coefficient of deviation of 0.06 while for the deflection, the deflection at the maximum moment was 100 mm and 93 mm, respectively. Correspondingly, for composite beam with partial shear connection labelled as CBP1, the moment-deflection relationship was shown a better agreement in between the experiment and FE model as indicated in the Figure 7. For moment, the ultimate moment in the experiment and FE model were 190.22 kNm and 181.34 kNm at the deflection of 100 mm and 98 mm, respectively, which were 0.05 of coefficient of deviation in ultimate moment. As both FE models for the composite beams CBF1 and CBP1 were shown close agreement with experimental models, the FE models can be used for further parametric studies.

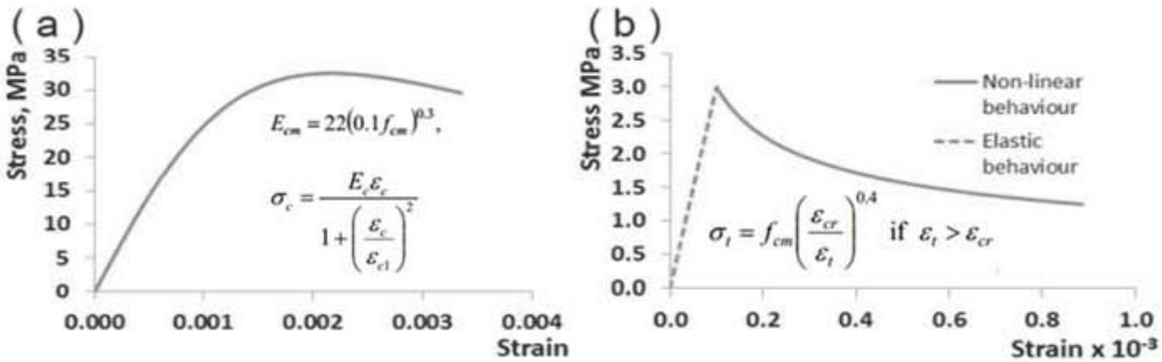

Figure 4: Concrete material property (a) Compressive behaviour (b) Tensile behavior

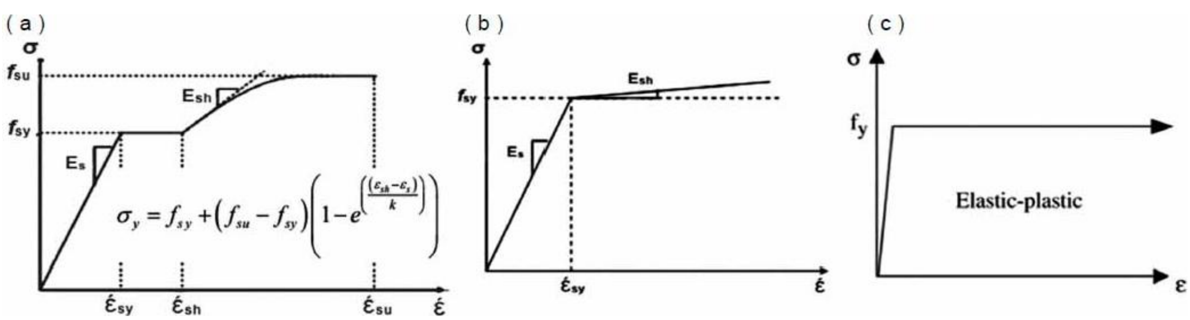

Figure 5: Material property (a) Steel beam (b) Reinforcing bars (c) Shear connectors

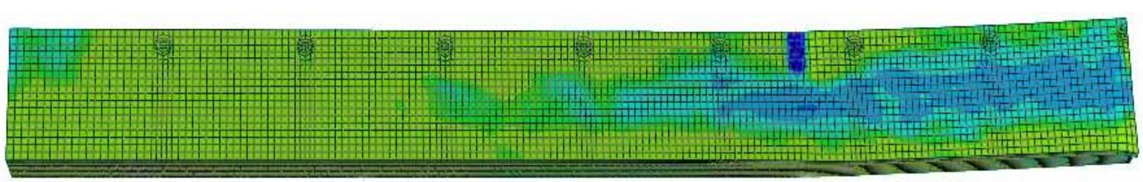

Figure 6: Failure mode - Concrete crushing at loading point of the composite beam with full shear connection

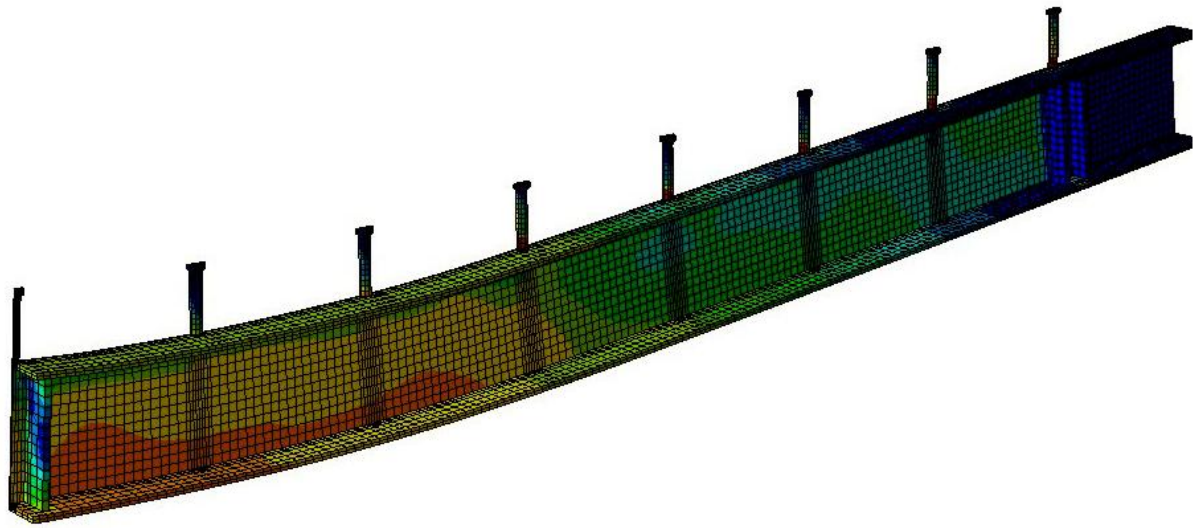

Figure 7: Failure mode – Shear connection failure of the composite beam with partial shear connection

4. Parametric Study and Disscussion

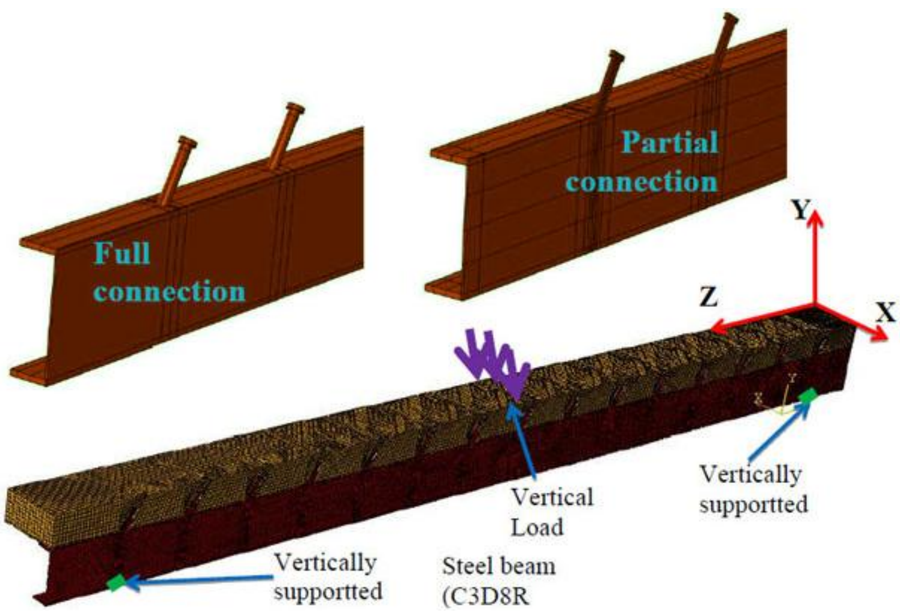

Figure 8: FE model of composite beam with inclined shear stud connectors subjected to full shear loading

The validated FE models were then included for parametric study such that the inclined shear stud connectors were applied instead of vertical shear stud connectors as shown in the Figure 8, while keeping the rest of the elements were same. In terms of the overall behavior, which was studied by Bavan et al (2016), while the similar responses of moment against deflection were predicted in the composite beam with full shear connection, an increase of 12.1% in the ultimate moment was predicted in the composite beam with partial shear connection as indicated in the Figure 9(a) and Figure 9(b), respectively. Shear stud connectors are a small mechanical device, which include a head and shank and the purpose of head is to avoid the separation in between the major material components of concrete slab and steel beam. In addition, the slip, which occurs due to the deflection of the composite beam, induces a gap in between the concrete slab and the shank of shear stud, and failure of the composite beam will be occurred due to the slip in between the concrete slab and steel beam. In terms of the separation and gap induced by slip, the resisting capacity of the shear flow forces, which is important in the determination of the capacity of the composite beam, is studied in this research study. It was observed throughout the analysis of the composite beam that the separation of the shear stud connectors and gap induced by slip occured either near the loading point or near supports. The nodes of the concrete and the shank of shear studs were monitored in the zone of gap induced by slip. In the elastic region of the shear stud component, there was no separation of gap in the composite beam with full and partial shear connection in both applications of vertical and inclined shear stud connectors . However, in plastic plateau, which was reached early in the application of vertical shear stud connectors rather than the application of inclined shear stud connectors in both cases of composite beam with full and partial shear connections. Figure 10 shows the separation behavior and gap induced due to the slip near the loading point of the composite beam with full shear connection and but, there is no gap induced while the separation initiated at the head of the stud in the application of the inclined stud of the composite beam with full shear connection, which is shown in the Figure 11.

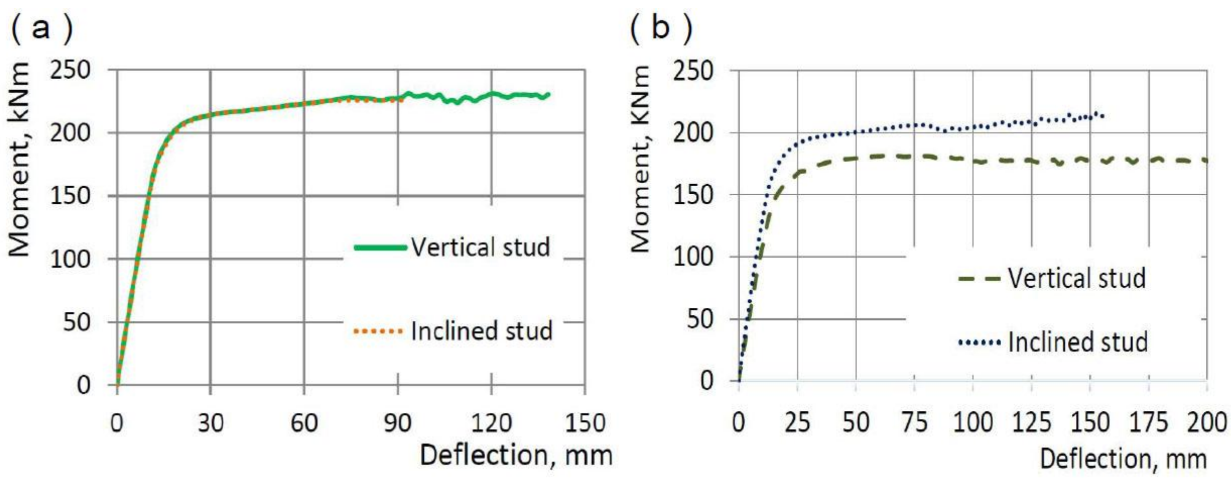

Figure 9: Moment deflection responses of the composite beam with inclined shear stud connectors (a) Full shear connection (b) Partial shear connection

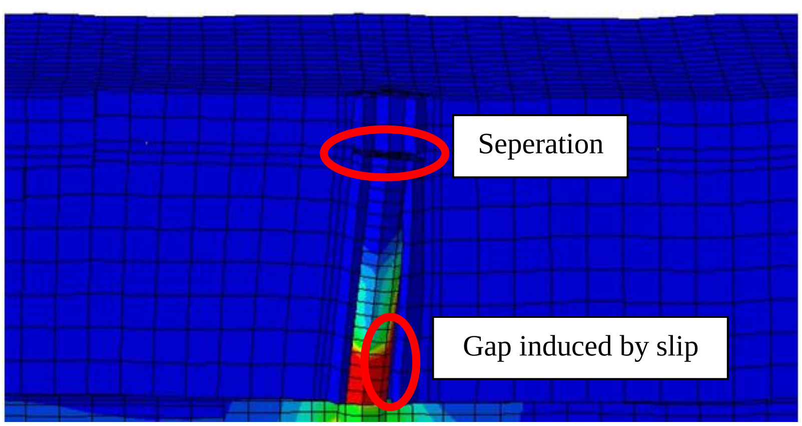

Figure 10: Separation and gap induced by slip at loading point of the composite beam with full shear connection in the application of vertical shear stud connectors

Meanwhile, in the application of vertical shear stud connectors to the composite beam with partial shear connection, the early separation of head of stud was predicted near to the supports with gap induced by slip as shown in the Figure 12. Figure 13 indicates the composite beam with partial shear connection in the application of inclined shear stud connectors and it was observed that there no gap induced while the separation was initiated in the similar condition where the failure occurred in the composite beam with partial shear connection in the application of vertical shear stud connectors. Indeed, the level of main failure zone occurred in the plastic plateau stage of connectors, where the critical flexural and shear stresses were resisted by the connectors, was reduced in the application of inclined shear stud connectors.

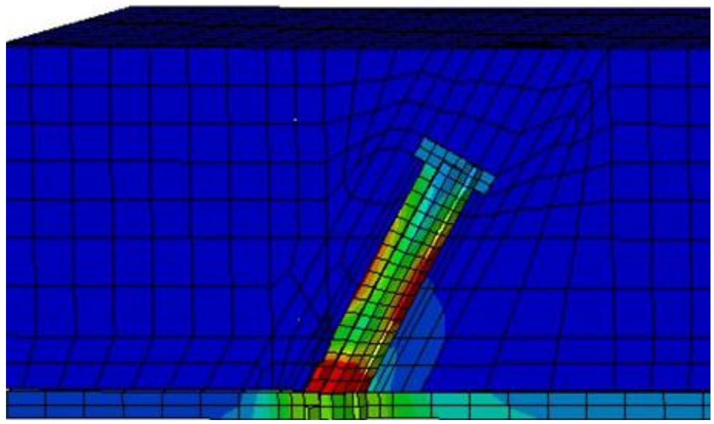

Figure 11: Loading point of the composite beam with full shear connection in the application of Inclined shear stud connectors

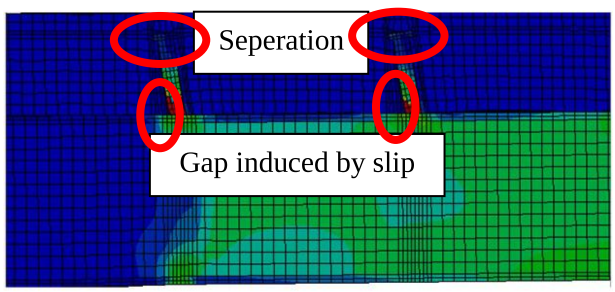

Figure 12: Seperation and gap induced by slip at loading point of the composite beam with partial shear connection in the application of vertical shear stud connectors

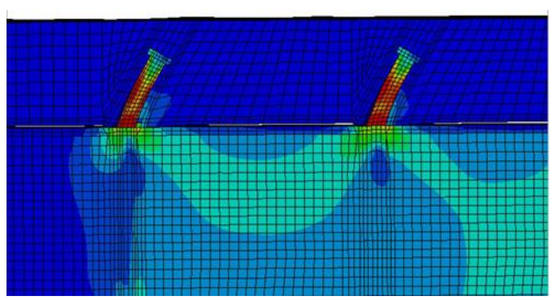

Figure 13: Loading point of the composite beam with partial shear connection in the application of Inclined shear stud connectors

5. Conclusions

This non linear FE model study was able to simulate the local behaviors of the steel concrete composite beam in the application of inclined shear stud connectors. The study concluded that when the application of inclined stud connectors, the internal stresses developed to failure are postponed. Moreover, the results showed that the separation in between the head of shear stud connector and concrete slab is delayed in the application of inclined shear stud connectors, which have to be additionally resisted the internal stresses and in such way, the ultimate bearing capacity of the composite beam is improved. In addition, the gap induced by slip is reduced in the application of inclined studs and in such way, the bearing zone is increased, which has to be resisted internal streeses additionally.

5. Acknowledgements

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SEISMIC VULNERABILITY ASSESSMENT AND RETROFITTING OF EXISTING REINFORCED CONCRETE BUILDING

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ABSTRACT:
The seismic assessment helps in identifying structural deficiencies related to lack of strength and ductility of both-certain elements or of the structural system as a whole. There are several methods of analyses that are primarily categorized into two groups-Elastic and Inelastic. A relatively simple nonlinear static procedure (pushover) is used for the seismic analysis of given structure. This method has the capability to provide passable information on seismic demands imposed by the design ground motion on the structural system and its different components. This paper focus on the detailed assessment and structural evaluation of seismic vulnerability of an existing office building located in seismic zone 2B (UBC97) as per ASCE-41 recommendations. The assessment has been carried out by review of the available reference documents related to seismic vulnerability accompanied by site visit, data collection, study of the detailed information of the different components of the structure like rebar percentage, detailing and actual strength of concrete, performance of computations corresponding to the modeled structure and interpretation of results. Moreover, reinforced cement concrete (RCC) jacketing of columns that has been deemed to be the most feasible and cost effective retrofitting technique is implemented to upgrade the seismic capacity of the structure. This has resulted in significant overall increase in the structural strength of the structure.

KEYWORDS:
ASCE-41, pushover analysis, deformation, performance point, capacity spectrum, retrofitting

INTRODUCTION

Performance based seismic approach is a technique used for realistic assessment and performance of primarily existing structures subjected to generalized or particular earthquake ground motion. While the existing code based approach does not account for many expected hazard and performance levels, the former has the capability to determine how the structure will perform under severe earthquake conditions.

A Tier-1 evaluation is conducted for the case study building in accordance with American Society of Civil Engineers document (ASCE-41-13, 2013) as applicable, of compliant/non-compliant statements related to structural, non-structural and foundation conditions as of Federal Emergency Management Agency, Washington, D.C document (FEMA 310, 1998). In this phase, quick checks are used to calculate the strength and stiffness of various building components. The intent is to identify the building type and the structural performance level as described in Table C2-4, ASCE-41-13.

Tier-2 analysis represents a rough approximation of the non-linear behavior of the actual structure and ignores numerous non-linear effects (FEMA 310, 1998). The linear static procedure is based on the linearly-elastic stiffness and equivalent viscous damping that is an approximation below yield point.

In Tier-3 phase, nonlinear static analysis is performed through which strength and stiffness deterioration associated with inelastic material and large displacements are easily worked out. The plot of the base shear as a function of the roof displacement is the “pushover curve” of the structure. The demand curve is also plotted between ground acceleration and time period of the structure that depends on the seismic zone and soil type. The intersection of the capacity (pushover) curve with the demand spectrum curve provides the performance or response point, which is the measure of expected damage in the building.

If the structure is found to be seismically vulnerable upon completion of the non-linear static analysis, various retrofitting strategies are then proposed for improving the seismic performance of the structure.

CASE STUDY DETAILS

Figure 1 shows the newly constructed six story office building under consideration for assessment. The building’s overall dimension is 155’ long by 60’ wide and is approximately 103’-9’’ tall with a raft foundation. The soil type is Sc i.e very dense soil and soft rock whereas seismic zone is taken as 2B according to the uniform building code UBC-97. The building type is found to be C3(a)- Concrete moment frame with shear walls and stiff diaphragm, according to ASCE-41-13. Since the structure is composed of very less amount of infill walls, therefore these are not considered in analyses.

Figure 1.Elevation of Building

SCREENING PHASE

In this phase of the seismic assessment, a site visit is carried out of the structure which is supplemented by a checklist. The checks depend upon a number of parameters that are- type of building, level of performance, level of seismicity and soil profile type/site class. The checklists are further categorized majorly into three groups-structural, geological site hazard and foundation and nonstructural.

The checklist based evaluation of the structure is carried out as per ASCE-41-03 Tier-1 recommendations. Through visual inspection, using engineering judgment and performing rough calculations to estimate the seismic forces resisted by primary lateral force resisting components, a number of deficiencies (non-compliant items) were found in the structure. These are summarized in the table 1

Checklist	Non-Compliant Items
Building System	Soft Story
	Captive Column
	Deterioration of Concrete
	Weak Story
	Torsional Irregularity
Lateral Force Resisting System	Proportion of Infill Walls
	Overall Construction Quality
	Strong Column/Weak Beam
Connection	Wall Anchorage
	Deflection Compatibility
Geological Hazards and Foundation	Overturning

Table 1.Summary of Tier 1 Assessment

LINEAR EVALUATION PHASE

A 3-D model of the building is developed in ETABS 2015 software. The beams and columns are modeled using linear elements whereas infill and reinforced concrete (shear) walls are modeled as membrane area elements. Table 2 shows the input parameters for the model to perform analysis. The Linear elastic analysis is performed which represents a rough approximation of the non-linear behavior of the structure and ignores redistribution of forces and other non-linear effects. It is based upon the linearly-elastic stiffness and equivalent viscous damping which is an approximation below yield point. Figure 2 shows the 3-D model of the building. There were some checks that were assumed to be non-compliant using visual observation and engineering judgment after tier-1 (screening phase) assessment but these are now accurately checked and calculated using results of linear static analysis. The items still found persistent to be deficient from Tier-1 phase include torsion irregularity, soft story and weak story. Moreover, the linear analysis results indicate that there are certain beams and columns with demand/capacity ratio (DCRs) greater than one, so the structure is expected to respond beyond the linear static range. The beams with DCRs greater than unity are due to combined torsion and shear effects. These are local failures and will not affect the overall building stability. Therefore, the critical vulnerable structural members are the columns as they are over stressed

Dead Load	Self-Weight
	3” thick finishes load
	8” thick slab
	12” thick Infill and Shear wall
	900 lb/ft wall load
	50 psf for office area

Live Load	100 psf for ramp,parking and storage places
Earthquake Load	
Z	0.20
C_a	0.24
Cv	0.32
Soil Type	S_c

Table 2.ETABS 2015 Input Parameters

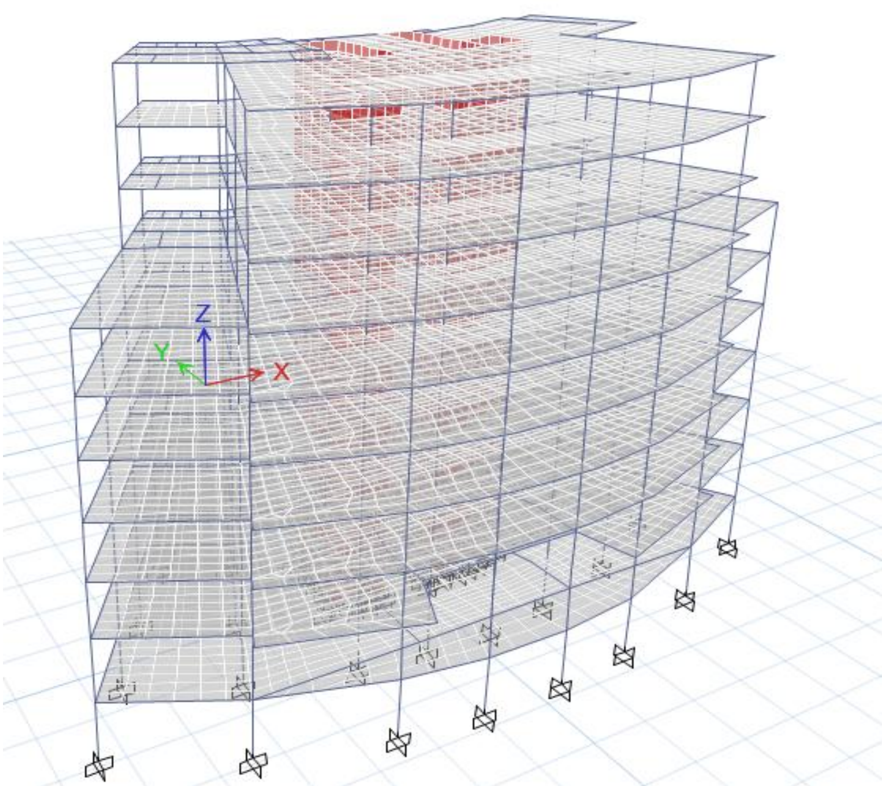

Figure 2.3-D FEM Model on ETABS

DETAILED NON LINEAR STATIC ASSESSMENT

The results of the linear static analysis of this building indicate that many columns have DCR > 1. This specifies that the building is expected to respond in the inelastic range. In detailed evaluation phase, pushover analysis according to ATC-40 and FEMA-440 criteria is adopted. The building model is now modified by providing discrete plastic hinge elements (limped plasticity model) as calculated from Response 2000 section analysis software at locations expected to experience nonlinear behavior, like column and beam ends. These locations are critical as maximum moments form here. Table 3 gives the geometric and material properties used in the model. ETABS 2015 software is used to generate the load-deformation or pushover curve which is plotted between base shear and roof displacement. It shows the incremental increase in the magnitude of structural loading in accordance with the pre-defined pattern (restart using secant stiffness for member unloading and P-Delta effects for geometric nonlinearity) as shown in Figure 3

Geometric Properties	
Beam	Width= 12”
	Depth= 24”
Column	Width= 24”
	Depth= 24”
	Length= 12’
Material Properties	fc’= 3000 psi for column and beam
	E_{con}= 3144 ksi for column and beam

Table 3.Properties of Nonlinear Model

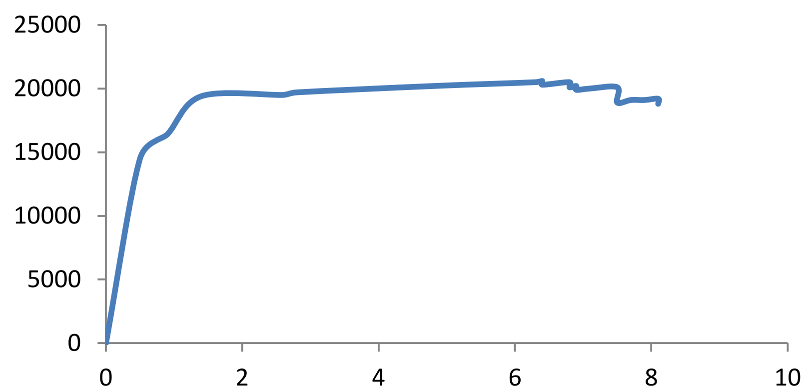

Figure 3.Pushover Curve In Major Direction

The pushover curve which is a measure of the building’s capacity is converted into capacity spectrum and compared to estimated demand as per Uniform building code document (ICB0, 1997). The point where they intersect each other is called the performance point. The fundamental requisite of the non-linear inelastic assessment is the magnitude of the target displacement at which seismic performance evaluation is performed. A life safety performance criterion is selected for the study building, because it is an office building and such buildings are typically evaluated for life safety. Figure 4 shows how the force-deformation relation for hinges are defined in FEMA 356 document where IO, LS and CP is Immediate

Occupancy, Life Safety and Collapse Prevention respectively, representing the acceptance criteria and performance states

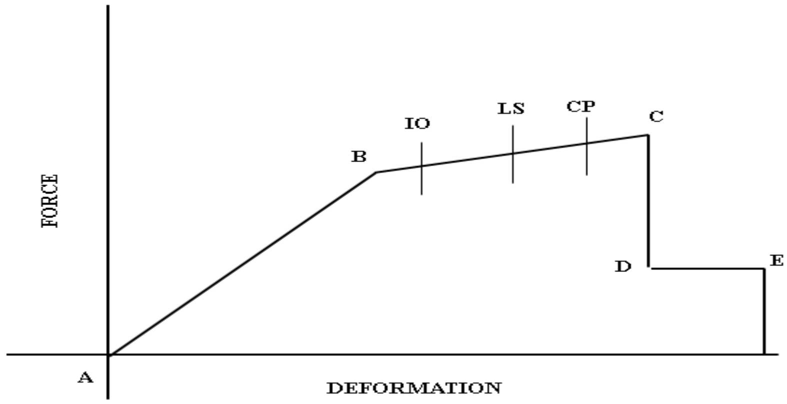

Figure 4.Back Bone Curve for Hinges (Reprinted from Public Domain Document, FEMA 356)

The model which is incorporating inelastic material response under non-linear static procedure is displaced to the target displacement and resulting internal forces and deformations are determined. The mathematical model of the structure is subjected to monotonically increasing lateral forces or displacements until either a target displacement is exceeded, or the building collapses. Using the target displacement, the performance point is achieved and the status of the building is observed. Figure 5 below shows the stresses on columns of the structure at its performance point. The status of the building does not conform to the life safety (LS) level of performance as weaknesses in the critical structural members of the structure are mathematically noticeable. There are 16 plastic hinge locations throughout the structure, all in columns, where the rotations have exceeded the life safety performance level. The red color circled columns have Demand/Capacity ratio much greater than unity and are identified to be significantly overstressed. Hence they are critical and need to be strengthened, whereas the blue color circled columns are slightly overstressed and can be ignored. This indicates that the energy dissipation and plastic rotations are unevenly distributed throughout the structure and the building is behaving poorly even in the non-linear range. Thus, it is evident that retrofitting needs to be done to overcome the weak strength of the structure.

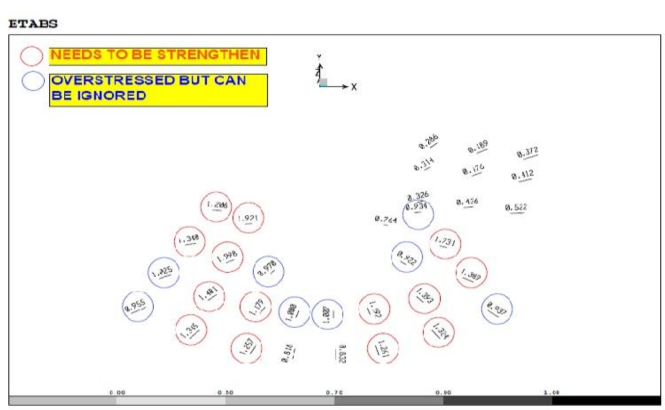

Figure 5.Demand/ Capacity Ratio of 1st Floor Columns before Retrofitting

RETROFIT SOLUTION

Conceptual Resolutions Considered

Initially, there are several solutions that are considered for retrofitting but since the building presents numerous challenges, therefore they must be well thought of. The first one is mass reduction. This can be achieved in several ways.

- Replacing the infill walls composed of solid blocks by hollow blocks
- Decreasing superimposed sustained dead loads on the structure like furniture etc.
- Removing story(s) from the structure

These all are not feasible because the building is a newly constructed office building. The replacement materials in walls will disturb the finishes whereas adequate amount of furniture is a basic requirement in these types of buildings and the owner is not willing to decrease any story from the structure. The project study case participants then considered addition of walls in the ground story but these caused failures to occur in story immediately above. So the walls need to extend into the upper stories making it highly time and cost consuming. Another option is to provide reinforced concrete (RC) shear walls but because of the associated foundation work, it is deemed highly expensive. Another choice is to provide steel bracing extending from one end of a column up to another end of a different column. The problem with this solution is that it will reduce the size of the rooms besides affecting the overall aesthetics of the structure. The authors also examined the use of steel and RC jacketing of columns as a retrofitting method, with the latter being finally recommended as the retrofitting technique to be implemented after careful and comparative analysis of all. The details of these are explained in the following sections.

Comparative Analysis between Use of Steel and RC Jacketing

There were many different options considered as described in the preceding section but ultimately the jacketing of columns is considered to be the best option as per the structure requirements. At the same time, many different versions of this type of solution exist and the inelastic analyses of all these types provide satisfactory results in improving the strength of the structure up to the required performance level i.e. Life Safety. Hence, further investigation needs to be done in order to select the best alternative. Figure 6 shows the comparison between software generated retrofitted models of the basic type of jacketing of columns. It is worthy to mention here that both pass the design check for failure so the pivotal aspect now is the cost of implementation.

Figure 6.Reinforced concrete and steel jacketing of a typical overstressed column.

<table>
<tr><th colspan="2">RC Jacketing</th><th>Steel Jacketing</th></tr>
<tr><td rowspan="4">Main Bar</td><td>Dia=1”</td><td rowspan="7">Using A-36 Steel Plate
Plate Thickness = 1”
Unit Weight of Steel= 490lb/ft^3
Cost= Rs 133394</td></tr>
<tr><td>Quantity=12</td></tr>
<tr><td>Length=10.5’</td></tr>
<tr><td>Cost=Rs 12825</td></tr>
<tr><td rowspan="4">Ties</td><td>Dia=0.375”</td></tr>
<tr><td>Quantity=21</td></tr>
<tr><td>Length=10’</td></tr>
<tr><td>Cost=Rs 3000</td><td rowspan="6">Labor + Others
Cost= Rs 16606</td></tr>
<tr><td rowspan="2">Concreting</td><td>Volume=0.07cum</td></tr>
<tr><td>Cost=Rs 700</td></tr>
<tr><td rowspan="2">Shear Connectors
(Drilling + Chemical)</td><td>Numbers of Drills=160</td></tr>
<tr><td>Cost=Rs 40000</td></tr>
<tr><td>Labor + Others</td><td>Cost=Rs 3475</td></tr>
<tr><td>Total Cost</td><td>Rs 60000</td><td>Rs 150000</td></tr>
</table>

Table 4.Cost Comparison between RC and Steel Jacketing of a typical Column

Implemented Retrofit Solution

The reinforced concrete jacketing of columns is eventually selected as the retrofitting technique to be executed for the case study structure. Initially, scanning is done to exactly locate the primary reinforcement as well as the ties in the existing column. Then, enlargement of the existing overstressed column is done by placing steel rebars around its periphery and then pouring high compressive strength concrete over it. The strength of the concrete poured must be greater than that of the existing column. The shear load connectors are used to transfer the load and resist any movement in the process. The holes for these load connectors are drilled and the necessary chemical is applied in the concrete in vacant spaces as specified from the scanning results. Figure 7 shows the column being jacketed.

Figure 7.RC Jacketing of Columns Being Executed On Site

The implementation of this method has significantly increased the member sizes, thereby its stiffness too. It has enhanced the confinement along with the shear and axial behavior increase in the columns. The columns with DCR greatest are retrofitted first and engineering judgment is needed to decide the number of columns to be retrofitted considering the location of columns with respect to load distribution. Figure 8 shows the up gradation of the capacity of the critical structural members that were previously overstressed.

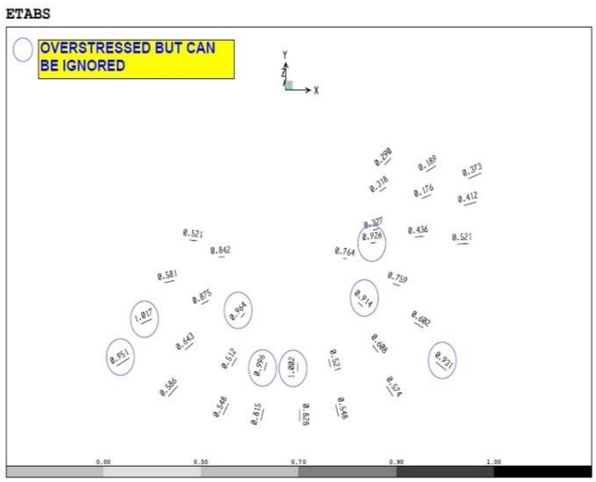

Figure 8.Demand/ Capacity Ratio of 1st Floor Columns after Retrofitting

CONCLUSION

Performance-based seismic vulnerability assessment of existing structures greatly aids in determining strength and stiffness deterioration associated with inelastic materials. The structures found deficient as a result of the inelastic analysis can then be retrofitted using different techniques. The reinforced concrete jacketing strengthening method assists in considerable amount of uniformly distributed increase in strength and stiffness of columns. In contrast to the steel jacketing technique where fire protection and corrosion resistance is required for exposed steel, this method results in the durability of the existing column without such measures. Unlike mass reduction, provision of RC shear walls or steel bracing systems that are uneconomical or impractical, this technique does not pose such issues either. Hence, of all the retrofitting techniques considered for the case study structure, RC jacketing has been found to be the most practicable, feasible and cost effective method and the same has been implemented. This has resulted in the improvement of the seismic capacity of the structure up to the required performance level.

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Estimation of Modal Shift for Bus Rapid Transit System (BRTS) in Quetta

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Abstract

This study is concerned with the perception and credence of commuters towards the proposed Bus rapid transit system. Study carried out to identify how commuters observe and value the offered BRTS and its attributes in view point of offering accessible and excellence service. Experiment encompassed paper questionnaire including revealed preference (RP) and stated preference (SP) exercise. A choice models was developed to capture the present demand of existing modes and future demand of BRTS. The present study employs an extensive variety of modelling approaches and inspects a series of traditional and advanced estimation and calibration techniques including multinomial logit model, mixed logit model and approach of combining RP/SP methods to incorporate advantages of both data sets for modal shift. The study confirms that on existing data commuters give preference to motorcycle and bus, whereas with the introduction of BRTS, commuters become very sensitive for their comfort, and willing to shift towards new service with up to 39% of mode share. Policy sensitivity analysis shows that to increase the ridership of BRTS the travel time is the key factor.

Keywords

Bus Rapid Transit System, Stated preference, Mixed Logit Model, Ridership, Policy sensitivity

1. Introduction

Urban form and urban transport system have an enormous impact on the way people travel. For many towns where effective public transport has been neglected, leaving mobility needs exclusively in the hands of private vehicles. Due to Rapid growing economics and population typically seen in developing countries, there is an increase trend of an urban sprawl and auto-mobilization. This has a direct effect on the level of transport demand and travel pattern. In the lack of appropriate implementation of planning measures, it also leads to additional cost for transportation infrastructure and its operation, while at the same time creating many environmental, economical, and social problems (Patankar et al, 2007)
Today, Bus Rapid Transit System (BRTS) is amongst the most effective solution in providing great quality transit facility in a cost effective basis to urban zones both in developed and developing countries. BRT is totally environmental friendly transport system and is effective in different cities of the world.

BRT is a service, which is providing, comfortable and cost effective mobility (Bus Rapid Planning Guide, 2007). An understanding of the attitude and behavior of travelers is the essential for the formation of the efficient transport system.(Vedagiri et al) BRT system is general built on routes where varied traffic congestion is already a problem, or where congestion is likely to happen in the near future, (Hossain 2006). Bus is the main transport system used in Pakistan and regularly its level of service is falling due to insufficient capacity and management problems. In absence of adequate and effective bus transit service, the possible bus users now are using private transport such as Cars, Motor bikes, Rickshaws.

2. Study Area and Existing Transportation

The City of Quetta is situated at the 66°41'40"-67°17'25" East longitudes and 30°01'29"-30°28'25" North latitudes at height of 1676.9 meters from sea level, bowl shaped, valley bounded by mountains having a total area of 2653 sq. km. Quetta is the largest district and the most populous city of Province Baluchistan and district population is 759,941 and estimating population for 2018 is more than 3 million the population annual growth rate is 4.13% (census 1998). Public transportation in City, presently consists of rickshaws and buses. Beside this transportation modes private modes user also contribute a lot in system. In city motorcycle share is more compare to other modes as it is cheap in time and cost

A corridor between uptown (Hazar ganji) to downtown central Business District (CBD) which is connected by Sariab Road and Zarghoon Road was selected as the study area, the length of selector corridor is about 16.6km. This corridor was selected because it is the main zone of city which mainly connect to the CBD and has congestion problem specifically in peak hour of day. The map of proposed corridor is presented in figure 1and the whole commuters who may use this corridor is divided into seven family monthly income groups as shown in Figure 2

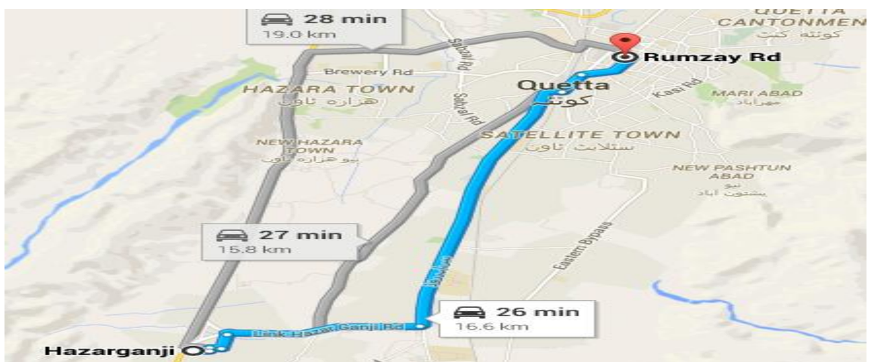

Figure 1. Proposed BRTS route

From survey it reveals that bike is the dominated mode of respondents, it is because mostly student who are sensitive to time and cost used bike as their first option. While bus on the other hand has also well enough percentage the peoples going for work and mainly students who do not afford bike use bus as their mode as shown in figure 3. Public transport is the predominantly by the mode captive and students for mandatory trips and peoples are concerned with cost and punctual service of the public transport (Mahale, 2010).

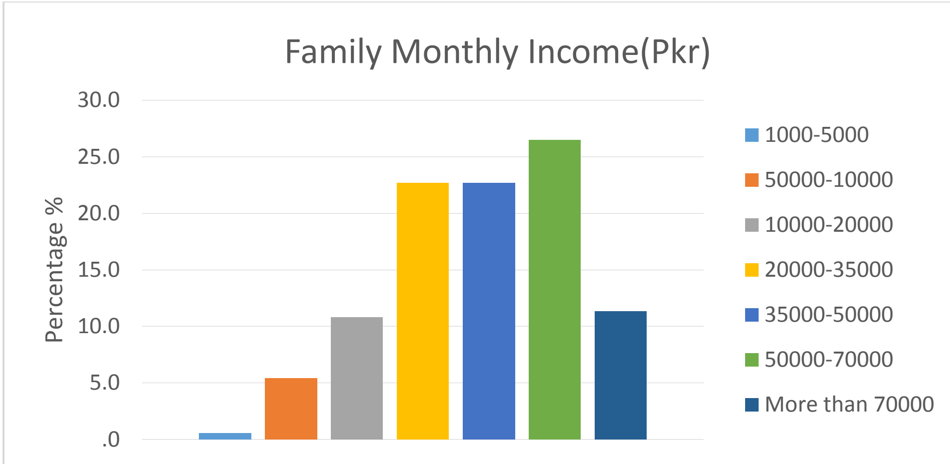

Figure 2. Family Monthly Income

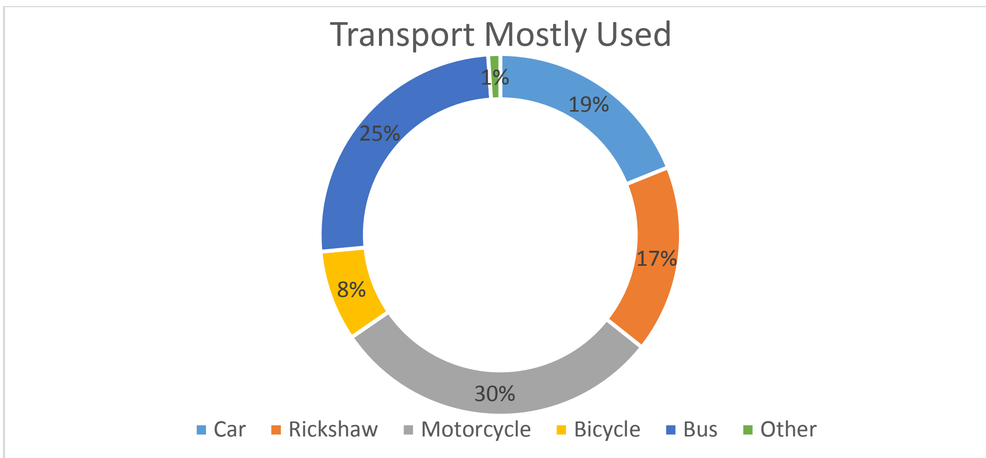

Figure 3. Transport Mostly Used

3. Data Collection and Methodology

The questionnaire developed in this study has divided into four parts. The very first part of questionnaire gathered the RP data for recently trip made by respondents. In second part consist of SP exercise of hypothetical scenarios of newly implemented Bus Rapid Transit System and the third and last part includes general questions traveler's characteristics respectively. The random sampling technique has been used to fill out 200 paper based questionnaires along the selected corridor. The methodology followed for the model development and its calibration is based on the combine revealed preference (RP) and stated preference (SP) survey for commuter's present modes preference and future for shifting on proposed BRTS, using RP and SP approach. In this Study Multinomial Logit Model (MNL), for RP and SP data were calibrated from the data using package Biogeme 2.2, whereas out of these models only selected best model are estimated as mixed logit and combine model for RP and SP data. The conceptual framework for model development is shown in Figure 4.

4. Non SP Content

In RP part of the survey the data was gathered for last previous trip on public transport (Bus) made by the respondents. An account of the trip was obtained by asking numerous questions including the transport mostly used, purpose of visit, worst and best aspects of transport, respondents was also asked for the distance for trip and the estimation of cost and time of their modes. The questionnaire also included general questions regarding safety, comfort, and respondent's personal information and characteristics.

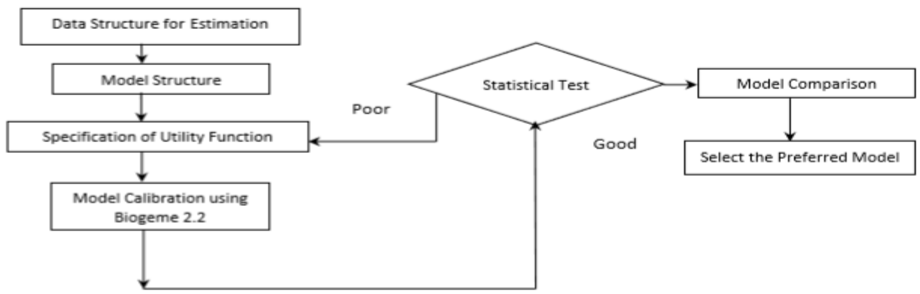

Figure 4. Conceptual framework for model development

5. SP Content

The Bus rapid transit system is true modernization in Quetta city, it is not suitable in this case to only forecast the influence of observations of existing services. However it is essential to obtain data from survey of hypothetical scenarios which comprises its attributes regarding BRTS to evaluate its influence on travel behavior as shown in table 1. A fractional factorial design was used in SP exercise to evaluate the scenarios and every respondents get 8 binary choices for BRTS1 and BRTS2 in SP exercise.

Table 1. Attributes and Attribute levels

Attributes	Attribute levels
Travel cost	20Rs, 25Rs, 30Rs
Travel Time	20min, 30min, 40min
Seats	Confirm, Not Confirm
Headway	03min, 05min. 07min
Walk time to bus stop	05min, 10min
AC/Heater	Available, Not Available

6. Multinomial Logit(MNL) Model on RP

Multinomial models of mode choice were calibrated using the RP data, the models were formulated with minimal specification, which comprises those variables considered as important for any sensible model with dependent variables being the car, bike, rickshaw and bus, while base case in this model is rickshaw.

The utility functions are as given by

1. **U(Car)** = ASC_Car * *one* + BETA_TC * *TC_Car* + BETA_TT * *TT_Car* + Male_C * *DMale* + Student_C * *DStdnt* + edu_C * *DEdu* + Govtemp_C * *DGempy* + AgeSnr_C * *Agesnr* + IncomeG2_C * *DIncomeG2* + PurtrE_C * *DPurE* + PurtrS_C * *DPurS* + PurtrW_C * *DPurW* + ImpSeats_C * *DImpSeats* + Comfort_C * *DImpCmfrt*+ Safety_C * *DSafety* + OptmjrnyT_C * *DOptmzjrny* + MinC_C * *DMnzCost* (1)

2. **U(Rickshaw)** = ASC_Rick * *one* + BETA_TC * *TC_Rick* + BETA_TT * *TT_Rick* *(2)*

3. **U(Bike)** = ASC_Bike * *one* + BETA_TC * *TC_Bike* + BETA_TT * *TT_Bike* + Male_Bi * *DMale* + Student_Bi * *DStdnt* + edu_Bi * *DEdu* + Govtemp_Bi * *DGempy* + AgeSnr_Bi * *Agesnr* + IncomeG2_Bi * *DIncomeG2* + PurtrE_Bi * *DPurE* + PurtrS_Bi * *DPurS* + PurtrW_Bi * *DPurW* + ImpSeats_Bi * *DImpSeats* + Comfort_Bi * *DImpCmfrt* + Safety_Bi * *DSafety* + OptmjrnyT_Bi * *DOptmzjrny* + MinC_Bi * *DMnzCost* *(3)*

4. **U(Bus)** = ASC_Bus * *one* + BETA_TC * *TC_Bus* + BETA_TT * *TT_Bus* + WT_B * *Wtng* + Male_B * *DMale* + Student_B * *DStdnt* + edu_B * *DEdu* + Govtemp_B * *DGempy* + AgeSnr_B * *Agesnr* + IncomeG2_B * *DIncomeG2* + PurtrE_B * *DPurE* + PurtrS_B * *DPurS* + PurtrW_B * *DPurW* + ImpSeats_B * *DImpSeats* + Comfort_B * *DImpCmfrt* + Safety_B * *DSafety* + OptmjrnyT_B * *DOptmzjrny* + MinC_B * *DMnzCost* *(4)*

The models results are presented in Table 2 whereas details of variables and coefficients used in all models are defined Annexure-1.The model constants or alternative specific constants were defined comparative to Rickshaw. The positive sign for car, bike and for bus implies that commuters are likely to go by these modes. This propose that commuters reveal an inclination to travel by these modes as this looks reasonable that there is no other competitive modes to show any aversion from existing modes. As normal, for travel time and cost in all models are negative which shows commuters favor to select that mode which offers the lowest travel time and cost. Negative sign income implies that higher income travelers naturally aversion to by bus, the positive sign for age which employs a senior citizen likely to go by car. Gender also effect as the share of modes, for males as they use all modes so on all modes the signs are positive.

7. Mixed Multinomial Logit(MMNL) Model on RP and Comparison

Mixed logit is a very flexible model that can approximate any random utility model. It resolves the three restrictions of standard logit by letting for random taste difference, unrestricted substitution patterns, and correlation in unobserved aspects over time (Kenneth train 2002).

The model utility function is same as of MNL but has one prominent difference that normally distributed coefficient specified for travel time to explain additional variation of the model. The utility functions are as given by.

1. **U(Car)** = ASC_Car * *one* + BETA_TC * *TC_Car* + BETA_TT [SIGMA] * *TT_Car* + Male_C * *DMale* + Student_C * *DStdnt* + edu_C * *DEdu* + Govtemp_C * *DGempy* + AgeSnr_C * *Agesnr* + ncomeG2_C * *DIncomeG2* + PurtrE_C * *DPurE* + Comfort_C * *DImpCmfrt* (5)

2. **U(Rickshaw)** = ASC_Rick * *one* + BETA_TC * *TC_Rick* + BETA_TT * *TT_Rick* *(6)*

3. **U(Bike)** = ASC_Bike * *one* + BETA_TC * *TC_Bike* + BETA_TT [SIGMA] * *TT_Bike* + Male_Bi * *DMale* + Student_Bi * *DStdnt* + edu_Bi * *DEdu* + Govtemp_Bi * *DGempy* + AgeSnr_Bi * *Agesnr* + IncomeG2_Bi * *DIncomeG2* + PurtrE_Bi * *DPurE* + Comfort_Bi * *DImpCmfrt* (7)

4. **U(Bus)** = ASC_Bus * *one* + BETA_TC * *TC_Bus* + BETA_TT [SIGMA] * *TT_Bus* + WT_B * *Wtng* + Male_B * *DMale* + Student_B * *DStdnt* + edu_B * *DEdu* + Govtemp_B * *DGempy* + AgeSnr_B * *Agesnr* + IncomeG2_B * *DIncomeG2* + PurtrE_B * *DPurE* +

Comfort_B * *DImpCmfrt*
(8)

The models results are presented in Table 2**,** and mixed logit model calibration improved the efficiency of the model with better t- statistics. The addition of the error section term enhanced the fit to the data. Normally distributed component are significant in standard deviation.

Table 2. Comparison RP Multinomial and RP Mixed Multinomial

File	MNL	MMNL
Number of Observations	170	170
Estimated Parameters	30	31
Converged	TRUE	TRUE
Rho^2	0.207	**0.213**
Rho^2(Adjusted)	0.08	**0.082**
Final log (L)	-186.837	-185.461
ASC_Bike	1.39(1.34)	1.37(1.27)
ASC_Bus	1.11(1.16)	**1.22(1.18)**
ASC_Car	2.46(2.45)	2.65(2.40)
ASC_Rick	Fixed	Fixed
BETA_TC	-0.00944(-3.04)	-0.00993(-2.91)
BETA_TT	-0.0139(-1.29)	
Mean of BETA_TT		**-0.0142(-1.31)**
S.D of BETA_TT		**-0.0302(-1.57)**
Male_C	1.39(2.36)	1.40(2.22)
Male_Bi	3.15(4.67)	2.21(4.63)
Male_B	1.30(2.44)	1.34(2.29)
WT_B	-0.0508(-1.37)	**-0.0523(-1.39)**
Student_C	-2.57(-2.36)	-2.63(-2.27)
Student_Bi	1.23(1.35)	**1.33(1.39)**
Student_B	0.930(1.04)	0.948(0.96)
edu_C	1.21(1.80)	**1.43(1.89)**
edu_Bi	0.106(0.19)	0.0232(0.04)
edu_B	-0.458(-0.82)	-0.420(-0.69)
Govtemp_C	0.0969(1.36)	**0.831(1.38)**
Govtemp_Bi	-0.0134(-0.02)	-0.0452(-0.06)
Govtemp_B	-0.379(-0.56)	-0.338(-0.38)
AgeSnr_C	0.547(0.81)	0.591(0.80)
AgeSnr_Bi	-1.26(-1.77)	**-1.36(-1.81)**
AgeSnr_B	-0.255 (-0.37)	**-0.661**
Comfort_C	0.509(0.87)	0.549(0.87)
Comfort_Bi	0.336(0.61)	**0.365(0.64)**
Comfort_B	-0.972(-1.82)	-1.15(-1.92)
IncomeG2_C	1.62(2.38)	**1.77(2.39)**
IncomeG2_Bi	0.751(1.33)	0.801(1.38)
IncomeG2_B	-0.540(-0.97)	-0.557(-0.93)
PurtrE_C	0.640(0.62)	**0.690(0.63)**
PurtrE_Bi	1.12(1.36)	**1.20(1.40)**
PurtrE_B	1.03(1.26)	1.12(1.24)

8. Multinomial Logit Model on SP

A model was developed where a respondents had to choose two alternative set of attributes. The MNL model was employed to estimate the preference of commuters towards BRTS and its attributes, Variables added to the model from simpler to complex incrementally dependent variables are BRTS1 and BRTS2 and base model in this case is BRTS1. The utility functions are given as

1. **U (BRTS1)** =ASC_BRTS1 * *one* + TC * *X11* + TT * *X12* + WT * *X15* *(9)*

2. **U(BRTS2)** =ASC_BRTS2 * *one* + TC * *X21* + TT * *X22* + WT * *X25* + HD * *X24* + AC2 * *X26* + SE2 * *X23* + Safety * *DSafety* + Male * *DMale* + Female * *DFMale* + PurtrS * *DPurS* + PurtrE * *DPurE* + PurtrW * *DPurW* + ImpSeats * *DImpSeats* + IncomeG2 * *DIncomeG2* + ImpCmfrt * *DImpCmfrt* + Agesnr * *AgeSnr* + *Gempy* * *DGempy* + Stdnt * *DStdnt* + ChngMds * *DChngMds* + unedu * *Dunedu* + Edu * *DEdu* + Optmzjrny * *DOptmzjrny* + Bsns * *DBsns* + MnzCost * *DMnzCost* *(10)*

The models results are presented in Table 3. More complex models had lower *t- statistics* less than simpler models. The *t-statistics* of some variables are not fair. Travel time and travel cost of all models were negative as expected. As some other attributes which were added to model are Headway and waiting time has negative Sign, while headway implies positive sign that commuters are sensitive to headway.AC/Heater and seats availability in BRTS, has also impact on commuter's preference and positive sign and higher t-statistics reveals that respondents are keen for their comfort in daily trips. Both male and female show preference towards BRTS accordance with different attributes of service which were given in choice set, governmental employee, illiterate and students show preference and has positive sign and show preference to travel by BRTS. Higher income commuters show natural aversion as they feel more comfortable in their own cars.

9. Mixed Multinomial Logit Model on SP, Panel Data and Comparison

Model which was finalized after comparison and with different tests was taken for the calibration on mixed logit model on SP data, normally distributed coefficient is specified with travel time. Secondly Panel multinomial mixed logit model is another model to identify to vary across individual to cope repeated measurement. The model utility function is same as of MNL but has prominent difference that normally distributed coefficient specified for travel time to explain additional variation of the model and have specified with another coefficient ZERO [SIGMA_BRTS] * *one* to cope with repeated measurement. The utility functions are as follow.

1. **U (BRTS1)** **=**ASC_BRTS1 * *one* + TC * *X11* + TT [SIGMA1] * *X12* + ZERO [SIGMA_BRTS1] * *one* *(11)*

2. **U(BRTS2)** =ASC_BRTS2 * *one* + TC * *X21* + TT [SIGMA1] * *X22* + WT * *X25* + HD * *X24* + AC2 * *X26* + SE2 * *X23* + Male * *DMale* + Female * *DFMale* + PurtrS * *DPurS* + PurtrE * *DPurE* + PurtrW * *DPurW* + ImpSeats * *DImpSeats* + IncomeG2 * *DIncomeG2* + ImpCmfrt * *DImpCmfrt* + Agesnr * *AgeSnr* + Safety * *DSafety* + ZERO [SIGMA_BRTS1] * *one* *(12)*

The models results are presented in Table 3 whereas details of variables and coefficients used in all models are defined in Annexure-1. The mixed logit model calibration not improved the efficiency of the model as expected but panel approach for the repeated measurement has improved the efficiency of model with better t-statistics values. The addition of the error section term enhanced the fit to the data. Normally distributed component are significant in standard deviation for panel approach, furthermore the t- statistics of travel time improved significant.

Table 3.Comparison SP Multinomial and SP Mixed Multinomial and Panel

File	MNL_SP	MMNL_SP	MMNL Panel
Number of observations	1480	1480	1480
Number of Individuals	1480	1480	185
Estimated Parameters	17	18	18
Converged	TRUE	TRUE	TRUE
Rho^2(ρ)	0.166	0.166	0.167
Rho^2(Adjusted)	0.15	0.149	0.148
Final log (L)	-855.41	-855.346	-854.19
ASC_BRTS2	1.41(2.52)	0.474(2.96)	1.40(2.54)
AC2	0.785(4.48)	0.792(4.40)	0.803(4.57)
HD	0.0663(1.11)	0.0676(1.14)	0.0729(1.22)
Male	0.0365(0.15)	0.0416(0.18)	0.0487(0.19)
SE2	1.71(14.18)	1.72(12.83)	-1.72(-14.20)
Safety	0.0140(0.12)	0.0162(0.13)	0.0185(0.15)
TC	-1.63e-013(-0.00)	9.46e-016(0.00)	-3.22(-0.00)
TT	-0.0483(-8.09)		
Mean of TT		**-0.0487(-7.71)**	**-0.0485(-8.10)**
S.D of TT		**-0.0189(-0.37)**	**-0.00903(-1.46)**
WT	-0.141(-5.91)	-0.142(-5.96)	-0.142(-5.94)

Female	0.132(0.52)	0.134(0.58)	0.147(0.58)
PurtrS	0.00518(0.03)	0.0048(0.02)	0.00387(0.02)
Agesnr	-0.0144(-0.09)	-0.0181(-0.11)	-0.0168(-0.10)
ImpCmfrt	0.00541(0.04)	0.00748(0.07)	0.0138(0.11)
ImpSeats	-0.307(-1.33)	-0.313(-2.09)	-0.313(-1.35)
IncomeG2	-0.00548(-0.04)	-0.00721(-0.06)	-0.0133(-0.10)
PurtrW	0.112(0.59)	0.114(0.62)	0.106(0.56)
PurtrE	-0.0755(-0.38)	-0.0739(-0.39)	-0.0794(-0.40)

10. Combined RP/SP Mixed Multinomial Logit Model and Panel

Two data sources Revealed Preference and Stated Preference were used, RP explores the present existing situation and behavior of commuters while on the other hand SP basically hypothetical scenario given to a respondents to choice their alternative with different assume attributes. The combined RP/SP model required both data sheet to cover the advantages of both sheets, it is however essential to test for any systematic variance between utilities. In order to combine data from both sources to formulate the mode choice model an artificial tree suggested by Bradley and Daly 1991 was used. The utility functions are as follow.

1. **U(Car)** =RP * (ASC_Car * *one* + BETA_TC * *TC_Car* + BETA_TT * *TT_Car* + Female_C * *DFMale* + AgeSnr_C * *Agesnr* + ZERO [SIGMA] * *one*) + SP * *THETA* * (ASC_Car * *one* + BETA_TC * *TC_Car* + BETA_TT * *TT_Car* + Female_C * *DFMale* + AgeSnr_C * *Agesnr* + ZERO [SIGMA] * *one*) (13)

2. **U(Rickshaw)** =RP * (ASC_Rick * *one* + BETA_TC * *TC_Rick* + BETA_TT * *TT_Rick*) + SP * *THETA* * (ASC_Rick * *one* + BETA_TC * *TC_Rick* + BETA_TT * *TT_Rick*) (14)

3. **U(Bike)** =RP * (ASC_Bike * *one* + BETA_TC * *TC_Bike* + BETA_TT * *TT_Bike* + Female_Bi * *DFMale* + AgeSnr_Bi * *Agesnr* + ZERO [SIGMA] * *one*) + SP * *THETA* * (ASC_Bike * *one* + BETA_TC * *TC_Bike* + BETA_TT * *TT_Bike* + Female_Bi * *DFMale* + AgeSnr_Bi * *Agesnr* + ZERO [SIGMA] * *one*) (15)
4. **U(Bus)** =RP * (ASC_Bus * *one* + BETA_TC * *TC_Bus* + BETA_TT * *TT_Bus* + Female_B * *DFMale* + AgeSnr_B * *Agesnr* + ZERO [SIGMA] * *one*) + SP * *THETA* * (ASC_Bus * *one* + BETA_TC * *TC_Bus* + BETA_TT * *TT_Bus* + Female_B * *DFMale* + AgeSnr_B * *Agesnr* + ZERO [SIGMA] * *one*) (16)
5. **U(BRTS1)** = RP * (ASC_BRTS1 * *one* + BETA_TC * *X11* + BETA_TT * *X12* + Female_B1 * *DFMale* + AgeSnr_B1 * *Agesnr* + ZERO [SIGMA] * *one*) + SP * *THETA* * (ASC_BRTS2 * *one* + BETA_TC * *X11* + BETA_TT * *X12* + Female_B1 * *DFMale* + AgeSnr_B1 * *Agesnr* + ZERO [SIGMA] * *one*) (17)

   U(BRTS2) =RP * (ASC_BRTS2 * *one* + BETA_TC * *X11* + BETA_TT * *X22* + Female_B2 * *DFMale* + AgeSnr_B2 * *Agesnr* + ZERO [SIGMA] * *one*) + SP * *THETA* * (ASC_BRTS2 * *one* + BETA_TC * *X11* + BETA_TT * *X22* + Female_B2 * *DFMale* + A geSnr_B2 * *Agesnr* + ZERO [SIGMA] * *one*) (18)

The models results are presented in Table 4 the model constants or alternative specific constants were defined comparative to Rickshaw. The positive sign for car, bike, BRTS1 and BRTS2 implies that commuters are likely to go by these modes while negative sign for bus implies natural aversion to go by bus as its travel time is high comparative to other modes. This propose that commuters reveal an inclination to travel by these modes as this looks reasonable but higher value for BRTS2 shows commuters are keen to go by BRTS2.Travel time and travel cost which are generic variables as expected are negative and significant in t-statistics, the improved which has been seen is the coefficient of cost which was not significant in SP model. Females are more likely to go by BRTS and car and have positive sign and showing aversion for bus, while t-statistic on other modes are not significant, senior citizens are show preference to BRTS2 and car but have negative impact for bus.

Table 4. Combine RP/SP Mixed Multinomial Logit Model and Panel

File	**Model_C3**
Number of Observations	1480
Number of Individuals	185
Estimated Parameters	19
Converged	TRUE
Rho^2	0.184
Rho^2(Adjusted)	0.177
Final log (L)	-2162.968
ASC_Bike	2.34(3.19)
ASC_Bus	-2.12(-3.00)
ASC_Car	1.15(1.90)
ASC_Rick	Fixed
BRTS1	1.54(1.95)
BRTS2	2.13(3.00)
BETA_TC	**-0.00380(-2.89)**
BETA_TT	**-0.00541(-1.57)**

SIGMA	18.9(3.96)
THETA	1.11(4.25)
ZERO	Fixed
Female_C	0.830(3.37)
Female_B	-0.533(-2.62)
Female_Bi	0.0001(0.0)
Female_B1	0.0001(0.00
Female_B2	0.853(3.69)
AgeSnr_C	1.89(3.92)
AgeSnr_B	-0.787(-2.45)
AgeSnr_Bi	0.661(2.15)
AgeSnr_B1	0.0001(0.0)
AgeSnr_B2	0.974(2.95)

11. Policy Sensitivity Analysis

In this part consideration was paid for best effective policy selection, which can cause the deviation effect on shifting the rider ship from other modes to BRTS2, combine model for RP/SP has been implemented for this analysis. For this determination incremental MNL model was used in place of direct calculation from the elasticity. For analyzing the policy sensitivity analysis incremental logit equation (Ben-Akiva and Lerman, 1985) has been used.

11.1. Decreasing Travel Time and Cost of BRTS2

Decreasing travel time of BRTS2 by 5% the model share of BRTS2 is increased by 2.26% as shown in table 5 further it is also observed that motorcycles user are less likely to shift, as it is reasonable that motorcycles user already take less travel time, while bus and car user show significant percentage of 27.95% and 32.95 % respectively for shifting, as bus take more time and car also due to congestion as shown in figure 5. Similarly, decreasing travel cost of BRTS2 by 5% the model share of BRTS2 is increased by 1.50 % as shown in table 6 further it is also observed that motorcycles and bus users are more likely to shift, as it is reasonable that motorcycles and bus are low income travelers so reducing cost directly affect the ridership, while car user show insignificant percentage as shown in figure 6.

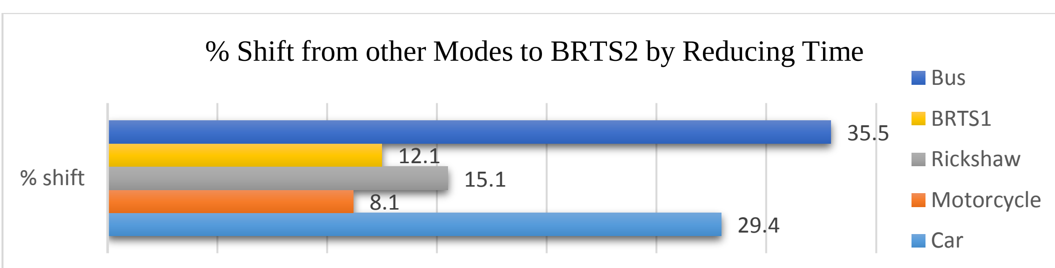

Figure 5. % Shift from other Modes to BRTS2 by Reducing Travel Time

Table 5. Impact on Modal Shift by Decreasing Travel Time of BRTS2

% Travel Time Decrease of BRTS2	Car (%)	Motorcycle (%)	Rickshaw (%)	Bus (%)	BRTS1 (%)	BRTS2 (%)	BRTS2 (+%)	% Shift from other competitive modes				
								Car	MC	Rick	BRT1	Bus
0	13.2	22.1	7.1	17.2	13.7	26.5						
5	13.0	21.9	6.9	16.8	13.6	27.1	2.26	26.3	14.1	17.2	13.1	29.1
10	12.8	21.7	6.5	16.4	13.6	28.2	6.31	27.6	12.3	15.4	12.3	32.3
15	12.5	21.7	6.3	16.0	13.4	28.9	8.8	27.8	12.1	.15.1	12.6	32.8
20	11.9	21.3	5.9	15.5	13.0	30.4	14.1	28.1	11.1	15.3	12.4	33.4
25	10.8	20.9	5.4	14.9	12.8	32.5	21.0	28.5	9.9	14.5	12.5	34.6
30	9.9	19.8	5.1	14.0	12.6	34.7	27.7	29.4	8.1	15.1	12.1	35.5

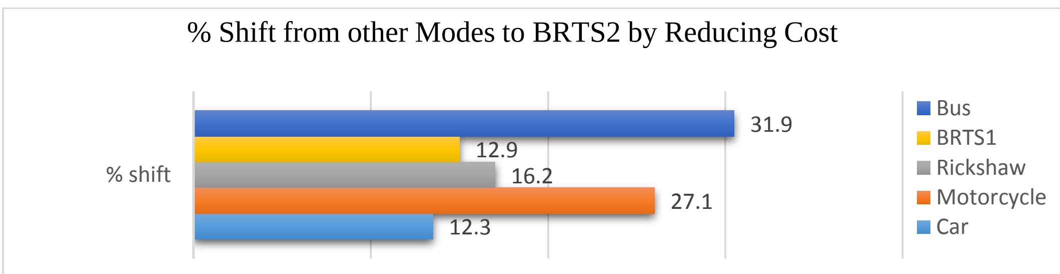

Figure 6. % Shift from other Modes to BRTS2 by Reducing Travel Cost

Table 6. Impact on Modal Shift by Decreasing Travel Cost of BRTS2

%Travel Cost Decrease Of BRTS2	Car (%)	Motorcycle (%)	Rickshaw (%)	Bus (%)	BRTS1 (%)	BRTS2 (%)	BRTS2 (+%)	% Shift from other Competitive modes				
								Car	MC	Rick	BRTS1	Bus
0	13.2	22.1	7.1	17.2	13.7	26.5						
5	13.1	21.9	6.9	17.1	13.6	26.9	1.50	14.3	24.7	17.1	15.0	29.2
10	12.9	21.7	6.7	16.9	13.5	27.2	2.6	14.1	25.1	17.2	14.7	29.5
15	12.8	21.4	6.6	16.7	13.7	27.7	4.43	13.9	25.7	17.1	14.5	30.1
20	12.5	21.2	6.3	16.5	13.7	27.9	5.15	13.8	26.2	17.3	14.1	30.9
25	12.5	20.9	6.1	16.1	13.5	28.1	5.86	13.1	26.8	16.9	13.5	31.2
30	12.9	20.1	5.9	15.9	13.9	28.3	6.57	12.3	27.1	16.2	12.9	31.9

12. Conclusion

As the aim of this research is to evaluate the existing and future mode share of different modes. The RP (Revealed preference) and SP (Stated preference) methods were used to estimate the present and future share of modes.
The main finding of the different developed models are as follow

- According to RP Model the mode share of different came out to be car 20%, motorcycle 33%, bus 27%, and rickshaw 18%.
- The estimated coefficients of travel time and travel cost were negative as expected in all models and results of mixed logit on RP and SP showed improvement in overall model parameters.
- SP models showed that availability of seats and AC/heater is important for commuters and this suggests that commuters are sensitive for comfort in their trips.
- The combined RP/SP model was developed to incorporate the advantages of both models, t-statistic value of travel cost and resulted in improvement of SP model which was not significant before
- The mode share after combined RP/SP model was 13% car, 22% motorcycle,7% rickshaw,17% bus, and 39% BRTS.
- Reducing travel time of BRTS by 5% increases modal share of BRTS by2.26%.Reducing travel cost of BRTS by 5% increases modal share of BRTS by 1.50 %.

ANNEXURE-1 Variables and Coefficients used in Models

Variables	***Coefficients***	***Definition***
BETA_TC	BETA_TC	*Cost, Generic, in Rupees*
BETA_TT	BETA_TT	*Time, Generic, in Minutes*
Agesnr	AgeSnr_N	*Dummy, if Age is greater than 49 = 1, otherwise = 0*
DImpCmfrt	Comfort_N	*Dummy, comfort important = 1, otherwise =0*
DGempy	Govtemp_N	*Dummy, Govt. Employ = 1, otherwise = 0*
DImpSeats	ImpSeats_N	*Dummy,important of seats, important =1,otherwise = 0*
DIncomeG2	IncomeG2_N	*Dummy,if income greater than Rs.35000 =1 otherwise = 0*
DMale	Male_N	*Dummy, if Male = 1, otherwise = 0*
DMnzCost	MinC_N	*Dummy minimizing cost, if minimizing travel cost = 1, otherwise = 0*
DOptmzjrny	OptmjrnyT_B	*Dummy optimizing trip in term of travel time, if*
DPurE	PurtrE_N	*Dummy, purpose of trip education = 1 otherwise = 0*
DPurS	PurtrS_N	*Dummy, purpose of trip shopping = 1 otherwise = 0*
DPurW	PurtrW_N	*Dummy, purpose of trip work = 1 otherwise = 0*
DSafety	Safety_N	*Dummy, Safety = 1 otherwise = 0*
DStdnt	Student_N	*Dummy, Student = 1 otherwise = 0*
Wtng	WT_B	*Waiting time of Bus, in Minutes*
DEdu	edu_N	*Dummy, Educated = 1 otherwise = 0*
X11	TC	*Cost, Generic, in Rupees*
X12	TT	*Time, Generic, in Minutes*
X21	TC	*Cost, Generic, in Rupees*
X22	TT	*Time, Generic, in Minutes*
X25	WT1	*Waiting Time, in Minutes*
X15	WT2	*Waiting Time, in Minutes*
X26	AC2	*Availability of Air conditioner =1, otherwise =0*
X24	HD2	*Headway Time of bus, in Minutes*
X23	SE2	*Confirm seats = 1, otherwise = 0*

DBsns	Bsns2	*Dummy, if Business man = 1, otherwise = 0*
DChngMds	ChngMds2	*Dummy, if changes modes = 1, otherwise = 0*
DFemale	Female2	*Dummy, if Female = 1, otherwise = 0*
	ASC_N	*is a parameter vector to be estimated for Alternative Specific Constant*
	Sigma	*Is a normally distributed error component*
	Theta	*Is a parameter vector to be estimated for scaling*

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An Account of Non-Parametric Analysis of Traffic Accidents at Unsignalized Intersections

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Abstract
Traffic accidents are a global phenomenon and their occurrence on unsignalized intersections is a major concern which is the focus of this study. Analysis of these accidents using distribution free methods or non-parametric techniques provides the flexibility to examine constrained data which does not follow or cannot be applied to any particular distribution. This paper aims to explore the accidents at unsignalized intersections using two non-parametric methods. One is the Mann Whitney test which is used to explore the association between the two groups of accident frequencies and traffic parameters. The other is Artificial Neural Networks (ANNs) which are used to predict the number of accidents based upon volume and speed characteristics of the intersections. The results of Mann-Whitney test indicated that variables with lower U-statistics showed greater arithmetical differences between the two groups of accidents. It was also observed that the application of regression model, as an accident forecasting model, was not feasible for the available data. On the other hand, application of ANNs and their ensemble provided more than 70% accuracy for the same dataset.

Keywords
Unsignalized Intersections, Traffic Accidents, Non-Parametric Analysis, Mann-Whitney Test, Artificial Neural Networks

1. Introduction

Traffic accidents are amongst the world's leading source of injuries and the number one cause of death among young people aged between 15 to 29 (WHO, 2015). They occur on almost all road networks and on most facilities. Unsignalized intersections are no exception to this phenomenon. They are part of every small and most large road networks. Their abundance amongst the fixed facilities makes them more prone to traffic accidents. Since unsignalized intersections operate on priority rule, it's the user which decides when to cross, rather than the system. This makes unsignalized intersections more hazardous as compared to signalized intersections.

In low and middle-income countries traffic data for specific facilities is either not available or too scarce and incomplete to carryout detailed statistical analysis. Sometimes the data obtained does not follow common statistical distributions specific to road accidents such as poison, poison gamma, log normal and negative exponential. Such data could only be analyzed using distribution free methods. They are also known as non-parametric methods. This paper aims to explore the accidents at unsignalized intersections using two non-parametric methods. One is the Mann Whitney test which is used to explore the association between the two groups of accident frequencies and traffic parameters. The other is Artificial Neural Networks (ANNs) which are used to predict the number of accidents based upon volume and speed characteristics of the intersections.

2. Literature Review

The basic elements of any road network are unsignalized intersections and are the most frequently found intersection type (Haleem, 2009). Different researchers evaluated the effect of various variables on its safety. Spek et al. (2006) conducted a very detailed analysis of the influence of speed on crashes and conflicts occurring at unsignalized intersections. Volume and speed were reported as important factors affecting safety at unsignalized intersections by Haleem and Abdel-Aty (2010) and Haleem et al. (2010). Apart from volume and speed, spacing/gap between vehicles was also analyzed by Ahmed et al. (2016). Other parameters such as the spacing and offset of trees planted along the median on roads leading to unsignalized intersections were studied by Chen et al. (2016) along with volume and speed. These researchers found that improperly planted landscape does cause higher crash and injury rates.

Similar to the t-test, the Mann-Whitney test is also used for the comparing between two datasets as a non-parametric alternative (Jevtić et al., 2015). Clabaux et al. (2012) used this test to prove whether a statistically significant difference exist in the mean speed of motorcyclists between two groups and applied it on 14 datapoints. Dotzauer et al. (2013) compared difference in mean intersection waiting-and-crossing time between groups, per trial ranks through Mann-Whitney test. Their dataset comprised of 18 points. Korošec and Papa (2013) used this test to evaluate the difference in performance of two algorithms, the Parameter-Less Evolutionary Search (PLES) and the Ant-Stigmergy Algorithm for solving a constrained transportation scheduling problem. The differences between various variables for drivers with and without Mild Cognitive Impairment (MCI) were analyzed using Mann-Whitney U tests by Devlin et al. (2012). Stigson (2009) used the Mann-Whitney test to examine the difference in crashes between subcategories of four-star roads in Sweden.

Two type of ANNs were employed, namely; linear and multilayer feedforward (MLFF). Linear ANNs are similar to regression models as they attempt to predict the dependent variable using the input signals (independent variables) by determining their optimum weights and biases (Sanger, 1989; Oja, 1992). MLFNN have been commonly used for prediction problems in many fields (Qian and Sajnowski, 1988; Gao and Er, 2005). These networks are comprised of multiple layers with signals (input vectors) processed in the forward direction through each layer.

3. Methods

The crash data was obtained from Malaysian Institute of Road Safety Research (MIROS). After processing of raw data a total of 14 sites were obtained for which information related to the desired parameters was available. The data pertinent to the geometric and physical parameters of intersections was gathered through field survey. Traffic data was collected using data loggers, the details of which can be found in Ahmed et al. (2016). The layout of a typical unsignalized intersection for this study is shown in Figure 1.

The two non-parametric methods used in this research are Mann-Whitney U test and ANNs. The Mann-Whitney U test is similar to the t-test, and is used to evaluate if there exist any statistically significant difference between intersections that had single and multiple accidents with respect to volume, speed, gap between vehicles and road width. Since, it is a test that compares the mean ranks between two unpaired groups, therefore; the data was divided into two groups. One group comprised of sites that experienced only one accident during the period for which the accident data was obtained. The other group contained sites that had multiple accidents during the study period.

ANNs were used to develop models for predicting number of accidents on intersections based upon the available data. The performance and effectiveness of these models was also compared to regression model. ANNs are a popular branch of Artificial Intelligence (AI) in the field of prediction modeling (Maier and Dandy, 1996, Zhang et al., 1998, Imrie et al., 2000, Duan et al., 2013). They provide the flexibility to work with noisy and restricted data (Thompson and Kramer, 1994) as is the case in this study.

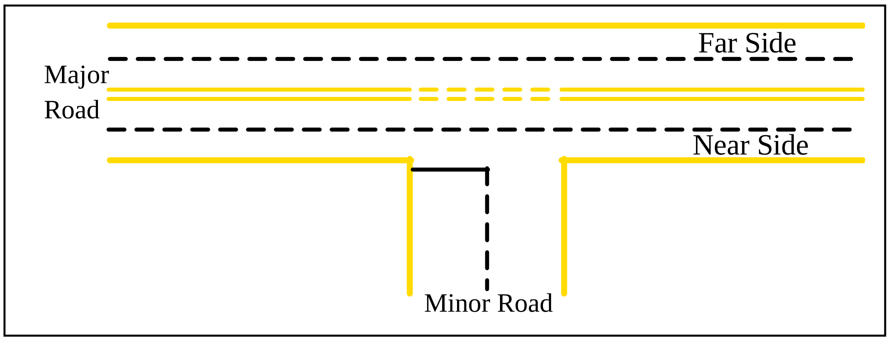

Figure 1: Layout of a Typical Unsignalized Intersection

4. Results and Discussion

4.1 Descriptive Analysis

The study sites were unsignalized intersections which are often employed in cases where a major highway crosses a minor highway or street. Accidents can occur at these situations due to the difference in the road design characteristics which means that the drivers from different directions would behave differently at the intersection. The variables used for the descriptive analysis are accident frequency, major road width (road width of the major highway for the specific intersection), near to far volume ratio, average speed near, average speed far, average speed all, and near to far gap ratio. Here the near to far volume ratio refers to the ratio between the major road traffic volume moving in the near side direction to the major road traffic volume moving in the far side direction, as shown in Figure 1. Similarly the near to far gap ratio refers to the ratio between the gap/spacing among vehicles moving in the near side direction to the gap/spacing among vehicles moving in the far side direction. The average speed near, average speed far and average speed all refers to the time mean speed of major road vehicles moving in the near side direction, far side direction and both directions respectively. Out of these seven variables, accident frequency is the dependent variable while the rest are independent variables.

The descriptive statistics related to them are presented in Table 1. It was found that sites having more than one accident were located on major roads with greater widths as compared to sites that had only one accident. The mean 'near to far volume ratio' was higher for sites with multiple accidents, i.e. the volume of traffic moving in the far side direction was greater on such sites in comparison to sites with single accidents. The average speeds were also higher; whether on the near side, far side or overall, for sites with multiple accidents as compared to sites with single accidents. The average of the gap or spacing between vehicles moving in the near side direction was less than far side direction, on sites with multiple accidents as compared to sites with single accidents. As a result the mean ‘near to far gap ratio’ was less than one on sites which experienced multiple accidents and was greater than one on sites which experienced single accidents. All the above results indicate that decrease in spacing/gap between vehicles and increase in major road width, speed, and volume increase the chances of accidents.

Table 1: Descriptive Statistics of the Variables Used in the Study

		Accident Frequency	Major Road Width	Near to Far Volume Ratio	Average Speed Near	Average Speed Far	Average Speed All	Near to Far Gap Ratio
Sites having single accidents (n = 10)	Mean	1	11.58	1.02	35.88	37.8	36.84	1.24
	S.D	0	2.73	0.48	10.66	9.4	9.88	0.67
	Min	1	7.1	0.43	19.2	25.45	22.33	0.49
	Max	1	15	1.9	55.65	59.46	57.56	2.61
Sites having multiple accidents (n = 4)	Mean	3	12.03	1.55	38.08	41.88	39.98	0.68
	S.D	1.15	2.7	0.4	5.02	8.56	6.53	0.15
	Min	2	8.3	1.08	30.57	31.05	30.81	0.53
	Max	4	14.5	2.01	40.81	49.52	45.16	0.88

S.D = Standard Deviation, Min = Minimum, Max = Maximum, n = number of sites

4.2 Mann-Whitney U Test

The Mann-Whitney U test explains whether there exists any statically significant relationship between the accident frequency and each of the six independent variables for the two groups. The ‘Null Hypothesis’ for road width states that ‘There is no significant difference of mean road width between the two accident groups’, its ‘Alternative Hypothesis’ is ‘There is significant difference of mean road width between the two accident groups’. The ‘Null Hypothesis’ for near to far volume ratio states that ‘There is no significant difference of mean near to far volume ratio between the two accident groups’, its ‘Alternative Hypothesis’ is ‘There is no significant difference of mean near to far volume ratio between the two accident groups’. In identical manner the null hypotheses and their alternate hypotheses for the other four variables are stated. A very small rejection region of 5% was selected. As a result the alternate hypotheses for all the variables were rejected because their exact significance was greater than 0.05. However the arithmetical difference between the means of the two groups varied with respect to each variable, as shown in Table 2. For ‘near to far volume ratio’ and ‘near to far gap ratio’ the difference was almost half. It is interpreted as the mean near to far volume ratio for sites which experienced multiple accidents was almost double than the sites which experienced single accidents. For the case of near to far gap ratio, the sites which experienced multiple accidents had mean gap ratio, between the vehicles moving in the near side direction to the far side direction, almost half than the sites which experienced single accidents. For the variables pertinent to speed and road width the arithmetical difference was not very high. Variables with lesser U statistic possess greater arithmetical difference among groups.

Table 2: Results of the Mann-Whitney U Test

	Major Road Width	Near to Far Volume Ratio	Average Speed Near	Average Speed Far	Average Speed All	Near to Far Gap Ratio
Mean for Multiple Accidents	12.03	1.55	38.08	41.88	39.98	0.68

Mean for Single Accidents	11.58	1.02	35.88	37.80	36.84	1.24
Difference between the means of Multiple to Single accidents	3.84%	51.81%	6.12%	10.79%	8.52%	-45.29%
Mann-Whitney U Statistics	18.5	8	16	12	12	10
Exact Significance	0.86	0.11	0.64	0.3	0.3	0.19

4.3 Predictive Modelling

The predictive modeling effort was started with the development of regression model for the given data. The regression model and its relevant accuracy measure (r-square) are given in Table 3. It can be observed from the data presented in this table that the regression model was impractical since all the variables had statistically insignificant effect in the model. This confirms to the finding of Mann-Whitney test where the significance for all variables was found to be statistically insignificant at 5% probability. Moreover, the R-square for the model was less than 0.5 while the percentage error was more than 40%. This means that the model was not effective in capturing the trend of the data.

Table 3: Regression Model

	Coefficients	Standard Error	t Stat	P-value
Intercept	3.548853	5.66546	0.626402	0.548504
Major Road Width	0.085695	0.206076	0.415842	0.688456
Near to Far Volume Ratio	0.12136	0.954527	0.127141	0.901966
Average Speed Near	-0.10089	0.133602	-0.75514	0.471812
Average Speed Far	0.044795	0.089775	0.498965	0.631233
Average Speed All	0	0	65535	Cannot be calculated
Near to Far Gap Ratio	-1.09362	1.492324	-0.73283	Cannot be calculated
R-square		0.22		
Mean Absolute Percentage Error (MAPE)		41.5%		

Considering the results of regression model, we also tried MLFF and linear ANN models and their ensembles for predicting number of accidents. The specifications of the models are shown in table 4 along with their accuracy measures. It should be noted that the available dataset, with all samples, was not sufficient to develop the regression model which resulted in statistically insignificant values of coefficients as shown in Table 3. But ANN models do not have such restrictions. Therefore, the accuracy measures for ANN models were calculated on the test dataset which was approximately 40% of the available dataset, selected randomly. The data in Table 4 shows that the MLFF models were able to give the best performance in terms of R-square and MAPE then all models tried in this study. Linear ANN model's performance was lower than MLFF; however, MAPE for this model was less than regression model. The ensemble was developed with the simple average of the two models without weighing factor. Hence, its performance was lower than MLFF. Hence, MLFF models are considered best in this case.

Table 4: ANN Models and Ensemble

Type of ANN	No. of Units	R-square	MAPE
MLFF	5	0.29	38.0%
Linear	N/A	0.20	38.3%
Ensemble	2 (The above models)	0.25	38.0%

5. Conclusion

This paper presented an account of the non-parametric analysis of accidents at unsignalized intersections. Mann-Whitney U test and Artificial Neural Networks (ANNs) were used for the analysis with accident frequency as the dependent variable and various independent variables. Although the result of the Mann-Whitney U statistic was not significant at 95% confidence level but the trend was similar to the practical observations on site. From the analysis, it is concluded that decrease in spacing/gap between vehicles travelling in the near side direction increases the chances of accidents. Increase in major road width, speed, and volume have a positive effect on accident frequency, thus an increase in such parameters will result in an increase in accident frequency. The results showed that the traffic parameter which had the least Mann-Whitney U statistic possessed the largest arithmetical difference between the two accident groups.

The regression model developed was found to be impractical since all the variables were statistically insignificant. Therefore, ANNs were employed using MLFNN and linear models. However, the MLFNN was found to provide better performance than linear ANN. Ensemble of models was also employed using the simple average of linear and MLFNN models and it was found to be having lesser accuracy.

It is recommended that unsignalized intersections lying on wider major roads with high speed traffic and more volume on the near side direction of travel should be evaluated for safety. Possible solutions to hazardous sites could be channelization, speed calming, geometric amendments and signalization. ANNs are recommended to be used for prediction of constrained data which is usually the case for crashes as they occur infrequently.

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Mapping Access To Public Transport Network And Assessing The Changes To Public Transportation Systems On Accessibility For Rawalpindi

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Abstract
Easy access to the public transport network (PTN) is key issue faced by the cities world over. A lot of work has been done to increase the access to PTN. Still, there lies a research gap in the case of Pakistan, where poor access to the PTN has caused people to buy more and more cars. Thus, adding millions of cars on the inadequate road network of larger cities, causing congestion and parking problems on both major and minor roads. Similar situation has emerged in the twin cities because of insufficiency of good accessible PTN. In this background, this paper maps the accessibility to the PTN for Rawalpindi. The method used for this purpose is the Public Transport Accessibility Levels (PTAL). After the mapping of accessibility, underserved regions have been identified. The paper goes on to suggest changes in the existing Public Transport Route network to evaluate the effects on accessibility. Population analysis has been performed for the original and the modified PTN to study the change after PTN change. The suggested changes in the PTN reduce the poor accessible region from 80.18% to 69.6%. The results of the analysis can be used by the local transport authorities to improve the existing PTN in a major way.

Keywords
Public transport, accessibility mapping, public transport network modification, population analysis.

1. Introduction

The urban planners and the transportation planners are focusing on the sustainable development of the cities. One of the major component in achieving a sustainable city is to provide mobility and good accessibility to PTN. To improve the mobility, quality and accessibility of the public transport, a lot of work has been carried out in the developed countries. Like other developing countries, in Pakistan too, the main focus of the transportation authorities is to improve the mobility and quality of public transport system. A little has been carried out in improving the accessibility of the public transport. Transport accessibility is a crucial factor, defined as the ease by which a transport service is accessed by the people from a given location. The accessibility to a public transport service is important for a sustainable PTN. The measurement of transport accessibility is important for both infrastructure management and for forecasting of transport ridership. Its measurement helps identify underserved regions, particularly in the developing countries, considering their limited fiscal resources. This research focuses on the calculation and mapping of the public transport accessibility and suggesting adjustments to improve accessibility.

2. Literature Review

The concept of accessibility was first coined by Hansen in 1959, as the "opportunity which an individual or type of person at given location possesses to take part in a particular activity or set of activities".

Hansen focuses on the relationship between accessibility and urban development. According to Liu et al., (2003) accessibility is “the ease with which activities at one place may be reached from another via a particular travel mode”. An accessible public transport system ensures equal opportunities for all people in the society. Guers and Wee (2004), define accessibility as “the extent to which the land use-transport system enables (group of) individuals or goods to reach activities or destinations by means of a transport mode”. The accessibility measures described by them include the infrastructure, location, person and utility based measures. Local index of transit availability (LITA), a methodology developed by Roods (1997) considers the frequency, capacity and route coverage of public transport to identify the accessibility. Land Use & Public Transport Accessibility Index (LUPTAI) Tool was developed by Pitot et al. (2006). It measures accessibility to common land use destinations by public transport and walking. LUPTAI quantifies and determines the accessibility by developing a composite index of measures. It uses the travel distance and time between the two locations. The measured values are applied through GIS to different locations. The Public transport accessibility level (PTAL) methodology was developed by London Borough of Hammersmith and Fullham in 1992. This method was employed by transport for London to measure density of the PTN at any location in London. In London, PTAL was used to develop the parking policies. The parking was discouraged in areas with high PTAL score and vice versa. Transport for London (TfL) have employed PTAL methodology to calculate the accessibility of public transport for the city of London. TfL is currently using this technique to measure the current accessibility and future accessibility. Out of all the methods described above, the PTAL methodology was selected to calculate the accessibility. The data requirements for the implementation of PTAL methodology is less as compared to other accessibility modelling methods. The results by PTAL methodology can be easily mapped by using GIS tool and are easy to interpret. Belinda and Julian (2003), conducted an analysis of city bus network in Northern Ireland. The study used the census and network data with the aid of GIS. The use of GIS allowed to study the effect of network changes on PTAL. A base map was created representing the city’s current bus network. Three different hypothetical maps were obtained by altering the routes on the base map. This helped generate various PTAL values in the base map and hypothetical maps to identify the best option. Parvathy, V S Sanjay and Bindhu (2013), did the accessibility analysis for Thiruvananthapuram city, India. The objective of the study was to calculate public transport accessibility index using PTAL and to develop regression models for public transport trips. To conduct a survey, the city was divided into 13 parts. The data was collected from 560 houses, which included questions based on household, personal and trip characteristics. The values of Accessibility indexes after the PTAL calculation and these were visualized on the city map. The regression models for different wards were obtained and t-test was conducted. The results showed that the trips by public transport mainly depend upon the walk time from origin to the bus stop and waiting time at the bus stop.

3. Study Area

Selected areas of Rawalpindi and Islamabad were considered for the analysis. The total area of the study was 214km^2, covering 68.17 km^2 of Islamabad and 146.18 km^2 of Rawalpindi. In the selected area 29 public transport routes were identified. These public transport routes employ high occupancy vehicles such as busses, low occupancy vehicles such as wagons and pickup Suzuki. The public transport map along with the identified area is shown in figure 1, with service access points (SAP). For detailed analysis, the study area was divided into three population zones as shown in figure 2. The division of the population zones was done on the basis of the town map of Rawalpindi and Islamabad. The population of the study area was taken from the Punjab Bureau of Statistics.

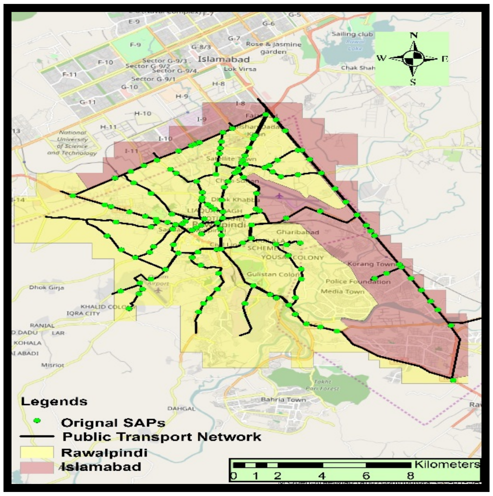


Figure 1: Study Area Map

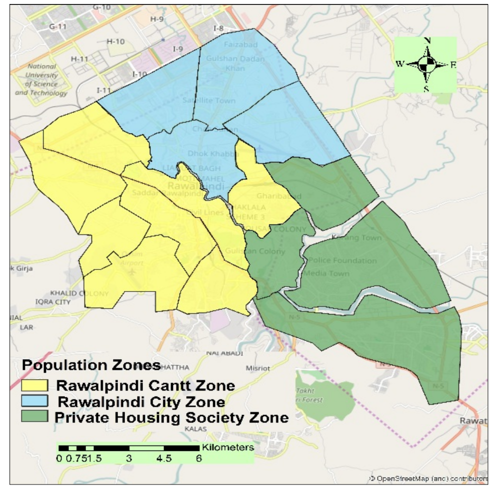


Figure 2: Population Zones

4. PTAL Methodology

For the PTAL analysis, the study area was divided into cells of equal areas of 0.25km² as shown in the figure 3. The centroid of each cell was considered as the Point of interest (POI). The public transport stops were considered as service access points. The steps of the PTAL methodology are:

1. The point of interest (POI) and nearest service access points (SAP) are identified.
2. Valid walking routes from POI to SAP are identified. The distance is calculated.
3. The walking speed as defined by the transport for London is 4.8km/h. For Rawalpindi, the speed is assumed to be 3.6 km/h and 4.8 km/h, considering the lack of proper walking network in Rawalpindi. The threshold value of distance was considered to be 1200 meters. The speed of 3.6 km/h or 60m/min takes 20 minutes to cover 1200 meters and for walking speed of 48km/h it takes 15 minutes to cover 1200 meters. Calculations are done by using both speeds:

   Walking time (WT) = (Distance from POI to Stop)/ (Walk Speed)
4. The frequency of different modes is considered and the mode with highest frequency is given weight 1. The rest are given weight 0.5. Scheduled waiting time at a bus stop is calculated. Headway was observed different for every public transport route:

   Scheduled waiting time (SWT) =0.5 x headway
5. The reliability (k) of public transport as per the transport for London is defined as the difference between the scheduled headway and the actual headway of the public transport modes.
6. The total access time to the service access point is the sum of SWT, WT and k:

   Total Access Time (TAT) =SWT+ WT + k
7. The next step is the calculation of equivalent doorstep frequency (EDF). The EDF is used to convert the total access time to a unit that the public transport service is available at the door step:

   EDF= TAT/30
8. The weights calculated in the step 4 are multiplied with EDF to get Accessibility indexes (AI):

   AI= weight x EDF
9. From the above step the AI for different steps are calculated. These AI for different modes are summed up to find the AI of a specific POI.
10. The accessibility index for POI is converted to PTAL value ranging from ‘0 to 6’ as shown in Table 1. The ‘0’ represents worst accessibility and ‘6’ represents the best accessible region.
11. Each PTAL value is assigned a different color. The POI has different PTAL values. These PTAL values are plotted in maps by using the Arc GIS Software. The colors with their associated PTAL bands are shown in table 1.

Table 1: PTAL Color Ranges

PTAL	Range of Index	Map Colour	Description

0	0		Very Poor
1	0-5		Very Poor
2	5-10		Poor
3	10-15		Moderate
4	15-20		Good
5	20-25		Very Good
6	25+		Very Good

A sample for the above step for the POI id 1 is shown in table 2.

Table 2 Sample Accessibility Calculation

POI ID	Mode of travel	Bus Stop Name	Distance	Frequency	Walk time	Sch waiting time	Reliability	Avg waiting time	Total access time	Equivalent doorstep frequency	Weight	AI
1	W1	GTS stop	640	8	8	3.75	4.5	8.25	16.25	1.84	1	1.84
	:	:	:	:	:	:	:	:	:	:	:	:
	:	:	:	:	:	:	:	:	:	:	:	:
	Wn	n^{th} Stop	540	5	6.75	6	4.5	10.5	17.25	1.79	.5	.86

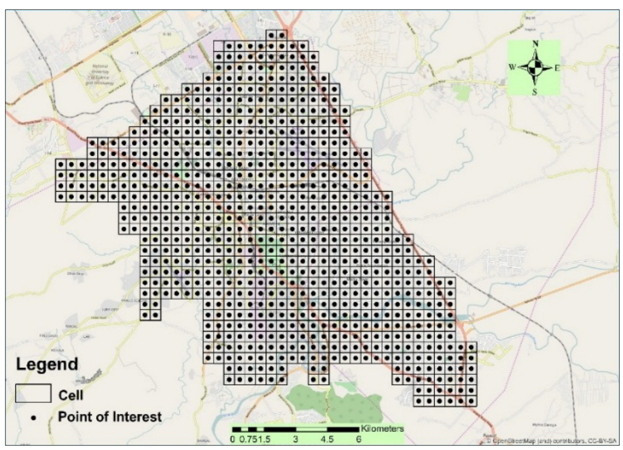
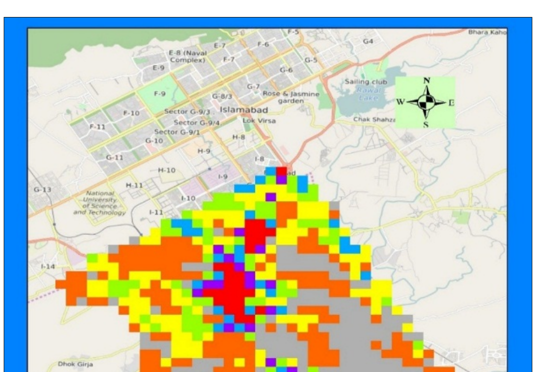


Figure 3: Point of Interest

5. PTAL Analysis

The study area was divided into 757 equal cells known as point of interest as shown in the figure 3, the accessibility index for each cell was calculated individually. After the calculation, the accessibility was mapped using the ArcGIS software. The PTAL model for the base case scenario i.e. for walking speed of 60m/min and 80m/min are shown in the figure 4 and figure 5.



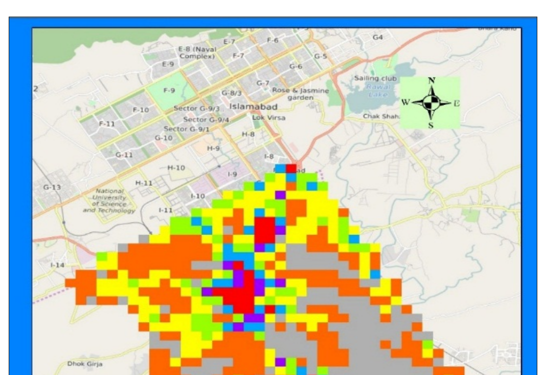


Figure 4: PTAL Model for 80m/min

Figure 5: PTAL model for 60m/min

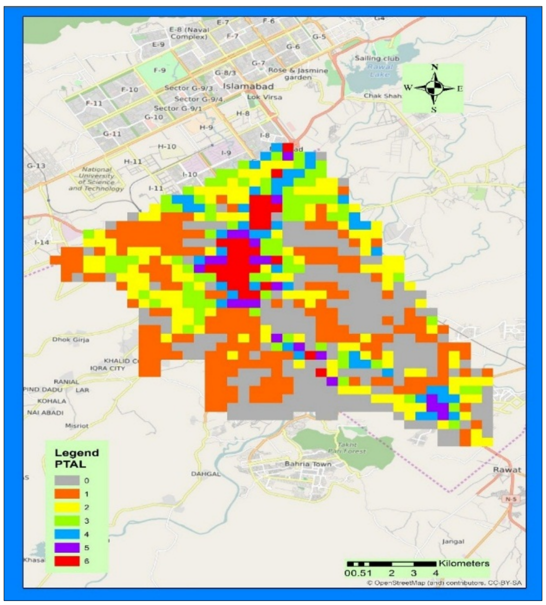


Figure 7: PTAL Model after Addition of New Routes 80m/min

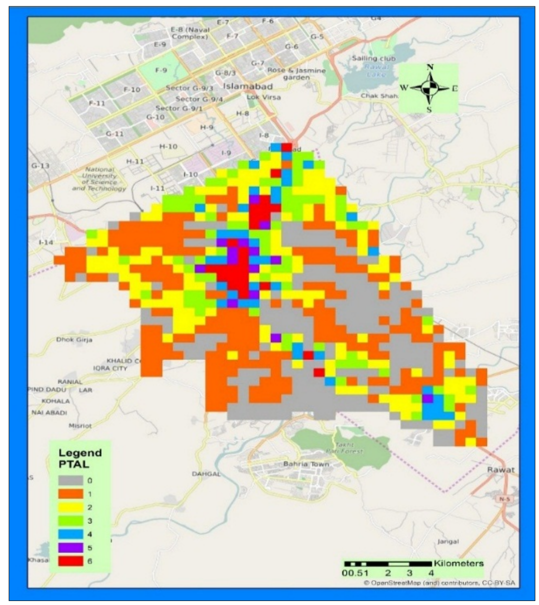


Figure 8: PTAL model after Addition of New Routes 60m/min

Based on the original accessibility scores, addition of the PTN was suggested. This addition was done for the underserved regions. The added public transport route map is shown in the figure 9. Accessibility was again mapped for this modified scenario. The modified accessibility map is shown in the figure 7 and figure 8 for both the walking speeds.

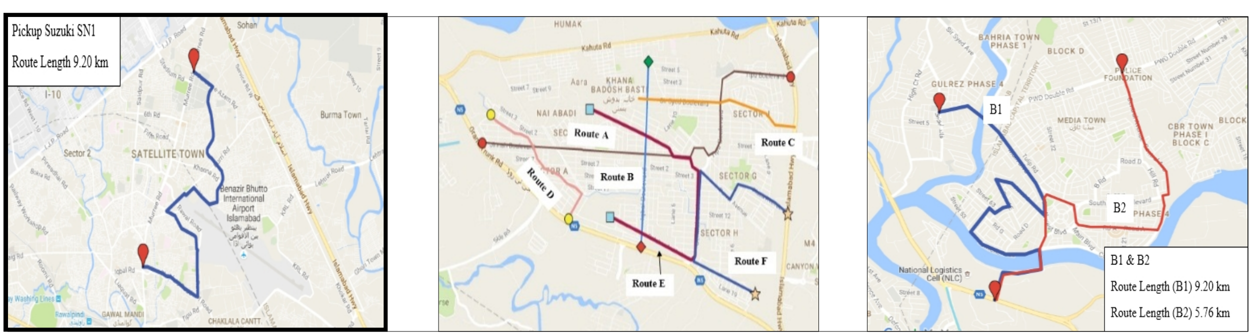


Figure 9: Addition of New Public Transport Routes

6. Results of Accessibility Analysis

The results of the analysis show that accessibility coverage for the walking speed of 60 m/min is less as compared to the walking speed of the 80 m/min. The results are shown in the figure 10, according to the results, the coverage area for the PTAL range of "0" is same for both the walking speeds. About 7% of the coverage area lies in the region with the best accessibility. After the addition of new public transport routes, the accessibility for both walking speeds improved. The results are shown in the figure 11 and 12, the area with poor accessibility for both the walking speeds has decreased. Poor accessibility area decreased by 11% and 10% for the walking speeds of 80m/min and 60m/min respectively. Whereas, the area for excellent accessibility has increased by 18.4% for the walking speed of 80m/min.

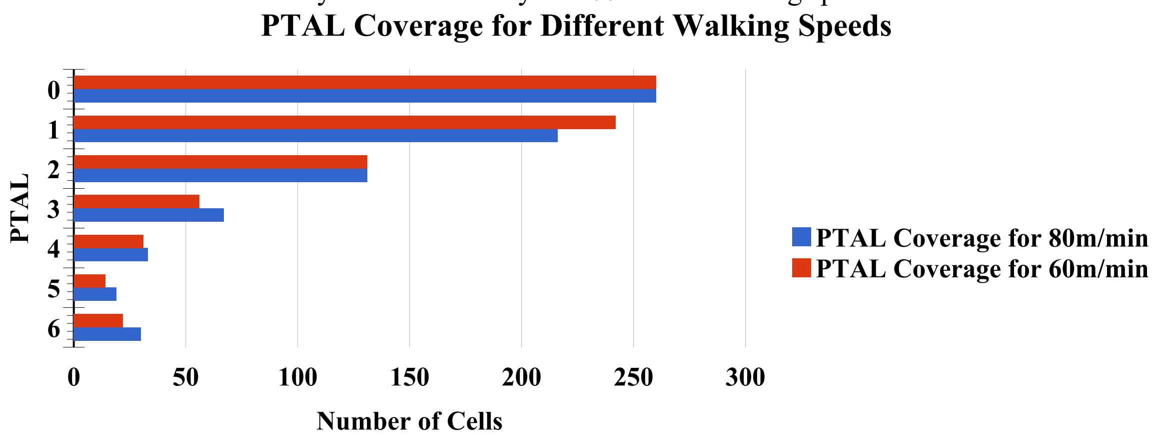


Figure 10: PTAL Coverage for Different Walking Speeds

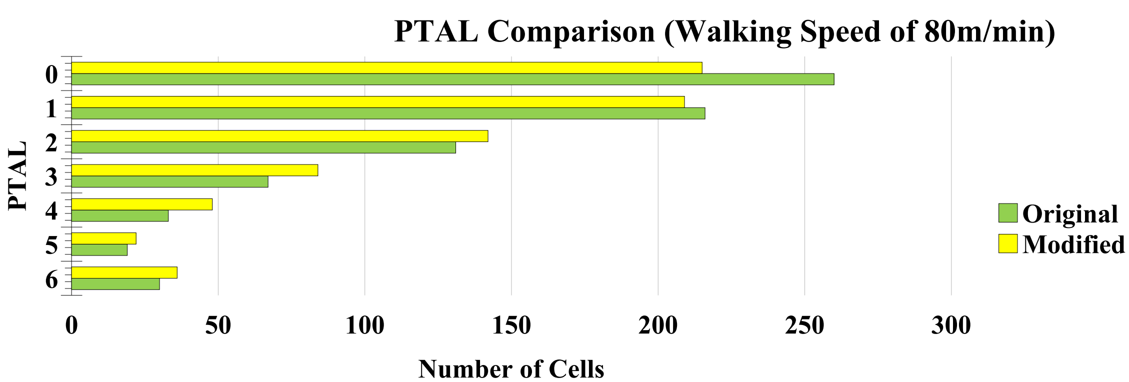


Figure 11: PTAL Comparison for Walking Speed of 80m/min

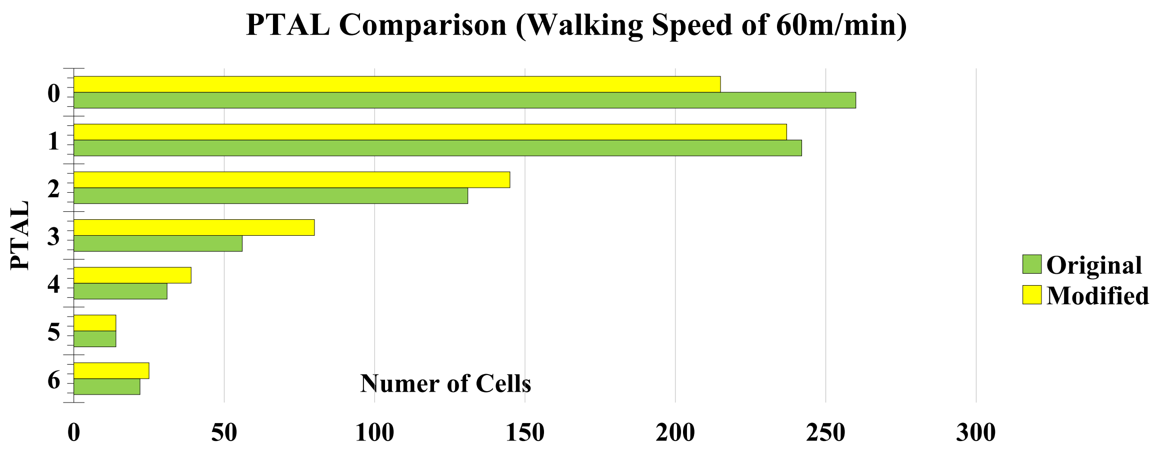


Figure 12: PTAL Comparison for Walking Speed of 60m/min

7. Population Analysis

After the accessibility analysis, the population was calculated for both the original and modified accessibility. The walking speed of 80 m/min was considered for the population analysis. New public transport routes were introduced in the Rawalpindi city zone and the private housing society zone. The results suggest that the population residing in the poorest accessibility region of private housing society zone has decreased by 33.6%. Similarly, the population being served by the very good accessibility region has increased by 136% in the private housing society zone. After the introduction of new public transport route in the Rawalpindi city zone, the population being served by good accessibility region has increased by 15%. The results of the population analysis are shown in figure 13 and figure 14.

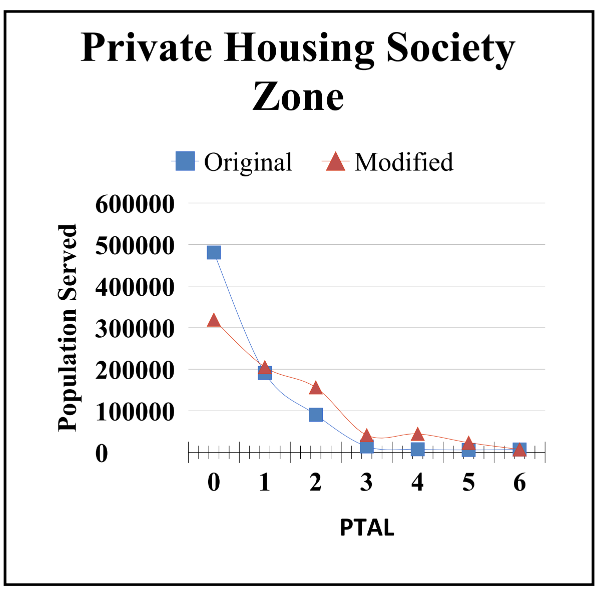


Figure 14: Population Served for Private Housing Society Zone

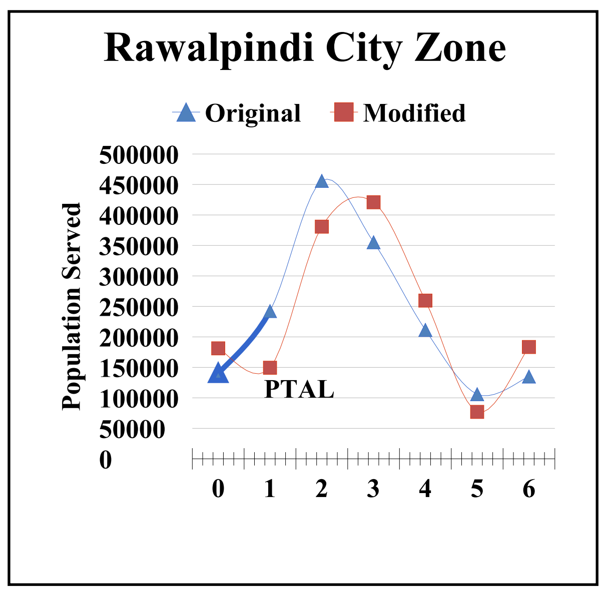


Figure 13: Population Served for Rawalpindi City Zone

8. Conclusions and Recommendations

The change in the walking speed has affected the accessibility coverage. The accessibility coverage of walking speed of 60 m/min is 36km^2 for very good accessibility region. After increasing the speed, the accessibility coverage for the same region increased to 49km^2.After the addition of new routes, the very poor region's accessibility decreased from 476 km^2 to 424 km^2, which indicates a change of 10%. Whereas, the accessibility coverage of very good region has increased from 49 km^2 to 57 km^2.The private housing society zone includes PWD, Bahria town phase I to VI and DHA phase-II, it has the poorest accessibility score. The additions suggested in this study which include the addition of new public transport services in the private housing society zone should be implemented by the local transport authority. Also the local transport authority should ask for public transport plan before approving the construction of new societies. The PTAL analysis show that the business area of Rawalpindi city zone has the best accessibility. Parking restrictions should be imposed in the business area of Rawalpindi city zone.

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Lane Changing Behavior and Effects on Volume and Speed: A Modal Analysis

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Abstract
Lane changing (LC) is a leading cause of traffic congestion, yet its effects on urban networks and modal volumes have received scant attention. Traffic flows at two locations on an urban arterial (Rashid Minhas Road, Karachi) were studied for lane changing and modal volumes. Lane changing was observed for four traffic modes (passenger cars, motorbikes, rickshaws, and buses/trucks), and was weighted to reflect the different impact each mode has on the surrounding vehicles when it changes lanes. The LC counts were weighted with respect to vehicle size and extent of lane changing. Only those LCs which involved the middle and fast lane were recorded. The troughs and peaks in the modal LC and volume curves were then compared. The results showed that lane-changing by passenger cars, rickshaws and the total volume over time is highly synchronized with the volumes of the respective modes, while motorbikes, buses and trucks display randomized behavior. Periods of near-congestion can be identified, and a clear impact of location and traffic patterns on the parameters that signal congestion is also evident.

Keywords
Lane changing, volume, mode, congestion, urban

Background
Lane changing is known to act as a micro-trigger for congestion: excessive lane changing can cause a breakdown in traffic flow (OECD/ECMT Transport Research Center, 2007). Ahn and Cassidy (2007) observed that lane changing is the main cause of congestion on freeways. Laval and Leclercq (2007) described how a lane-changing vehicle is a moving bottleneck that impedes the trajectories of the following vehicles. The impact of a vehicle changing lanes on the surrounding vehicles may be determined by its size, speed and the extent of the lateral lane changing movement. Driving on the lane marker may also cause congestion by leaving too little space for the vehicles on either lane to navigate easily. While extensive empirical and modeling work has been carried out (Ahmed, 1999), the analysis of lane-changing by vehicle class (mode) and the unique impact each mode has on surrounding traffic while changing lanes has been neglected. Furthermore, the overwhelming majority of surveys have been conducted on freeways (Cassidy and Rudjanakanoknad, 2005; Cassidy and Bertini, 1999; Bertini and Leal, 2005), with no examination of urban roads. This study captures the effect of modes unique to urban networks such as rickshaws and their variants and public buses, and the network characteristics themselves.

Methodology
Two locations on Rashid Minhas Road, a 2-way, 3-lane arterial in Karachi, were chosen for study. One observation point, near Aladin Park, was on a pedestrian crossing above a straight road while the other, near Shafique Mor, was on a pedestrian crossing just after a U-turn. The road width at Aladin Park was

10.2m and at Shafique Mor was 7.5m. These locations were well away from any traffic signals, lacked significant pavement deterioration, jaywalking and encroachment, and were about 200m after a U-turn which eliminated any queues and made them ideal for the study of only lane changing on traffic congestion and flow. Videos were recorded on both locations, from 10:30 till 18:45 at Aladin Park and from 11:35 till 19:00 at Shafique Mor. These videos were later analyzed for volumes and lane changes (LCs) over time in 5-minute segments for passenger cars, motorbikes, rickshaws, and buses and trucks. At both locations, only one direction (a 3-lane road) was recorded.

For the analysis of lane changing behavior, only those LCs were counted that involved the middle lane and fast lane. This was done because the LC behavior of vehicles switching between the middle and slow lanes was not representative of the LCs involving the middle and fast lane. Vehicles rarely switched to the slow lane, in order to either turn left into a service road or to temporarily park (such as buses to let passengers disembark), while vehicles switched to the fast lane to increase their speed. To reflect the variable impact of lane changing on surrounding traffic when different amounts of transverse movement occur while changing lanes, the counting was done in the following manner:

- Every time a vehicle crossed the lane marker, a score of 2 was assigned to the vehicle's class.
- If the vehicle was simply driving over the lane marker, a score of 1 was assigned.
- If the extent of a vehicle driving over or crossing the lane marker was marginal, especially during low levels of surrounding traffic, the scores awarded were lowered by 1, since LCs made into small spaces between vehicles are more likely to transform smooth flow into stop-and-go flow (Ahn and Cassidy, 2007). Accordingly, for very abrupt lane changes with/or long transverse movements, the scores awarded were incremented by 1.

The level of disruption that a vehicle causes to the surrounding traffic while changing lanes is proportional to its relative speed and size. However, in an urban arterial, the smaller headways and multiple ingress points do not allow large speed differentials between the vehicles (as opposed to freeways), and vehicles accelerate or decelerate while changing lanes. A speed-based parameter would therefore require continuous recalculation, therefore only the size differential was considered. A factor was assigned to each vehicle class. This was done by comparing the box-volumes (length × width × height) of a typical vehicle for each category (car, motorbike, rickshaw, bus and truck) with that of a car. This yielded a multiplication factor greater than 1 for vehicles bigger than a car, and less than 1 for vehicles smaller. The LC score for each vehicle category is simply multiplied by this factor to yield a weighted score for each 5-minute period. Since vehicle speed is often inversely related to vehicle size in urban networks and slower vehicles act as moving bottlenecks (Munoz and Daganzo, 2004), a larger factor is highly likely to represent a vehicle that causes more disruption while changing lanes.

The passenger car was chosen as the reference vehicle since other vehicles rarely use indicators or have functional indicators, and so their lane changing can be assumed to be more unexpected and disruptive to surrounding traffic. These factors served only to make the combined LC score of a 5-minute segment more representative of the different modal volumes that the segment witnessed, and did not mean anything on their own. Vehicles were counted over the duration of the video. Pilot observations revealed that few LCs involved the slow lane (since it was mostly used by rickshaws and buses). It was assumed that the LCs involving the middle and fast lanes affected (or were affected by) the entire 3-lane volume for a given segment, as Laval and Daganzo (2005) asserted that the disruption caused by a lane change incites more lane changes. As a result, vehicles were counted for all the lanes.

The dimensions of the standard vehicle were used to compute the weightage for each class. For each 5-minute segment, the volume and the weighted lane changing score for each vehicle class was computed. Curves were plotted for volumes and lane changing (LC) scores over time for all vehicle classes. Since the data had already been grouped over 5-minute intervals, to prevent further loss of accuracy the curves were not smoothed out in any way. Buses and trucks were aggregated into one mode because of two reasons. Firstly, it was very difficult to ascertain a particular behavior for buses simply through visual observation, since a small number of buses (such as almost all minibuses) were public buses, which meant that they routinely weaved through the middle and slow lane. Most of them, however, were intercity or private buses/vans which traversed only on the fast lane. Furthermore, there were numerous varieties of trucks with variable speeds. These diverse modes were aggregated to represent any large vehicle that travels at slow to medium speed and creates a significant moving bottleneck for the surrounding traffic. As a result, irregular and rare vehicles such as tractors were also added to this mode. Using a few samples to determine the average distribution of trucks to buses, this mode was assigned a composite LC score.

The two curves (volume and LC) were then analyzed for concurrent behavior. Observation of the traffic at both locations revealed that it never came to a complete stop, with low densities characteristic of most of the non-peak hour movement. It was therefore highly unlikely that a high or low amount of volume or LC (represented by a peak or trough in the curves) at either location would affect that location's traffic for anything beyond a brief period. Connections were therefore only made between peaks or troughs occurring within a maximum of 10 minutes of each other. To classify between the different amounts of lag, when a trough appeared in one curve, it was connected with a trough in the other curve that appeared

1. within 5 minutes, (solid connection)
2. between 5-10 minutes ahead of the trough, (dashed connection, or positive lag)
3. between 5-10 minutes behind the trough (square dotted connection, or negative lag)

The same was done for peaks. Finally, those events in which an LC peak coincided with a volume trough (and vice versa), or inversions, were recorded. The purpose behind making and counting these different connections was to identify any possible cause-effect relationship between LCs and volume. At any time, two things can happen: LC can increase volume by increasing density, in effect increasing the cross section of the platoon at the expense of speed, or volume can increase LC by increasing the number of vehicles (thereby increasing the probability that one of them will change lanes) or by reducing the speed (thereby inciting drivers to increase their speed by changing lanes).

It is important to understand the implications of grouping the readings into 5-minute intervals. It may be assumed that the readings follow a Gaussian distribution within an interval, in which case the bulk of lane changes (resulting in a trough or peak) in one interval occur, on average, 5 minutes before those in the next interval. However, it is perfectly possible that the bulk of LCs occurred towards the very end of one interval and the beginning of the next, which means that even though the time elapsed between these two concentrations may be less than 5 minutes, they will be clumped together in the category of delayed connections (occurring 5-10 minutes before or after the corresponding feature in the complementary curve).

Observations

Shafique Mor (average 5-minute volume = 470) experienced a steady progression of total volume almost throughout the video duration. However, it had mostly lower volumes than Aladin Park (average 5-minute volume = 523), exceeding it only at around 19:30. As the road section was narrower by 2.8m at Shafique Mor, higher densities could be observed here. Aladin Park volumes peaked from 12:00 noon till 14:00 (due to the closing of schools) then dipped sharply, recovering only moderately at 16:00 due to the closing of offices, which may suggest that this route served more school-going passengers than office workers.

Mode-wise, at Aladin Park cars displayed volume trends similar to that of the total volume. Motorbike and rickshaw volumes peaked from 12:00 noon till 14:00 and 14:40 till 14:45 respectively, and then stayed steady. At Shafique Mor, cars increased from 13:20 till 15:30 and from 17:10 till 18:30.

Motorbikes and rickshaws increased steadily throughout. Buses and trucks experienced a decrease in volume over time at both locations. This may be attributed to scheduling of these slower modes (more volume allocated to non-peak hours to increase mobility).

Volume and LC Correlation

Standalone analysis for modal LCs showed little other than a very slight progression with time at both locations for motorbikes and cars at Shafique Mor. However, when correlated with volume, significant synchronization was visible between the LC and volume curves for all modes on both locations.

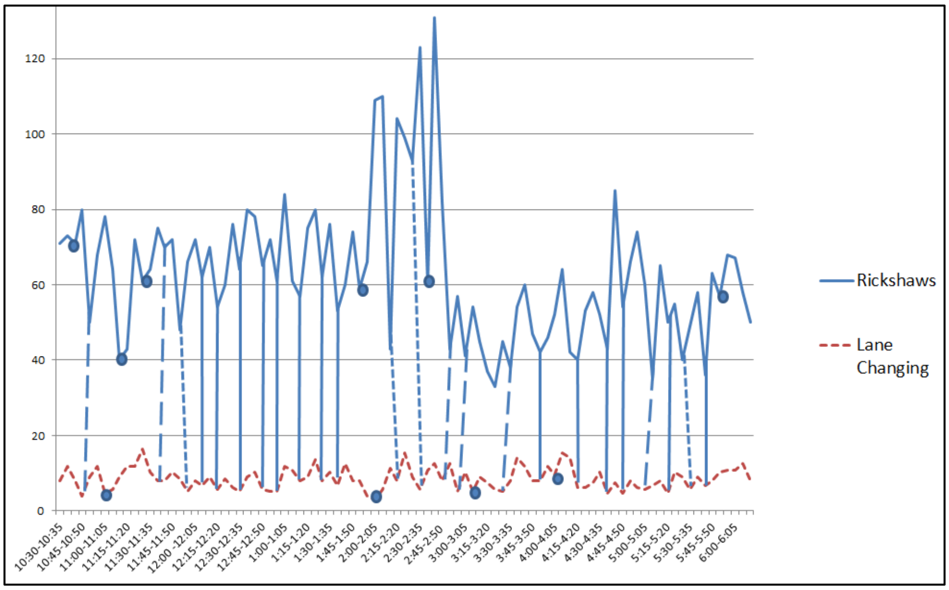


Figure 1: Correlation between troughs on rickshaw volume (solid curve) and LC (dashed curve) for Aladin Park.

Figure 1 shows the manner in which the LC and volume curves were analyzed in the case of rickshaws at Aladin Park. The two curves are connected using three different kinds of connections:

1. Solid lines: connecting troughs occurring within 5 minutes of each other
2. Square dotted lines: connecting volume troughs that occur 5-10 minutes before an LC trough
3. Dashed lines: connecting volume troughs that occur 5-10 minutes after an LC trough

Additionally, dots were used to mark troughs that were not found to occur within 10 minutes of a trough in the other curve. After connecting the corresponding peaks and troughs for all modes at both locations as described above, it was clear that for most modes at Aladin Park, a peak in LC occurred within 5 minutes of a peak in volume (or a trough in LC occurred within 5 minutes of a trough in volume) the vast majority of the time. Cars and rickshaws seemed to experience this correlation the most, with 48.6% of all peaks in volumes and LC (and 41.8% of all troughs) occurring within 5 minutes for cars, and 45.9% of all peaks and 50.8% of all troughs occurring within 5 minutes for rickshaws. These peaks or troughs were connected with solid lines. Roughly the same trends were observed at Shafique Mor, but with a greater percentage of lagged correlations (peaks or troughs in one curve occurring between 5-10 minutes of a peak or trough in the other curve).

There was also a small degree of quantization between the distribution of different connections in some of the modes at both locations. At Aladin Park, cars exclusively had connections with less than 5 minutes of lag (as shown by the solid connections) from 10:30 till 11:30 – a low volume, high speed phase. This was followed by a 1-hour phase of almost exclusively connections with a positive lag of 5-10 minutes (dashed connections). Aladin Park, a location with a straight, wider road, and higher volume with more volume fluctuation, had much more quantization than Shafique Mor. Although the quantization of lag within 5 minutes often occurs in periods of low, constant volumes, while a positive lag of 5-10 minutes appears to favor periods of high volume and fluctuation, the process on the whole appears randomized, especially for negative lag of 5-10 minutes (square dotted connections). Table 1 shows a summary of the curve correlation data.

The first data point for Aladin Park may be interpreted in the following manner: assuming that the different lag amounts are randomly distributed (there is no quantization), there is a 48.57% chance that a peak in car LC and a peak in car volume will occur within 5 minutes of each other. There is a 25.71% chance that a car LC peak will be followed by a car volume peak within 5-10 minutes. Occasionally, one point (peak or trough in either the LC or volume curve) was found to have 2 connections (for example, a peak in volume occurred within 5 minutes of an LC peak and between 5-10 minutes of it). These double connections may be taken to be a source of error, with only one of these connections considered valid.

Table 1: Summary of correlation between LC and volume curves at both locations

Vehicle		Unaccounted Peaks/Troughs				Connection Types								Percentages									
	SM = Shafique Mor	Volume		LC		Solid		Dashed		Square Dotted		Double		Percent unaccounted		Percent Solid		Percent Dashed		Percent Square Dotted		Percentage double connections	
	AP = Aladin Park	SM	AP	SM	AP	SM	AP	SM	AP	SM	AP	SM	AP	SM	AP	SM	AP	SM	AP	SM	AP	SM	AP
Car	Peaks	6	7	6	11	12	17	6	9	5	0	0	2	20.69	25.71	41.38	48.57	20.69	25.71	17.24	0.00	0.00	5.71
	Troughs	4	1	7	6	6	14	11	16	9	0	2	3	17.46	10.45	19.05	41.79	34.92	47.76	28.57	0.00	6.35	8.96
Bike	Peaks	6	8	4	3	9	7	6	9	7	7	1	0	18.52	19.30	33.33	24.56	22.22	31.58	25.93	24.56	3.70	0.00
	Troughs	6	6	5	3	10	13	6	8	6	2	0	0	20.00	16.36	36.36	47.27	21.82	29.09	21.82	7.27	0.00	0.00
Rickshaw	Peaks	4	6	5	3	12	14	5	10	3	2	1	0	18.37	14.75	48.98	45.90	20.41	32.79	12.24	6.56	4.08	0.00
	Troughs	2	6	3	4	12	15	5	6	3	5	2	0	11.11	16.13	53.33	48.39	22.22	19.35	13.33	16.13	8.89	0.00
Bus/Truck	Peaks	4	6	6	8	10	7	7	9	7	6	0	2	17.24	24.14	34.48	24.14	24.14	31.03	24.14	20.69	0.00	6.90
	Troughs	5	10	6	9	10	4	7	8	6	4	0	0	19.30	37.25	35.09	15.69	24.56	31.37	21.05	15.69	0.00	0.00
Combined	Peaks	5	5	3	4	7	17	8	6	7	2	3	0	15.38	15.25	26.92	57.63	30.77	20.34	26.92	6.78	11.54	0.00
	Troughs	5	4	4	7	4	9	10	8	5	6	2	0	19.15	19.30	17.02	31.58	42.55	28.07	21.28	21.05	8.51	0.00

Inversions

If a trough in the LC curve is connected with a peak in the volume curve (or vice versa) in the same 5-minute time segment (i.e. with a vertical line), it is counted as an inversion. It may or may not be an unaccounted trough/peak. Inversions are a counter-argument to the theory that LC and volume are proportional in their responses to each other (an increase in one causes an increase in the other, and same for decreases). By recording instances where inversions occur, those periods where this theory breaks down may be identified. A possible explanation for inversions is presented in Figure 2 below:

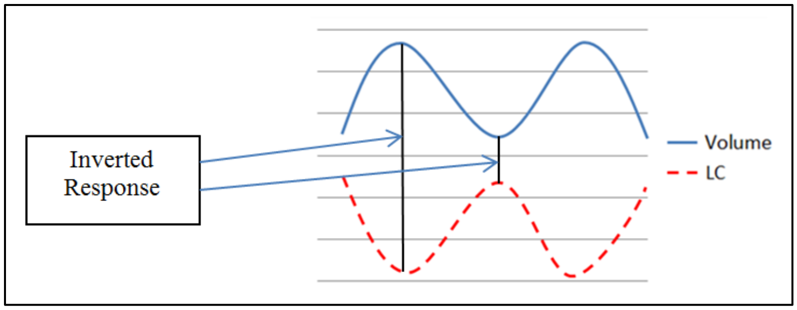


Figure 2: As LC increases, speed is reduced and congestion occurs. This causes volume to fall. With no space to maneuver, LC falls and volume improves. This cycle is typical during stop-start motion.

Inversions across modes at Aladin Park were largely synchronized. The inversions for cars, motorbikes and the total volume were concentrated in the same time periods, from 12:50 till 14:00 and from 16:30 till 18:00 (volume peaks during both periods due to school and office closure). Bus and truck inversions were concentrated from 15:30 till 18:35. At Shafique Mor, car inversions largely coincided with a sharp decrease in volume (14:00 till 16:20) and the evening peak (18:10 till 18:40). Motorbike inversions were staggered, but appeared concentrated at a low volume period from 11:40 till 12:15 and at the evening peak. Rickshaw (Figure 3) and total volume inversions were mostly found in the period from 15:15 till 17:00. Bus and truck inversions were much more abundant and mainly found from 13:30 till 15:35.

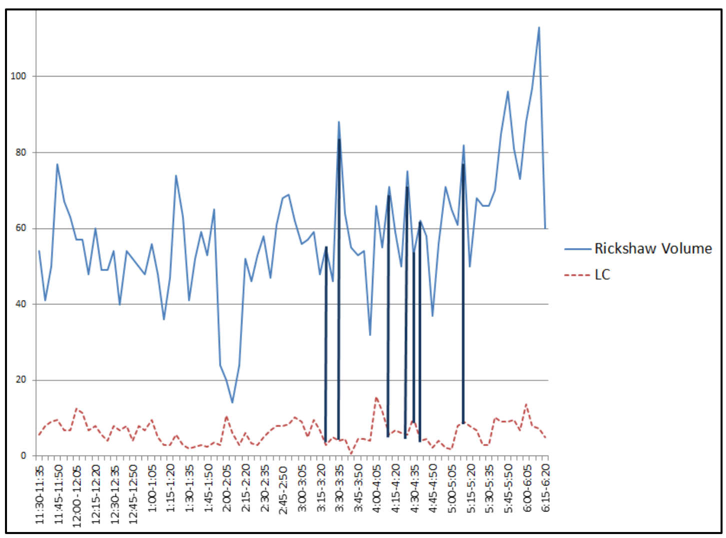


Figure 3: Inversions between rickshaw volume and LC at Shafique Mor

Modal Analysis

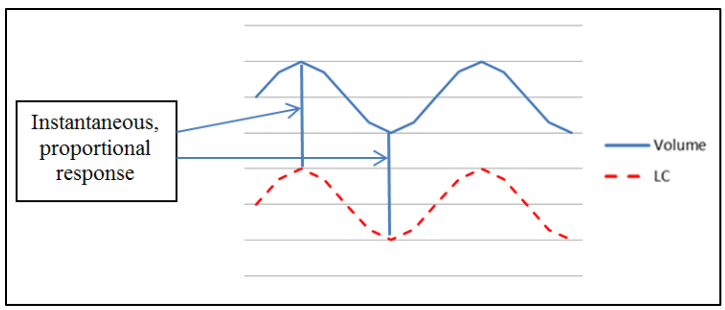
The observations for LCs and volumes have been found to vary not just across modes but also with location. There have also been several consistencies between the trends followed by different modes at both locations, especially between cars and rickshaws. The following analysis attempts to interpret and explain the trends described above:

Cars

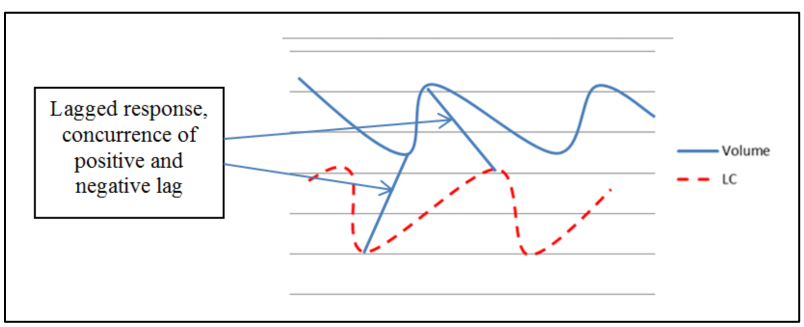
For cars at both locations, most of the connections between the LC and volume curves have less than 5 minutes of lag and between 5-10 minutes of positive lag. Corresponding peaks in the two curves mostly occur within 5 minutes (solid), while corresponding troughs are mostly between 5-10 minutes (dashed). This hysteretic behavior may be because of high volumes during these connections. At Aladin Park, most of the dashed connections occurred during 11:20 till 12:20 and 12:55 till 12:50, which was when the highest total volumes were recorded. At Shafique Mor, dashed connections are bunched from 13:30 till 15:00, which is when car volumes were at their highest. There is also a concentration of square dotted connections (5-10 minutes of negative lag) in the same period, some of which are double connections. The double connections mean that an LC trough may correspond with a volume trough 5-10 minutes later or with a volume trough that occurred 5-10 minutes earlier. Either way, assuming that the corresponding troughs are joined by a cause-effect relationship, this suggests that during periods of high car or total volume, the period between corresponding features in both curves increases. A possible explanation is given through a proposed model of volume-LC interaction in Figure 4.

Since at both locations traffic does not ever come to a complete stop, it follows that a trough in LC may be caused by a fall in volume. In other words, no cars are changing lanes because too few car drivers are on the road, with no desire to change lanes since speeds are high. However, in a congested scenario, an opposite traffic state may also generate troughs in the LC and volume curves simultaneously: If no vehicles are moving due to gridlock, volume and LC will both be minimum.

It is postulated that at both locations, conditions similar to state (b) in the figure below prevailed during the majority of dashed connections due to high volumes, and they may be the reason for the increased lag between troughs in the two curves. This is supported by the fact that most of the square dotted connections for cars occur together with dashed connections at Shafique Mor when car volumes are at their peak.



State a: Low density, high speed conditions. Increased lane changing causes peaks in volume; resulting voids in traffic stream reduce density, decreasing the volume and the need for lane changing, causing troughs in both curves.



State b: High density, low speed. LC falls abruptly when congestion emerges, contributing to the gradual decrease in volume. Volume improves abruptly as congestion clears up, and drivers resume lane changing after a while, when sufficient speed has been regained. As a result, there is a lag between the two curves.

Figure 4: Traffic flow states during which different types of lag will be observed

Car inversions at Aladin Park are concentrated from 12:55 till 14:30 and from 16:25 till 18:00. Four events occur during both these periods:

a. an increase in motorbike, rickshaw and total volumes,
b. a concentration of inversions for motorbikes and total volume,
c. an increase in car LC, and
d. a gradual decrease followed by recovery in car volume coinciding with a gradual increase followed by decrease in car LC.

Interestingly, there is no increase in motorbike or rickshaw LC, with total volume LC increasing only in the second period. At Shafique Mor, car inversions are concentrated from 13:55 till 16:20 and 18:10 till 18:40. During the first period, rickshaw, bus and truck, and total volume inversions are also concentrated, although no clear correlation with volume can be observed. During the second period, all modes have peak volumes (although car volumes had a higher peak at around 14:00), and motorbike and bus and truck inversions are concentrated. Furthermore, there is a high coincidence between square dotted and dashed connections (13:30 till 16:15) and inversions (13:55 till 16:20). If the above model is correct, this suggests that when the traffic approaches a state of congestion, an increase in both inversions and lagged connections (square dotted and dashed) occurs.

Motorbikes

Motorbikes at both locations displayed high levels of deviation from the trends observed in other modes. They had the highest total numbers of unaccounted peaks and troughs (an average of 36% over both locations) and the lowest levels of volume-LC connections within 5 minutes (solid connections) compared to other modes. As expected, motorbike LC is not very responsive to hourly fluctuations in volume. Due to the maneuverability of the motorbike, its LC depends little on the prevailing traffic conditions – other than during peak hours and low volume, the LC remains almost constant.

This extra maneuverability may also explain the other trends. At both locations and across modes, motorbikes have the highest amount of negative lag (square dotted connections). However, since negative lag appears during both peak hours and low flows, quantized and dispersed, it is unclear what exactly triggers it. The high level of dispersion may suggest that motorbike flows switch between near-congested and free flow very fast. This makes sense – even during periods of stop-start or congested flow, motorbike volumes are maintained due to their ability to filter between other modes. Other lag connections are also present in considerable amounts, but there is little quantization between similar connections. Inversions are also staggered and appear during periods of both low and high volume at both locations. The scattered nature of these findings reflects the anomalous nature of the motorbike compared to other modes, particularly its ability to maintain volumes and LCs during congestion. Furthermore, at both locations the relative distribution of connections and inversions is fairly consistent, suggesting that motorbike behavior is independent of location as well.

Rickshaws

At both locations, rickshaws exhibited similar distributions in lag, and had the highest amount of solid connections among all the modes, particularly during periods of moderate, stable volume. From 12:10 till 13:40 at Aladin Park and 11:45 till 13:55 at Shafique Mor, rickshaw volumes were moderate and connections were uniformly solid, suggesting that state (a) prevailed during these times. At Shafique Mor, peaks and troughs had almost equal distributions of each type of lag, and inversions were tightly grouped from 15:20 till 15:35 and from 16:10 till 16:40, in sync with total volume inversions, bus and truck, and some car inversions. Rickshaw volumes during this period experienced a sudden increase, and car LC was at its peak. A concentration of dashed/square dotted (lagged) and double connections was visible during this period.

At Aladin Park, lagged connections mostly occurred during periods of volume fluctuation. Inversions were grouped together from 10:45 till 11:40 (during which moderate fluctuation in rickshaw volume and moderate volumes of other modes were observed) and from 14:00 till 15:30 (a period marked by a huge peak and fall in rickshaw volumes). While the first period is unremarkable, the second shows a strong correlation between fluctuations in volume and rickshaw inversions. It is notable that no rickshaw inversions were present during peak hours at both locations, suggesting that it is change in volume rather than absolute volume that triggers rickshaw inversions.

Buses and Trucks
This mode included all vehicles that were larger than those in the car category. Further classifying these vehicles into separate categories proved to be difficult since there existed many varieties of trucks and buses. Buses included contract carriages (which mostly traversed the fast lane) and public buses (which routinely weaved between the slow and middle lanes to pick and drop passengers). This diverse LC behavior would mean that counting all buses as one category would introduce many errors, and it was impossible to tell these two types apart upon visual inspection. All these groups were therefore lumped together in this category to represent all large vehicles with average to above average speed.

On both locations, similar trends in volume and LC were apparent – steady volumes with a noticeable decrease centered at 13:00 and 16:00. LC was higher in the periods with low volume, suggesting that state (b) (high density, low conditions) prevailed during this time. The different lag types for this category most closely resemble that of motorbikes, both in distribution over time and relative to each other. While the disparity in size and maneuverability between these two categories is substantial, larger modes are often yielded to during congestion, which allows them to have better flow than cars and rickshaws. This means that motorbikes and trucks/buses have better than expected mobility in congested states.

Both categories also less sensitive to speed loss due to potholes (motorbikes are able to maneuver around potholes more easily, while buses/trucks have heavier suspensions which allow them to move faster than other modes on poor roads). Inversions are also coincident with dashed and square dotted lag connections, although there is little correlation with inversions or LC/volume anomalies in other modes. Therefore, even during the early part of the day when state (a) clearly prevailed, the inversions and lag for this category indicated that their flow was stop-start. This problem was solved by reviewing the videos – the speeds of buses and trucks were significantly lower than that of other modes. Therefore, the lag connections and inversions for buses and trucks indicated state (b) even though the flow is congestion free.

Conclusions

While the curves appear to have correlation (Table 2), it is problematic to declare any cause-effect relationship between LC and volume. This is illustrated by the conflicting nature of the following well-known traffic behaviors:

1. High volumes (resulting in low average speed) may cause vehicles to change lanes in order to increase their speed (increasing volume increases LC).
2. The greater the volume, the greater the probability that some of the vehicles will change lanes for reasons other than increasing speed (increasing volume increases LC).
3. The sudden influx of vehicles at peak hours, which has nothing to do with lane-changing.
4. On the other hand, lane changing also increases volumes by decreasing density (increasing LC increases volume).

If, for example, LC peaks are attributed as the cause of volume peaks, it will be impossible to explain the several instances when LC peaks occur after the corresponding volume peaks. Rather than simple cause-effect, the relation may be one of interdependency, as illustrated in state (a) and state (b) above.

Table 2: Correlations between car and rickshaw inversions and other parameters during periods of high inversions. A tick for LC or volume signifies that the inversions occurred during a period of unusually high or low count for the appropriate parameter.

Rickshaw Inversions																
			Rickshaw		Motorbike			Car			Bus and Truck			Total		
Location	Period	Lagged (Dashed/Square Dotted) Connections	Volume	LC	LC	Volume	Inversions	LC	Volume	Inversions	LC	Volume	Inversions	LC	Volume	Inversions
Aladin Park	10:45 – 11:40	✓	✓			✓							✓			
	14:00 – 15:30	✓	✓			✓			✓		✓	✓	✓		✓	✓
Shafique Mor	15:20 – 15:35	✓						✓	✓	✓		✓	✓			✓
	16:10 – 16:40	✓				✓							✓		✓	✓

Car Inversions																
			Car		Motorbike			Rickshaw			Bus and Truck			Total		
Location	Period	Lagged (Dashed/Square Dotted) Connections	Volume	LC	LC	Volume	Inversions	LC	Volume	Inversions	LC	Volume	Inversions	LC	Volume	Inversions
Aladin Park	12:55 – 14:30	✓		✓		✓	✓		✓						✓	✓
	16:25 – 18:00	✓		✓		✓	✓		✓		✓		✓		✓	✓
Shafique Mor	13:55 – 16:20	✓								✓		✓	✓			✓
	18:10 – 18:40		✓			✓	✓		✓		✓	✓	✓		✓	

The following conclusions may be drawn from the above analysis and observations:

1. Inversions will coincide with periods of sustained lagged connections (dashed and square dotted). These connections commonly occur during periods of congestion.
2. Steady volumes, whether high or low, will favor solid connections.
3. Regardless of location, bus/truck and motorbike connections and inversions will be highly randomized.
4. Buses and trucks will have high levels of inversions and lagged connections even during periods of low flow due to low speed. Cars, rickshaws and the total volume have similar distribution of connections.
5. For solid and dashed connections (especially dashed), the proportion of peaks to troughs is often comparable. In some cases (most of which were at Aladin Park), there is a large difference in the relative distribution of the two, but it is almost always less than 2:1.
6. A conglomerated analysis of the traffic stream reveals that locations with high speeds and volumes and low densities (in this case Aladin Park) have predominantly solid connections, while low speed, high density locations have predominantly lagged connections.

These findings shed light on key aspects of urban traffic behavior that has rarely been measured in quantitative form, and may help predict traffic congestion. As information about speed and density variation is combined with this analysis, and the modes are defined and counted more accurately with smaller time intervals, a significant improvement in this study area can be expected.

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Policy Development For the Integration of Building Information Modeling Into Construction Management Education

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Abstract

The need of teaching innovative technologies and tools in Construction Engineering and Management (CE&M) programs has been stressed globally and cannot be ignored. The literature review of the international construction industry and academia indicates the significance of teaching the innovative Building Information Modeling (BIM) technology in CE&M education. A recent study shows that academic institutions are generally slow to adopt changes, especially if it is forced by a continuous change of new technologies. The aim of this research is to develop an education policy which invites all the Architecture, Engineering and Construction (AEC) stakeholders, particularly from higher education institutions to develop their action plan for the integration of BIM into CE&M education. A detailed literature review was performed for this purpose and identified those factors that affect the successful integration of BIM. The output of this research will provide some key policies and their implementation strategies to the higher education institutions so that future students can meet the local and international industry needs.

Keywords

Building Information Modeling (BIM), Construction Engineering and Management (CE&M), Construction Industry, Construction Education, Policy Development

1. Introduction

Nowadays various design firms, construction management firms, consultants, and other organizations use Building Information Modeling (BIM) in their construction offices to facilitate Construction Engineering and Management (CE&M) processes and services (Smith 2014). Most of the Architecture, Engineering, and Construction (AEC) firms rely on BIM knowledge and skills to compete for business, which motivates academicians to implement BIM in CE&M programs (Lee and Yun 2015). Many researchers have also recognized the need to teach BIM to equip CE&M graduates to meet the requirements of the construction industry and to teach complex CE&M topics such as constructability analysis, life-cycle costing, etc. as well. A research study (Wang et al. 2014) has revealed that CE&M graduates often face difficulties to use classroom knowledge while dealing with the complexities and uncertainties of real life projects. As a solution, the incorporation of BIM to CE&M curriculum will provide an opportunity to students to use simulation and visualization tools of BIM to the experience the real-world projects.

Considering this benefit and also to satisfy the AEC industry needs, several universities worldwide are offering BIM related courses in CE&M programs, and many are in the process of integrating BIM-based knowledge into their curriculum (Ahn et al. 2013). On the other hand, research studies (Badrinath et al. 2016) and (Suwal et al. 2014) have highlighted the impediments that need to be considered before incorporating BIM in CE&M programs. To learn more, the authors conducted a literature review to ascertain the status of BIM education, particularly of CE&M programs in Australia, in the USA, and in Pakistan. Research studies have identified numerous barriers, and those have been combined into four major categories as shown in Table 1.

Table 1: Factors Affecting the Successful Integration of BIM into CE&M Curricula

Research Factors	Reference(s)
Factor 1. BIM Awareness and Academic Environment	
• Lack of knowledge of BIM in Academia	(Panuwatwanich et al. 2013)
• Rigid attitudes of faculty members to change the existing curriculum	(Lee and Yun 2015)
• Lack of suitable materials for BIM related training	(Sacks and Pikas 2013)
Factor 2. Lack of Resources to Integrate BIM into Academic Environment	
• Shortage of trained BIM faculty members	(Abbas et al. 2016)
• Little information about the content, principles, and methods of instruction of BIM	(Sacks and Pikas 2013)
• Lack of time and resources to prepare a new curriculum	(Gerber et al.2011, Sacks and Pikas 2013)
• Unavailability of space in conventional CE&M curriculum to include new courses	(Panuwatwanich et al. 2013)
• Lack of financial resource as BIM teaching requires high cost IT equipment	(Hedayatia et al. 2015)
• Deficiency of accreditation standards	(Gerber et al. 2011, Panuwatwanich et al. 2013)
• Paucity of BIM Handbook & training material	(Panuwatwanich et al. 2013)
• Small number of interested student	(Gerber et al. 2011)
Factor 3. Collaboration with Industry Stakeholders	
• Less understanding of skills needed in the industry	(Sacks and Pikas 2013)
• Lack of industry involvement	(Panuwatwanich et al. 2013, Wu and Issa 2013)
Factor 4. Complex BIM Tools	
• Modeling demands expert construction knowledge	(Magiera 2013)
• Constant software upgrades	(Gordon et al. 2009)
• Intricacy of new software tools	(Lee et al. 2013)

All above-mentioned barriers are obviously affecting the integration of BIM in construction programs in Pakistani universities. Also, the research on BIM related topics is in its beginning in Pakistan (Hussain and Choudhry 2013). A recent study (Abbas et al. 2016) noted that most of the Pakistani AEC universities are interested in teaching BIM through their CE&M programs, but in fact, only 41.37 percent of the total AEC universities teach or discuss BIM content at the undergraduate, graduate level. The most of the universities are struggling because there is a shortage of faculty members interested in BIM. Moreover, lack of industry involvement and conventional CE&M education structure are the other top barriers to teaching BIM.

All of the above-mentioned research indicates that CE&M universities are facing challenges in the BIM integration process; therefore, the CE&M universities desire a clear policy plan to fully integrate BIM into the curriculum. Research studies of (Hedayatia et al. 2015) and (Demirdoven 2015) also reiterated that although several research efforts highlight the importance of BIM education, its benefits, and the barriers to implementing BIM tools, the practical strategies to eliminate these obstacles have rarely been discussed. The proposed BIM policy plan and implementation strategies in this paper will serve as a strong foundation for BIM implementation and to overcome a hurdle in teaching and learning BIM. The main objective of this research is to develop an education policy plan based on the expectations of AEC construction industry professionals working on BIM related tasks. This study will provide a foundation to educators who desire to include BIM in CE&M programs and also provide a guideline that will be helpful in the implementation of BIM content so that CE&M students can polish their skills up to the industry requirements.

2. Objectives and Scope

The scope of this research is to develop a policy plan, which would suggest all the AEC stakeholders, particularly AEC universities to develop their action plans for the integration of BIM into CE&M education. The following objectives were set for this study:

1. To identify the barriers that affect the higher institutions in the integration of BIM into construction management education.
2. To develop a policy for minimizing these barriers.
3. To develop implementation strategies according to the policy.

3. Methodology

To develop a policy plan, a preliminary study leading to detailed literature review was done. The main barriers to integrating BIM into CE&M education were identified from the literature and were combined into four major categories such as BIM awareness and academic environment, lack of resources to integrate BIM in an academic environment, collaboration with industry stakeholders and complex BIM tools. Afterward, policy outline and strategies were developed for the key individual category.

4. Research Results - Policies and their Implementation Strategies Discussion

After applying the policy plan development steps as mentioned previously, this research has resulted in the following aspects.

4.1 BIM Awareness and Academic Environment

4.1.1. Policy statement

The AEC universities will make increased efforts to promote awareness about the significance of BIM technologies and tools and integrate BIM tools in estimating, planning and scheduling, safety, and project management subjects at the undergraduate and post-graduate level.

4.1.2. Strategies to implement policy

The traditional resistance during the integration of BIM in higher education will be eliminated by creating awareness and with the help of organizing different workshops in AEC universities that will produce some audio-visual material. Also, all these workshops course content, study materials, tutorials, and lectures will be uploaded to the conference website. At an initial step, AEC universities can present different short duration training to faculty members and CE&M students regarding BIM that will pave the way to increase awareness in CE&M programs. To conduct introductory training of the CE&M students and faculty members, the authors proposed a crash-course on BIM of 20-hour duration as shown in Table 2.

Table 2: BIM Crash-Course Contents

Lecture #	Duration	Course Contents
1	3 hours	Lecture on Introduction to BIM + Introduction to Autodesk Revit User Interface
2	3 hours	Architectural Modeling
3	3 hours	Structural Modeling
4	3 hours	Mechanical, Electrical, and Plumbing (MEP) Modeling
5	3 hours	Project Documentation + Quantity Take Off + Sheet Generation
6	3 hours	Autodesk Navisworks Introduction + 4D Scheduling + Clash Detection
7	2 hours	Energy Analysis

The major learning objectives of this BIM training course are to comprehend the concept and functions of BIM and its tools including:

- Develop architectural, structural and Mechanical, Electrical and Plumbing *(*MEP) BIM Models.
- Perform Analyzes such as Clash Detection, Energy Analysis, Solar Study, Lighting Analysis, Quantity Take Off, 4D Scheduling using BIM tools
- Project Documentation for design, execution and facility management operations using BIM tools.
- Collaborate and Coordinate with multiple disciplines involved in a construction project using BIM tools.

4.2 Lack of Resources to Integrate BIM in Academic Environment

4.2.1. Policy statement

The higher professional accreditation bodies will ensure appropriate measures to improve the quality of education. It will take appropriate measures to include BIM-based contents coherent with the local and international AEC industry requirements. Teachers will be given a particular training regarding the BIM tools.

4.2.2. Strategies to implement policy

Universities will establish a monitoring and evaluation practices to ensure continuous improvements in curriculum and to achieve the learning outcomes accordingly. Also, it is extremely important that AEC universities develop a BIM course or other best strategy to incorporate BIM into CE&M curricula. However, integration of BIM in CE&M is not easy as the learning, and teaching approaches can vary depending on the level at which the BIM skills are being educated. So, in the light of the recommendation of Barison and Santos (2010b), the authors highlighted three basic skill levels such as an introductory, intermediary and advanced level for BIM education learning and teaching purpose. At the introductory level, CE&M students can learn basic BIM tools and can investigate basic concepts of modeling and can further comprehend different manners and can easily communicate different types of information. At the intermediary level, CE&M students can learn about BIM tools and advanced modeling techniques which can help to build structural systems and to determine more features in BIM tools. And at the advanced, CE&M students should have knowledge of BIM professional practice, construction methods, and construction material level.

Further, the universities cannot achieve the desired outcomes without developing an accreditation BIM standard so to gain the desired results the higher education bodies can implement incentive/ penalty based system to warn the CE&M universities to integrate the latest innovative BIM-based content into their AEC programs. To equip the faculty members according to the international standards, higher education bodies can start the competency and short-term foreign faculty hiring programs with the local and the international supports.

4.3 Collaboration with Industry Stakeholders

4.3.1. Policy statement

The AEC universities will collaborate with construction industry stakeholder and use all possible resources to minimize the gap between the construction industry and academia.

4.3.2. Strategies to implement policy

The yawning gap between academia and state-of-the-art industry demands can be filled by conducting a various research to capture the current AEC industry needs and by undertaking some changes in traditional construction program structures. Further, the mindset of construction industry professionals can be changed by realizing the potential benefits of BIM adoption over 2D drafting, and professional education expert committee can play the vital role to achieve this task and can bridge the gap between industry and academia.

4.4 Complex BIM Tools

4.4.1. Policy statement

The AEC universities will develop national standards for the integration of BIM tools in CE&M education.

4.4.2. Strategies to implement policy

The BIM integration guidelines will provide a direction to update the CE&M education curriculum. However, the integration of BIM in CE&M curriculum does not only mean to provide the software training and theoretical knowledge, but even more to developed other skills such as communication skills, teamwork skills, and management skills within a BIM model to support the construction industry.

5. Conclusions

The main contribution of this study is a BIM integration roadmap for CE&M programs, so that when university graduates enter into a professional career, they have sufficient knowledge of the latest innovative technologies and tools according to the current industry demands. This research is expected to provide foundations to the educators in higher education institutions where they have already integrated BIM or under the process of developing BIM-based education for CE&M programs. The BIM will enhance the skills of students of plan reading, cost estimation and solving analytical problems. Such knowledge of BIM will enable graduates to get good jobs in the local and international market and facilitate society by providing latest BIM innovative tools based solutions as a bright side of civil engineering.

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Suitability of Various Roof Materials as Thermal Insulation

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Abstract
Global warming is an international issue of the present era. The plains of our country Pakistan are tropical region where temperature soars even beyond 52 ^{0}C in summer causing a lot of discomfort and even deaths because of sun stoke during the months from May to August. As a matter of fact the major source of heat inside a building is the radiation through the roof which is directly exposed to sun shine. Hence ways and means must be adopted to resist this radiation. Attempts have been made to keep the room temperature within tolerable limits with the help of different materials such as clay, ceramics, marble dust and lime as insulators. However, the effectiveness of hollow blocks is not quantified so far. Therefore this study is aimed at finding the suitability of hollow blocks/sections as heat insulation. During this experimental study hollow blocks with different sizes were cast with OPC, white cement with marble dust and white cement with ceramics powder, because white materials reflect the heat more readily than absorbing it. Total twenty seven blocks were cast and tested. In some cases space in the hollow sections was filled with wood saw-dust. The results were encouraging. A minimum temperature difference of 09^{0}C was achieved in case of hollow space without wood chips, when top temperature was 52 ^{0}C and bottom temperature was 44 ^{0}C. Filling the hollow space with wood saw-dust showed significant effect on temperature difference which was observed to be as high as 11 ^{0}C. Here we consider the peak hours of the day i.e. from 2:00 pm to 3:00 pm.

Keywords
Clay; ceramics; marble dust; lime and hollow block

1. Introduction

Now-a-days it is talk of era that global warming is increasing day by day because the average temperature of earth is increasing. Our country is tropical region where buildings need great capability to protect against day time heat and night chill. In the plains of this country the temperature some times exceeds 50 degrees Centigrade. It is very hard to survive in this situation without bringing down the temperature within tolerable limit. Every year many deaths are reported from various parts of Pakistan because of scorching heat during the months of May to August. Since last few years Nawabshah has remained the hottest place in Pakistan. The major source of heat inside the buildings is the heat radiation through the roof because of direct exposure to sun-shine. Therefore structure should be so designed that it is the most suitable for the climatic conditions of the region.

A lot of industrially produced materials are available in the market to provide thermal insulation. Thermo-pore, Timber sheets, or artificial sheets like plywood, chipboard veneer board and hard-board are all good heat-insulators and can therefore be used for the prevention of heat, but there are so many short-comings. A false ceiling, beneath the original one, is the most common and major method to prevent the conduction of heat from the top. Sometimes, a thick layer of clay and tile-flooring of baked clay are provided on the roof for the same purpose. Felts of various types, originally designed as water–proofing media for roof, also partly help to reduce the temperature inside the rooms. However, all these methods are quite expensive and beyond the reach of the common man.

In the past, buildings were kept cool by constructing ceilings of baked clay tiles, supported by a skeleton of wooden strips and beams or steels bars and girders. A thick layer of straw was placed over the tiles, upon which several inches of clay layer was laid and then it was plastered over with clay-mortar or sometimes lime-mortar. This was quite effective but, nowadays, RCC slabs are constructed, which conduct heat more readily, causing the inside temperature to shoot up strikingly. The natural material, i.e. clay could probably prove to be the most economical and effective material to prevent the heat summer form affecting the room-temperature through direct exposure of the roof to the sunshine.

Experimental studies are being conducted for suitability of white materials like lime, white cement, white marble dust, for their suitability as heat-reflectors to be employed on the roof. However it is an established fact that white materials reflect heat rather than absorbing or transferring it to the inner atmosphere of buildings. An attempt was made to study the suitability of common clay tiles, both sun-dried and baked as thermal insulation (Mahmood *et al*). However, the clay is not reflecting material. Therefore, it was expected that materials like ceramic powder, marble-dust and lime could give better results in this regard. The main object of this study was to find the best possible combination of locally and cheaply available materials of construction and the ways and means to produce precast ceramic and marble dust tiles. Which could be readily used for thermal insulation in the tropical regions in order to reduce the thermal conductivity. A substantial work has already been done on various types of materials and techniques such as clay, ceramics, marble dust and wood chips etc. (Mahmood *et al* 2004, 2004 & 2005). However, hollow blocks have not been tried in order to achieve thermal insulation. Main idea of using the hollow blocks is that the difference between outside temperature and inside temperature should be maximum. The room temperature should be reduced to such an extent that any need of air-conditioning or air coolers be avoided resulting energy savings which is so vital for a country like Pakistan where there is already acute shortage of electric power. This will have an over all favourable effect on the economy of the country. It may be mentioned here that Samo and others conducted study on prediction of the cooling energy requirement in buildings using the degree-days method (Samo *et al* 1999 & 2000).

2. Present Study

The authors have been involved for many years in conducting experimental studies (Mahmood et al, 1992, 1994, 1995, 1996, 1996, 1999, 1999, 2000, 2000, 2002 & 2004) on indigenous materials of construction to determine various properties, which are not only important from structural point of view, but several other aspects as well. The present experimental study was carried out continuously for a number of months during summer from 02 P.M. to 03 P.M. The size of tiles was 4.5 x 12 in. however, thickness was the major parameter. The second parameter was whether the tiles were unbaked or baked. The third parameter was the combination of these two tiles. A number materials like marble-dust, lime and ceramics powder have also been tried. More than 75 ceramic, marble dust and lime specimens with different mix ratios were cast and tested. The size of the tiles was 4.5” x 12”. The thickness of all the tiles was kept constant at 2”. Not only ceramic, marble dust and lime tiles were tested but un-baked and baked clay tile were also placed beneath the other tiles to achieve better results in terms of thermal insulation. Experimental study was also conducted mainly to study thermal insulation of various hollow blocks having different sizes of hollow sections and with different materials. The major parameter of this study is to investigate the effectiveness of hollow space as thermal insulation and also its best suitable size.

The other parameter is the type of material, while third parameter is the effect of wooden chips filled in the hollow space on heat insulation. Twenty seven hollow blocks were cast and tested for heat insulation with and without wooden chips filled inside the hollow space. The blocks were placed outside directly exposed to sunshine on stand specially manufactured for this purpose in order to have temperature measurements on the top and bottom of the blocks. The temperature measurements were recorded continuously from 2.00 P.M to 3.00 P.M. with time interval of one hour. Each test was conducted for three days. Electronic LCD type display system thermometer was used for these measurements. it i apparent that during hottest period, which was more commonly from 02.00-03.00 p.m., the temperature soared up to 47-50 degrees centigrade.

Table 1: Details of the temperature readings recording at peak hours of the day for various types of un-baked clay tiles

Time	½"			¾"			1"			1½"			2"		
	Top ^{0}C	Bott: ^{0}C	Diff: ^{0}C	Top ^{0}C	Bott: ^{0}C	Diff: ^{0} C	Top ^{0}C	Bott: ^{0}C	Diff: ^{0}C	Top ^{0}C	Bott^0 C:	Diff: ^{0}C	Top ^{0}C	Bott: ^{0}C	Diff: ^{0}C
02:00 P.M	45	44	1	45	43	2	47	45	2	47	43	4	47	42	5
03:00 P.M	45	44	1	45	43	2	47	44	3	47	43	4	48	42	6

T**able 2: Details of the temperature readings recording at peak hours of the day for various types of baked clay.**

Time	½"			¾"			1"			1½"			2"		
	Top ^{0}C	Bott: ^{0}C	Diff: ^{0}C	Top ^{0}C	Bott: ^{0}C	Diff: ^{0}C	Top ^{0}C	Bott: ^{0}C	Diff: ^{0}C	Top ^{0}C	Bott^0 C:	Diff: ^{0}C	Top ^{0}C	Bott: ^{0}C	Diff: ^{0}C
02:00 P.M	45	45	0	45	45	0	47	46	1	47	45	2	47	44	3
03:00 P.M	45	45	0	45	45	0	47	45	2	47	44	3	48	45	3

Table 3: Details of the temperature readings recording at peak hours of the day for various types of combination of un-baked & baked clay.

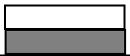
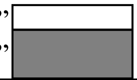
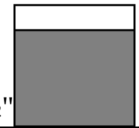
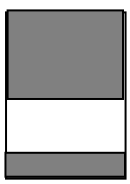
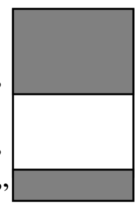
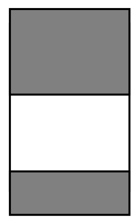
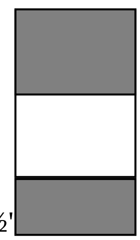
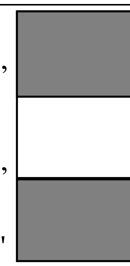
Time	 ½” ½”			 ½” ¾”			 ½” 1"			 ½” 1½"			 ½” 2"		
	Top ^{0}C	Bott: ^{0}C	Diff: ^{0}C	Top ^{0}C	Bott: ^{0}C	Diff: ^{0}C	Top ^{0}C	Bott: ^{0}C	Diff: ^{0}C	Top ^{0}C	Bott^0 C:	Diff: ^{0}C	Top ^{0}C	Bott: ^{0}C	Diff: ^{0}C
02:00 P.M	45	43	2	46	42	4	46	42	4	46	42	4	47	42	5
03:00 P.M	46	43	3	46	42	4	47	43	4	46	41	5	48	43	5

Table 4: Details of the temperature readings recording at peak hours of the day for various types of combination of un-baked & baked clay.

Time	 2” 2” ½”			 2” 2” ¾”			 2” 2” 1”			 2” 2” 1½'			 2” 2” 2"		
	Top ^{0}C	Bott: ^{0}C	Diff: ^{0}C	Top ^{0}C	Bott: ^{0}C	Diff: ^{0}C	Top ^{0}C	Bott: ^{0}C	Diff: ^{0}C	Top ^{0}C	Bott^0 C:	Diff: ^{0}C	Top ^{0}C	Bott: ^{0}C	Diff: ^{0}C
2:00 P.M	47	41	6	47	40	7	47	39	8	47	39	8	47	39	8
03:00 P.M	47	41	6	48	41	7	48	39	9	48	40	8	48	40	8

3. Results and Discussions

The results are presented in tabular form in Tables 1 to 4. These tables give a complete account of all the temperature-values measured at the top and the bottom of the tiles, along with timings. The tiles were placed outside, directly exposed to sunshine on stands, so that temperature-measurements could be recorded. Electronic LCD (Liquid Crystal Diode) type display-system thermometer was used for these measurements. Temperature-difference in each case was also determined and has been included in the tables. Not only pure clay was tried, but tiles made from a combination of clay and pit-sand in the proportion of 30 to 70 percent were also prepared. Form the tables, it is apparent that, during the hottest period, which was commonly form 2 to 3 PM, temperature soared up to 44 to 48 degrees centigrade.

Form table 1, it can be observed that thickness of unbaked clay tiles produced considerable effect on the conduction of heat; the bottom temperature was reduced by 1 degrees in case of half-inch thick tiles and this reduction went up to 6 degrees centigrade when the thickness was 2 inches.

Form table 2, it is observed that thermal insulation of baked clay tiles is less than that of unbaked clay tiles.

Tables 3 & 4 show a combination of baked and unbaked clay tiles. Here a maximum temperature difference of 5^{o}C is achieved when the baked clay tile is 2” thick, upon which a half-inch unbaked clay tile is placed. This difference goes on increasing and reaches a maximum value of 9^o when 2 inch thick tiles combination, baked + unbaked +baked, are placed. This seems to be the best possible arrangement because, when the outer temperature was 47^oC, the inner temperature was only 38^oC which may not cause too much discomfort if the electric fan is turned on. Therefore, it can be concluded that, in houses where tier-girder system is used, a total of six inches of roof- thickness is essential for keeping the rooms relatively un-affected by the heat from direct exposure of the roof to the sun. For this, first a baked clay tiles is placed between the steel tiers, upon which the layer of clay could be placed, and upon that a pavement of baked clay floor-tiles could be constructed in such a way that total thickness is not less than 6 inches. Since unbaked clay is a more effective insulation, the thickness of upper tiles could be reduced, while the thickness of clay-layer could be increased for the maximum benefit. The thickness of the lower tile may not be decreased due to gravity loads. This acts like a beam and resists flexural stresses particularly tension in the extreme bottom fibre at Centre, which is under tension and , being un-reinforced, tiles are weak in tension.

It is observed that during the hottest period which was more commonly form 02.00-03.00 pm, the temperature soared up to between 44^oC and 49^oC. Form the Tables 5-6 it can be observed that the ratio of white cement used as binding material dose not have pronounced effect on the thermal conductivity of tiles in all the cases. Comparing the temperature difference between top and bottom for tiles with marbl-dust with those of ceramics and lime, it appears that the thermal conductivity of all these three materials is about the same. when the thickness of tiles was only 2”. However, if this value is compared with previous study (mahmood *et al* 2000) for the same layer thickness of 2” of unbaked and baked clay tiles. This seems considerably higher.

Table 5: variation of temperature for 2” thick marble dust tiles

Time	Cement: Marble-Dust 1:1			Cement: Marble-dust 1:2			Cement: Marble-Dust 1:3		
	Top ^oC)	Bott: (^oC)	Diff: (^oC)	Top (^oC)	Bott (^oC)	Diff: (^oC)	Top (^oC)	Bott (^oC)	Diff: (^oC)
02:00 PM	47	42	5	47	41	6	48	41	7
03:00 PM	48	42	6	49	42	7	49	42	7

Table 6: variation of temperature for 2” thick ceramics tiles

Time	Cement: Ceramics 1:1			Cement: Ceramics 1:2			Cement: Ceramics 1:3		
	Top ^oC)	Bott: (^oC)	Diff: (^oC)	Top (^oC)	Bott: (^oC)	Diff: (^oC)	Top (^oC)	Bott (^oC)	Diff: (^oC)
02:00 PM	47	40	7	49	42	7	48	41	7
03:00 PM	48	41	7	49	41	8	49	41	8

From Tables 7-8 it appears that use of ½ “ thick layer of pure clay and ½ “ baked clay tiles show a very favorable effect in combination with white material tiles.

Table 7: variation of temperature for a total 3" thickness of cement marble dust, pure clay and baked clay tiles

Time	2" Cement : marble Dust (1 : 1) / ½" Pure Clay / ½" Baked Clay			2" Cement : Marble Dust (1 : 2) / ½" Pure Clay / ½" Baked Clay			2" Cement : Marble Dust (1 : 3) / ½" Pure Clay / ½" Baked Clay		
	Top °C)	Bott: (°C)	Diff: (°C)	Top (°C)	Bott: (°C)	Diff: (°C)	Top (°C)	Bott (°C)	Diff: (°C)
02:00 PM	49	38	11	48	38	10	48	37	11
03:00 PM	49	38	11	48	37	11	49	38	11

Table 8: variation of temperature for a total 3" thickness of cement ceramic, pure clay and baked clay tiles.

Time	2" Cement : Ceramics (1 : 1) / ½" Pure Clay / ½" Baked Clay			2" Cement : Ceramics (1 : 2) / ½" Pure Clay / ½" Baked Clay			2" Cement : Ceramics (1 : 3) / ½" Pure Clay / ½" Baked Clay		
	Top °C)	Bott: (°C)	Diff: (°C)	Top (°C)	Bott (°C)	Diff: (°C)	Top (°C)	Bott (°C)	Diff: (°C)
02:00 PM	49	37	12	49	37	12	48	37	11
03:00 PM	49	36	13	49	37	12	49	37	12

Form this it can be concluded that use of white materials which include marble-dust, ceramic powder, lime and white cement are far more effective in reducing the thermal conductivity of roof as compared with common clay. Table 9-11 presents the temperature measurements of concrete blocks observed for three days. It is apparent from this tables that temperature difference between top and bottom is more in concrete blocks with more hollow space and decreases with decrease in hollow space. The maximum temperature difference observed is 11 ^{o}C in case of hollow space of 87.5 mm and 81.25 mm at the time of hottest atmospheric temperature. Table 12 shows temperature measurement of blocks cast from white cement with marble dust. More or less it shows same trend of heat difference with respect to hollow space as it was in concrete blocks. However, the top of these blocks are becoming less hot as compared to the concrete blocks, which was expected, because of white colour of the blocks. It can be observed from the table 13 that the temperature difference between top and bottom in this case is less than that observed in previous case. The maximum difference noted here is 7 degrees centigrade. Tables 14 presents the temperature measurements of white cement with ceramic powder hollow blocks. More or less its behaviour is identical to the case of

Table9: Temperature measurements of cement hollow block.

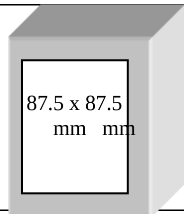
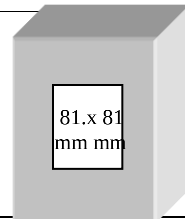
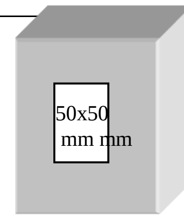
Time Hours									
	Top °C)	Bott: (°C)	Diff: (°C)	Top (°C)	Bott: (°C)	Diff: (°C)	Top (°C)	Bott (°C)	Diff: (°C)
02:00 PM	49	38	11	49	39	10	48	42	6
03:00 PM	50	39	11	49	38	11	49	42	7

Table10: Temperature measurements of white cement with marble dust hollow block.

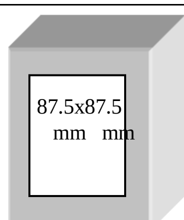
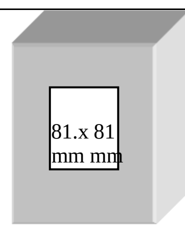
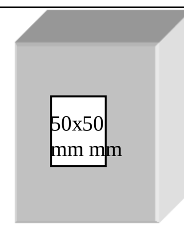
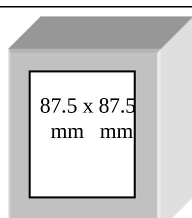
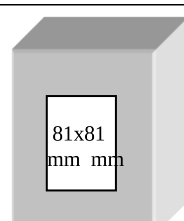
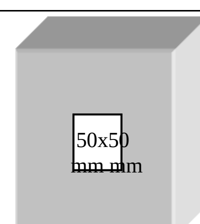
Time Hours									
	Top °C)	Bott: (°C)	Diff: (°C)	Top (°C)	Bott: (°C)	Diff: (°C)	Top (°C)	Bott (°C)	Diff: (°C)
02:00 PM	50	37	13	50	37	13	50	38	12
03:00 PM	51	37	14	51	38	13	51	39	12

Table11: Temperature measurements of white cement with ceramics powder hollow block.

Time Hours									
	Top °C)	Bott: (°C)	Diff: (°C)	Top (°C)	Bott: (°C)	Diff: (°C)	Top (°C)	Bott (°C)	Diff: (°C)

02:00 PM	47	40	7	47	39	8	47	39	8
03:00 PM	47	41	6	47	39	8	47	39	8

Temperature observation for the hollow blocks filled with wooden chips is presented in tables 12 to 14. It is apparent from these tables that the maximum difference of 14 degrees centigrade can be achieved in case of cement blocks with higher value of hollow space while it decreases to 13 degrees in case of lowest value of hollow space. However, this difference is less in the cases of white cement with marble dust and ceramic powder, which is similar to the case of these blocks without wood chips.
The concrete blocks with wooden chips seem to be best possible technique. The blocks should be placed over the top of roof so that temperature difference at the top of roof of a room should be 14 degrees. It is expected that the temperature at the bottom of the roof inside the room would further decrease by a few degrees. Therefore the total temperature difference could be up to 15 degrees between outside and inside the room.

Table12: Temperature measurements of cement concrete hollow blocks filled with wooden chips.

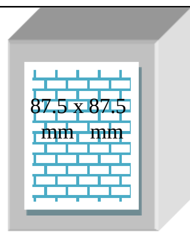
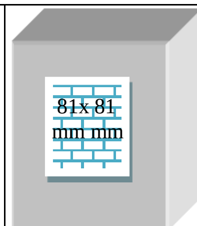
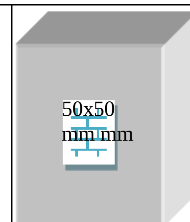
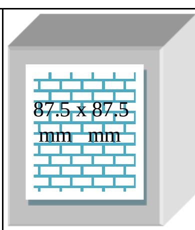
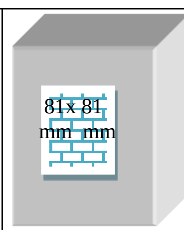
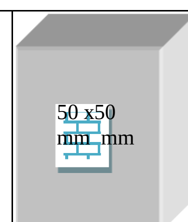
Time Hours									
	Top ^{o}C)	Bott: (^{o}C)	Diff: (^{o}C)	Top (^{o}C)	Bott: (^{o}C)	Diff: (^{o}C)	Top (^{o}C)	Bott (^{o}C)	Diff: (^{o}C)
02:00 PM	50	3	14	50	37	13	50	38	12
03:00 PM	51	37	14	51	38	13	51	39	12

Table13: Temperature measurements of white cement with marble dust hollow blocks filled with wooden chips.

Time Hours									
	Top ^{o}C)	Bott: (^{o}C)	Diff: (^{o}C)	Top (^{o}C)	Bott: (^{o}C)	Diff: (^{o}C)	Top (^{o}C)	Bott (^{o}C)	Diff: (^{o}C)
02:00 PM	48	39	9	48	38	10	48	39	9

03:00 PM	47	38	9	47	38	9	47	38	8

Table14: Temperature measurements of white cement with ceramics powder hollow blocks filled with wooden chips.

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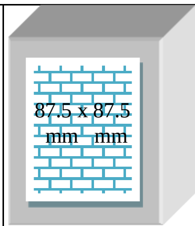
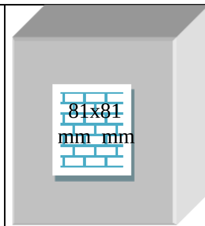
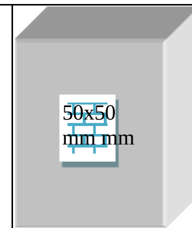
Time Hours									
	Top ^{o}C)	Bott: (^{o}C)	Diff: (^{o}C)	Top (^{o}C)	Bott: (^{o}C)	Diff: (^{o}C)	Top (^{o}C)	Bott (^{o}C)	Diff: (^{o}C)
02:00 PM	49	39	10	49	40	9	49	41	8
03:00 PM	48	38	10	48	39	9	48	39	9

4. Conclusions

(i) Unbaked clay is more effective as thermal insulation than baked clay tile.

(ii) A total of six inches thickness of roof is required to give reasonable good temperature difference, greatly preventing transfer of heat from ceiling inside the room.

(iii) A maximum temperature difference of 13"C was achieved on a day when maximum temperature on the top due to direct exposure to sun shine was 47^{0} While inside temperature was 32°C.

(iv) Marble-dust ceramic and lime tiles are far more effective as the thermal insulation than Clay and Baked Clay tiles.

(v) A total thickness of 2” of Marble Dust, Ceramic and Lime tile caused reduction of temperature averagely by 5.8^{o}C which is insufficient.

(vi) A maximum temperature difference of 11^{o}C was achieved on the day when the maximum temperature on the top due to direct exposure to the sun-shine was 49^{o}C while inside temperature was 38^{o}C for a thickness of only 2”.

(vii) Hollow blocks of concrete are more effective as thermal insulation in terms of temperature difference between up and bottom.

(viii) Hollow blocks of white cement with marble dust and ceramic powder are effective in terms of top temperature because of its white colour.

(ix) Temperature difference increases with increase in hollow space however, it is marginal, but in the case off white cement with marble dust and ceramic powder it is effect less.

(x) A minimum temperature difference of 11 ^{o}C achieved in case of hollow space without wood chips. Where top temperature was 53 ^{o}C and bottom temperature was 42 ^{o}C. Filling of hollow space with wood chips has significant effect on temperature difference, which was observed as high as 15 ^{o}C.

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Success Parameters of Bidding A Construction Project

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Abstract

Bidding is a key factor for success of any construction project. It pre-defines the execution strategy of a project. Clients have options for the selection of contractor; however, it is the contractor who vies to win the contract by satisfying client's requirement while keeping the targeted profit.

Ineffective bidding strategies are the clear cause of project failure, leading to economic decline of Pakistani construction industry. This economic downfall is propelling the contractors to bid for low or no profit.

The purpose of this research is to distinguish and identify various parameters involved in successful bidding of a construction project. Questionnaire will be developed for collecting data from response of (Large, medium and small sized) contractors. Using statistical response, data will be analyzed as Importance Index and Spearman's rank correlation.

Highlighted parameters if incorporated to fill in the gaps which are short falling in current bidding strategies, will not only benefit the contractor in winning and execution of a project, but eventually be in favor of client and construction industry of Pakistan.

Keywords

Bidding, parameters, contractor, project, construction.

1. Introduction

A successful bidding of a construction project requires a lot more than accurate pricing of material, labor and equipment. According to (Barr et al.,1990) "The procedure of bidding is expensive as it involves direct cost for information search, determinations of specifications, subcontractor solicitation, and proposal preparation." Contractor cannot take risk by randomly bidding a project. There must be some strategy for economical approach to accomplish the required work with sufficient volume at maximum profit. Usually the biddings are loosely defined and the parameters which are having higher contribution to bid are mostly ignored by contractors. At this point, there is a need to identify a set of certain factors which are having a greater contribution in formulation of those bidding strategies.
The research focuses on Pakistani Construction industry which has always been a source of economic and social benefit to the country. Pakistani construction industry is comparatively a slower contributor in National and International market, which is compelling the construction contractors to miss out the targeted profit while bidding a construction project. This study covers the success parameters for bidding a construction project from a contractor's perspective in favor of a contractor. Business, Institutional

markets and government usually vie for economical bidding and price negotiations. Highlighting the chief factors having higher impact on bidding, can be a way out for progression of effective bidding. This article pursues to magnification of viable parameters that should be taken into account while constructing a bidding approach which in turn will strengthen the construction sector in local and global markets.

1.1 Objectives

1. To review various bidding parameters used for bidding a construction project.
2. To identify the success parameters for bidding a project.
3. To highlight relative importance of success bidding parameters.

1.2 Scope

Bidding is an initial step for commencement of any project. In construction, it has a significant importance as its ignorance may eventually be the cause of project delays and cost overruns. This study aims to identify the top ranking factors leading to successful bidding of a construction project, and to pinpoint those factors having similar importance with respect to small, medium and large sized contractors as per Pakistan Engineering Council (PEC) Categorization.

2. Literature Review

Bidding is a most dominant mechanism for allocating contracts of a construction project which contributes to vigor the economy. Bidding is a legal requirement to award public and private sector contracts. To assign a contract, client compares the competitiveness of the in-line contractors. Improving competitiveness should be contractors' foremost interest. Customarily, lowest tender price is a core factor for assessment and selection of any contractor. Conversely, past study shows that lowest price weakens the projects' viability, quality and extends the duration of execution. It also emphasizes that tender price and contract time are not enough for evaluating a contract and thus their experiences and current capacity should be measured which in-turn would indicate the contractor's strength towards management, techniques, finance and reputation (Shen et al., 2004).

An extensive literature review was conducted during study period, not much information could be gathered that related to contractor's bidding strategies contributing to winning of tender.

Factors that improve the bidding strategies can be uncovered by an in-depth study (Ahmad et al., 1988).

Contractor's part is an essential element for accomplishing a project with targeted profit. Competitive contractors are more appealing to the client for effective execution of a project. As contractors' qualification is directly proportional client's goal of maintaining the schedules, budgets and quality (Virendra et al., 2016).

To indicate the importance of factors influencing bidding, there must be a proper way to gather information from the contractors who are involved in construction. One way to collect data for highlighting success parameters in favor of a contractor is to conduct questionnaire based survey. The survey's evaluation maybe based upon the scaling of 1 to 5 (indicating very low impact to very high impact). General factors such as clearness and detail of specification, past experience with similar work, workload, duration of project, competitors, market and external conditions, are shaping the bidding strategies (Barr et al., 1990).

3. Methodology

Questionnaire was first converted to online form and then was emailed to 83 professionals and experts belonging to contractors sized as large, medium and small as per PEC categories. Out of 83 floated questionnaires, 49 were received from respondents. Hence, respondents experience and perception has been documented in this study through the survey.

Table 1: Detail of conducted survey

Questionnaires Distributed	83
Questionnaires received	49
Half-filled questionnaire	4
Complete and Valid Questionnaires	45
% of questionnaire used for the study	54%

An extensive review of the literature divulged numerous factors affecting bidding of construction projects. After detailed review discussions with the professionals, involved in contract pricing, were held. This sought input into the pertinence of factors already identified to pick up the ones falling into the context of the Pakistani Bidding Systems.

Thus, based on the findings based on literature reviews (Barr et al.,1990) (Dwarika et al., 2014) (Virendra et al., 2016) (Chan et al., 1997) (Ahmed et al., 1988) (Memon et al., 2011) and interviews with various experts resulted in the identification of 42 success parameters, which were further subdivided into eight categories as project related factors, organization related factors, management related factors, labor related factors, and plant/equipment related factors, material related factors and external factors and each core category was consisting of the relevant factors.

The questionnaire was divided into three sections: Demographic information of the respondents and their firm was sought in the first and second part; third section contained 42 factors that are affecting the bidding strategies and the respondents were asked to rate each factor based on their experience from a scale ranging from 1 to 5. In this scale, 1 represents "Very Low Impact", 2 represents "Low Impact", 3 represents "Moderate Impact", 4 represents "High Impact" and 5 represents "Very High Impact". A pilot study was conducted at the preliminary stage on a sample of respondents to know the degree of clarity and to ensure the transparency of the questions along with the duration of completion of the project. After the responses were collected we made further procedure by evaluating the Importance index by using formula:

$$\text{Importance Index} = \frac{5\text{n}1 + 4\text{n}2 + 3\text{n}3 + 2\text{n}4 + \text{n}5}{5(\text{n}1 + \text{n}2 + \text{n}3 + \text{n}4 + \text{n}5)}$$

Where n1 denotes the number of respondents who answered "very High Impact", n2 denotes the number of respondents who answered "High impact", n3 denotes number of respondents who answered "Moderate impact", n4 denotes the number of respondents who answered "Low impact" and n5 denotes the number of respondents who answered "Very Low Impact". (Thomas et al., 2014).

The spearman's rank correlation was used to check the statistical relation between two variables. In this study, the relationship among the different contractors was measured ranging from +1 to -1. Where +1 indicates strong agreement/relationship and -1 presents the negative/weak relationship. The spearman correlation coefficient was used to measure and compare the ranking relation existing between large, medium and small size contractors for the eight major categories

The spearman coefficient r_s can be calculated by the following formula:

$$\text{rs} = \frac{6\sum d^2}{N(N^2 - 1)}$$

Where,

rs = Spearman's rank correlation coefficient between two groups of contractors.

d = The difference in ranking between ranks assigned to variables for each cause (large contractors and medium contractors, large contractors and small contractors, medium contractors and small contractors), and

N = the number of ranks, equals to 42 and 8 for all the success parameters and for the main categories of Construction bidding, respectively. (Ayudhya et al., 2011).

4. Data Analysis

From the sample population consisting of contractor's experience regarding the impact of forty two factors, was then compiled for calculation of Importance Index. The factors were ranked further according to their Importance indices. The ranking was formerly done regardless of any contractor's category and later it was ranked as per Large, Medium and Small contractors.

Table:2 Comparison of factors with Importance Index and Raking

Main Categories	Successful Bidding Parameters	Large contractors		Medium contractors		Small contractors		Overall	
		I	R	I	R	I	R	I	R
project related factors	Job start time	0.623	38	0.720	32	0.778	24	0.676	37
	Duration of execution	0.777	8	0.780	14	0.800	18	0.782	13
	Addressing the tender requirements	0.685	25	0.740	26	0.733	32	0.707	27
	Portion of work to be subcontracted	0.623	38	0.720	32	0.600	42	0.640	39
	Certainty in the Estimate	0.731	17	0.740	26	0.844	11	0.756	17
	Clearness and detail of specifications	0.808	2	0.820	8	0.911	3	0.831	3
	Project complexity	0.800	4	0.740	26	0.800	18	0.787	12
	Meeting green building standards	0.562	42	0.720	32	0.733	32	0.631	41
	Degree of hazard	0.654	32	0.700	36	0.822	13	0.698	30
Organization related factors	Availability of qualified staff	0.731	17	0.780	14	0.889	5	0.773	14
	Prequalification requirement	0.715	21	0.780	14	0.800	18	0.747	20
	Current work load	0.685	25	0.720	32	0.778	24	0.711	24
	Confidence in your Workforce	0.731	17	0.820	8	0.822	13	0.769	16
	Type and number of supervisory persons available	0.646	35	0.820	8	0.822	13	0.720	21
	Reliability of subcontractors	0.662	29	0.760	20	0.756	30	0.702	29
Management related factors	Expertise in management and co-ordination	0.800	4	0.880	2	0.933	2	0.844	2
	Relationship with architect/owner	0.777	8	0.900	1	0.889	5	0.827	4
	Understanding client's requirements	0.838	1	0.860	5	0.956	1	0.867	1
	Interaction with client	0.808	2	0.800	13	0.867	8	0.818	5
	Credibility of committing contract	0.777	8	0.880	2	0.822	13	0.809	7
	Job related Contingency	0.715	21	0.740	26	0.644	38	0.707	27
	Competitive (your strength in the industry)	0.754	13	0.820	8	0.778	24	0.773	14
Finance related factors	Cash flow requirement	0.785	7	0.820	8	0.911	3	0.818	5
	Tax liabilities	0.692	23	0.740	26	0.733	32	0.711	24
	General overheads	0.731	17	0.760	20	0.644	38	0.720	21
	Capital availability	0.738	16	0.860	5	0.889	5	0.796	10
	Contractor's link with the market	0.769	11	0.880	2	0.800	18	0.800	9
	Adaptability and flexibility	0.746	14	0.780	14	0.733	32	0.751	19
Labor related factors	Type and number of laborers available	0.692	23	0.760	20	0.756	30	0.720	21
	Labor environment (union/nonunion/Cooperative)	0.654	32	0.640	41	0.822	13	0.684	36
	Safety hazards	0.638	37	0.660	40	0.867	8	0.689	35
Plant/Equipment related factors	Type and number of Equipment required	0.792	6	0.760	20	0.844	11	0.796	10
	Type and number of Equipment available	0.769	11	0.840	7	0.867	8	0.804	8
	Selection of plant/equipment	0.746	14	0.760	20	0.778	24	0.756	17
Material related factors	Effects of inflation	0.646	35	0.760	20	0.778	24	0.698	30
	Market condition (busy or slow)	0.669	28	0.680	39	0.800	18	0.698	30
	Proportion of off-site prefabrication.	0.608	40	0.640	41	0.644	38	0.622	42
	Contractor's own material manufacturing/supplies	0.685	25	0.700	36	0.800	18	0.711	24
External factors	Confidence in external events (interest rates, inflation, etc.)	0.662	29	0.780	14	0.711	36	0.698	30
	Response to environmental aesthetics	0.592	41	0.780	14	0.622	41	0.640	39
	Government regulations	0.654	32	0.740	26	0.667	37	0.676	37
	Social and political influence	0.662	29	0.700	36	0.778	24	0.693	34

Table:2 illustrates the comparison of each size of contractors with overall contractors on the basis of importance index and ranking. The factors having similar importance index were ranked equally and thus, expert's opinion was considered in identification of more significant factors. Out of these, top ten factors were pinpointed and then were compared with ranking of each contractor's size.

Table:3 Top ten factors of bidding a Construction project

	Top 10 factors of Large sized contractors	Top 10 factors of Medium sized contractors	Top 10 factors of Small sized contractors	Overall top 10 factors
1	***Understanding client's requirements***	***Relationship with architect/owner***	***Understanding client's requirements***	***Understanding client's requirements***
2	Interaction with client	***Expertise in management and co-ordination***	***Expertise in management and co-ordination***	***Expertise in management and co-ordination***
3	***Clearness and detail of specifications***	Credibility of committing contract	***Clearness and detail of specifications***	***Clearness and detail of specifications***
4	Project complexity	Contractor's link with the market	Cash flow requirement	***Relationship with architect/owner***
5	***Expertise in management and co-ordination***	***Understanding client's requirements***	Availability of qualified staff	Interaction with client
6	Type and number of Equipment required	Capital availability	***Relationship with architect/owner***	Cash flow requirement
7	Cash flow requirement	Type and number of Equipment available	Capital availability	Credibility of committing contract
8	Credibility of committing contract	***Clearness and detail of specifications***	Interaction with client	Type and number of Equipment available
9	***Relationship with architect/owner***	Confidence in your Workforce	Safety hazards	Contractor's link with the market
10	Duration of execution	Type and number of supervisory persons available	Type and number of Equipment available	Capital availability

After the identification of top ten factors, four factors were recognized to be common which are as follows:

1. Understanding client's requirements
2. Expertise in management and co-ordination
3. Clearness and details of specifications
4. Relationship with architect and owner

These factors are having different ranking as per contractor's category as is shown in Table:3.
These sub factors are the part of main categories as, understanding client's requirements, expertise in managemant and co-ordination and relationship with architect and owner are the sub factors of management related factors where as clearness and details of specifications is a sub factor of project related factors.
The project team looked forward towards measuring the relationship between the contractors and eight main categories of successful bidding parameters. For this purpose spearman rank correlation was used to check the agreement among the respondents in ranking of factors.

Table: 4 Spearman Rank Correlation for large, medium and small sized contractors.

Main Categories	Large Sized-Medium Sized	Large Sized-Small Sized	Medium Sized-Small Sized
Project related factors	*0.775* ***strong***	*0.7416* ***strong***	*0.4333* ***moderate***
Organization related factors	*0.057* ***Weak***	*0.4285* ***Moderate***	*0.7428* ***Strong***
Management related factors	*0.2857* ***Weak***	*0.875* ***Strong***	*0.5178* ***Strong***
Finance related factors	*0.7142* ***Strong***	*0.6857* ***Strong***	*0.6285* ***Strong***
Labor related factors	*0.5* ***Strong***	*-1* ***Very Weak***	*-0.5* ***Very Weak***
Plant/Equipment related factors	*0.25* ***Weak***	*0.5* ***Strong***	*0.75* ***Strong***
Material related factors	*0.4* ***Moderate***	*0.9* ***Strong***	*0.1* ***Weak***
External factors	*-0.8* ***Very Weak***	*0.9* ***Strong***	*-0.9* ***Very Weak***

In Table:4, the agreements between the contractors are characterized as strong, moderate and weak as per the following strategy:
If Spearman Rank Correlation ranges from 1 to 0.5 is categorized as strong, 0.5 to 0.3 is categorized as moderate, 0.3 to 0.1 is weak, and -0.1 to -1 is very weak.

5. Discussion

After the analysis of the compiled data four factors were recognized similar among the categorized contractor out of the top 10 enlisted factors, which are discussed as follows.

Understanding client's requirements:
Approaching for a contract without understanding a client is useless, as contractors should be more sensitive towards this point. This will not only assist in omitting the disputes but also lead to winning of a tender.
Client is always looking for a contractor who is capable enough to ensure them the fulfilling of their requirement and how the project will be delivered, and this can only be done by understanding their requiements and expectations. Knowing the client's need and concerns may enable the contractor to structure and schedule the tender in such a manner that the bid stands out from the competitors.

Relationship with architect/owner:
A positive relationship of contractor with owner/architect is important because it helps a construction project bidding to be successful and smooth completion of a project on time becomes possible. Their positive relationship and effective communication results in minimized disagreements and disputes on site and project can move from one phase to the next.

Clearness and detail of specification:
Specifications are mainly involved in different types of large and small scaled projects. It is essential to have a clear and detailed image of specifications when a project is utilizing a public bidding process. Although they are not enough to address all the issues that may come up during construction but they typically highlight the solutions of dealing with issues on site and predefine the project with greater details than drawings and contracts.

Expertise in management and co-ordination:
Good coordination leads a project towards team work and sense of responsibility. When the different domains involved in a project are well integrated, eventually comes up with effective communication, and problem solving. The higher the interaction the higher the success. Experts and specialist leads to the linearity of the learning curve and completion of project on time without and overrun or errors. Client's

are mostly attracted with effective and efficient inter department coordination and management, and this gives a win-win situation to the contractor.

6. Conclusions

The intent of this study was to identify the successful parameters of bidding a construction project. Factors were ranked and reduced by the expert's advice for their significance and capability of influencing the project outcoms. This study has been conducted to illustrate those factors in the perspective of large, medium and small contractors as per categorical regisration by Pakistan Engineering Council (PEC). Importance index was evaluated which enabled the factors to be ranked and prioritized according to there values as criteria with highest importance index was considered to be at the top most rank and the factors with lowest importance index were kept at the bottom. After getting importance and ranking of those recognized factors top ten factors were elaborated for 3 categories of contractors as: Large-Medium , Large –Small and Medium-Small contractors. Furthermore, a list of top ten overall factors was also identified and those chief factors are: Expertise in management and co-ordination, Understanding client's requirements, Capital availability, Relationship with architect/owner, Interaction with client, Cash flow requirement, Type and number of Equipment available, Credibility of committing contract, Contractor's link with the market and Clearness and detail of specifications. After comparison of three of the categories of contractors, four common existing factors were found as: Understanding client's requirements, Expertise in management and co-ordination, Clearness and details of specifications and Relationship with architect and owner.

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ASSESSMENT OF BARRIERS IN MARKETING PRACTICES -THE KARACHI CONSTRUCTION INDUSTRY EXPERIENCE

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Abstract

Marketing is considered to be an investment for creating a competitive edge and allowing competitive advantage. Hardcore marketing, which was mostly developed for the manufacturing industry provides little help to the architectural engineering and construction (AEC) industry. It has forced the AEC industry to take research in marketing seriously. This main focus of this paper is to assess barriers in marketing practices faced by the AEC industry of Pakistan. For this purpose a survey was conducted from 45 different AEC firms based in Karachi. The survey constituted a questionnaire which was based on the marketing barriers; hypothesis testing was done on the collected data. After the hypothesis testing the low profit margin, insufficient financial budget and gross misunderstanding of market were found to be most significant barriers in marketing practices adopted by AEC firms of Pakistan. The overcoming of these barriers will be helpful in improving the marketing strategies followed by AEC firms.

Keywords

Marketing, Barriers, Construction Industry, Assessment

1. Introduction

The shortcomings of construction firms regarding their ability to adopt modern management systems and techniques has been written about extensively. In particular, the ability of the construction industry to innovate and manage change has been widely debated over the years. Lansley (1987) argued that in an environment characterized by competitive change, there is need to create systems and procedures that can develop slowly and in sympathy with incremental changes in the environment and will ensure that skills are updated regularly and appropriately. Oglesby et al. (1989) stated that construction firm owner's do not seem to be aware of the economic payoff to be derived from the appropriate use of modern management systems and are, as a consequence, unwilling to incur the cost of operating these systems on their construction projects.

On the point of the industry's attitude to management, many writers point out that although marketing has been established in manufacturing and many services industries, in construction it has largely either been ignored or grossly misunderstood. For example, Fisher (1989) revealed that there were many deeply held misconceptions about the appropriateness and value of general management skills and marketing skills in particular.

Project focused industries, such as construction, were not as customer-focused as some other industries (Bryde and Robinson, 2007). Egan (1998) stated that within five years of his study, the construction industry should “deliver its products to its customers in the same way as the best customer-led manufacturing and service industries,” as cited in Bryde and Robinson (2007). The low priority given to marketing in industry is reflected in the paucity of reported research and helpful general literature on the subject. Until this aspect of the operations of construction businesses is researched and good practice and their benefits are identified and disseminated, the desired improvements cannot be achieved.

2. Literature Review

Marketing is typically understood to be creating and promoting products and service s to consumers and organizations (Kotler 2004). The basic purpose of marketing is to raise the demand curve for organization products or services and, consequently, to rise profits. The most common theme running through the available literature is that the construction industry has performed very poorly in marketing its services and products. Many reasons have been advanced to explain this shortcoming. The principal ones are as follows.

At a general level, compared to other construction management functions such as estimating, scheduling, and cost control, literature on marketing in construction is very sparse. This means the industry's professionals are being educated without a systematic study of this important aspect of management. Harris (1991) noted that professional education and training have always been streamlined and narrowed down to production of highly scientifically trained professionals from the universities with little or no management training. Lack of training in marketing is part of this wider problem.

One of the main reasons argued by Friedman(1984) is that perhaps the contractors and professionals have never in the past met with difficulties in obtaining the required level of works to maintain survival and profit. They are therefore inclined to think that their reputation and the quality of their work will continue to win new orders. Pearce (1992) points out that, in many cases, contractors and professionals alike believe that the most important part of the organization is the production side, i.e., they are production-oriented rather than marketing-oriented. They look for opportunities that fit their capabilities as contractors, rather than adapting their capabilities to suit current and future market opportunities.

There is a wide perception that only clients can create demand for work, and that the firms themselves cannot do so (Bell 1981). Others argue that the nature of the industry is such that it is not capable of being planned, i.e., its dynamic environment prevents any long- and medium-term planning (Moore 1984; Pearce 1992). This argument is in-valid because changing environmental factors affect all industries which make planning all the more important so that organizations can anticipate and cope with changes that may affect business. There are, however, a good number of enterprises that are increasingly beginning to put their experiences to use by venturing into areas such as property development and other joint-venture projects as well as other strategic alliances with a great deal of success(Bell 1981; Moodley 1994; Friedman1994).

Bell (1981) examined and compared attitudes toward, and organization of, marketing within construction firms within the United Kingdom. He thought at the onset of the research that marketing strategy could be developed based on the distinctive service-industry features. But the search for a distinctive set of service-marketing features relevant to the construction industry was not fruitful. Similarly, Hardy and Davies

(1984) found out in their research that many firms exhibited an indifferent attitude to marketing. Fisher (1989) also commented on the unbalanced view of marketing, and found that to a surprisingly large number of firms, marketing appears to be synonymous with selling, business lunches, and "double-glazing hype." Yet marketing, the science and business philosophy, he argued, is taught as a serious subject to the cream of business managers and other professionals in almost every other industry. Morgan and Morgan(1990) concluded from a study of marketing communication in the U.K construction industry that marketing is still a new phenomenon viewed with skepticism. Shearer (1990) highlighted the problem of conceptualizing marketing in construction. That research also found a prevalence of the view that marketing in construction is in essence selling promises, because the client is normally being asked to buy some-thing that does not exist. Pheng (1991) observed that marketing has attracted only little attention among construction contractors and professionals alike. Similarly, Morgan and Burnicle (1991) noted that the U.K. construction industry has been slow in adopting marketing principles.

The conceptual difficulty is only a part of the wider debate about the meaning of marketing. For example, Namo and Fellows (1993) found a wide variety of definitions of marketing even among marketing professionals, and supported their argument with two definitions by Kotler (1984) and Ohmae (1988). Kotler defined marketing in terms of human activity directed at satisfying needs and wants through exchange, and Ohmae saw it as discovering what customers want and orienting the firms to satisfy those wants. Payne (1988) viewed marketing orientation to be the degree of responsiveness of an organization to its market needs.

Construction researchers had also run into similar problems with the definition of marketing. For instance, Arditi and Davis (1988) described marketing as consisting of activities such as finding new markets; evaluating job potentials; establishing contacts with potential clients; gaining information regarding market conditions, potential customers, and projects; prequalifying with clients; estimating project cost; submitting proposals; entering into contracts; negotiating changes and claims; and, finally, developing new technology or different contract forms. More recently, Betts and Ofori (1992) observed that competition in the construction industry has increased considerably in recent years due to influences resulting from changes in technology as well as changes in client desires as a result of variation in taste, aspiration, and purchasing power. Asa response to these external influences, competition within the construction industry has become more intense and sophisticated as firms adopt practices and procedures to help them survive. It is obvious from the literature review that there is a need of assessment of barriers in marketing practices specific to AEC firms of Pakistan.

3. Methodology

Flow chart for the methodology adopted for study is shown in Fig. 1. Firstly comprehensive literature review was done to get acquainted with the marketing practices in AEC industry from different research papers, books and publications, and then based on gathered knowledge, the next step was to discover and identify marketing barriers (Yisa et. al, 1995) as study variables to be analyzed in the field. After identification of the barriers, study was moved towards the next step: formulating questionnaire to be used as a statistical data collection method for the study. Questionnaire comprised of barriers as shown in table 1, response was taken on a Likert Scale from 1 to 5 based on the significance of the barriers. The target population was the local AEC industry. The responses were gathered from 45 different firms with different roles in the AEC industry as shown in the fig.2 and fig. 3. The next step was the distribution of these questionnaires among different industry members as shown in the figure 3, it can be seen that response was collected from all levels of different organizations and later collecting and sorting them out for analysis. After following these steps the data was extracted from the questionnaires handed over by respondents and were tabulated for evaluation and analysis.

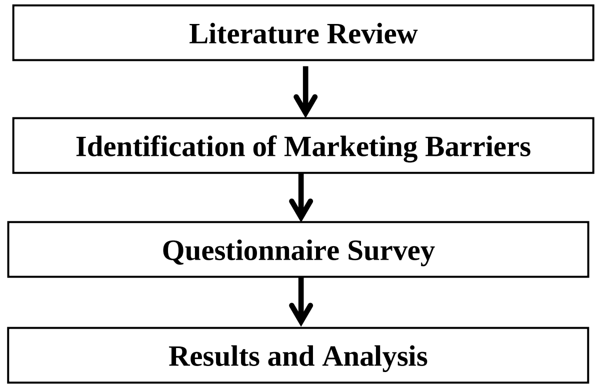

Fig. 1 Methodology

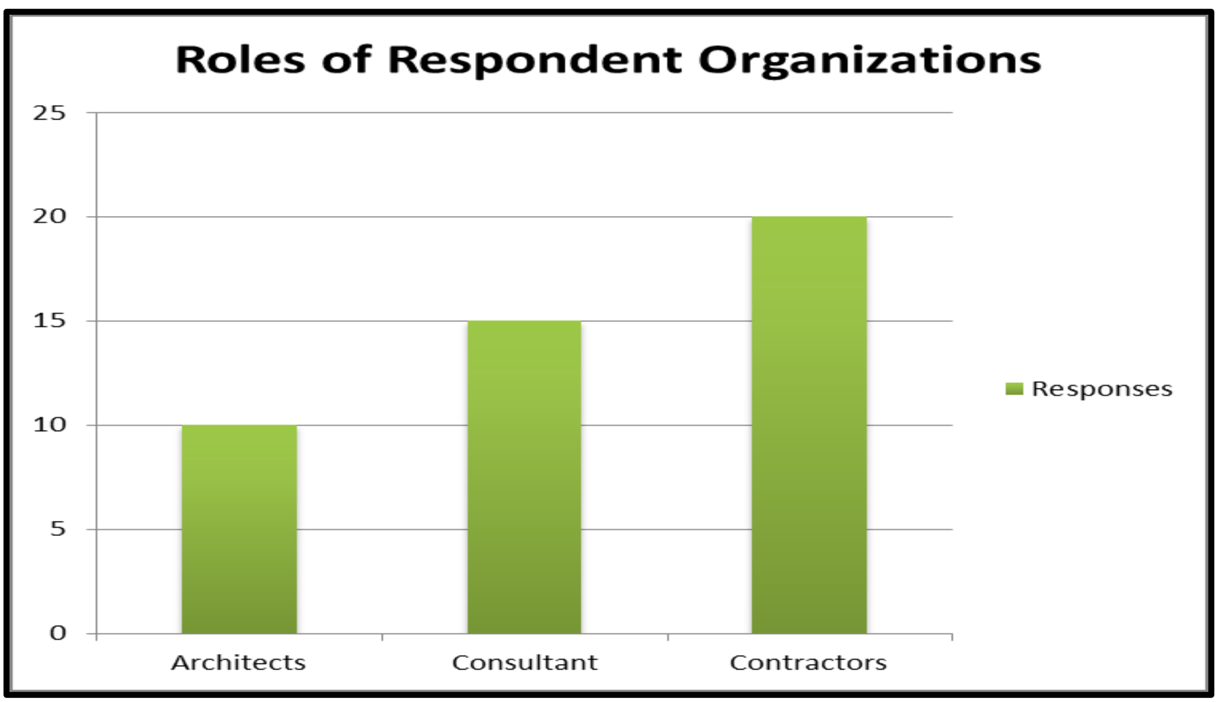

Fig. 2 Respondents Divisions w.r.t Roles in AEC Industry

From figure 2 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.

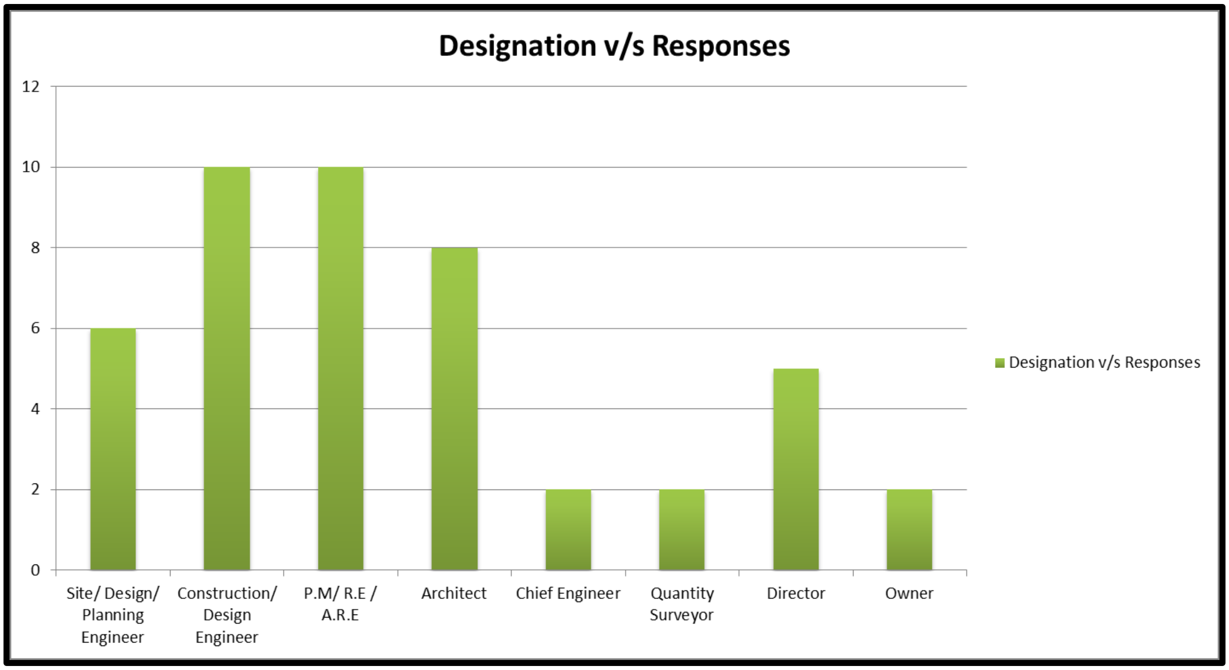

Fig. 3 Respondents Divisions w.r.t Positions in AEC Industry

In Fig. 3, it can be seen that response was collected from all levels of AEC firms because marketing concepts need to be understood by engineers and architects at each level of firm.

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.

4.1 Z Test

A Z-test is the statistical test for the mean of the population. Where n = No. of respondents

Formula for Z-Test Analysis

$$\mathbf{z = \frac{x - \mu}{\frac{\sigma}{\sqrt{n}}}}$$

Where,

X = sample mean

μ = population mean

σ = standard deviation

n = sample size (No. of respondents)

Null hypothesis (H_o) and Alternative hypothesis (H_1) for Marketing Barriers:
For aforementioned marketing barriers two hypothetical tests have been performed; one for Most significant barriers and other for average significant barriers. Following are the hypothesis defined for each of the marketing barriers.

Table 1: Null and Alternative Hypothesis

Marketing	Most Significant	Average Significant
Barriers	$H_{\circ}$= Not most significant H_1=Most significant	$H_{\circ}$= Not average significant H1= Average Significant

4.2 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 i.e. If P-value "<" or "=" 0.05, we accept $H_{\circ}$ otherwise accept H1 and results were generated. The results are shown in the table 2.

Table 2: Results of Hypothesis Testing

HYPOTHETICAL TEST FOR SIGNIFICANCE OF MARKETING BARRIERS		
MARKETING BARRIERS	P - Value	P - Value
	Most Significant	Average significant
Misunderstanding of Marketing Practices	0.051921276	2.00959E-10
Inability to stop executing and think strategically	9.52E-14	0.36822343
Inadequate customer insight	1.31E-15	0.138861344
Lack of Clarity on goals and objectives	1.37E-14	0.061379821
Insufficient Financial Budget	0.131454969	1.50253E-09
Shortage of Resources	1.46E-14	0.128795631
Low Profit Margin	0.177099898	6.35589E-14
Ambiguity in goals and objectives	1.37E-14	0.061379821
Poor Alignment between Sales and Marketing	7.08E-15	0.220147351

5. Results

Following results can be drawn for hypthetical testing of the data collected, barriers were divided into two categories as discussed below.

5.1 Most Significant Barriers

Following barriers were found to be most significant by AEC firms,

5.1.1 Misunderstanding of Marketing Practices

“Misunderstanding of Marketing Practices” was rated as most significant marketing barrier in AEC industry. AEC firms find it unnecessary to spend on marketing. It was found out from discussion and survey that most of people find their industry different from other industries and feels that spending in marketing will not help them in getting new projects and compete with other firms.

5.1.2 Insufficient Financial Budget

In local AEC industry, firms don’t have enough budgets for marketing. Firms require different amounts of capital to get up and running. For example, businesses might need to purchase facilities, buy the materials needed to execute projects and pay various fees associated with licensing and regulations. Only those within certain income brackets can usually start businesses, unless they manage to acquire a loan or sell off shares of the business.

5.1.3 Low profit margin

AEC industry has rated “low profit margin” as a significant marketing barrier. In AEC industry, smaller firms have not reasonable profit margin so that can’t spend on advertising, on marketing strategies and on training on their resources.

5.2 Average Significant Barriers

These are the average significant barriers:

5.2.1 Shortage of Resources

“Shortage of Resources” was found to be an average significant barrier. Shortage of resources in staffing, budgeting or time is hindrance to their marketing success.

5.2.2 Inability to stop executing and think strategically

Many marketing organizations operate under a “Because Marketing” strategy. This means they do what they do “because” they have to, or “because” they’ve always done it that way, or “because” that’s what they are paid to do. There are no goals, no real strategy, and no objectives. This would be similar to driving your car aimlessly with no planned destination simply “because” you own a car. If you do not have an end goal or objective, then you are marketing aimlessly.

5.2.3 Inadequate Customer Insight

The above listed barrier has been rated as an average marketing barrier. The study indicates that marketers struggle because they don’t have sufficient customer insight. The fact is that most organizations have more content for specific target customers then they realize. Websites, collateral, sales playbooks, etc. are all filled with subject matter that marketers can use. So when it comes to content, the problem is not its availability. The problem is that most organizations don’t know how to use content effectively.

5.2.4 Poor Alignment between Sales and Marketing:

This is certainly barriers to AEC industry`s success. It has been rated as average significant barriers. The reason for poor alignment between sales and marketing according to industry, it is not the core problem that industry pays attention to.

5.2.5 Ambiguity in Goals and Objectives

There is thinking in industry that marketing and sales should have the same ultimate objective: Supposed to contribute to pipeline and generate revenue. Each has a unique role in meeting those objectives, but the goal should be the same. This clarity is not visible very much in local AEC industry and is commonly found in the industry as a barrier.

5.2.6 Variation in Public Demands

AEC industry considers “variation in Public Demands” as an average barrier. It’s characteristic of construction industry that there are discontinuities in demand of projects.

6. Conclusions

It is evident from study that misunderstandings about marketing, presence of financial constraints, and low profit margins are the most significant barriers in marketing practices of AEC firms. While shortage of resources, inability to stop executing and think strategically, inadequate customer insight, poor alignment between sales and marketing, ambiguity in goals and objectives, variation in public demands are the average significant barriers. It can be concluded from study that standard marketing practices need tuning for AEC industry because there are industry specific barriers that must be taken into account. Once succeeded in overcoming these barriers, AEC firms can generate more business by committing effective marketing practices that will in turn also help in growth of construction industry.

7. Recommendations

This study was carried out in Karachi that is considered to be financial hub of Pakistan but for generalized results about marketing barriers 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 practices in AEC industry of Pakistan.

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The Role of Energy Efficient Building to Promote Sustainable Construction

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Abstract
Pakistan is facing an immense energy crisis and demand of energy is increasing day by day. The sustainable solution for this energy demand is to conserve energy and produce its smaller portions from non-renewable sources and the remaining from renewable sources since Pakistan has more than 300 full light days, wind and a treasure of water sources. Buildings are the prime consumer of energy and there lies a capacity in buildings to conserve energy if designed intelligently. The buildings can be smartly designed to take maximum advantage of natural sources for lighting, heating and cooling purposes inside the building while minimizing the electricity use. The purpose of this paper is to highlight the importance of energy efficient buildings for a better, economical and energy saving sustainable construction. A case study of textile mill is discussed where energy efficient system was installed resulting in a reduced electricity consumption by more than 35%. The investment on transforming a conventional building to an energy efficient building returns in the form of lesser electricity bills and healthier production.

Keywords
Energy Efficient Building, Sustainable Construction, Energy Crisis, Passive Design

1. Introduction

The construction industry bears the potential to make future of the earth more sustainable. The global concern about development is not only raising the level of economy rather the focus is how living standards are influenced in long run (Ding, 2008). Sustainable construction is like a three-legged stool; the economy, society and ecosystem that need to be balanced concurrently (Young, 1997). Pakistan is facing energy crisis which has adversely affected its agricultural, industrial and economic sector resulting in increased rate of inflation, poverty, unemployment, social evils and frustration in people (Faheem, 2016). The demand of energy is an ever-increasing phenomenon and for sustainable future, energy conserved is better than energy produced. The efficient energy use in domestic sector of Pakistan is strongly required since it is the prime consumer of energy (Khurshid et al., 2014). The life cycle energy analysis of buildings has revealed that 80-90% of energy is consumed in their operation phase and 10-20% is embodied energy. The energy used during demolition process of building is insignificant as compare to the total life cycle energy of the building whereas life cycle energy distribution of building encounters the over-all energy use during construction, operation, maintenance and disposal. Therefore, reduction in operating energy is the instinct of an energy efficient building design (Adalberth K, 1997). The concept of green construction is strongly appealing the interests of developers and stakeholders due to environmental impacts of construction. The terms green construction, green buildings, high performance buildings, sustainable construction and sustainable buildings are interchangeably being used in literature (Amos and Albert, 2016). The objective of green construction is to minimize the environmental and ecological impacts of construction, to promote renewable energy use and building performance by means of efficient energy use, indoor air quality, safety and comfort (Ding, 2008). The Pakistan Green Building Council Pakistan Green Building Council (PGBC) is working for the development of green building technology in Pakistan. Recently the National Energy Efficiency and Conservation Act 2016 is passed by the Senate and National Assembly of Pakistan to prescribe the energy conservation codes for buildings. This ENERCON Act 2016 along with Pakistan Engineering council will try to implement energy conservation codes by law (PGBC).

This paper has focused on how energy can be saved by varying the building design. It has been found that the efficient use of sunlight can reduce the electricity demand of a building. The building can be designed passively to maximize day light harvesting. Moreover, a case study of textile mill in Pakistan has been discussed in detail whose energy consumption was very high. This mill was converted to an energy efficient building by increasing the use of ambient sources thus the electricity bills were reduced significantly.

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2. Energy Efficient Building

The worldwide energy consumption indenture buildings as the main consumer and the total resource consumption and emission in the world are also greatly affected by the buildings. A building throughout its life cycles consumes energy directly from construction, operation, maintenance and destruction of the building and some amount of energy is indirectly utilized during the production of construction materials (Sartori and Hestnes, 2007). There is a definite potential of energy conservation in buildings but it is not a one night process. There are multiple options to reduce the operating energy in buildings. These energy efficient buildings are being adopted all over the world. A reduction of 50% in energy use by 2020 is the goal of Dutch government while European Union is expecting 20% reduction in its building energy use by the year 2020 (Hoppe et al., 2011). In a research, it has been revealed that energy efficient buildings can save more than 30% energy as compare to traditional building (Zainordin et al., 2012). The design of energy efficient building involves the resourceful use of active and passive design techniques to get the best output from natural, renewable and non-renewable sources of energy. The essential drive of design is to make building comfortable in terms of air, temperature and light requirements. The active design of building uses the purchase energy which is electricity, natural gas, solar panel or wind energy to make building comfortable providing it with HVAC (heating, ventilation and air conditioning) system, electric lights etc. whereas the passive design techniques employ the ambient energy sources. Envisioning the solstices, the best use of sunlight inside building is made possible for heating and cooling purposes. It also includes radiant panels and heat pumps. Designing passively means working with the nature instead of working against it. Energy codes of Pakistan categorize the building into five energy systems namely Building envelope, lighting, mechanical and electrical systems, HVAC and service water heating. The building envelop has a potential of saving 40% of energy while lighting can save up to 29% (ENERCON). Following techniques can be adopted to design an energy efficient building (Alamgir, 2008)

2.5 Thermal Insulation

Thermal mass of the building plays a vital role in heat gain and loss of the building. Thermal insulation is provided to retain the heat space of the build and to avoid the excess heat gain as well. This acts encountering the conduction, radiation and convention phenomenon (Autodesk). The cooling load of building can be reduced to 30% by providing thermal insulation which is also possible by sandwiching the polystyrene sheet (Alamgir, 2008).

2.6 Surface to Volume ratio and Ground Surfaces

The surface to volume ratio affects the gain and loss of heat. Along with thermal mass the S/V ratio needs to be minimized which will sequentially curtail the heat lost in cold weather and gain in hot weather (Alamgir, 2008). The temperature of ground surface and rooftops can be controlled by reflective surface or vegetation.

2.3 Building Energy Index (BEI) and Return on Investment (ROI)

BEI is expressed in kWh/ m^2/ year of a building, it describes energy consumption of a building in one year. The total energy for the complete year in the form of electricity is divided by the total area of the building to get building energy index (Muhammad et al., 2015). BEI is representative of the energy performance of the building and during design phase it can be estimated for the proposed building to set a benchmark for its energy performance (Mclean at el., 2014). The installation of energy efficient construction requires more initial investments as compared to traditional construction. If return on investment for traditional and energy efficient building is evaluated and compared, it may reinforce the plan to invest in energy efficient building. ROI gives an idea about time in which additional initial investment of newly installed energy efficient tools is returned. Techniques should be selected in such a way that ROI period is as less as possible.

2.4 Life Cycle Cost Analysis of Energy Efficient Building

The trend of analyzing building project in terms of cost has shifted towards the complete life cycle cost analysis where the cost of construction, operation, maintenance and disposal of the building is analyzed targeting to reduce the operational cost together with initial cost. In case of Green building or energy efficient building the additional initial cost may not convince the owner to build such type of structure. Sometimes, a more economical solution may not necessarily be sustainable (Ding, 2008). The life cycle cost savings of energy efficient buildings by lowering the maintenance and utility cost encourage the owners to construct such buildings (Ms. Alexia et al., 2008). For instance, a building project of $5 million will cost up to $100,000 for incorporating green features and it will save $1 million in its overall life of the building (Greg and Capital, 2003).

3. Case Study of Textile Mill in Pakistan

The following is case study of textile mill where the consumption of electricity was very huge and some passive energy measures were taken to reduce its electricity use for lighting, cooling and humidity system. Following is the detail of issues and their solutions.

3.1 Lighting Issues and Solution

Too many lights were installed in spinning unit. Most of them were energy savers and tube lights which consume more electricity. During initial visit, it was recommended that lights should be replaced with LED's. Some of the LED's installed were of low grade and low quality which do not assist proper saving specially after first few hours of operation. Only good quality lights needed to be installed at Spinning

Unit. A combination of COB, LEB Tube Lights, SMD’s, LVD’s, High Bay and Street Lights had to be used for enhanced energy saving.

3.2 Day Light Harvesting

The facility was simulated for day light harvesting with the help of Design Builder. 12 Sky lights of dimensions (8ft x 6ft) were fitted on the ceiling, the working plane was fixed at 1.5 meter and found that the average Lux introduced in the building by Sun was around 55 Lux which is almost equal to what the artificial lights were creating averagely. To keep the heat from the sun entering the facility from the sky lights, it was recommended to use Low E-glass (tampered).

It was highly advisable to go for Sky lighting to save energy consumed by the artificial lighting i.e. approximately 30-35 KW for day timings. For night remaining 10-15 KW for good quality LED’s and efficient fans can be obtained by Solar Light. Resultantly 40 to 50 KW of lighting could be saved. All tube lights, energy savers and other lights were replaced with LED (COB, High Bay, Led tube lights, SMDs, Street Lights) as per the area requirement. When going towards day light harvesting, airtight aluminum or Upvc windows with Low-E glass were used to reflect back the heat that is coming from sun.

3.2.1 Cooling

The actual cooling requirement was around 571 KW, if the HVAC (heating, ventilation and air conditioning) was properly designed. But HVAC that was running in the factory was not designed as per the facility and Spinning textile. After the expansion of the factory the current design became invalid and because of this the cooling and humidity requirement yield higher power than it was required in case of proper design.

3.2.2 Interior lighting

With proper lighting design and use of optimized LEDs, the requirement came to 12KW as compared to the 40 KW previously being used that could even go lower if the day lighting was properly harvest. Air intake areas were small and not properly designed. Expansion of air intake areas resulted in lower energy consumption of cooling system. To further reduce the energy consumption of the cooling system, fall ceiling was placed in the ring section too. On average, the humidification plants in textile consume about 15% to 25% of the total energy of the plant. The control system consists of variable speed drives for supply air fans, exhaust air fans and pumps in addition to control actuators for fresh air, recirculation and exhaust dampers. Energy savings in the range of 25% to 60% is possible by incorporating control systems in the plants depending on the outside climate. The entire system can be controlled through a central computer. It was calculated that additional 1% of the energy can be saved if the building envelope is properly insulated so as to prevent the escape of heating and cooling in the building.

With above mentioned steps 30% to 60% reduction was achieved in cooling system consumption, which is the major energy consumer in spinning unit and 70% reduction in lighting energy consumption was achieved.

4. Conclusions

This paper emphasis on the importance of energy efficient building since Pakistan has been facing energy crisis for a decade in the form of load shedding. Energy has direct relation with economic development because industries in Pakistan mostly depends on electricity. A unit of electricity saved is better than a unit generated. Currently conservation of energy is a chief concern of nations. More than 50% of electricity generated in Pakistan is consumed inside buildings for lighting and HVAC. It is concluded from the literature and case study discussed in this paper that energy use for lighting and HVAC can be reduced by passively designing the buildings to use ambient sources. Textile Industry is the backbone of Pakistan economy and its electricity consumption is very high that can be controlled via energy efficient techniques. The new lighting appliances are very effective to conserve energy, moreover, the use of incandescent convert 90% energy to heat which also increase the cooling demand. The new fans with plastic body are more efficient as they require less electricity compare to old heavy steel body fans. Properly designed windows with glazing glass are helpful for efficient HVAC.

5. Recommendations

Worldwide trend in green building construction shows that the hindrance to its adoption is lack of awareness and more initial cost. The life cycle cost analysis of energy efficient system can satisfy the owner for additional cost. Pakistan government should take measures for the awareness of construction stakeholders about energy efficient buildings and also for the implementation of National Energy Conservation Act 2016. Moreover, an incentive system can also be devised to rate the buildings and provide certificates for energy efficiency which will also help people to switch from traditional to energy efficient buildings.

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Analysis of Missing Data for River Flow Using Statistical Tool Group Method of Data Handling (GMDH)

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Abstract
Water resources management requires handling of large datasets. Transforming the large water resources datasets into the information requires preprocessing to deduce it into the valuable information. While transforming the data, researchers face following problems: 1) the datasets are not sufficient; 2) hard to locate; 3) and are inconsistent or non-compatible. Recent development in the data driven modeling has given researchers to overcome these problems. This research paper presents the quantitative analysis of the missing flow data of **Guddu Barrage** using group method of data handling (GMDH). This study is aimed at finding the missing data using the most appropriate approach of statistics in GMDH and to compare the missing data with actual data for checking and validation of model in GMDH. Flow data having upstream (US) and downstream (DS) flow of sixteen years (i.e. 1998 to 2013) is trained in GMDH using Curve Fitting(CF) and Load Forecast(LF) Analysis. After data mining, accuracy of mined data is assessed using available statistical indices in GHDH i.e. mean absolute error (MAE), root mean square error (RMSE) and coefficient of determination (R^2). It is observed from analyzed data that: for US flow data, R^2 is 0.942 and 0.089, MAE is 9793.85 and 63488, RMSE is 25216.8 and 100371 in LF and CF respectively. On the other hand, for DS flow data, R^2 is 0.959 and 0.0974, MAE is 6013.8 and 53967, RMSE is 19581 and 92853 in LF and CF respectively.

Keywords
Missig data; Water Resource Management; Statistical Parameters; Data driven modelling

1. Introduction

Problems related to water management have previously been viewed as responsibility of a narrow set of experts while the complexity of it hasn't been realized until later where role of citizens and political representatives has been found important. All stakeholders, players and decision makers need to understand that eventually negotiated solutions will help to come up with satisfactory solutions even if they might not be completely satisfied involving everyone. Management and consensus on the water issues are only possible to be achieved, when the information among all the stakeholders are shared in an equitable way with support of knowledge based tools (Cunge, Jean A., and Marc Erlich, 1999). The notion of knowledge based tools is that the organization provides processed information that enables the consumer to take the actions that he/she would not take without that information (Abbott, M. B., and Z. Vojinovic, 2009**).**

Water resources management requires handling of large datasets. Transforming the large water resources datasets into the information requires preprocessing to deduce it into the valuable information. While transformation of the data, researchers face following problems: 1) the datasets are not sufficient; 2) hard to locate;3) and are inconsistent or non-compatible. Recent development in the data driven modeling has given researcher to overcome these problems.
It is common about river flow data that due to mismanagement, flood or any other reason data is not recorded in database. This missing data creates trouble when exported into water resource management software for knowing the behavior of historical data and prediction of future upcoming river flow for risk and flood management. Therefore, raw data which contains missing values should be mined so that decision makers can have refined and useful data for mitigating the risk, disaster or any unexpected event. Different approaches are being used for data mining like interpolation, extrapolation, normal ratio method, correlation method etc. But accuracy and trained of data is concerned due to variation in data with respect to seasons i.e. summer (Kharif) and winter (Rabi). If available data in Kharif is considered for analyzing the missing data of Rabi then it will not give accurate or good results due to variation in trend. In the last few decades, various types of data mining techniques and models have been recognized and they show the stochastic nature of hydrologic processes which led to an increasing interest in artificial neural network (ANN) and fuzzy logic techniques. These techniques consider the nonlinearity in the rainfall-runoff process and the utilization of soft computing techniques such as support vector machines, expert systems, and genetic algorithms are grouped under the general description “artificial intelligence” (Dastoran M.T et al., 2010).

These new machine techniques, especially ANN and neuro-fuzzy methods, have been used to solve different hydrology and water resources problems during the last decades they presented very good results. The ANN technique is widely used in different areas of water resource management. (Bhattacharya and Solomatine, 2000) used this technique to evaluate stage–discharge relationship; (Dawson and Wilby, 1998a**)** applied ANN for rainfall-runoff modeling**;** (Dawson and Wilby, 1998b) also compared the application of different types of ANN for river flow forecasting; (Hsu et al., 1995) evaluated the application of ANN for rainfall-runoff process; (Karunanithi et al., 1994) predicted river flow using adaptive ANN; (Luk et al., 1998) tried to forecast rainfall events using ANN; (Minns and Hall, 1996), and (Tokar and Johnson, 1999) employed this method as a tool of rainfall-runoff modeling; (Dastorani and Wright, 2004) employed ANN to optimize the results of a hydrodynamic approach for river flow prediction;(Dastorani and Wright, 2003) completed a research project on flow estimation for ungauged catchments using a neural network method; and (Dastorani and Wright, 2002) used ANN for real-time river flow prediction in a multi station Catchment.

Back propagation learning algorithms have allowed a rapid development of artificial intelligence and neural networks and research in its application has observed an increasing trend. However, neural networks unlike statistical methods need a substantial amount of information regarding the model’s structure. The information regarding the quantity and quality of the input arguments is important to be known to experts other than the number of hidden layers and neurons. Such an approach requires not only the background knowledge about the theory but also rules needed to be followed for the translation of this knowledge into neural networks (J.A. Muller et al. 1998). The approach adopted for the determination of network architecture corresponds to a subjective choice of the final model, which in the majority of the cases will not approximate the ideal. Comparison of behavior of neural networks to statistical methods in data analysis reveals that it is not appropriate to be view them as competitors since they have many overlaps (W.S. Sarle, 1994).

Ecological processes are one of the areas of modeling where GMDH algorithms have been popularly used. Most of these processes are generally highly nonlinear, where noisy data and statistical models fail to produce accurate results. A combined method of the principle of inductive self-organization and physical laws for the mathematical formulation of general dynamic models proposed by (Dolgopolov

(1986) describes the evolution of the magnetic field of active solar regions. The optimal structure of the model is determined by the GMDH on the basis of a complete description in the form of a system of differential equations (i.e. the analogue of differential equations in GMDH is difference equations) of magneto hydrodynamics and a source function which takes into account forces of a potential nature. The concept of predicting the solar activity is discussed in (A.G. Ivakhnenko et al., 1988) where both harmonic GMDH and the method of analogue complexing are discussed. The method of analogue complexing as a more robust approach is producing best results. Due to the long history a single analogue is found to produce more accurate results in comparison to the combination of analogues.

(Krotov et al., 1987) used the multiplicative-additive GMDH algorithm to predict tree-growth rings while (Valenca and Ludermir, 1998) compared the Box-Jenkins approach with neural networks (NN) with active neurons for the forecast of daily river flows. It was proved that NN with active neurons rise up the accuracy of short-term forecast and hence increase the lead-time of step by step long term forecast. (F-J. Chang and Y-Y. Yuan, 1999) are dealing with another important ecological problem, the flood forecasting. A stepwise regression similar to the one introduced by (Tamura and Kondo, 1980) is proposed to tackle the multicollinearity problem and the algorithm is known as SGMDH. Additionally, a new recursive algorithm introduced by (J.A. Muller, 1995), which reduces computation time and estimates the coefficients at each step in an adaptive way is proposed for real time forecasting. The latter algorithm is known as RESGMDH and performs better in cases where the rainfall histograms between calibration event (training data) and verification event (testing data) are significantly different.

2. Methodology

Historical flow data in cubic feet per second (cusec) available in Microsoft Excel spreadsheet for **Guddu Barrage** at upstream (US) and downstream (DS) contains number of missing values due to damage of gauging system installed at barrage or any other reason. Missing value of any day in aforementioned dates is marked as “N/A” (not available) and then imported to GMDH for checking of statistical parameters of this raw. Raw data contains missing values and other garbage values are then trained by removing the outliers such as flow data of DS was greater than US due to mishandling during processing data at barrage site. The trained data then again imported in GMDH for post processing for computation of missing values and best fit model which tends toward the actual data. In this regards two methods of data mining are used i.e. Load Forecast and Curve Fitting Analysis for the temporal of 1998 to 2013. Furthermore, these both methods are based on time series analysis. Available statistical parameters in GMDH like mean absolute error (MAE), root mean square error (RMSE) and coefficient of determination (R^2) were checked for accuracy of post processed data.

Table 1: Statistical parameters of preprocessed flow data for US and DS

Variables	US	DS
Numeric values	4595	4564
Text values	0	0
Missing values	1250	1281
Unique values	3477	3181
Zero values	0	0
Most frequent	40032	3400
Min. value	9767	109
Max. value	1148738	1148738
Std. deviation	104876.79	97725.4808
2σ outliers	163	157

3σ outliers	78	77
4σ outliers	50	48

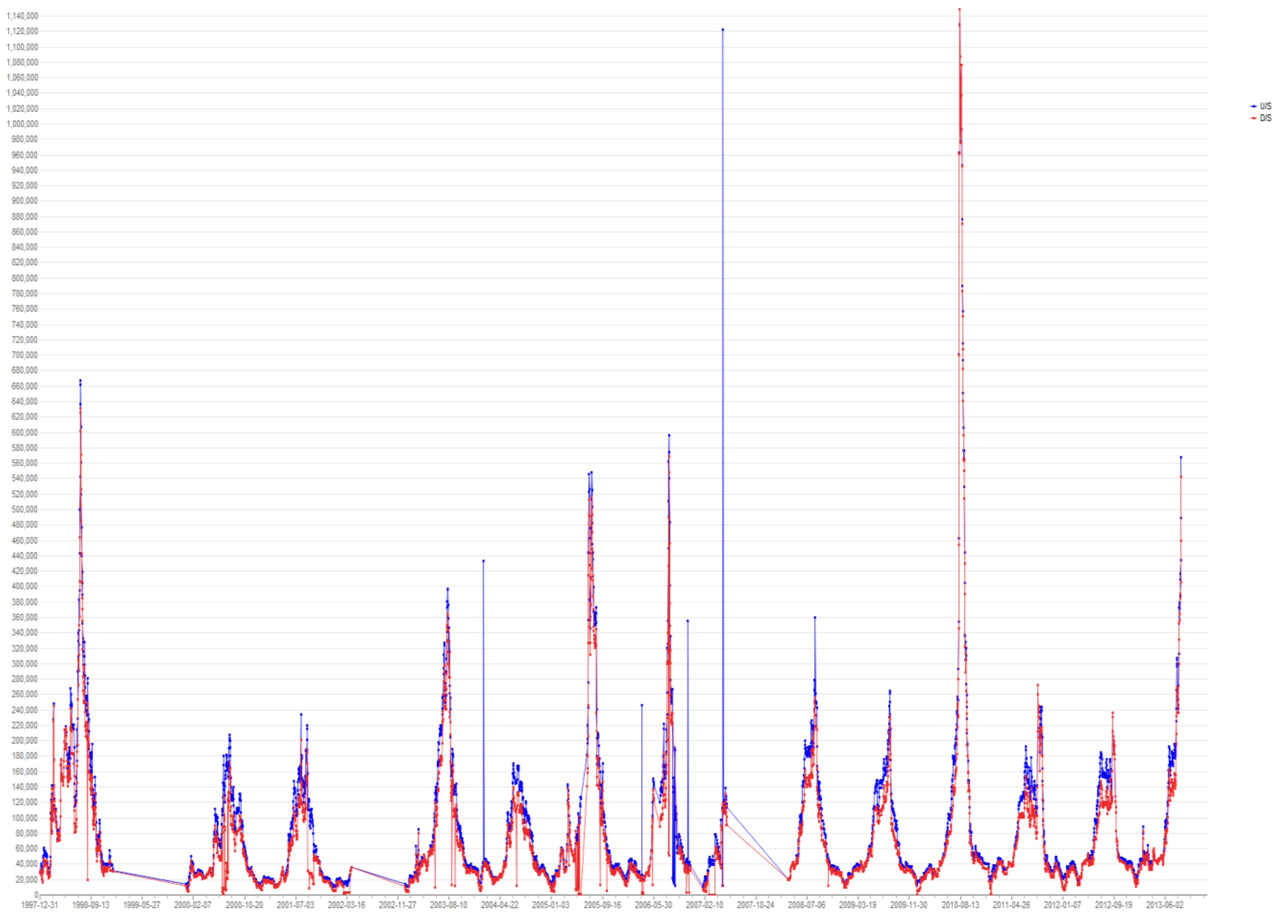
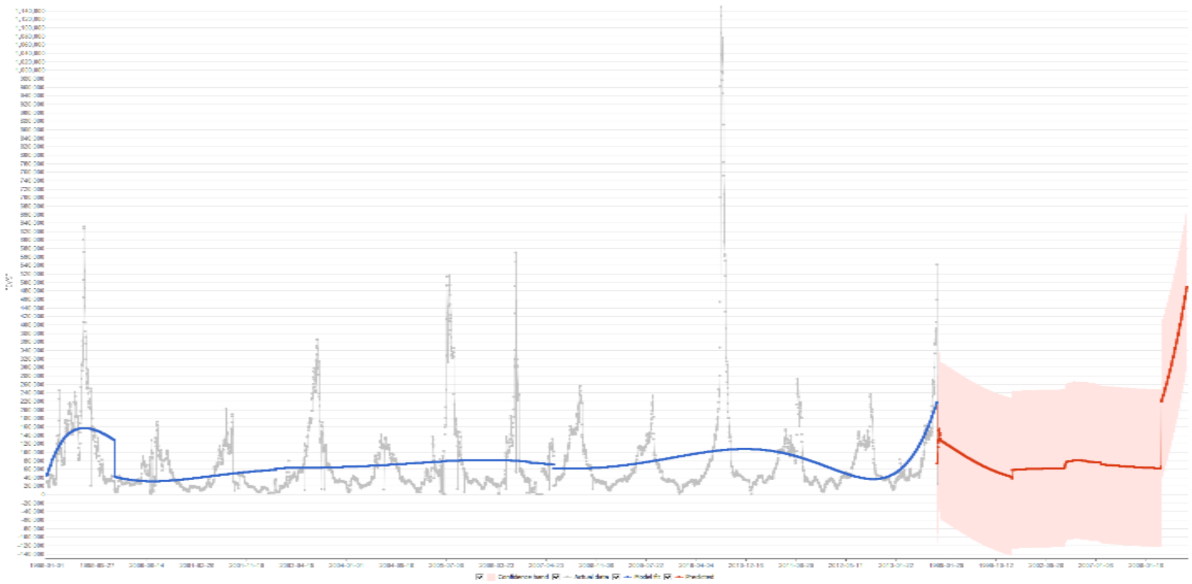


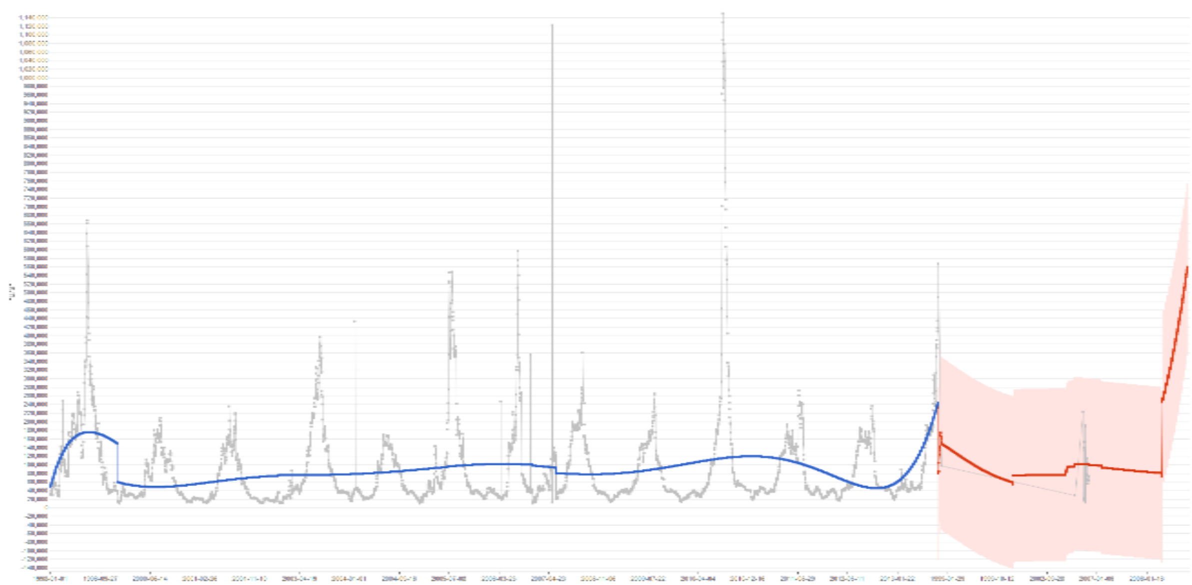
Figure 1: Representation of preprocessed US and DS flow data

3. Results and Discussions

GMDH analyzes the whole data including mising values by fitting the best fit model that represents the most accurate predicted values as compared to actual data.This technique uses different approches of mathematical modeling like curve fitting and load forcast; accuracy of predicted values varies from model to model. Not only accuracy varies but also simulation time for same data in both models varies. Load forcast takes more time as compared to curve fitting. Summary of raw data is listed in Table 1 which throughly enumarates the status of raw data. While Figure 1 (time in days on x-axis while discharge in cusec on y-axis) shows the pre processed flow data in time series graph. Blue color represent US flow while red color represent DS flow. All flow values are in cubic feet per second (cusec). While flow value of each day of thirteen years i.e. total 5845 days which contains both available and missing flow data are incorporated in time series graph.

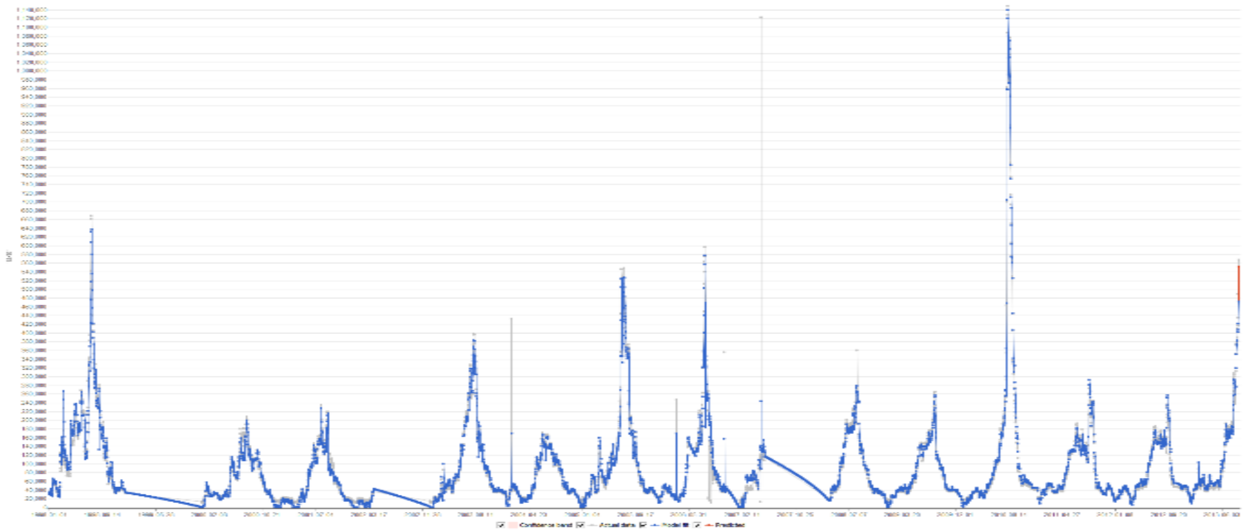


(a)

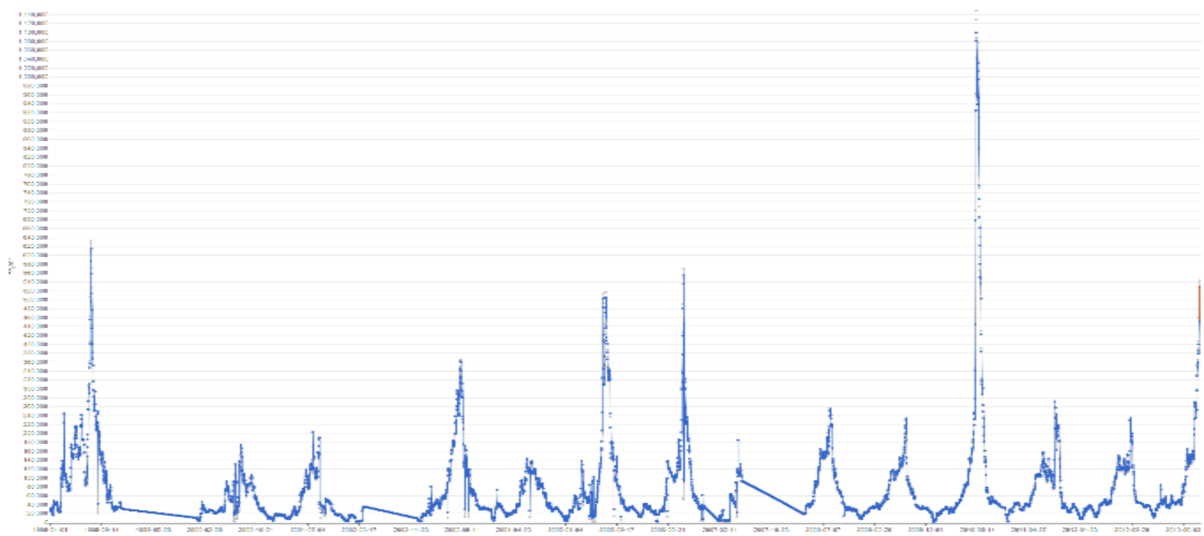


(b)

Figure 2: Post processed data using Curve Fitting Analysis: (a) US flow data; (b) DS flow data



(a)



(b)

Figure 3: Post processed data using Load Forecast Analysis: (a) US flow data; (b) DS flow data

Table 2: Accuracy of model for post processed flow data for US and DS (in cusec)

Statistical Parameters	Load Forecast		Curve Fitting	
	US flow	DS flow	US flow	DS flow
MAE	9793.85	6013.8	63488	53967
RMSE	25216.8	19581	100371	92853
R^2	0.942	0.959	0.089	0.0974

Table 2 represents the post processed statistical results and Figure 2 and Figure 3 (time in days on x-axis while discharge in cusec on y-axis) represent the time series flow data of both raw and processed data using curve fitting and load forecast analysis respectively. For figures 2 and 3 dim curve shows the actual flow while blue and red curve shows the predicated missing flow values. After data mining, best fit model is generated that can be used for particular barrage at specified location i.e. at US or DS. GMDH generated model for load forecast and curve fitting methods are shown in Equatation 1 and 2 respectively. Moreover, this model can also be used for river flow forcasting. The accuracy of forcasted flow depends on the history of previous data used for the analysis. The more historical data, the more accurate will be the model.

Y1[t] = -103454 + ""D/S"[t-1]"*1.25888 + ""U/S"[t-1], cubert"*5290.07 + ""U/S"[t-1], cubert"*""D/S"[t-1], cubert"*(-68.9674) **Eq 1.**

Y1 = 47124.8 + time*1514.84 + time^2*(-6.02728) + time^3*0.00983156 + time^4*(-8.64387e-06) + time^5*4.54784e-09 + time^6*(-1.49247e-12) + time^7*3.0655e-16 + time^8*(-3.80717e-20) + time^9*2.59354e-24 + time^10*(-7.35767e-29) **Eq 2.**

Where
Y1 = predicated output value of model
t = time in days
U/S = input variable (upstream flow in cusec)
D/S = input variable (downstream flow in cusec)

4. Conclusion

From data analysis in GMDH it is observed that:

1. Coefficient of determination (R^2) for upstream river flow data using load forecast and curve fitting method is 0.942 and 0.089 respectively.
2. Coefficient of determination (R^2) for downstream river flow data using load forecast and curve fitting method is 0.959 and 0.0974 respectively.
3. Load forcasting method for data analysis is more accurate than curve fitting method.
4. Simulation time required for data analysis in load forecast is more as compared to the curve fitting method.

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Assessment of Environmental Risks and Opportunities in Operation Phase of Hydropower Plants

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Abstract

Environmental risk assessment in the operation phase of hydropower plants, as one of the main sources of renewable energy, is a novel research subject. Hence, the aim of this study is assessing environmental risks and opportunities in the operation phase of Bakhtiari hydropower plant. Firstly, risks and opportunities were identified by reviewing literature and interviewing with experts. Then, they were analyzed using fuzzy FMEA method (considering probability, severity and detection parameters). Due to existence of a higher degree of uncertainty in the experts' opinions for the severity parameter, four criteria (i.e. compensation cost, persistence time, environment sensitivity and individual) were considered to determine this parameter more accurately. Criteria weights were obtained by combining fuzzy ANP and fuzzy DEMATEL methods. Finally, equipment oil and grease leakage to the river, development of nearby constructions and ingress of dust and animals into the plant were identified as the most important risks, respectively; and the most important opportunities were found to be the improvement in the situation of Dez downstream power plant, decrease in fossil-fuel consumption and increase in stability of electro power network, respectively.

Keywords

Hydropower Plants, Environmental Risk and Opportunity Assessment, Operation Phase, Fuzzy ANP, Fuzzy DEMATEL

1. Introduction

Availability of energy in any region of the world affects its economic development substantially (Hosseini and Jenab, 2002). Demand for electrical power is quickly developing in industrialized countries (Najafpour et al., 2010). This may be owing to the rapidly decreasing conventional energy resources through time. Herein, looking for green, clean and renewable-energy sources with respect to the requirements of environmental issues is recognized as a necessity (Saedi et al., 2014).

Hydropower stations are the general term for constructions, which can convert general water energy to electric power and have the capability to control, utilize and protect water resources and environment. From the engineering point of view, obvious vagueness and uncertainties in various economic, social and environmental factors exists in hydropower stations. However, risk analysis in renewable-energy projects, especially for hydropower plants, is very limited (Kucukali, 2011). Therefore, a comprehensive risk

assessment method is necessary to determine priority ranking, overall evaluation and decision-making (Wang et al., 2004).
In other words, these great projects are always exposed to environmental risk. These risks are not limited only to the construction phase. The operation phase of dams and hydropower plants is also an important issue that should be investigated in detail. In this regard, it should be noted that environmental risk assessment is a step beyond conventional risk assessment, and it makes it clear that hazards how intensely threaten human and natural environment.
Hence, the aim of this paper is to assess environmental risks and opportunities in operation phase of hydropower plants.

1.1 Hydropower Stations

According to McManus, a hydroelectric generating station has a dam that traps a large quantity of water, a spillway for controlled release of surplus water and a powerhouse. The powerhouse contains channels guiding water through turbines that convert the linear water flow into a rotating flow. Since the turbine and generator are joined together, the rotating turbine causes the generator rotor to rotate. The electric power potential from water flow is related to water mass, the fall height and gravitational acceleration. The mass depends on the amount of water available and its rate of flow. Power station design determines the height of the water. The majority of designs take in water from the top of the dam to discharge it at the base into an existing downstream river bed. This optimizes height while ensuring controlled water flow (McManus, 2011; Saedi et al., 2014).

2. Materials and Methods

First, by a comprehensive and accurate literature review, environmental risk and opportunities related with the operation phase of hydropower plants accompanied by their causes and consequences will be collected. Then, the identified items will be screened and approved by university professors and experienced experts within the field.
In the next step, risk assessment will be conducted using Fuzzy FMEA (FFMEA). Here, according to the FMEA method, Risk Priority Number (RPN) will be obtained by multiplying three parameters, i.e. probability of failure occurrence (O), severity degree (S) and the probability of not detecting the failure (D) (Wang et al., 2009).
In this study, due to lack of access to past reliable records, these three parameters will be obtained by a questionnaire survey among experts in hydroelectric power plants (questionnaire number one). Due to existence of a higher degree of uncertainty in the experts' opinions for the severity parameter, four criteria (i.e. compensation cost, persistence time, environment sensitivity and individual) will be considered to determine this parameter more accurately (questionnaire number two). Criteria weights will be obtained by combining Fuzzy ANP (FANP) and Fuzzy DEMATEL (FDEMATEL) methods.

2.1 Risk Assessment Using Failure Modes and Effects Analysis

Failure Modes and Effects Analysis (FMEA) is a systematic tool based on team work. It is an engineering technique which can define, identify and eliminate known and/or potential failures, problems, and errors from systems, designs or processes (Ardeshir et al., 2015).
In other words, FMEA is an analytical method in risk assessment that tries to limit potential hazard in areas where the risk assessment is done as well as the causes and impacts associated with it to identify and score them (Meng Tay and Peng Lim, 2006).
In this technique, the RPN will be calculated according to Equation (1).

$$RPN = S \times O \times D \quad (1)$$

The conventional FMEA method has some shortcomings. Hence, to overcome these shortcomings, fuzzy logic is combined with this method in this paper. That is to say, instead of using crisp numbers for measuring S, O and D, fuzzy numbers will be incorporated. These fuzzy numbers may be linguistic variables based on experts' judgment or outputs of other techniques.

2.2 Analytic Network Process

Analytic Hierarchy Process (AHP), developed by Saaty, is essentially the formalization of our intuitive understanding of a complex problem using a hierarchical structure. The AHP enables the decision maker (DM) to structure a complicated problem in the form of a simple hierarchy and to evaluate a large number of quantitative and qualitative factors in a systematic manner with conflicting multiple criteria (Tavakkoli et al., 2011).

Saaty suggested the use of AHP to solve the problem of independence among alternatives or criteria, and the use of ANP to solve the problem of dependence among alternatives or criteria (Dagdeviren et al., 2008).

The ANP is a generalization of the AHP. Whereas AHP represents a framework with a unidirectional hierarchical AHP relationship, ANP allows for complex inter-relationships among decision levels and attributes. The ANP feedback approach replaces hierarchies with networks in which the relationships between levels are not easily represented as higher or lower, dominant or subordinate, direct or indirect (Meade and Sarkis, 1999).

2.3 Decision Making Trial and Evaluation

Decision Making Trial and Evaluation Laboratory (DEMATEL) technique, which has been developed by the Science and Human Affairs Program of the Battelle Memorial Institute of Geneva between 1972 and 1976 and used for research and solving several groups of complicated and interdependent problems. DEMATEL not only can convert the relations between cause and effect of criteria into a visual structural model, but also can be used as a way to handle the inner dependences within a set of criteria. This method has been applied in various fields most recently. DEMATEL uses the knowledge of experts to lay out the structure model of a system. It not only provides a way to visualize causal relationships between criteria through an impact-relationship map but also indicates the degree to which criteria influence each other. As a result, total direct and indirect influences of each factor (indicator) are obtained as each factor's influence given to other factors, but as well influence received from other factors over others. In addition, according to the DEMATEL method, the levels of interdependences of factors do not have reciprocal values, which are closer to the real system (Zhou et al., 2014).

2.4 Pairwise Comparisons Using Language Scale

In network analysis methods and DEMATEL technique, due to the existence of uncertainty in the pairwise comparisons made by experts, Triangular Fuzzy Numbers (TFN) equivalent to experts' opinions will be applied.

In these pairwise comparisons the importance of the element "i" on element "j" for decision-maker "k" will be: (Farrokh et al., 2016).

$i, j = 1, 2, 3, \ldots, \mathrm{n} \qquad \mathrm{k} = 1, 2, 3, \ldots k$

If $i = j$ then $\tilde{a}_{ijk} = (1,1,1)$ (2)

If $i \neq j$ then $\tilde{a}_{ijk} = \left(l_{ijk}, m_{ijk}, u_{ijk}\right)$ (3)

Moreover, the importance of element "j" on element "i" for decision maker "k" will be obtained using Equation (4).

$\tilde{a}_{ijk}^{-1} = \left(1/u_{ijk}, 1/m_{ijk}, 1/l_{ijk}\right)$ (4)

Chang scale (based on Table 1) will be applied for scoring and conducting pairwise comparisons in the FANP (Zheng, 2011). Furthermore, Table 2 will be used for scoring and performing pairwise comparisons in the FDEMATEL method (Attari et al., 2012; Wu et al., 2007).

Table 1: Linguistic Variables and Their Equivalent Fuzzy Numbers for FANP Method.

Linguistic scale for importance	Linguistic variable	Triangular Fuzzy Number	Reverse Fuzzy Number
Equally importance	Very low	(1,1,1)	(1,1,1)
Weak importance of one over another	Low	$(1/2, 1, 3/2)$	$(2/3, 1, 2)$
Strong importance	Moderate	$(1, 3/2, 2)$	$(1/2, 2/3, 1)$
Very strong importance	High	$(3/2, 2, 5/2)$	$(2/5, 1/2, 2/3)$
Absolute importance	Very high	$(2, 5/2, 3)$	$(1/3, 2/5, 1/2)$

Table 2: Linguistic Variables and Their Equivalent Fuzzy Numbers for FDEMATEL Method.

Linguistic Variable	Triangular Fuzzy Number
Very high impact	(0.7,0.9,1.0)
High impact	(0.5,0.7,0.9)
Moderate impact	(0.3,0.5,0.7)
Low impact	(0.1,0.3,0.5)
No impact	(0.0,0.1,0.3)

2.5 Testing the Reliability and Consistency of Questionnaires Data

The Cronbach's alpha coefficient will be used to measure the reliability of respondents' opinions for the FMEA method (i.e. questionnaire number one). The normal range of this coefficient is between 0 and 1. Closer to one alpha values represent more reliability, and alpha values greater than 0.6 will indicate a good reliability (Cheung et al., 2015).
Furthermore, to check the consistency of gathered data from returned questionnaires (questionnaire number two), the Gogus and Boucher method, presented in 1997, will be used for calculating the degree of consistency of fuzzy pairwise comparison matrices. In this regard, if the consistency rate is less than 0.1, pairwise comparison's consistency will be acceptable (Gogus and Boucher, 1997).

2.6 Defuzzification of Fuzzy RPNs

Defuzzification is an important step in fuzzy systems. In these systems, the results of an approximate reasoning usually will be obtained under the form of one or more fuzzy numbers. In these cases, it is necessary to convert the system's fuzzy outputs to a crisp (non-fuzzy) number. In this step, to perform a better comparison between the results, the calculated fuzzy RPN will be converted to a crisp RPN.
There are several methods such as Middle of Maximum (MOM), Center of Gravity (COG), Center of Area (COA) etc. In the current study, the average fuzzy defuzzification method will be applied according to equation (5) (Basaran, 2012).

$$FRPN = (a_1, a_2, a_3)$$

$$RPN = \frac{a_1 + 2a_2 + a_3}{4} \tag{5}$$

3. Case Study

The Bakhtiari Dam is an arch dam currently under construction on the Bakhtiari River within the Zagros Mountains on the border of Lorestan and Khuzestan Provinces, Iran. At a planned height of 325 metres (1,066 ft) it will be the world's tallest dam once completed and withholds the second largest reservoir in Iran after the Karkheh reservoir. The main purpose of the dam is hydroelectric power production, and it will support a 1,500 MW power station. By trapping sediment, the dam is also expected to extend the life of the Dez Dam 50 km (31 mi) downstream (Mohammadi, 2016).

The preliminary studies of Bakhtiari dam began in 1992, and the design phase lasted until 2009 under the supervision of consultants and experts. Currently, the project is at the beginning of the construction phase. Since Bakhtiari power plant is located in a pristine and undeveloped area and also due to problems in approaching sediment surface elevation to the inlet valve of Dez hydropower station in its downstream, valuable environmental issues are present for study (Mohammadi, 2016).

Figure 1: Location of Baktiari Dam and Hydropower Plant in South West of Iran.

4. Results

After identifying the risks, causes and consequences of them and conducting the filtering process, eight environmental opportunities and 32 environmental risks related to the operation phase of the Bakhtiari hydroelectric station were considered for assessment. They were classified into four categories, i.e. ten physicochemical risks, ten economic, social and cultural risks, ten health and safety risks and two natural risks. In this classification process, effort has been made to prevent from considering risks which are the outcome or cause of other risks.

In the next step, the required information was collected using questionnaires number one and two. The questionnaire number two was distributed among Bakhtiari hydroelectric power plant experts. Among the responses of ten experts, three were inconsistent; hence, the output result was calculated based on seven remaining consistent responses. Then, by combining fuzzy pairwise comparison matrices based upon the responses of the experts, the consistency rate for each matrix, i.e. CR^m and CR^g were determined 0.0344 and 0.0839, respectively. Since these values are less than 0.1, the consistency rates of experts' preferences are acceptable. After that, by combining the results of the FANP and FDEMATEL methods, criteria weights with regard to severity of risks were obtained. The results are presented in Table 3.

In the next step, questionnaire number one for FFMEA was prepared and the validity of it was confirmed by a number of experienced university faculty members. Then, this questionnaire was distributed among 15 experts of the Bakhtiari hydropower plant. The calculated Cronbach's alpha was 0.624, which proved the reliability of the questionnaire with a good approximation. Finally, by combining the results of FFMEA, FANP and FDEMATEL methods, the data were analyzed and evaluated. The output results are presented in Table 4 and Table 5.

Table 3: Un-normalized and Normalized Criteria Weights.

Criteria	Un-normalized Weight	Normalized Weight
Compensation cost	0.7216	0.21
Persistence time	0.8087	0.24
Environment sensitivity	1.0000	0.30
Individual	0.8424	0.25

Table 4: RPNs and Ranking of Environmental Risks of the Bakhtiari Hydropower Plant.

Risk	RPN	Ranking	Risk	RPN	Ranking
Equipment oil and grease leakage to the river	0.16594	1	Failure of power plant components by sediments	0.09314	17
Development of nearby constructions	0.15895	2	Falling from height	0.09270	18
Ingress of dust and animals into the plant	0.15838	3	Lack of training for repair, maintenance and operation	0.09016	19
Decomposition of waste and residual woods behind the dam	0.15507	4	Operator's slide in the plant's damp and wet environments	0.08984	20
Failure to identify faulty equipment during operation and contact with it	0.15476	5	Water leakage into the oil	0.08623	21
Discharge of sanitary wastewater of staff camp to the river	0.15121	6	Increase in unemployment	0.08171	22
Population growth in the region	0.14623	7	Landslides	0.08079	23
Overloading to the plant	0.12961	8	Seismicity	0.07470	24
High humidity in the plant's internal environment	0.12699	9	Fluctuation in the downstream water level	0.07446	25
Terrorism	0.12082	10	Fish collisions with turbines	0.07428	26
Reduction in inflow discharge	0.10520	11	Washing lower classes with detergents	0.07405	27
Power plant exhaustion	0.09714	12	Failure of power plant components by water salinity	0.06654	28
Staff electrocution	0.09680	13	Electrocution of people residing around power lines	0.06453	29
Electric or magnetic fields produced inside or near to the plant	0.09609	14	Increase in noise pollution	0.06186	30
Working in confined environments	0.09422	15	Birds collisions with power line wires	0.05353	31
Transformation of indigenous cultures	0.09400	16	Oxygen reduction in downstream water	0.05332	32
Average of RPNs			0.10194		

Table 5: RPNs and Ranking of Environmental Opportunities of the Bakhtiari Hydropower Plant.

Opportunity	RPN	Ranking
Improvement in the situation of Dez downstream power plant	0.32849	1
Decrease in fossil fuel consumption	0.32079	2
Stability in electro power network	0.21921	3
Increase in reliability of providing downstream water rights	0.21567	4
Improvement in quality of life	0.17737	5
Decrease in nuclear fuel consumption	0.16553	6
Setting up the network during blackout period	0.11761	7
Peak supply for network	0.10485	8
Average of RPNs	0.20619	

5. Conclusion

Environmental risk and opportunity assessment enables us to identify critical environmental risks, to find a solution for mitigating them and to try for improving the opportunities. This evaluation revealed that the highest RPNs belonged to equipment oil and grease leakage to the river, development of nearby constructions and ingress of dust and animals into the plant with 0.16594, 0.15895, and 0.15838 values, respectively. Moreover, highest RPNs among opportunities belonged to improvement in the situation of the Dez downstream power plant, decrease in fossil-fuel consumption and increase in stability of electro power network with 0.32849, 0.32079, and 0.21921 values, respectively.

On the other hand, in the operation phase of this hydroelectric power plant, the average RPN of environmental risks and opportunities were 0.20619 and 0.10194, respectively. Applying fuzzy logic and modeling the inherent uncertainty in the experts' opinions in the FMEA method (as a part of this research methodology) enabled us to prevent from the evenness of RPNs for two different risks. Furthermore, by taking into account four criteria for calculating the severity parameter in the FMEA technique, the obtained value for the severity parameter was closer to reality.

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Drought Forecasting For Tropical Countries by Comparing Of Precipitation Data Patterns with Astronomical Positions

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Abstract
South Asia, Brazil Indonesia Australia, Sahel and many other countries lie with in the tropical boundaries. The rain fall data maintained by so many countries are analyzed by hydrologists for their agricultural events. The seasons are traditionally defined by wind patterns and the cultivation seasons for the year. Sri Lanka has Yala, Maha seasons with monsoons. Our Hydrologic year begins in October.
India Pakistan begins the hydrologic year in June as Rabi and Kariff in December.
Sri Lanka has gained 1deg C increase in temperature from 1950 onwards. Heavy dust gathering is in the ocean atmosphere. Changes in hydrology are recorded under these circumstances. Droughts are frequent. Precipitation data and planetary movements are studied and found some periodic episodes.
Full Moon days are rainy.
Moon has a common plane episode with Earth in 1974, 1983, 1992, 2001, 2009/10, 2019 and the author predicted droughts in 1993 and made it true. 2001 was a severe drought. Brazil was badly affected.
Another discovery was the droughts coming in 1996, 2006, 2016 (Sri Lanka , Bolivia) was another planetary episode caused by Jupiter, Saturn and Sun causing a right angled triangle in these years as predicted by author.

Key words
Drought forecasting, Astronomical analysis, Water resources

INTRODUCTION

Tropical countries are periodically subjected to wind generated by annual Earth orbital motion. Monsoon is the wind carrying precipitation in South Asia and Southeast Asia. Recently due to global warming in climate, major changes took place in rainfall patterns. Earth orbital motion with fixed inclination of NS axis creates monsoon in Dec-Jan from NE and May-Jun from SW to precipitate water to Sri Lanka. In addition inter-monsoons caused by crossing the Sun over Sri Lanka come in September and April. Around the same time frontal type cyclones develop with heavy floods. Traditionally combined effect is used to compute the uncertainty of floods.

Global warming is a phenomenon in all tropical countries. South Asia has increased mean temperature by 1^0C during the last 60years.
Astronomical features observed during the same time revealed many cyclic events leading to floods and droughts. The common plane episode of Moon and Earth leads to a prolonged drought in tropics as happened in 1974, 1983, 1992 and 2001, 2010. The short-term effects of heavy planets like Jupiter and Saturn are causing floods in half year when the earth is in between them and Sun. Full moon days are rainy in Sri Lanka. The shift in precipitation made Hambanthota dry and Colombo wet.

Discovery in droughts and floods is common to tropics. Brazil had the same drought in 2001 as predicted by the author in 1993. Astronomical features appeared in cyclic form during the last 36years when the global warming increased in tropical countries. Variation in rainfall patterns is a classic example of sources to identify exceptions. Flood hydrology is disturbed when the normal rainfall is concentrated to less duration and increased droughts cause insufficient water in the reservoirs.

1. SRILANKA GEOGRAPHY

Sri Lanka is an island in the Indian Ocean closer to South India with a high seasonal precipitation. Most of the precipitation is drained through the catchment basins at a faster rate and creates flooding at narrow locations of the river system. The main land is formed with a lower plateau of less than 300m of elevation. A mid plateau of 100-300m is forming the central mountains. A high plateau of 1000-2000m is in the central steep hill region.

2. CLIMATE

The island is situated closer to Indian Peninsular and hence NE and SW monsoons are activated due to the revolution of the Earth round the Sun with an angle of inclination of 23.5^0 between the normal to the plane of revolution and NS axis of the Earth. The movement of the Earth in that inclination creates movement of the Sun from Tropic of Cancer to Tropic of Capricorn. Heated wind streams rise up to create further motion. Cooler winds are attracted with high potential when the Sun is at extremes. Hence SW monsoons are in full swing in June. It begins in May and reaches Sri Lanka with high precipitation. Then it enters India and for June- September period provides rains.

3. CONDITIONS OF RAIN FALL

Presence of dust for nucleation, presence of water vapour in the air mass and dynamic cooling of air mass are essential conditions of rain fall. Dust form an essential role in rain formation. Presence of dust leads to nucleation of water drops and dynamic cooling of wind accelerates nucleation. Then it reaches point of saturation and releases drops of water. Insufficient dust storm is not able to form sufficient nuclei and the cloud passes the locality unsuccessfully

4. GLOBAL WARMING

Global warming is due to burning of fossil fuel in large quantities to produce high accumulation of green house gases. Solar energy is the source coming in short waves. Heat stores in the daytime using gases in the troposphere and water in the reservoirs and landmass in the continent. Emission in long waves slowly takes place in the afternoon and night. Cloud cover can reflect long waves back and form a green house to warm up the troposphere. This warming process is slowly building up during the last 60 years in the tropics by 1deg C.

5. CERTAINTY CHANGES

5.1. Tsunamis

Earth as a planet contributes to variations of hydrosphere due to its volcanic eruptions located around plate boundaries. The dangerous tsunamis are caused by under water volcanic eruptions. In 1883 Krakato disaster was noted. A very same type disaster caused damage from Andaman and killed 300,000 on December 26, 2004 full moon day.

5.2. Floods

Global warming has increased the intensity of floods. The flood occurred in May 2003 on a full moon day was more devastating. A 330mm one day rainfall was showered on southwest hill country due to a frontal type cyclone. In May 2015 severe rains in one day caused land slides in Aranayaka killing people and burying villages.

5.3. Droughts

Global warming has created a cycle of droughts over tropical countries. Usually there are four seasons to bring rainfall in to the island. The SW and NE monsoons are major rainy seasons. Inter monsoons are benefiting to begin the cultivation. Rainfed cultivations are postponed if inter-monsoons are delayed or absent. If monsoons are in short duration or absent, then total cultivation fails. Wet zone perennial crops die in the drought. Droughts in concentrated areas occur due to seasonal changes. In 2003/4 Maha season heavy drought affected Kurunegala District

6. CHANGES IN HYDROLOGY

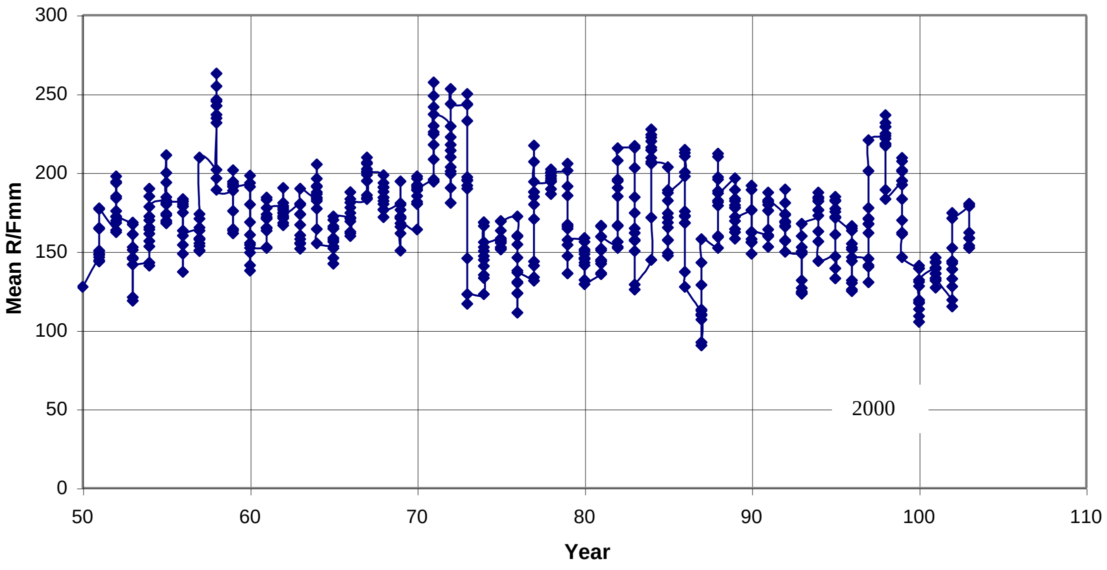
6.1. Kirindi Oya

Observations in precipitation in each district of Sri Lanka are available and some data was from 19th century. Kirindi oya basin in 1930 was rich with water and the estimate for irrigation was 24000ha and this was dropped to 4000ha under climate change. Kirindi Oya reservoir fully dried in 2001.

6.2. Kurunegala

The average monthly precipitation was 150-200mm in the period 1950-1975 but it dropped to 120-200mm in the subsequent period 1975 to 2003. In addition it dropped to 80mm in 1987. It shows stabilization and gradual rise and fall in 30 years above 100mm. This area is far away to get high rainfall

Figure 1: 12Month Running Average Precipitation in Kurunegala 1950-2003



6.3 Puttalam

Annual precipitation is very low in Puttalam District. It is around 1000mm. But in drought periods it drops to 820mm.

Puttalam has gained exceptional rains in 2002 and 1984. This was in contrast to the pattern shown by other locations. These two years were common drought years and heavy precipitation has recorded after the drought. The change in rainfall patterns in Sri Lanka is accommodated with a shift in NW direction in these exceptional years.

6.4. Dry Zone

Dry zone area has commonly dropped precipitation but shows stability. The 1850mm isohyet demarcates the wet zone boundary. About one third of Sri Lanka falls in to wet zone. This wet area is shrinking. Matara city now falls out of wet zone. The Sinharaja natural forest lost many trees and reduced to scrubs from SE direction. Cooler areas of Nuwaraeliya, Bandarawela lost morning mist and coolness in morning. Only Colombo and Matara districts had slight increase from the mean of 1960-90 and dry zone had the worst hit. NE monsoons lost 19% and next Inter monsoons lost 10%. Matale district reduced rains by 410mm during 50 years as the worst. From 1880 to 2015 nearly 30% of precipitation was reduced.

7. WEATHER PATTERNS

Sri Lanka has a very good record of hydrology data for the last 200 years. Historical records show tsunami, storm surges, droughts, floods and epidemics. Beminitiya was a drought existed for 12 years from 45BC. This led the need to write down hither to by hearted (oral succession) Buddhist texts on ola leaves at Matale Alu vihara Temple.

7.1. Observed pattern

Rainfall patterns are usually changing but mean of Galle rainfall data is maintaining close to 2300mm. Annual rainfall in 1974, 1983, 1992, 2001 showed a drop when compared with rest of the years. Galle is facing the sea and SW monsoons are directly reaching without any obstruction. **The research conducted by the author discovered this phenomenon in 1993 and forecasted the drought in 2001 and 2010.** This pattern is no doubt attributed to astronomical features. The drought becomes so serious due to global warming. There was no serious report on drought before 1973. Stable atmosphere causes high-pressure situation. High pressure is causing drought in the island.
(Seneviratne, 1993)

7.2. Destabilizing forces

Tides are caused by gravity pull of Sun and Moon. When Earth and Moon taken together the resultant center of gravity lies over the earth surface. Rotation of Earth in the eastward direction rotates the center of gravity westward with same speed of 1600km/hour. This center drags water towards it and in doing so reach a peak in 6 hours and ends in 12hours. Direction changes in 12 hours and low tides begin. It reaches a peak in 6 hours and ends the cycle in another 6 hours.

Sun is the other object to alter the tides. Its effect is about 10% of that of the Moon. On the full moon days the two forces are canceling each other. On new moon days the two forces are added together. We can see rainfall differences on these different days. Full moons are usually rainy days. When Earth comes to the middle water has free movement and rain is more.

Tides are changing according to the declination of Moon. Direction of tides is changing accordingly. Declination envelope of Moon is changing from $18,5^0$ to $28,5^0$ in 18.55year cycle. In this cycle it reaches the coplanar position in 9.25years. This episode came in drought years in Sri Lanka as noted above. The coplanar activity of Earth and Moon taking place nearly 1.5 year period is very calm and clouds are not seen in the sky. Usual showers are not matured to give sufficient rains. All the tropical areas are affected to this drought. Brazilian drought came in 2001 in the same period as in Sri Lanka, [Seneviratne, (1993), 2005].

7.3. Short-term effects

Droughts lasting for 3month period and excessive floods appearing in tropical countries can have astronomical reasons. Declination and position of Jupiter and Saturn is important. Jupiter and Saturn are heavy objects in the solar system. Jupiter balances the system by pushing Sun back and has some influence on water. Its period of revolution is 12 years. Every year half the period it is visible in the outer orbit in the night. Rest of the year it moves to the other side of the Sun and is not visible in the night. When it is visible in the night we are in between Jupiter and Sun. Hence it is a chance for free movement of water and rainfall continues. Same case is repeated for Saturn, which has a period of 30 years. These two planets come near in 20 years. The impact of combined planets is greater in years around 1961, 1981, 2001 than the years around 1971, 1991and 2011 due to positioning. Precipitation in 1980, 1981and 1982 September months was more than January February months of same years.

7.4. Long Term Effects

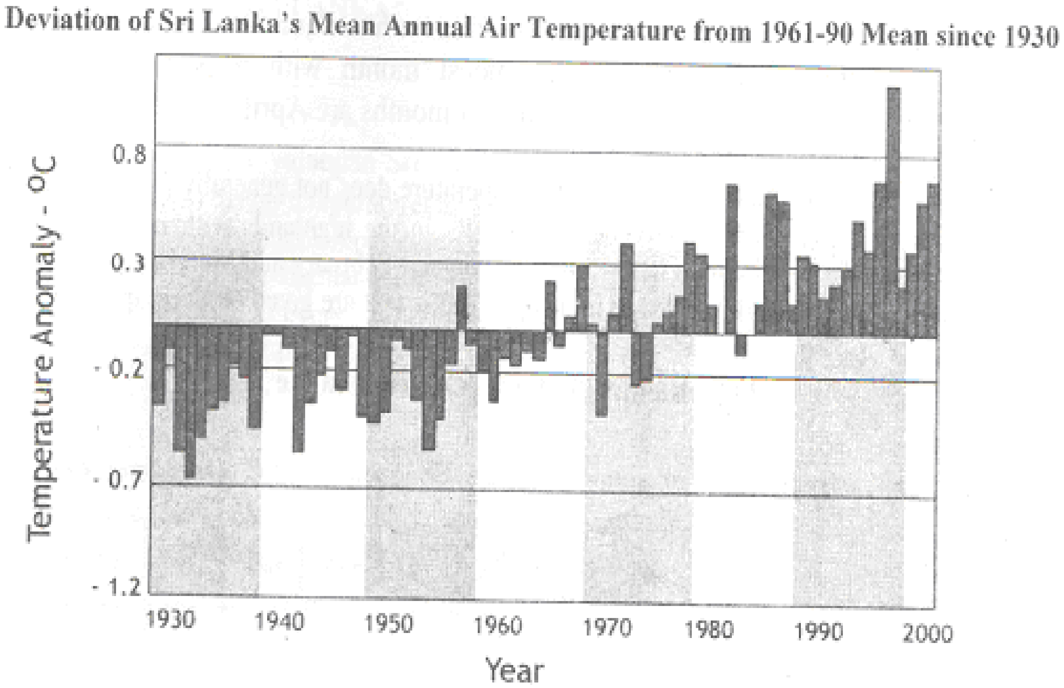
The effect of Moon creating a drought when the two planes are coinciding is a long-term effect. Some times the drought extends to 18 months. Process has shown a gradual extension from 1974 to 2001. This can be attributed to global warming. Tropical countries are subjected to this event. Astronomical features are similar to those lands with same geographic conditions and subjected to atmospheric pressure changes due to wind.

Table 1: Drought Comparison for 2015 and 2016 -Sri Lanka- Rain fall in mm

STATION	YEAR	JAN	FEB	MARCH	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
Jaffna	2015	1	18	15	52	216	38	0	60	47	0	758	380
	2016	16	6	2	0	286	12	1	21	42	2		
Anuradhapura	2015	16	161	26	288	265	10	0	180	149	389	452	296
	2016	32	28	9	114	463	8	0	35	45	75		
Kurunegala	2015	6	34	207	367	160	76	20	74	125	283	422	275
	2016	2	29	151	307	738	18	2	12	21	70		
Colombo	2015	33	123	221	268	169	240	37	91	631	235	526	377
	2016	65	107	91	186	752	81	12	21	102	65		
Galle	2015	68	115	138	171	282	160	249	153	474	316	354	94
	2016	41	250	39	76	441	45	101	67	42	160		

2016 is a drought year and the country is suffering from lack of water resources. Power cuts are seen in industry. Small hydro projects are badly affected. Reservoirs dried and the crop production is reduced. Drinking water facilities are operated using water bowsers to affected villages. Tea, rubber, coconut industries are badly affected. Rice yield was dropped. This drought was forecasted by the author as identified by the Jupiter Saturn Sun forming a right angled triangle. How ever mean sea levels remain unchanged. One major storm comes in after drought season. Temperature rise by 1 deg C is shown in the figure 2. A kidney disease is spreading in the dry zone areas due to poor water quality. Use of wrong weed killers is the reason for poor water quality in dry zone.

Figure 2: Mean Annual Air Temperature from 1930 to2000 Colombo, Sri Lanka



It is noteworthy to mention that the global mean surface temperature has increased by between 0.6 ± 0.2 °C since the late 19th century which has been attributed to increasing concentrations of greenhouse gases due to rapid industrial growth (IPCC, 2001).

Table 2: Rainfall and Astronomical Data for Sri Lanka and Observations

YEAR	GALLE 6N,80E	COLOMBO 7N,80E	KURUNEGALA 8N,81E	APURA 9N,81E	MOON DECLINATION ENVELOPE	JUPITER SATURN GAP
	Mm	Mm	Mm	Mm	Degrees	Degrees
1978	2346	2301	2404	1194	18.5	54
1979	2805	2450	1886	1371	18.5	36
1980	2171	1999	1550	1123	19	18
1981	2086	1681	1913	1059	20	00
1982	2563	1825	2587	1037	22	18
1983	**1549**	**1750**	**1547**	**1115**	**23.5 drought**	**36**
1984	2245	2493	2484	1877	25	54
1985	2219	2390	2245	1273	26	72
1986	1693	1456	1529	1104	28.5 drought	90
1987	2466	2559	1893	1176	28.5	108
1988	2391	2035	2244	967	28	126
1989	2325	2265	2064	1104	27	144
1990	2085	2341	1949	1298	26	162
1991	2188	2096	2191	1456	25.5	180
1992	**2210**	**2575**	**1796**	**929**	**23.5 drought-flood**	**162**
1993	2472	2547	2011	1578	21.5	144
1994	2598	2483	1726	1405	21	126
1995	2321	2398	2055	849	20	108
1996	1970	2161	1757	987	19 drought	90
1997	2150	2530	2645	1330	18.5	72
1998	2290	2387	2199	1216	19	54
1999	2964	2888	1755	1293	20	36
2000	2516	2012	1584	1245	22	18
2001	**1771**	**1932**	**1599**	**1262**	**23.5 drought**	**00**
2002	1536	2199	2046	1264	25	18
2003	1894	2030	1228	1192	26 flood	36
2004	2252	1958	1998	1444	28.5	54
2005	1879	2814	2006	793	28.5 drought	72
2006	2552	2722	2396	1824	28	90
2007	3091	2084	1618	1380	27	108
2008	2776	2626	2399	1484	26.5	126
2009	2011	2230	1979	1315	25	144
2010	1671	2000	1528	893	23.5 drought	162
2011	**Rainy**	**Rainy**	**Rainy**	**Rainy**	**22 floods**	**180**

8. CONCLUSION

The drought predicted in 2010 seriously occurred in the January April period for all tropical countries. This astronomical position of Sun Jupiter conjunction existed in first part of year 2009 and 2010 caused severe hot drought in tropics. As the Saturn is not in conjunction with Jupiter and placed opposite in these years drought was not continuous. But in 2001, 2000 Jupiter and Saturn were in conjunction and the prolonged drought was resulted especially during periods of Sun coming in to conjunction with Jupiter and Saturn. The common plane episode of Earth and Moon occurring in 9.25 years cycle creates drought subjected to conjunction of Sun Jupiter and Saturn. The effect is either serious or soft depending on conjunction and period of the year. This effect is common to all tropical countries.

The countries in the sub tropics also experience heavy floods as in Pakistan after 23years. Tornados in USA, floods in China, Australia and Philippines intensified during this period 2014/2015. Brazil had a drought in the same period. The common plane episode of Earth and Moon occurred in December 2010 and after this date January- March period severe floods came in all over Sri Lanka in devastating conditions and in 2011. These floods coming after the droughts became so intensified and rated as 100 year flood. Analysis is given in Table 3.

As the Earth in January 4^{th} is farthest from Sun and slows down, the crust of Earth become weaker. On the full moon day Earth is pulled from both sides and becomes highly weak. Eruptions are likely in this period as happened in 2004 December full moon day. Tropical areas show more freedom for raining during full moon days.

Previous records show that all common plane episodes of Earth and Moon had droughts except in 1928. Floods are followed by the droughts. These episodes repeat in 2019/2020.

Verification of common plane episode in the early period is commensurate with the 1965(66), 55(55), 47(47), 37(38), 18(18), 1909(08/09). However in the middle a seasonal drought occurs as in 1979, 1976, 1968, 1958, 1950, 1945, 1934, 1923, 1914/15 which resulted for other reasons other than astronomical. Little drought came in 1928. Hydrology records in the other tropical countries need a comparison for this episode. 1996, 2006, 2016 droughts verify Jupiter Saturn Sun right angled triangle.

1 Common plane episode of Earth and Moon causes severe droughts in the interval of 9.25 years. In the descending phase drought is followed by the flood in tropical countries. In the ascending phase drought is coming after the episode.
2 Sun Jupiter and Saturn are large bodies which causes droughts when they are in conjunction behind the Sun or forms a right angled triangle.
3 Full moon days are usually rainy in the tropics as the Earth is in between Sun and Moon and volcanic eruptions occur.
4 When planets are visible in the night Earth comes in between Sun and planet and precipitation occurs.

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Utilization of Steel Slag in Concrete for Sustainable Development

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Abstract

Bangladesh is urbanizing rapidly and the demand for concrete in construction industry and thus the demand for coarse aggregates are increasing. If some of the waste materials are found suitable in construction practices, safe disposal of waste materials can be achieved and cost of construction also can be cut down. Therefore, this investigation was carried out to explore the possibility of utilization of steel slag in concrete as coarse aggregate. The aggregates were tested for different physical and mechanical properties. Cylindrical concrete specimens (100 mm by 200 mm) were also made with different W/C ratios. Brick aggregates were replaced by mixed steel slag and tested with different cement content of 340, 395, 400 and 450 kg/m^3. The concrete specimens were tested at 7, 28 and 90 days. Also, ultrasonic pulse velocity (UPV) test was conducted prior to crushing of the specimens.
Results show that the compressive strength of concrete increases by using 400 kg/m^3 of cement content and mixed slag. Thus, utilization of steel slag will help towards sustainable development of construction materials.

Keywords

Concrete, Steel slag, Compressive strength, Cement content, UPV

1. Introduction

Cement concrete is the most used material in the construction process. Almost 50% of the volume of concrete is composed of coarse aggregates while the coarse aggregate fraction is that retained on 4.75 mm sieve. The recycling of industrial waste material is a major content of sustainable development. Steel slag, which is different form blast furnace slag, is also the unavoidable by-product which is 15-20% of the production of steel in steelmaking industries. Large amount of steel slag was always disposed of as waste which leads to waste of resource, environmental pollution and also ecological destruction. There is no specific regulation of the utilization of steel slag as aggregates in concrete. However, the probability of their use in concrete with technological and ecological advantages has been studied by different researchers. Steel slag is a by-product of the steel making process in which steel cannot be formed in the Basic Oxygen Furnace (BOF) or in an Electric Arc furnace (EAF) without forming its by-product. Several slag types may be listed as: basic oxygen-furnace slag (BOS), blast furnace slag (BFS), ladle-furnace basic slag (LFS), electric arc-furnace oxidizing slag (EAFS), primary desulfurization slag (DS) and hearth furnace slag (OHS). There are at present many researchers studying suitable and quality applications for each slag type. The construction sector will without a doubt utilize most of these by-products.

In our research we used induction-furnace basic slag. In Bangladesh the total production of steel is estimated at 8 million tons per annum and production of slag is about 1, 20,000 to 1, 60,000 metric tons per annum (Nieri 2013). Because of high temperatures (about 1500°C) during the generation process, slag do not have any organic substances. The slag protects the metal from oxygen and controls temperature through a kind of lid formation. As slag is lighter than the liquid metal, they float and are easily removed. Slag is produced in a parallel route along the main processes of hot metal production and therefore the slag production process is considered as a part of the complete steel production process. In Bangladesh we obtained secondary steelmaking slag during melting process. No investigation on utilization of furnace slag as coarse aggregate in concrete has been carried out yet in Bangladesh as a complete replacement of brick coarse aggregate. Therefore, this study has been planned to find out the suitability of utilization of the slag aggregates in concrete as coarse aggregate with varying cement content for sustainable development.

2. Methodology

The slag aggregate was collected from a local steel manufacturing company. It was found that some slag aggregate were light in weight with a lot of voids and some were heavier with a little or no voids. Therefore, the slag aggregates were termed as the mixed slag aggregate (SM). The slag aggregate was tested for grading, unit weight, abrasion, specific gravity and absorption capacity and abrasion as per ASTM standard. Different types of aggregates investigated in this study are shown in Figure 1. Natural river sand was used as fine aggregate. The physical properties of coarse and fine aggregates are summarized in Table 1.

Table 1: Physical Properties of Coarse and Fine Aggregates

Type of Aggregate	Specific Gravity	Absorption Capacity (%)	SSD Unit Weight (kg/m^3)	Abrasion (%)	FM
BC (Brick Chips)	2.14	19	1211	38.8	Controlled as per ASTM- C33
SM (Slag-Mixed)	2.65	2.62	1550	35.2	
Fine Aggregate (Sylhet Sand)	2.59	3.30	1520	-	

Figure 1: Different Types of Aggregates (Left-Mixed Slag, Middle- Brick Chips, Right-Sylhet Sand)

The grading curves of coarse and fine aggregates satisfy the requirement of ASTM C33 are shown in Figure 2. CEM Type II/A-M cement (as per BDS EN 197–1:2000) was used. Tap water was used for mixing and curing of concrete. 100 mm by 200 mm cylindrical concrete specimens were made with s/a ratio (0.40); W/C ratio (0.40 and 0.45), and cement content was 340,395,400,450 (kg/m^3). Total 24 independent cases were investigated. A total of 264 cylindrical specimens were made. The mix design and case plans are summarized in Table 2 and Table 3 for each different replacement ratios (0%, 50% and 100%).

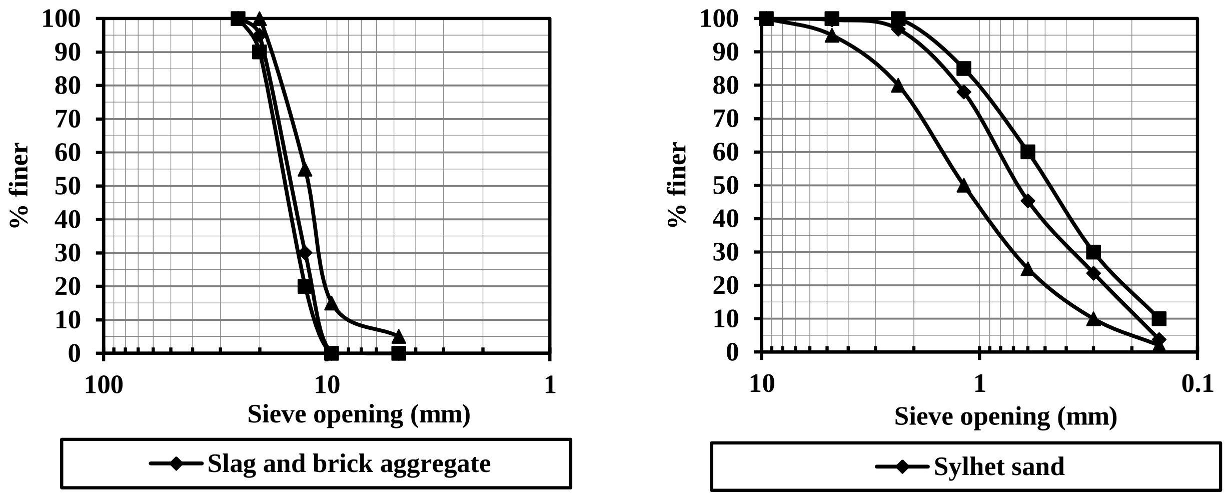

Figure 2. Grading Curve of Aggregates (Left - Coarse Aggregates, Right- Fine Aggregate)

After mixing concrete, slump was measured and then concrete specimens were made as per ASTM C31M–03. The specimens were cured under water till the testing for compressive strength. Before crushing of the specimens ultrasonic pulse velocity (UPV) was measured by using Pundit (UPV meter) according to ASTM C 597–02. Then the specimens were crushed by a compression machine. The specimens were tested at 7, 28 and 90 days.

Table 2: Case Plan and Mix Design

Cement Content Variations	s/a	W/C	Case ID	Unit Content, (kg/m3)			
				Cement	Water	Coarse Aggregate (Mixed Slag)	Fine Aggregate (Natural Sand)
340	0.44	0.40	RA-1	340	136	1079	828
		0.45	RA-2	340	153	1053	809
395		0.40	RA-3	395	158	1018	782
		0.45	RA-4	395	178	988	759
400		0.40	RA-5	400	160	1012	777
		0.45	RA-6	400	180	983	754
450		0.40	RA-7	450	180	957	735
		0.45	RA-8	450	203	924	709
Number of Case = 8 Cylinder Per Case = 3 × 3 (f_c'at 7, 28 and 90 days) + 2 ($f_{t\ at}$ 28 days) = 11 Total Cylinder = 8 × 11 = 88							

3. Result and Discussion

3.1 Workability

The effect of different cement content on the workability of concrete for different cement content (cc) and W/C ratio is shown in Figure 3.1. In this case, Induction Furnace (IF) slag used as coarse aggregate. The workability of concrete increases with an increase in the different cement content. It can be said that at a given slump, the water requirement increases as the cement content increases and total aggregate content decreases. It can be observed that, concrete made with brick aggregate has lower workability because of its high water absorption rate compared to the steel slag aggregate.

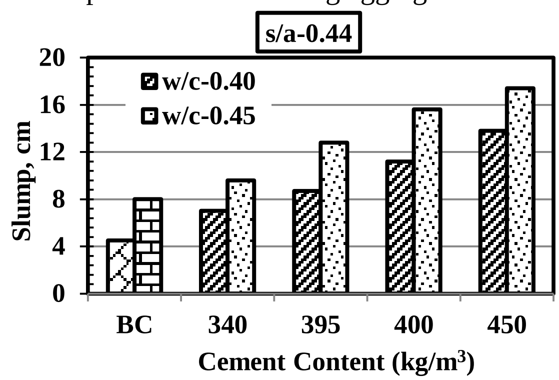

Figure 3.1: Effect of Different Cement Content on Workability of Concrete

3.2 Compressive Strength of Concrete

The effect of different cement content on 28 days compressive strength of concrete is shown in Figure 3.2. Four cement contents of 340, 395, 400 and 450 kg/m^3 were used in this study. Based on Figure 3.2, it can be summarized that, for W/C ratio of 0.40 and 0.45, the compressive strength increases with an increase of cement content. At high cement content, the failure initiates in the interface as well as within the aggregate, and visual inspection suggests combined failure. The improvement was due to good adhesion between crystallized slag aggregate and cement paste due to rough surface of slag aggregate (Nadeem and Pofale, 2012). On the other observation from the following plot, concrete aggregate made with brick chips has much lower strength at 28-days than concrete made with steel slag aggregate with different cement content. This is because of the high abrasion percentage of the aggregate which indicates its vulnerability under heavy loading.

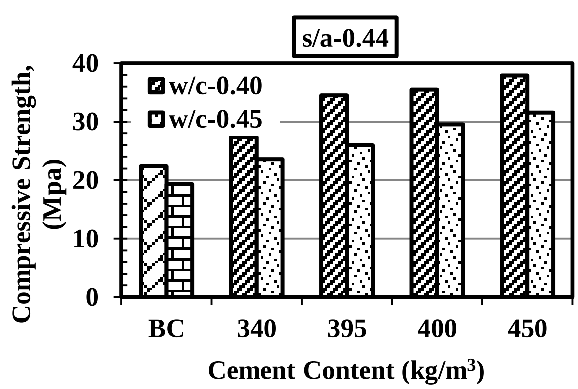

Figure 3.2: Effect of Cement Content on Compressive Strength of Concrete

3.3 Splitting Tensile Strength of Concrete

The effect of cement content on splitting tensile strength of concrete is shown in Figure 3.3. From Figure 3.3, it is evident that the splitting tensile strength of concrete increases with an increase of cement content and decrease with increasing W/C ratio (Demirboğa and Gül, 2006). Tensile strength test revealed that concrete made brick chips aggregate show lower strength than concrete made with steel slag aggregate. The increase in strength was due to the excellent rigidity of slag aggregate which ensured strong bonding and adhesion between aggregate particles and cement paste (Nadeem and Pofale, 2012).

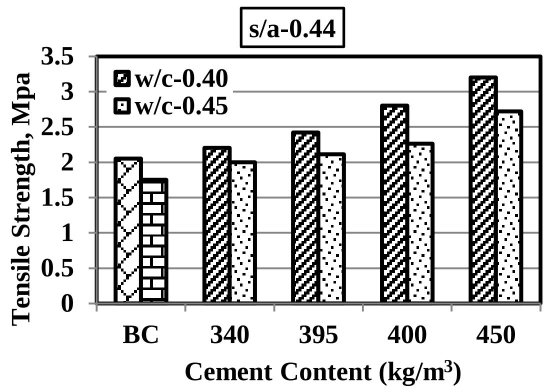

Figure 3.3: Effect of Cement Content on Splitting Tensile Strength of Concrete

3.4 Ultrasonic Pulse Velocity

The effect of different cement content on UPV through concrete is shown in Figure 3.4. It is clear that with the increasing of cement content the strength and the UPV for concrete tends to increase. Steel slag is an outcome of steel industry, it contains significant amount of iron content for an aggregate. So, concrete made with steel slag shows a faster UPV than concrete made with brick aggregate.
There is a relationship between UPV and Compressive strength of concrete with different cement content shown in Figure 3.4. It is clear that with the increase of cement content UPV and compressive strength of concrete tends to increases irrespective of s/a ratio and W/C ratio.

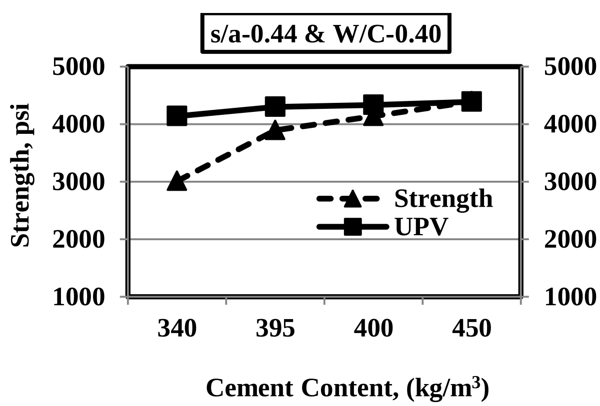

Figure 3.4: Strength and UPV Variation for Different Cement Content

3.5 Young's Modulus of Concrete

The effect of variation of cement content from 340 kg/m^3 to 450 kg/m^3 on Young's modulus of concrete is shown in Figure 3.5. As young's modulus is an indicator of how elastic the concrete is, thus it is very

important to understand the behavior of concrete made with particular type of aggregate. Based on Figure 3.5, it can be showed that the variation in Young's modulus due to variation in cement content is relatively more for concrete made with steel slag aggregate. On the other hand, with the increase of W/C ratio, Young's modulus of concrete is lower compared to higher W/C but with the increase of cement content Young's modulus of concrete increases.
Literature reveals that concrete with higher compressive strength gives higher Young's modulus (Yildirim 1995, Neville 1997).

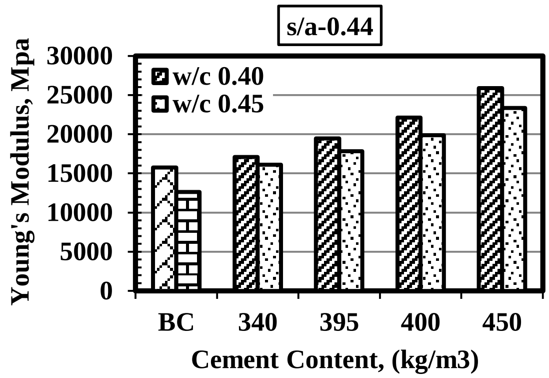

Figure 3.5: Effect of Cement Content on Young's Modulus of Concrete

4. Conclusions

From the extensive experimental investigation carried out on partial replacement of brick chips in concrete by steel slag, the following conclusions are drawn:

1. Steel slag in aggregate show better workability than brick chips aggregates.
2. Strength of concrete increases by using 400 kg/m^3 of cement content and mixed slag.
3. Concrete aggregate made with brick chips has much lower strength at 28-days than concrete made with steel slag aggregate with different cement content.
4. Concrete made with steel slag shows a faster UPV than concrete made with brick aggregate
5. With the increase of cement content UPV and compressive strength of concrete tends to increases irrespective of s/a ratio and W/C ratio.
6. With the increase of W/C ratio, Young's modulus of concrete is lower compared to higher W/C but with the increase of cement content Young's modulus of concrete increases.

5. Acknowledgement

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Estimation of GHG Emissions from Diesel-based Freight Transport

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Abstract

Freight and passenger transport accounts for approximately one third of total transport greenhouse gas emissions. Emissions from vehicles need to be accurately estimated to ensure that the air quality management is implemented appropriately. At present, Pakistan is lack in proper measurement and estimation of vehicular emissions. This study provides a framework for development of GHG emissions inventory.

This research aims at identification of procedures for estimation and monitoring of GHGs on road emissions, identification of significant pollutants from tailpipe emissions of diesel-based freight transport and to develop a relationship between vehicular emissions and speed. Emission data from four diesel buses were collected in Karachi. On-road measurements of vehicle emissions were acquired by using a portable instrument (IMR-2800). Exhaust emissions oxides of nitrogen (NO_x) and carbon dioxide (CO_2) have been reported in this study. Statistical analysis provided that both pollutants have non-linear relationship with the vehicle speed. Tailpipe emission rate initially decreases with the increase in speed and then after reaching at the optimum speed,NO_x and CO_2 concentration increases with increase in speed. The methodology developed may be utilized for in-depth analysis of GHG emissions from diesel-based freight transport which contributes large portion of urban and infrastructure traffic.

Keywords

Tailpipe Emissions, Diesel, Speed, Emission inventories, and PEMS (Portable Emission Measurement System).

1. Introduction

Diesel freight transport has been recognized as a one of the sources of emissions in Pakistan. The continuous growth in demand of diesel freight has led to an increase in fuel consumption and emissions, negatively impacting the environment and public health. Air pollution is a major risk to health and environment. (Arsalan and Mehdi, 2005). Outdoor air pollution is estimated to cause 1.3 million annual deaths worldwide (WHO, 2011). Emissions from freight transport account for approximately one third of total transport GHG emissions (Maykut *et al*, 2003). The share of transport emissions is continuously increasing and could reach more than 30% of total emissions by 2020 if no action is taken.

Transportation is a major source of global pollutants and contributing 21% in greenhouse gases worldwide (Gorham 2002). Transportation contributes 37.1 million tons CO_2 equivalent in 2008 and if it follows the same pattern emissions will be 66.6 million tons CO_2 equivalent in 2020 (Ministry P&D, UNDP). At present, traffic induced exhaust emission and energy consumptions is becoming one of the severest challenges for urban transportations in many metropolitans including Karachi. Accurately and effectively estimating and controlling urban traffic emissions and fuel consumption has increasingly attracted attentions from transportation professionals. The emissions resulting from freight transport can be estimated if emission inventory is known. The emission inventories have been established by many studies for developed countries. However, the emission inventory for conditions of Pakistan will be significantly different from the emission inventories used in developed countries. Therefore, one of the main objectives of this study is to identify the emission estimation technique based on our local conditions for the population of diesel buses used in freight transportation in Pakistan. Pakistan's healthy economic growth between 1990 and 2008 has led to significant increases in road freight traffic and this increasingly pollute climate .The transportation sector in Pakistan using road infrastructure is mainly informal and fleet of diesel vehicle mainly consists of old and altered diesel-based freight. These alterations in diesel buses bodies and engines result in higher emissions from road-based freight transport

2. Objectives:

Following are the objective of the study

- Identification of procedures for estimation and monitoring of GHGs on road emissions;
- Identification of significant pollutants from exhaust emissions of diesel-based freight transport;
- Develop a relationship between vehicular emissions and speed.

3. Identification of procedures for monitoring and estimation of GHG emissions

Environmental impacts of transport, such as atmospheric pollution, with its contribution to both local air quality and global warming have been prominent in research. Emissions can be characterized, regulated, or controlled only if they can be accurately measured. The increased health and environmental concerns about diesel emissions resulted in the development of a wide range of measurement techniques. Suitable methodological frameworks are required to enable the impacts to be assessed, and these methodologies require development, validation and testing. To measure vehicle emissions various techniques have been employed. These techniques include on-road measurements; mathematical model based estimation and fuel consumption based estimation of emissions and details of various techniques to measure emission from vehicle are presented in Table 1.

Table 1: Emission Measurement Techniques

Emission Measurement Technique	Explanation
On-road measurement of emissions	
Tunnel bore field sampling	(Miguel *et al ,1998*) conducted a study in San Francisco to measure vehicle emissions in a tunnel. During August, 1996 field sampling was conducted at the Tunnel. Separate samples were collected for uphill traffic in the two tunnel bores, one was for heavy-duty diesel trucks and second bore was for light-duty vehicles. Gas filter correlation spectrometry was used to determine concentrations of Carbon monoxide (CO), Carbon dioxide (CO_2) and oxides of nitrogen (NO_x).
Portable Instrument	(Frey *et al* ,1995) conducted a research in North Carolina in which on-road Measurement of Vehicle Tailpipe Emissions were estimated by using a

	Portable Instrument. For this purpose a study design procedure was developed and demonstrated for the deployment of portable on-board tailpipe emissions measurement systems for selected highway vehicles fueled by gasoline and E85.
Volume sampling	(Jayaratne *et al*, 2009) conducted a research in Australia to estimate average diesel and CNG emission rates. Total 22 test vehicles were chosen and tested out of which 13 were CNG and 9 were diesel buses. Vehicle speed was maintained at 60km/h. Measurements of emissions were carried out at four steady state engine loads. The system employed a continuous volume sampling method, where the entire exhaust from the bus was channeled into a flexible tube of diameter 300 mm. The concentrations of CO_2 and NOx were determined by dedicated gas analyzers.
Remote sensing	(Bishop *et al* 2001) conducted a study in the US and Europe to measure on-road emissions and the effects of altitude on Heavy-Duty Diesel Truck. The trucks were divided into two categories, loaded and unloaded. Measurements were collected through remote sensing at five different elevations.
SEMTECH-D instrument	(Chena *et a l*,2007) conducted a study in Shanghai for on-road emission measurement of heavy-duty diesel vehicles. Experiments were carried out in Shanghai on an urban highway, an arterial road and several residential roads. SEMTECH-D instrument was used to measure emissions of nine heavy duty diesel trucks
Chassis dynamometer	(Ristovski *et al* ,2005) measured the particle and carbon dioxide emissions from a fleet of 11 in-service new Ford Falcon Forte passenger vehicles in Australia. Measurements were made on the same group of vehicles in February, June, August and November. The vehicle emissions were monitored on a chassis dynamometer at five operating modes defined by road speeds 40 km/h, 60 km/h, 80 km/h, 100 km/h and idle speed.
On-board Measurement of Emissions	
Web-based support system	(Li *et al* 2009) presented a web-based support system in the U.S, to estimate and visualize the emissions of diesel transit buses where a micro-scale Vehicle Specific Power approach is used to estimate emissions based on global positioning system data.
A SEMTECH-D PEMS	(Liu *et al* 2009) conducted a study in Beijing and Xi'an, China, using a Portable emission monitoring system (PEMS). The goal of this research was to measure emissions from diesel trucks under actual on-road driving conditions using a PEMS and a total of 75 vehicles were tested. A SEMTECH-D

	PEMS was used for this test. The SEMTECH-D employs a flame ionization detector (FID) to measure THC, a non-dispersive ultraviolet (NDUV) analyzer to measure NOx, a non-dispersive infrared (NDIR) analyzer to measure CO and CO2, and an electrochemical sensor to measure oxygen (O2).
Mathematical Model based Estimation of Emissions	
Vehicle specific power (VSP)	The concept of vehicle-specific power (VSP) is a formalism used in the evaluation of emissions. (Zhai *et al* 2009) conducted a research in North Carolina with the aim of estimating emission rates of diesel fueled transit buses using VSP
Virginia tech (VT-Micro) model	(Rakha *et al* 2004) developed a framework by using microscopic emission models for estimating vehicle emissions. The original Virginia tech (VT-Micro) model was developed using a chassis dynamometer (A chassis dynamometer, sometimes referred to as a rolling road, measures power delivered to the surface of the "drive roller" by the drive wheels.
Comprehensive Modal Emissions Model (CMEM)	(Barth and Boriboonsomsin, 2009), conducted a study to carry out a variety of vehicle emission and energy studies. The development of the Comprehensive Modal Emissions Model (CMEM) began in 1996.
Fuel Consumption-based Estimation of Emissions	This technique calculates emission factors in grams of pollutant per unit of fuel used (kg, gallons or L) from remote sensing measurements. Combining these factors with fuel use data, available from concern authorities, yields a fuel based emission inventory.
Fuel Consumption-based Estimation of Emissions	
Remote sensing for Fuel-based Estimations	(Singer and Harley, 1996) carried out a research study in California to develop fuel based inventory for calculating vehicle emissions. A fuel-based methodology was developed and applied to calculate emissions of CO from cars and light/medium duty trucks, emission factors are normalized to fuel consumption and expressed as grams of pollutant emitted per gallon of gasoline burned.
Link-Based Estimation of Emission	
Single-wavelength Aethalometer	The link-based emission estimation method is another method for emission estimation rate. This approach is recommended to couple heavy-duty vehicle emission inventory estimation with transportation demand models. (Frey *et al* ,2008) conducted a study to estimate roadway link-based emission rates for heavy-duty trucks to be used in the emission inventory estimation and to quantify the impact of factors affecting truck Emissions.

Emissions model	The emissions estimates, then are compared with emission rates from EPA's emissions model (MOBILE5b) CARB's emissions model (EMFAC2000), MOVE MODEL and COPERT 4

4. Significant pollutants in diesel engine

When hydrocarbon fuel is burned with the correct amount of air in a diesel engine, the gases that are left are predominately water vapor, carbon dioxide, and nitrogen However, deviations from this ideal combustion lead to the production of some VOCs (volatile organic compounds), CO, NOx and PM (particulate matter). Diesel engines are substantial emitters of PM, CO_2 and NOx, but only small emitters of CO and VOCs (Bacha, 2001) . These gains in energy efficiency have however not been sufficient to outweigh the growth in emissions caused by larger transport freight volumes, due to a strong increase in global trade.

The concentrations depend on the engine load, with the content of CO_2 and H_2O increasing and that of O_2 decreasing with increasing engine load. None of these principal diesel emissions have adverse health or environmental effects. Diesel emissions include pollutants that can have adverse health and environmental effects. Most of these pollutants originate from various non-ideal processes during combustion, such as incomplete combustion of fuel, reactions between mixture components under high temperature and pressure, combustion of engine lubricating oil and oil additives as well as combustion of non-hydrocarbon components of diesel fuel, such as sulfur compounds and fuel additives. Total concentration of pollutants in diesel exhaust gases typically amounts to some tenths of one percent this is schematically illustrated in Figure 1.

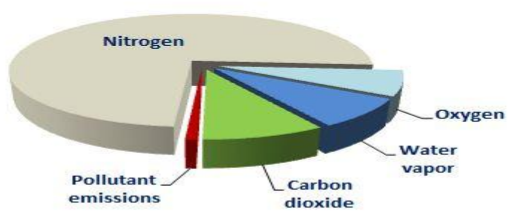

Figure 1: Relative Concentration of Pollutant Emissions in Diesel Exhaust Gas

This study quantify the significant pollutants from exhaust emissions emitted from diesel based freight transport and to drive a relationship between vehicular speed corresponding to estimated emissions.
It will furnish deliverable in form of freight transportation policy reforms along with recommendation for betterment of environment.

5. Methodology

5.1 Pilot study:

A pilot study has been done before the collection of real time data to identify the significant pollutants in diesel based vehicle. Two test vehicles has been selected, one has old engine and the other has new engine were tested in two conditions i.e. accelerated and non-accelerated. The Portable instrument, IMR-2800 was installed on back seat of bus. The device was connected to laptop computer to record second by second emission data. The emission recording probe was inserted in tailpipe of the test buses and the connecting pipe was attached to IMR-2800 device. The instrument measures continuously six gases: O_2, CO, SO_2, CO_2 and NO_2 with measured NO_X and temperature sensor measures gas temperature and ambient temperature. It was observed that among all pollutants the concentration of NO_x is higher. Observed data is presented in Figure 2

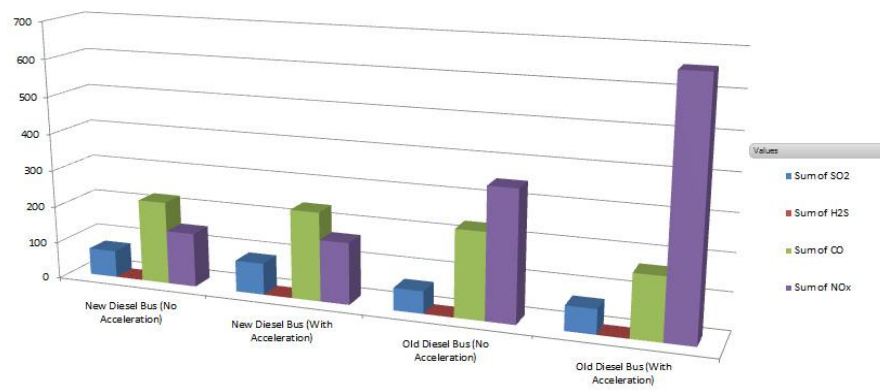

Figure 2: Relative Concentration of Pollutant Emissions in Test Vehicle

5.2 On-Road Emission Testing

In order to obtain the emission data under real-world traffic conditions, the selected route must be a regular bus route with passengers on-board. Real time emission data is collected when the selected bus goes out on its regular specified route. Vehicle emissions are measured from the main arterials in Karachi. Four diesel- buses were selected. The testing is carried out in such a way that one bus was tested for each day. The driver of the vehicle were asked to accelerate to their desired speed (maximum speed at which driver feel safe for a given road geometry and traffic condition).Exhaust emission data is measured by portable instrument IMR-2800 and speed data from Global positioning system (GPS) simultaneously with time interval of 2 minutes .All trips were made during free flow traffic condition.

6. Data Analysis:

In order to assess the effect of speed of test buses on tailpipe emission, regression analysis has been done between two selected pollutants (NO_X and CO_2) against vehicular speed. It is observed from data analysis that at the starting of the bus the emission is high whereas emission decreases with the increase in vehicle speed, when reaching to speed 60km/hr. concentration of CO_2 is increasing likewise NO_x is following the same trend. Figure 3 and 4 presented the trend of graph.

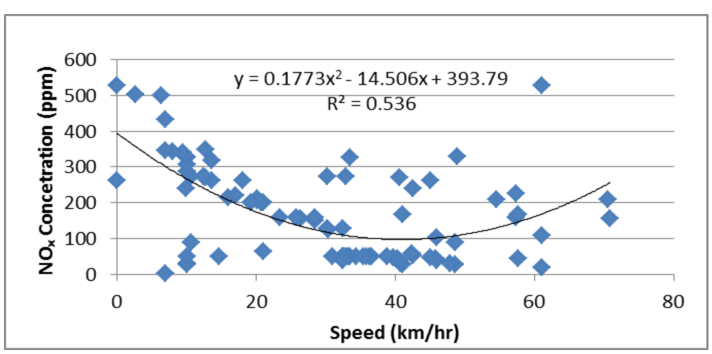

Figure 3: Estimated graph between NO_x and Speed values
Regression modal: y=0.177x^2-14.506x+393.79
y= NO_X Concentration (ppm)

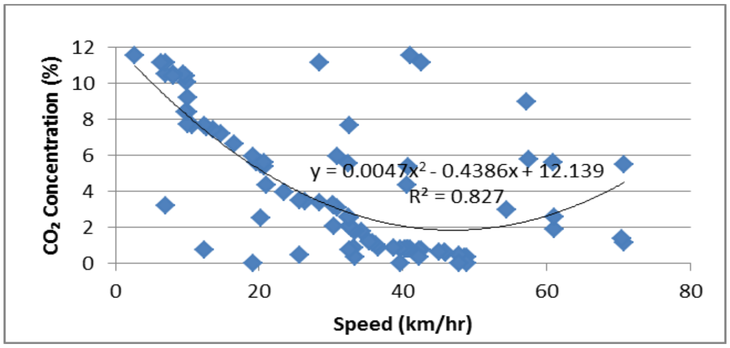

Figure 4: Estimated graph between CO_2 and Speed values
Regression modal: y=0.0047x^2-0.438x+12.13
y=CO_2 Concentration (%)

7. Conclusion and Recommendation

The first objective of this study is to identify the existing methodologies for estimation of emissions from vehicles are analyzed and studied through literature review. At present there is no inventory for estimation of GHG emissions from freight transport developed in Pakistan. Emissions in Pakistan are increasing heavily every year. This study concluded that mathematical model based emission estimation methods are most suitable for Pakistani scenario. Second objective of this study is to identify the pollutants from diesel based-freight transport .It is concluded that NOx acquire highest concentration among all GHG pollutants from the diesel-based freight transport .It can be reduce by installing the emission control devices and by adding the fuel additive (emission control catalyst).Third objective of this study is to analyze measured vehicle emissions in detail and in order to understand the measurement variation and the influence of operational variable (speed) on exhaust emissions from diesel based freight, a statistical analysis are carried out on the measured data. The real world exhausts emission concentration from the transit buses equipped with IMR-2800 and GPS and fueled with diesel are measured. Conclusion can be drawn from this study is vehicle tailpipe emission (NO_X and CO_2) is sensitive to vehicle speed tailpipe emission initially decreases with the increase in speed however; it increase further with increase in speed.

Driving patterns greatly influence the amount of vehicle emissions. Frequent acceleration and deceleration tend to generate more emissions than smoother driving. An effective traffic signal timing plan can smooth traffic flow in a manner that reduces the emissions. In addition, well planned transportation projects or activities can change the driving patterns of vehicles in the city at a more macroscopic level.

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Estimation of Passenger Car Units for Capacity Analysis Using Simulation Technique

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Abstract
In developing countries traffic flow is highly heterogeneous in nature, with vehicles of varying static and dynamic parameters. In traffic engineering different design and control features require to convert this traffic into equivalent traffic. In these conditions Passenger Car Unit (PCU) plays a significant role in converting mixed traffic flow into equivalent traffic flow for various planning and design calculations. The present study provides PCUs of different type of vehicles found on urban roads in Quetta at different volume-capacity (v/c) ratios. Traffic simulation software VISSIM is used to generate traffic flow and vehicle speeds under different conditions. A network model is created in VISSIM to replicate field conditions of study area and important VISSIM parameters are adjusted to reflect heterogeneous traffic conditions of study area which is further validated with field data. The study found out that PCU of different type of vehicles is dynamic in nature, and changes with the change in traffic volume and vehicle proportion.

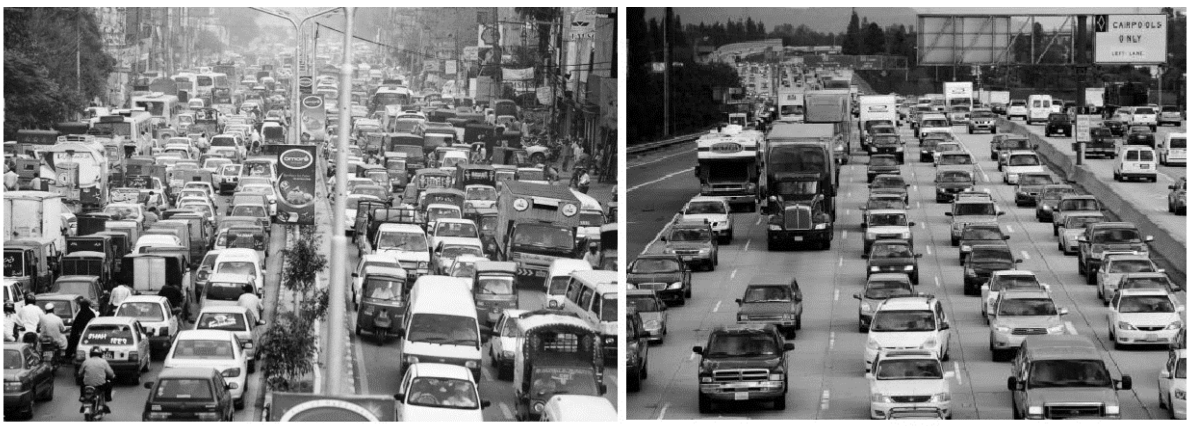
Keywords
Passenger Car Units, Heterogeneous traffic, VISSIM, Volume-capacity, Equivalent traffic

1. Introduction

Traffic on road consists of several types of vehicles, such as Passenger cars, Trucks, Bus, Light Commercial Vehicles (LCV), 3-wheelers, 2-wheelers, etc. In traffic engineering different design and control features require to convert this traffic into equivalent traffic. Passenger Car Units (PCUs) or Passenger car equivalents (PCE) is a vehicle unit to assess highway/road capacity. Road Capacity is expressed in passenger car per hour per lane. This is a measure of maximum number of vehicles that can pass through a point or a section of a road. The presence of heavy and small vehicles in traffic stream effects the capacity of road due to low or high performance as compared to passenger cars. Traffic volumes containing mix traffic must be converted into an equivalent flow of passenger cars using PCU values to estimate road capacity.

Quetta is the capital of Balochistan province and is located in southwest Pakistan close to its border with Afghanistan. Till 1947 Quetta was a small town, but rapid population growth in terms of rural-urban migration and influx of Afghan refugees turned Quetta into an over populated city. The city, which is home to approximately 759,941 people according to census 1998, however the current population of Quetta city increases up to 1.5-2 Million with a growth rate of 4.3%.

In Pakistan and other developing countries, traffic is heterogeneous in nature, consisting of various types of vehicles of different sizes and also has a lot of variation in speed, so fast moving vehicles cannot attain greater speeds. Moreover due to loose lane discipline situation becomes worse creating severe congestion, Different methods are in use in developed countries for determining PCUs, based on delays, headway, density, and queue discharge etc. But these methods are only suitable for homogeneous traffic conditions. On the contrary due to heterogeneous traffic conditions in Pakistan these methods for estimation of PCUs are not suitable thus modified or new methods should be used which are developed for heterogeneous conditions. The difference between heterogeneous and homogeneous traffic is shown in Figure 1a and 1b respectively.



(a) (b)

Figure 1. (a) Heterogeneous Traffic (b) Homogeneous Traffic

2. Review of Earlier Studies

The concept of PCE/PCU was first discussed in the Highway Capacity Manual (TRB, 1965) which introduced equivalent factor *(E_t)* for trucks and defined it as, “The number of passenger cars displaced in the traffic flow by a truck or a bus, under the prevailing roadway and traffic conditions”. (Werner & Morrall, 1976), used headway as performance measure to estimate PCUs of heavy vehicles on level terrain, that study is further modified by (Seguin, et al., 1982), by using spatial headway methodology and formulated the PCEs as the ratio of the average headway of vehicle type to the average headway of passenger cars. (Cunagin & Messer, 1983) used relative delay as performance measure to estimate PCUs on multilane rural highways, and also examined the effect of grades on PCU/PCE. (Aerde & Yagar, 1984), devised a method to calculate PCEs for two lane highways using relative speed reduction. Field data and Speed-Flow relationship curve was used to define multiple linear regression models, which were further used for estimation of percentile speed and speed reduction coefficients of different vehicles. The ratio of speed reduction coefficients of reference vehicle (passenger car) to the subject vehicle gives PCUs of subject vehicle. (Tanaboriboon & R. Aryal, 1990), used average headway method to find the effect of large vehicles on road capacity. (Al-Kaisy, et al., 2005), evaluate the PCEs for vehicles using queue discharge flow as a performing measure, the main assumption of his work is that during queue discharge the capacity is constant and v/c ratio is equal to 1. Density method is useful to estimate PCUs as it uses field data in terms of lane width and traffic density, and devised for pure homogeneous traffic conditions. (Tiwari, et al., 2000), modified the density equation for heterogeneous traffic conditions in India on 6 different highways. (Chandra & Kumar, 2003), studied the effect of lane width on PCU values on two lanes highways. (Mallikarjuna & Rao, 2006), model the Passenger car equivalency under heterogeneous traffic conditions in India. In the study Cellular Automata (CA) based simulation model was used to estimate the PCEs for different vehicles. (Arasana & Arkatkarb, 2011), derivate the capacity standards for intercity roads carrying heterogeneous traffic using traffic-flow simulation model HETEROSIM. (Bains, et al., 2012), presented the Modeling of Traffic Flow on Indian Expressways using Simulation Technique to estimates the passenger car units of different vehicles at different velocity-capacity (V/C) ratios using micro-simulation traffic software VISSIM. (Mehar, et al., 2014), estimated the Passenger car units at different level of services for capacity analysis of multilane interurban highways in India with the help of microscopic traffic simulation software VISSIM.

3. The Simulation Model

Planning Transport Verkehr (PTV) Groups Karlsruhe, Germany develops VISSIM for modeling and analyzing of complex traffic flows under different conditions. It is time step and driver behavior based simulation model designed for urban and highway traffic, public transport operations and pedestrian flows. VISSIM has the ability to perform under different constraints such as lane configuration, vehicle composition, traffic signals etc. VISSIM is based on car-following algorithm model developed by Wiedemann, i.e. Wiedemann 74 & 99. The basic idea behind the Wiedemann model is the assumption that a driver can be in one of four driving modes i.e. free driving, approaching, following and braking. In present study VISSIM 6.0 car following model Wiedemann 99 was used for calibration of software, which is based on 10 parameters, CC0-CC9 to define different scenarios of driving behavior. In this study parameters CC0 (stopping distance) and CC1 (time headway) was adjusted, that effects the individual driver behavior (Chitturi & Benekohal, 2008).

4. Study Area & Data Collection

The road section selected for the study is an urban road in Quetta. The study section was selected after preliminary survey such that it must satisfies following conditions: 1) The study section should be straight, 2) The width of roadway and shoulders should be uniform, 3) No direct access on the section from adjoining areas, and 4) availability of vintage point. The selected study road is a 4 lane divided road with a central median of 3 feet, the total width of roadway in one direction is 26 feet (8m) without any lane marking.

Traffic flow and traffic speeds were collected for two hours using video recording method after setting a video recorder camera at best available vintage point. A trap length of 90 feet was marked on the road section in front of vintage point. Average traffic volume observed at study location was 2212 vehicles per hour. Speed of each vehicle type observed in the count was estimated by measuring the time taken by each vehicle to cover the trap length. The maximum, mean and minimum speed for all vehicle categories with percent share and dimensions are shown in Table 1.

5. Calibration & Validation of Model

Calibration is a process in which simulation software parameters are defined or adjusted so that the simulation model replicates the observed field conditions to a sufficient level. Simulation model represents the original field condition in software, therefore its capability of replication must be verified before implementation. Calibration of simulation model is very important and difficult task, as the default model parameters in VISSIM are not calibrated for Pakistan conditions therefore model parameters were adjusted according to local observed conditions. In this study following parameters were adjusted for model calibration: Desired speed distribution, lateral distances between vehicles, and car following model parameters based on wiedemann-99. Two parameters of car following model, CCO (standstill distance) and CC1 (desired headway) were considered for this study as these two parameters have the direct influence on the traffic flow characteristics (Mehar, et al., 2014) and all other parameters remain at their default values. The calibration process of model is shown in Figure 2. The VISSIM was calibrated at low value of parameter CC0 (standstill distance) and CC1 (headway time) as shown in Table 2. Low values of parameters suggests the aggressive behavior of drivers in Pakistan as compare to developed countries.

Table 1. Traffic Composition and Speed Distribution

Vehicle Category	Percent Share (%)	Observed Minimum Speed (km/h)	Observed Mean Speed (km/h)	Observed Maximum Speed (km/h)	Dimension (m)
Passenger Car	13.29	28.29	37.39	56.43	3.5 - 4.7 x 1.4 - 2.0
Rickshaw	21.15	13.01	28.65	41.75	2.90 x 1.20
Bike	34.49	15.96	35.69	57.58	2.0 x 0.85
Bicycle	20.5	13.22	18.56	28.66	1.9 x 0.76
Tractor	0.29	16.85	19.125	20.11	7.7 x 2.2
Bus	1.67	14.69	27.02	38.8	10.13 x 2.5
Truck	0.79	18.47	26.72	36.44	10.13 x 2.5
Mini Bus	1.74	23.68	35.15	44.38	4.7 x 1.69
LCV	5.56	25.28	33.97	51.3	4.7 x 1.7
Animal driven cart	0.49	9.43	10.85	13.37	3.5 x 1.9

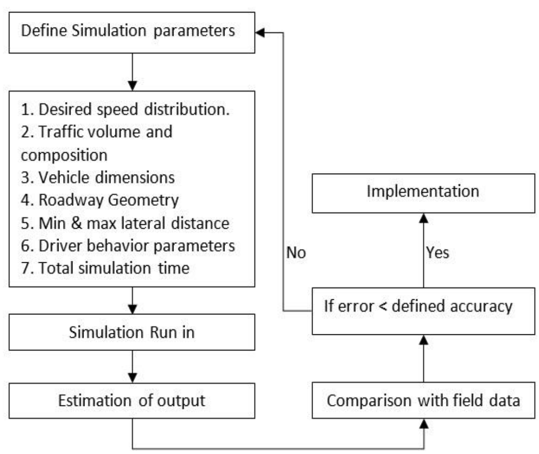


Figure 2. VISSIM Calibration Process

The other parameter which was adjusted is general vehicles lane change behavior, by default it was 'slow lane rule' but for replicating the field conditions it was selected as 'free lane selection'. The desired position at free flow speed was set as 'any position' instead of 'middle of lane' as observed during the data collection on study location. On the other hand the presence of small vehicles in gaps between two large vehicles on roads, the lateral distances was reduced from 1 to 0.40 at zero speed and 0.70 at speed 50km/h.

Validation is a process in which simulation results obtained from calibrated model is verified by comparing with field observation and data. Vehicles average speed and flow was set as performance measure for the validation of VISSIM. To check the validity of model the results obtained from simulated model was compared with field data as shown in Figure 3. It can be seen that observed and simulated speeds and flow are closer for all vehicle categories. This implies that simulation model in VISSIM is successfully calibrated according to field road and traffic condition.

Table 2. Adjusted VISSIM Parameters

Parameters for calibration	**Default Values**	**Calibrated Values**
CC0 (Standstill Distance)	1.50 m	0.70 m
CC1 (Headway Time)	0.90 sec	0.65 sec
General lane change behavior	Slow lane rule	Free lane selection
Desired position at free flow speed	Middle of lane	Any position
Minimum lateral distance @ 0 km/h with other vehicles	1 meter	0.40 meter
Minimum lateral distance @ 50 km/h with other vehicles	1 meter	0.70 meter

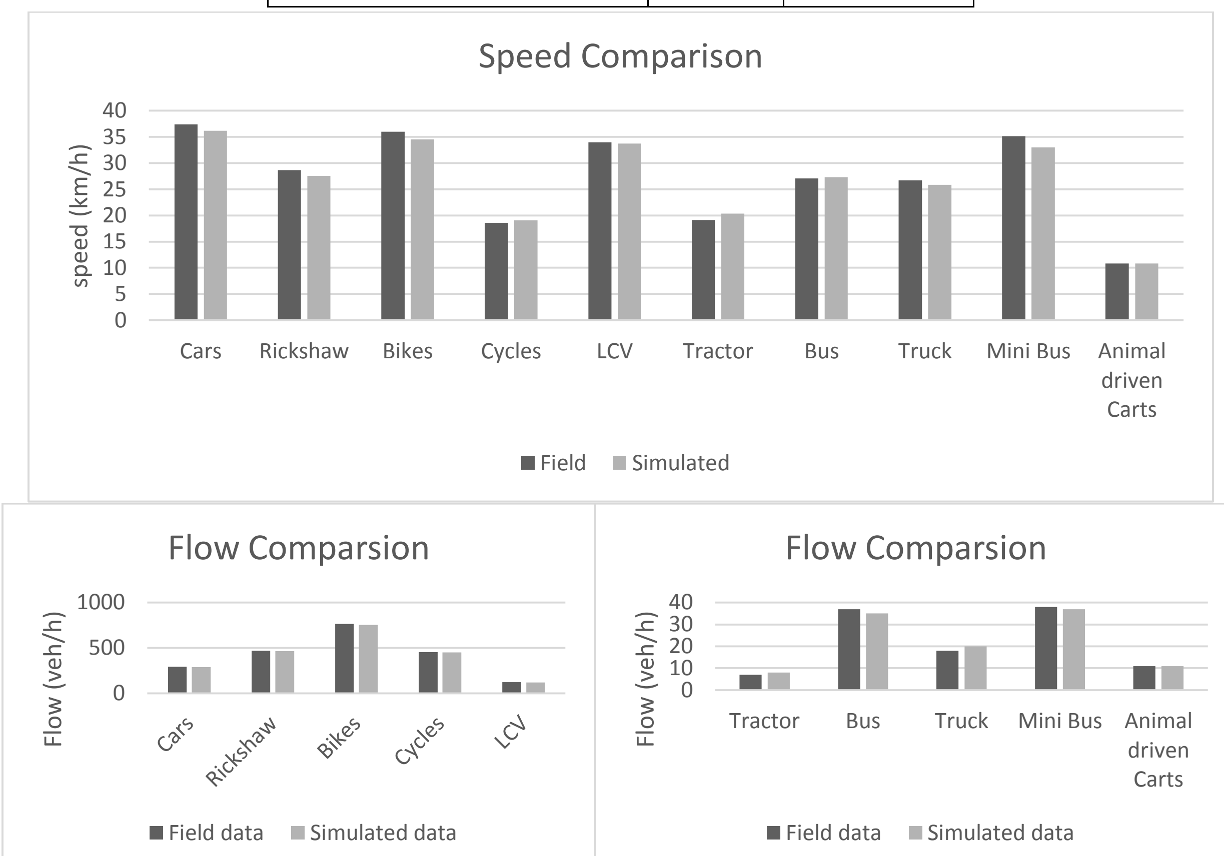


Figure 3. Comparison of Field and Simulated Data

6. Implementation of Model

After successful calibration and validation of model in VISSIM can be applied for different scenarios having various traffic conditions. In this study, first road capacity has been estimated by developing relation between traffic volume and speed under mix traffic flow as observed in field. It has also been used to estimate the impact of vehicle categories on traffic flow in terms of PCUs at different traffic flow levels. Further the model has been used to assess the effect of vehicle composition on PCUs.

7. Speed Flow Relationship and Capacity

The capacity of roadway under mix traffic condition was estimated with calibrated simulation model in VISSIM. Speed-volume curve was developed by increasing input traffic flow starting from 400 vph. As the input volume was increased the output volume at the end of simulation run is also increased comparatively, but after successive simulation runs the increase in input traffic volume have no effect on the output volume at the end of simulation run, this implies that the facility has reached at their capacity level. The capacity of road was estimated 3800 veh/hour in one direction under mix traffic conditions having same proportion of vehicle as observed in field. Figure 4 illustrate the stream speed-flow curve, where stream speed is the average speed of all vehicle categories obtained from VISSIM output results.

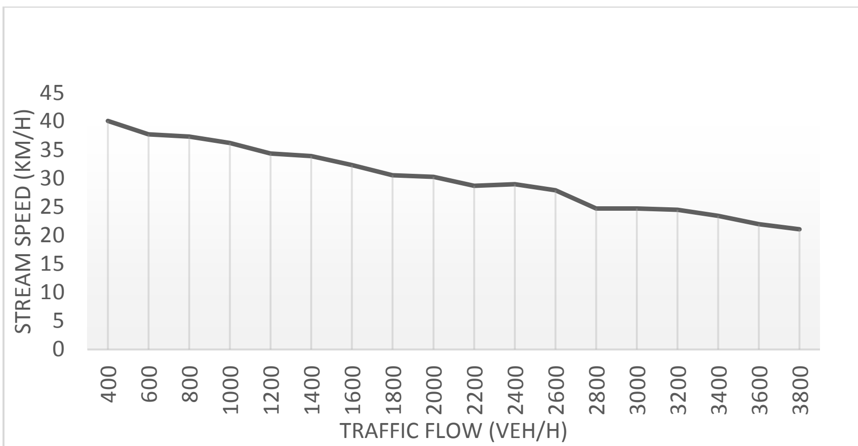


Figure 4. Stream speed and flow relationship for mix traffic

8. Estimation of PCUs

As discussed earlier the capacity of road under heterogeneous traffic conditions is best expressed in terms of PCUs per hour. To find the impact of different vehicle categories on traffic flow in terms of PCU (Chandra & Kumar, 2003) suggested equation as:

$$PCUi = \frac{Vc/Vi}{Ac/Ai} \qquad (1)$$

Where PCUi is the passenger car unit for vehicle type i, V_c is averaged speed of passenger cars, V_i is the average speed of vehicle type i, A_c is the projected area of passenger car, and A_i is the projected area of vehicle type i.

PCU is dynamic in nature and depends upon road and traffic parameter. To find the effect of traffic volume on PCU values, calibrated VISSIM simulation model was run at 0.125, 0.25, 0.375, 0.5, 0.625, 0.75, 0.85, and 1 volume to capacity (v/c) ratios. Output obtained from simulation model in terms of average speed served as input for equation 1. Table 3 shows the PCU values for all vehicle categories at different v/c ratios while passenger car is taken as reference vehicle.

It can be seen in Table 3, heavy vehicles such as Bus, Truck and Tractor shows highest values. It is due to the fact that heavier the vehicle, lesser its maneuverability, greater hindrance to other vehicles, hence larger PCU values. Further it can be observed in above Table that PCU values decreases as volume-capacity ratio increases. This phenomena occurs as speed differences between subject vehicle and reference vehicle (Passenger car) decreases as v/c increase due to congestion. Small vehicles like bike have less impact on passenger cars in terms of PCU as shown in Table 4. As small vehicles penetrates between two large vehicles and also have the ability to maintain higher speed as compare to other vehicles. Animal driven carts also shows higher PCU values due to very low speed as compare to passenger cars.

Table 3. PCU at v/c ratios

Vehicle Category	0.125 v/c	0.25 v/c	0.375 v/c	0.5 v/c	0.625 v/c	0.75 v/c	0.875 v/c	1 v/c
Rickshaw	1.48	1.18	0.95	0.82	0.72	0.68	0.67	0.64
Mini-Bus	2.10	1.79	1.56	1.30	1.26	1.22	1.43	1.37
Bus	9.39	8.25	7.02	5.71	5.19	5.07	5.12	4.76
Bike	0.47	0.39	0.33	0.30	0.28	0.28	0.28	0.26
Bicycle	0.94	0.75	0.60	0.51	0.42	0.39	0.38	0.34
Truck	9.75	7.66	7.02	6.15	5.19	5.21	4.80	4.80
LCV	2.33	1.98	1.70	1.56	1.54	1.47	1.38	1.38
Tractor	9.58	8.50	6.15	5.71	5.45	4.51	4.19	4.35
Animal driven cart	7.62	6.12	4.94	4.14	3.39	2.93	2.95	2.41

9. Conclusion

The main focus of this study was to estimate Passenger Car Units for all vehicle categories observed on urban roads of Quetta at different volume-capacity ratios. In present study many interesting conclusions were yielded. PCU is dynamic in nature and changes with the change in traffic volume and composition. From study it was concluded that PCU values for all vehicle categories decreases as the traffic flow increases, and at the capacity of road lowest values of PCU was observed under heterogeneous traffic conditions. Further it was observed that at the same traffic flow level, PCU values decreases as the proportion of subject vehicle increases. Microscopic traffic simulation software VISSIM is capable to simulate heterogeneous traffic flow up to satisfactory level. The model was calibrated at low values of standstill distance between vehicles (CC0) and time headway (CC1) which shows aggressive behavior of drivers in Quetta.

10. Acknowledgement

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Technological Advancements and Smart Transportation Planning in Asian Mega Cities

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Abstract

By 2050, there would be more than 50 megacities in Asian developing countries, including China, India & Pakistan. The prognosis of IEA concerning car ownership in Asian developing countries is about twenty-fold between 2000 and 2050. The mega cities in Asia, with more than 10 million people without apposite rail transit systems caused roughcast cataclysmic traffic congestion and producing unacceptable levels of CO2 emissions. Whereas, the technological advancements and smart transportation planning in Megacities across the globe are quite accessible manifest and it requires a paradigm shift in Asian mega cities to adopt smart urban transport planning approaches by considering the available technological advancements and its financing systems. This paper aims at addressing these transportation planning issues in Asian megacities by analyzing the available technological advancements and smart technologies concerning transportation planning in developed world and process of applying the learned lessons in Asian Cities of China, India and Pakistan. The scope of findings include the public, informal and private sector motorized and non–motorized transport, along with goods transportation planning solutions for Asian megacities.

Keywords

Mega Cities Planning, Mega Cities Management, Mega Urban Engineering, Urban Transport Planning, Smart Transport Planning

1. Introduction

Asian Megacities are quite exclusive urban contexts in terms of rapid urbanization, motorization and future contests for governments and urban transport authorities to deal with indefinite urban mobility demands. The condition of urban transport in Asian megacities is getting worse and policy makers grasps the gravity of the problem and have taken various measures to improve the situation. However, severe resource constraints are quite evident and inevitably there is huge demand supply gap of public transport and mass transit facilities in some Asian megacities. Due to some explicit individualities of Asian megacities, the successful policy solutions tested somewhere else may not produce similar results in this part of the world. This calls for exploring innovative policy measures by considering special features of Asian megacities and at the same time learn from the experiences of megacities in developed countries.

In this regard it is important to develop short-term, medium term and long-term scenarios while making any transportation planning policy or strategies for addressing the demands of growing urbanites in Asian megacities. It is argued by the governments and urban transport authorities that, investment for urban rail system, are quite resource intensive and not feasible at the early stage. Therefore they take stop gap less resource intensive short term measures.

As a repercussion the situation degrade further and continues for a very protracted time, which in turn witness more technologies advancements in urban transportation around the globe during that period and urban transport solutions that were just expensive, becomes more expensive or from luxurious to lavish and beyond the affordability levels of local governments and transport authorities in Asian megacities. Therefore it is necessary that localized alternative technological options must be explored. Another significant aspect of Asian megacities is complex urban form and unpredictable status of settlements and trend of their physical growth and development. The general lack of regard to traffic rules and regulations and an apathetic behavioral pattern of people in some Asian megacities is another evident fact that provides solid reason to develop a 'do-nothing scenario' for transportation planning. The 'do-nothing scenario' means keeping the business as usual due to financial constraints, technological inadequacies, priorities of decision makers and because of socio-political realities of an urban context which varies to a greater extent in Asian megacities. Therefore, a short term strategy is adopted to provide a temporary relief to citizens and in this way urban transportation planning issues are addressed in Asian megacities.

However, at the same time the urban transport authorities and institutions do not remains static but clearly indicates a realization about the urban transportation needs of urbanites and develop medium term 'muddle through scenarios' as an appropriate transportation planning choice. The 'muddle through scenario' means exploration of possibilities within constraints, inadequacies and priorities of an urban context. This is a kind of medium term strategy to provide urban transportation facilities and evaluate its impacts for a specific period of time. In 'muddle through scenarios' the lessons are learned from the applied urban transportation solutions and taken further steps ahead with adoption of new solutions. In this way regular muddle through process for urban transportation planning continues in Asian megacities. In this regard this paper present arguments for a paradigm shift as 'desired scenario' for Asian megacities by reviewing technological advancements and smart transportation planning techniques across the megacities around the world until this point of time. This would lead to find out the long term strategy for planning urban transportation in Asian megacities. The 'desired scenario' means necessity for the future generations or exploring the possibility of doing everything by delineating short term and long term goals with wide-ranging resolve and grit to find a solution for urban transportation planning in Asian megacities.

2. Method

The method adopted by this paper is based on the knowledge domains of urban engineering, urban transportation planning and megacity planning and management. It is ensured to be focused on globalized and localized technological advancements that emerged through innovation based knowledge corridor. It include detailed studies of transportation planning in megacities in Asia and around the world via literature review of recently published books, research papers and reports concerning technological and social advancements in urban transport planning and smart urban transportation planning approaches adopted in the cities of developed and developing countries. In addition to that, a common citizen's perspective concerning; public sector urban transport, informal and private sector motorized and non–motorized transport, and goods transportation planning solutions is also deliberated with local city planners and urban engineers to project an all-inclusive future urban transportation planning solution for Asian Megacities. The lessons learned from these studies are outlined in the following results.

3. Results

The provision of high quality social services, amenities, and metropolitan life style are beyond the control of governments but achieved by the competitive nature of megacities. In current era of globalization and knowledge-based economy, livability of city is the key component to entice highly skilled people to megacities and generates potential for new industries. As a result efficient urban transport system emerges that further enhance the livability of a city and create a long-term urban competitiveness and resilience.

3.1 Technological Advancements in Urban Transport

In recent times various technological advancements are taking place in urban transport sector of megacities around the globe. According to UN in 1950, there were just two megacities of 10 million people i.e. New York and Tokyo. In 2015, from total 29 megacities 16 exists in Asia. By 2030, there will be 41 megacities around the globe and Asia and Africa will be the main destination of these emerging megacities. Within this perspective discussing about technological advancements in urban transportation sector in each megacity is quite unmanageable. However, it would be appropriate to explain how technological advancements would transform the way people and goods move over the coming decades and it would definitely depends on the life-cycle of planning within Asian megacities and the range of impacts of transportation investments made by the national, regional and global financial powers. The megacity planners and urban transportation engineers around the globe anticipates that in future five categories of technological innovations may change the way people and goods move in Asian megacities.

At first it is the improvements in the driving capabilities of people where individual driver would have technological support and need less driving skills to safely drive their vehicles. It is a common observation that, many automatic cars are running on the roads with many technological features such as cameras to easily reverse the cars, way finding to find routes and destination and the automatic safety and safekeeping of their vehicles. Secondly in future new technology improve the taxi services and public transit modes with the use of mobile phones and computer internet applications. As evident in almost all megacities that, people hire taxies by a telephone call and sms services. E-ticketing is quite common feature of latest bus services, railways and air travel. Bookings for seats, its confirmation and cancellation takes place via email and mobile phone services. In megacities of developed countries there are transportation network companies who arrange for all kinds of transport required by travelers and goods moving from one place to another. Similar services may also be developed in Asian megacities. The third domain of technology application is for freight traffic and urban goods movement from port to wholesale market and from wholesale market to retailers. Now-a-days people do online shopping and goods are delivered to their doorsteps. Similarly a simple shopkeeper at any location can buy and sell their goods from international markets and all kinds of import and export of goods can takes place through use of cell phone and internet. The fourth sphere where future technology making its mark is the mass mobility of people and goods. For instance mass transit railways are already in practice in the megacities of developed countries (See Fig 1). The final realm of technological advancements in urban transport is record keeping about quantity and nature of goods, number of travelers, their management and timely reach in homes and workplaces in megacities.

The aforementioned technological perspective about urban transport in megacities of developed countries clearly indicates that there is a potential of adaptation and adoption of appropriate processes from these technological directions with localized solutions in Asian megacities. These technological advancements are an inevitable reality of the future and quite accessible manifest if appropriate policy is made and implemented with wholeheartedness in Asian megacities.

3.2 Smart Urban Transportation Planning Approaches

Cervero (2013) clarified that, in last two decades smart growth and compact city development are main urban transport policy determinants in North America, Europe, and Australia. The reduction in greenhouse gas emissions (GHG) and fuel consumption is considered very significant because Brazil, China and India, shown an American-style of suburbanization patterns, car ownership, and travel. By 2050, urbanites will be 70 percent of total inhabitants in the world and 90 percent of this growth will be in the Global South. Each year new mega-cities will emerge for the next 20 years with a proliferation of urban slums, wide income disparity and demands for basic urban services i.e. food, clothes, shelter, education, health and transport.

Morichi (2005) explained that, the distinctive facet of Asian urbanization is the absorption of populace, trade and industry in megacities which caused heavy demand on urban transport. The study of urbanization experienced in Japan indicates that, Japanese government along with their businessmen developed housing for the rural migrants and generated jobs for their sustenance. Japan Public Housing Corporation developed housing development projects and avoided the problem of urban squatter settlements as currently evident in some Asian megacities with unsound institutional mechanism to deal with this reality. Tokyo, Osaka and Nagoya were the three major urban centers that emerged as engines of economic development in Japan which faced population pressure and it was Japan's national development plan that paved way forward to technological advancement and smart technologies for their urban transportation development. However the repercussion of developing technologically advanced smart transportation in megacities also caused income inequality in contiguous rural regions of Japan. It was addressed by relocating manufacturing plants to peripheral regions and development of urban and regional transportation took place with second job generation for farmers in rural regions which in turn increased their income and reduced economic disparity. By 1990's the rapid motorization witnessed by Asian megacities however, in developed countries, extensive urban rail networks were made well before the motorization gained popularity among urbanites. Developed countries made available the high quality public transport to populace of megacities and stifled their speed of motorization. On the other hand Asian megacities witnessed no urban rail network and their main public transport mode was orthodox bus, the service level of which is not comparable to the comfort and convenience of mass transit railways.

This situation further accelerated motorization since car and motorcycles use appears to be a necessity rather than a choice for dwellers of Asian megacities. It is projected that by 2050, the number of motorized vehicles across the globe will reach up to 2.6 billion and the majority of which will be in developing countries, especially China, India and other Asian countries. China alone will have 800 million private automobiles by 2050 which is currently two-thirds of today's global car ownership. Rapid motorization inevitably shifted future travel from sustainable public transport i.e. non-motorized modes of walking and cycling to private vehicles. Today private vehicles make up half of all urban trips around the globe (Pourbaix 2011). The projected daily trips in urban areas by private cars may rise 80 percent from 3.5 billion in 2005 to 6.2 billion in 2025 (Pourbaix 2011). Much of this growth will be in developing countries. In some rapidly emerging economies like India and Pakistan the number of cars, trucks, long vehicles and motorcycles in the city roads growing at the rate of more than 20 percent annually (Pan et al. 2011). The huge demand for urban transport infrastructure in Asian megacities is not possible without necessary fine-tuning in infrastructure investments and making available the adequate financial resources. The narrow tax base in Asian megacities generates far below government revenue in comparison to required level of meeting the contending demands of urban mass transit railways.

This situation worsened further since mid of 1990s, because the major international financial institutions and donors shifted their priorities from investing in infrastructure sector development projects to development of social sectors in the Asian megacities. The major reason for this policy shift was their anticipated or rather assumed growing potentials of Privately Financed Initiatives (PFI) to invest in the infrastructure sector of Asian megacities. However, they remained unable to make any impact and failed to develop urban transport infrastructure in Asian megacities and their inadequacy created severe situation on urban roads. Calcutta, Shanghai and Bangkok are three main examples where road space has critically reduced to accommodate growing motorization and travel demand of metropolitan citizens (Morichi, 2005). Megacities of the developing world suffer from the worst poverty, congestion and airborne pollution (Suzuki et al. 2013). Cervero (2013) spell out that, whatever is done to improve urban transportation in Asian megacities must be pro-poor and it would require land-use integration with mode of urban transport. The accessible urban activities must be attractive with safe walking and cycling environs which are particularly vital to the welfare and prosperity of urbanites in the Asian megacities. Some megacities relies on flexible simple forms of mass transportation i.e. auto rickshaws, motorcycle taxis, and private cabs serve the multi-directional, less lineally focused travel patterns. In this way few roads and restricted geographies may translate into high ridership in high-capacity vehicles.

Therefore, trade-off between road space, urban form, transit provisions and transit usage is quite necessary for Asian megacities (Cervero, 2013). The motorization trends, contrasting urban forms, higher densities and rapid decentralization created poor road hierarchies and spatial mismatches in built environments of Asian megacities which demands for mass rapid transit. It need understanding of 5Ds i.e. density, diversity, design, distance to transit, and destination accessibility to shape the travel of people and goods (Cervero, 2013).

Common Citizen's Perspective:

The common citizens requires an integrated pedestrian and vehicular urban transportation network and its essential infrastructure in the Asian megacities. The pedestrian-friendly environments are in short supply in Asian megacities that when they do exist, they are found to strongly influence how people travel. Walking quality has important age and gender dimensions. Environments designed with more street lighting and a mixture of land uses generate foot traffic of women, children and senior citizens (Meleis 2011). Expanded, improved, and better-connected pathways are sometimes important features of slum upgrading programs. Design features like smaller city blocks can also encourage foot travel in developing megacities (Suzuki et al. 2013). The best idea for advancing urban mobility in fast-growing parts of the world need political will, institutional capacity and ability to manage and respond ever-increasing demands for urban travel (Dimitriou 2011). The major problem for citizens in some Asian megacities is the rapidly occurring land use changes and its link with public transport. Concerning public sector urban transport common citizens of Asian megacities requires a mass transit railway because it changes the whole city structure through an organized building system and also creates new land use pattern and economic opportunities for the people.

Similarly, in Asian megacities there exists an informal sector that develops alternative urban transport, and encroach upon road space and hamper the formal sector's efforts for provision of urban transport solutions. The citizens requires a regulatory mechanism to curb this informal situation and ensure formal apt urban transport in Asian megacities. Another issue of citizens is smart parking facilities required for their vehicles and control over private sector motorized vehicles i.e. old busses, minibuses, coaches, rickshaws, and other non-typical modes of transport that dominates the urban roads and cause congestions in Asian megacities. In addition non–motorized transport i.e. animal driven cart, push carts and other such vehicles also dictates the speed of vehicular traffic in historical districts of Asian megacities. Finally it is commonly observed in some Asian megacities that, very large trucks, long goods trawlers and other heavy vehicles use major urban roads. Whereas, a generic disregard to traffic rules and road use is also quite commonly evident in some Asian megacities. Therefore, citizens also requires an appropriate solution for goods transportation planning in Asian megacities. Based on aforementioned results discussion would abridge the paradigm shift by addressing technological advancements and smart transportation planning approaches as needed for the Asian megacities.

4. Discussions

At first by considering the technological advancements in urban transportation across the globe it is high time for decision makers in Asian megacities to develop technical expertise and manpower in above outlined five domains of innovation based knowledge corridor of urban transportation planning so as to compete with the world through using the potential of adaptation and adoption of appropriate processes for developing localized technological advancements. Secondly, city population is the key factor in determining about provision of a mass transit railways in megacities of developed countries. The recent population increase, physical size and morphological spread of some Asian megacities demands for introducing metropolitan railways at present. However, they remained too late to decide about this reality on time as their population growth remained unaccounted for while making urban transportation planning decisions.

The optimal timing for making decision about mass transit railways in Asian megacities is very significant because too early investment causes financial difficulties in operation and maintenance and too late decision makes it unfeasible forever. In order to make mass transit railways successful balanced distribution of population and economic activities among different urban regions and districts is necessary. The urban form of megacities evolves over a long period of time and transport infrastructure is an important element that influences the evolution city structure. If the urban form is polycentric it become further complex and inconsistent to develop railway based urban transportation system. The experience of Tokyo and Hong Kong may offers very pertinent and germane lessons because their dominant mode for commuting is urban railways with enormous number of urban commuters as desired in Asian megacities.

The strategies they adopted in urban transportation planning is decentralization, creation of suburban centers and boosting relocation of vital urban functions in outskirts. A significant issue that need understanding and comprehension is the monocentric one nuclei based urban morphology of some Asian megacities where a solitary city center contains all the vital urban functions such as central workplaces, urban business districts, major government and private sector offices and chief shopping areas or all kinds of key wholesale markets. This creates a demand of a centralized urban transportation plan on highly pressurized major urban roads of megacity with enormously increasing number of motorized and non-motorized vehicles. Whereas, multiple nuclei Asian megacities requires flexible forms of urban mass transportation to serve the multi-directional, less lineally focused travel patterns. In Europe and US, burdens for all capital costs and part of operation costs for urban mass transportation are taken by the government. Whereas, for Asian megacities the 50 percent of capital cost and operation costs can be reduced with public private partnership through active participation of the local businessmen and citizens (Litman, T. 2015).

The large number of population with high density settlements and high volume of railway demands in Asian megacities makes it technically feasible. Another factor is the cost of civil works for railway construction which are quite lower in Asian megacities than in developed countries. The cost of railway vehicles, signals and high-tech equipment is same. Whereas, overall cost of railway construction and operation is not much different from developed countries and this may be another confidence measure to develop mass transit railways in Asian megacities. In rapidly urbanizing Asian megacities, transportation departments are more often preoccupied with responding to everyday crises than planning to prevent travel demand from occurring in the first place. Whereas, planning and coordination of land use and transportation and across different transport modes need particular attention. Institutions rarely have sufficient time or funds to expand transport infrastructure fast enough to accommodate the growth in travel. Therefore, smart transportation planning approach of decision makers would be to develop mass transit railway at present time via public private partnership with local business community and people because currently population and economic activities are rapidly transforming and this is the correct time and best way to deal with informal sector urban encroachments in Asian megacities.

Finally by considering the citizen's perspective the decision makers in Asian megacities must decide to integrate pedestrian and vehicular urban transportation network for people and goods, provide its essential infrastructure with creation of apt built environment for urbanites by enforcement of town planning and building byelaws and improving law and regulatory control. The integration of short distance trips, long distance trips and very long distance trips by people and goods with appropriate mode of transport is the key approach for smart transportation planning and apt response to changing urban mobility culture in Asian megacities.

5. Conclusions

Based on desired scenario presented in the aforementioned discussion about technological advancements and smart transportation planning for Asian megacities, following conclusions are drawn:

There are five major streams of technological advancements in urban transportation planning of megacities i.e. first is people centered technologies to make people more efficient in urban mobility, second is advancements in taxi services and public transit modes to mobilize people efficiently, third is improvements in freight traffic movement and its infrastructure to efficiently mobilize goods in cities and ensure good business, fourth is mass mobility of people and goods via mass transit railways for efficient trade and commerce and fifth is record keeping about people and goods for their efficient management and timely reach in their homes and workplaces.

The smart transportation planning requires understanding of 5Ds i.e. density, diversity, design, distance to transit, and destination accessibility to shape the travel of people and goods. In addition integration of short distance trips, long distance trips and very long distance trips by people and goods with appropriate mode of transport is the key to address changing urban mobility culture in Asian megacities.

The provision of Mass Transit Railway is an inevitable element for Asian megacities if appropriate urban mobility is to be ensured and apt development is desired in future.

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Figure 1: Technological Advancements and Smart Transportation

Effect of Land Use on Traffic Congestion for Selected Arterials of Karachi

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Abstract

Land-use and transportation are interdependent. Transportation is a result of the activities defining core of the land-use at two ends. Land-use is defined in terms of the predominant activities performed, population density, urban size. Land-use-transportation interaction effects frequency of public transport alternatives and distance to facilities for shopping, etc. Urban areas having higher accessibility through diverse transport systems connecting land use demonstrate higher land values. Major land-use factors that affect transportation include land-use type, urban density, and accessibility-mobility requirements. Effective utilization of land stimulates urban activities and pressurizes roads traffic and other transportation facilities to allow minimizing spatio-temporal interactions. An understanding of transportation-land-use interaction would therefore benefit decision makers trying to improve transportation as well as urban governance.

This paper seeks to analyze effect of land use on road traffic congestion for selected major arterials of Karachi. The major arterials of Karachi selected involve mixed types of land-use activities. With the help of Google Earth, different types of land-uses of the areas are identified and the area occupied by them. Identification of the congestion spots on the arterials was done and typical traffic feature of Google Maps was used to make chart of congestion data. From the analysis it is observed that on University Road the commercial type of land-use contributes to the highest percentage i.e. 24.7%.

Keywords

Land-use Mix, Transportation, Connectivity, Travel Pattern, Accessibility

Introduction

Transportation and land use planning decisions interact. Transport planning decisions affect land use development, and land use conditions affect transport activity. These relationships are complex, with various interactive effects.

Land use is defined in terms of the characteristics of the residential location of the individuals; population density, urban size, the proximity and frequency of public transport alternatives and distance to facilities for shopping, services, leisure etc. Land use patterns affect accessibility, people's ability to reach desired services and activities, which affects mobility, the amount and type of travel activity (Litman 2003). Different land use patterns have different accessibility features. Urban areas have more accessible land use and more diverse transport systems, but slower and more costly automobile travel. Suburban and rural areas have less accessible land use and fewer travel options but driving is faster and cheaper per mile.

Cities and traffic have developed hand-in-hand since the earliest large human settlements. The same forces that draw inhabitants to congregate in large urban areas also lead to sometimes intolerable levels of traffic congestion on urban streets and thoroughfares. (ECMT 2007:5).

Congestion is the level at which transportation system performance is no longer acceptable due to traffic interference**.** The level of acceptable performance can vary by the type of transportation facility, by location within the region i.e. land use, and by time of day. For instance, commuters typically expect and are generally willing to accept a certain amount of traffic during morning and evening "rush hours". However, they may not be willing to accept that same level of performance in the middle of the day.

In general, highway congestion results when traffic demand approaches or exceeds the available capacity of the highway system. The level of traffic demand can vary significantly depending on the season, the day of the week, and the time of day. Also, the capacity of the highway system, which is usually thought of as constant, can change because of weather, work zones, traffic incidents, or other non-recurring events.

Mobility and accessibility are declining rapidly in most of the developing world. The issues that affect levels of mobility and possibilities for its improvement are varied. They include the rapid pace of motorization, conditions of local demand that far exceed the capacity of facilities, the incompatibility of urban structure with increased motorization, a stronger transport land use relationship than in developed cities, lack of adequate road maintenance. Developing cities have lessons to learn from developed cities as regards roles of new technologies, forms of institutional management and the long term consequences of different de facto policies toward the automobile. In the large cities of the developing world, travel times are generally high and increasing, and destinations accessible within limited time are decreasing (Gakenheimer, 1999).

Literature Review

Land use and transportation are interdependent. Effective utilization of land stimulates urban activities, and roads and other transportation facilities are maintained so as to allow for new transportation-related activity.

Communication through transportation is one of the important features in an urban area. It is not therefore wrong to say that urban traffic sustains urban activities.

The enormity of the urban planning challenges in developing countries is daunting. Last year, planet Earth became home to seven billion inhabitants, the majority of whom lived in cities. By 2050, urbanites are expected to make up 70 percent of total inhabitants (World Bank 2009). Ninety percent of this growth will be in the Global South.

Although most land use factors have modest individual impacts, typically affecting just a few percent of total travel, they are cumulative and synergistic.

Different land use patterns have different accessibility features. Urban areas have more accessible land use and more diverse transport systems, but slower and more costly automobile travel. Suburban and rural areas have less accessible land use and fewer travel options but driving is faster and cheaper per mile. Employment density affects commute mode share more than residential density (Barnes 2003).

Land use patterns can be evaluated based on the following attributes:

- Density **-** the number of people, jobs or housing units in an area.
- Clustering - whether related destinations are located close together (e.g., commercial centers, residential clusters, urban villages, etc.).

- Mix - whether different land use types (commercial, residential, etc.) are located together.
- Connectivity – the number of connections within the street and path systems.
- Impervious surface – land covered by buildings and pavement, also called the footprint. Greenspace – the portion of land used for lawns, gardens, parks, farms, woodlands, etc. The Green Area Factor or Green Area Ratio (GAR) refers to the percentage of land that is greenspace.
- Accessibility – the ability to reach desired activities and destinations.
- Non-motorized accessibility – the quality of walking and cycling conditions.

Land Use Features

Table 1: Land use Features

Features	**Urban**	**Suburb**
Public services nearby	Many	Few
Jobs nearby	Many	Few
Distance to major activity center (downtown to major mall)	Close	Medium
Road Type	Low speed grid	Higher speed arterials
Road & path connectivity	Well connected	Poorly connected
Parking	Sometimes limited	Abundant
Sidewalks along streets	Usually	Sometimes
Local transit service quality	Very good	Moderate
Site / building orientation	Pedestrian oriented	Automobile oriented
Mobility management	High to moderate	Moderate to low

Major Land Use Categories Are Listed Below

Table 2: Major Land Use Categories

Built Enviornment	Open Space
Residential (single and multi-family housing)	Parkland

Commercials (stores and offices)	Agricultural
Institutional (schools, public offices etc.)	Forests, chaparral, grasslands
Industrial	Wildlands (undeveloped lands)
Brownfields (old, unused and underused facilities)	Shorelines
Transportation Facilities (roads, paths, parking lots, etc.)	

Transportation planning decisions affect land use, both directly by determining which land is devoted to transport facilities such as roads, parking lots, and ports, and indirectly by affecting the relative accessibility and development costs in different locations (Kelly 1994; Boarnet, Greenwald and McMillan 2008; OTREC 2009).

Transportation planning decisions can be difficult to determine the exact land use impacts of a particular transport planning decision, particularly indirect, long-term impacts. Impacts are affected by factors such as the relative demand for different types of development, the degree to which a particular transportation project will improve accessibility and reduce costs, and how a transportation policy or project integrates with other factors.

Land Use and Transportation Connection

There are many factors that influence the connection. Two major factors are land use patterns and travel patterns. Other factors include historic land development patterns, economic factors, and local development markets. It focuses on the major factors; land use patterns and travel patterns.

<u>Land Use Patterns</u>
Land use patterns refer to the typical location and arrangement of different places (land uses) in the region, such as homes, stores, offices, parks, and factories. These patterns are guided by local zoning codes and subdivision requirements. The land use patterns require a certain type of transportation system because of the locations and distance between certain land uses.

<u>Travel Patterns</u>
Land use patterns have a major influence on the region's travel patterns. Travel patterns include where people travel, at what time, and how frequently. They also include what form of transportation is being used, such as car, bus, bike, or on foot. Travel patterns play a major role in determining the transportation system needed for the efficient movement of people and goods.

Land Use Factors

Table 3: Land Use Factors

Factor	Definition	Mechanism
Regional Accessibility	Location relative to regional centers,	Reduces travel distances between

	jobs or services.	regional destinations (homes, services and jobs)
Density	People, jobs or houses per unit of land area (acre, hectare, square mile or kilometer)	Reduces travel distances. Increases destinations within walking and cycling distances. Increases sidewalk, path and public transit efficiencies. Increases vehicle congestion and parking costs
Mix	Proximity of different land uses (residential, commercial, institutional, etc.). Sometimes described as jobs/housing balance, the ratio of jobs and residents in an area.	Reduces travel distances between local destinations (homes, services and jobs). Increases the portion of destinations within walking and cycling distances.
Centeredness (centricity)	Portion of jobs, commercial and other activities in major activity centers.	Provides agglomeration efficiencies and increases public transit service efficiency.
Connectivity	Degree that roads and paths are connected and allow direct travel between destinations.	Reduces travel distances. Reduces congestion delays. Increases the portion of destinations within walking and cycling distances.
Roadway design and management	Scale and design of streets, to control traffic speeds, support different modes, and enhance the street environment.	Improves walking, cycling and public transit travel. May improve local environments so people stay in their neighborhoods more.
Parking supply and management	Number of parking spaces per building unit or hectare, and the degree to which they are priced and regulated for efficiency.	Increased parking supply disperses destinations, reduces walkability, and reduces the costs of driving.
Active transport conditions	Quantity and quality of sidewalks, Crosswalks, paths, bike lanes, bike parking, pedestrian security and amenities.	Improves pedestrian and bicycle travel, and therefore public transit access. Encourages more local activities.
Transit accessibility	The degree to which destinations are accessible by high quality public transit.	Improves transit access and supports other accessibility improvements.
Site design	The layout and design of buildings and parking facilities.	Improves pedestrian access.
Mobility management	Various strategies that encourage use of alternative modes	Improves and encourages use of alternative modes

Study Area

Karachi is the largest city of Pakistan, having a population of approximately 20,000,000. It is the economic hub of the country. It was found that traffic congestion cost of Karachi in 2013 is 688 million USD per year and it is 2% of the total revenue of Pakistan. For an urban city of developing countries, traffic congestion cost may be around 1-2% of the GDP that particular city is contributing (M.S. Ali et al., 2013). Land use contributes a major portion in traffic congestion for a city.

The study area selected are the major arterials of Karachi as it involves mixed types of land use activities i.e. University Road, Rashid Minhas Road, Shahrah-e-Faisal, Shahrah-e-Pakistan, Karsaz Road, Korangi

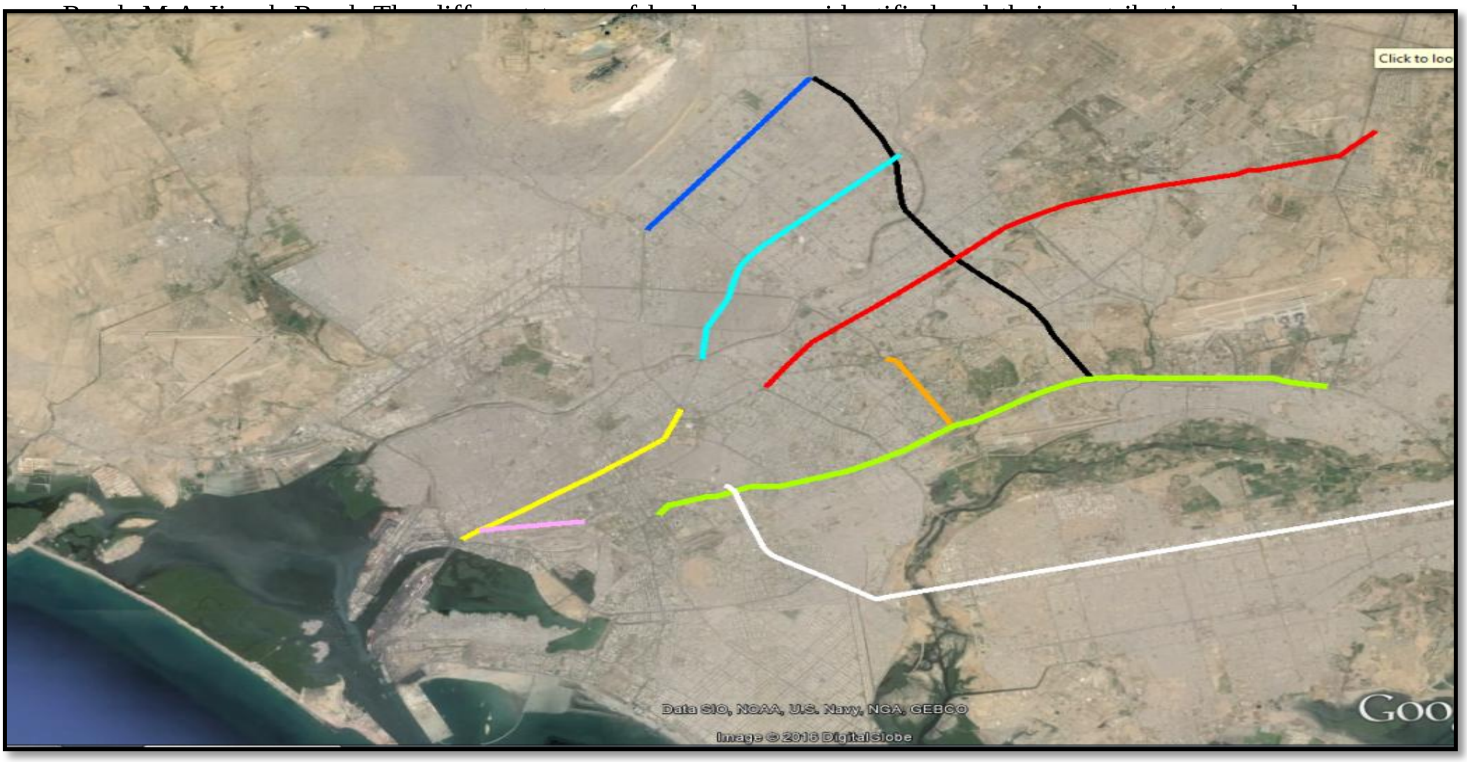

Figure 1: Map of Selected Arterials of Karachi

Key: White – Korangi Road

Green – Shahra-e-Faisal

Yellow – M. A. Jinnah Road

Orange – Karsaz Road

Red – University Road

Cyan – Shahra-e-Pakistan

Black – Rashid Minhas Road

Methodology

The major arterials of Karachi city are selected and with the help of Google Earth the different types of land uses of the areas were identified i.e. residential, commercial, open space, institutional, commercial + residential and recreational and the area occupied by them was also measured then accordingly it was scale down from 0.5 to 5(0.5 means 50m of area whereas, 5 means 500m of area).

Identification of the congestion spots on the selected arterials was done with the help of Google Maps. The Typical Traffic feature of Google Maps was used to make a chart of congestion data from 7 a.m. till 10 p.m

Data Collection

Land Use of the Study Area

Table 4: Types of Land Use & its Area

LAND-USE									
UNIVERSITY ROAD									
Location	Direction	Road	Land-use Types						
Jail Chowrangi - Wildlife Aquarium			Residential	Commercial	Open space	Institutional	Commercial + Residential	Recreational	50m=0.5
	Jail Chowrangi to Safoora	University Road							100m=1
jail						218.74			150m=1.5
showrooms				170.83					200m=2
aashi aprtmnt + shops							109.56		250m=2.5
shops				78.02					300m=3
shops				67.04					350m=3.5
shops + flats							74.11		400m=4
wild life park								141.86	450m=4.5
									500m=5
				3		2	2	1.5	
Wildlife Aquarium - Babar Hospital			Residential	Commercial	Open space	Institutional	Commercial + Residential	Recreational	
askari park	Jail Chowrangi to Safoora	University Road						301.16m	
resd + shops							201.93		
shops+ hotel				174.86					
open area					98.81				
				2	1		2	3	
Babar Hospital - PIA Garden			Residential	Commercial	Open space	Institutional	Commercial + Residential	Recreational	
mosque	Jail Chowrangi to Safoora	University Road				197.32m			
open space					138.76				
shops				147.47					
apprt + shops							174.09		
apprt + shops							141.67		
				1.5	1.5	2	3		

PIA Garden - Bank Al-Islami			Residential	Commercial	Open space	Institutional	Commercial + Residential	Recreational
shops resturant	Jail Chowrangi to Safoora	University Road		185.3				
shops				219.71				
open area					109.46			
				4	1			

Sir Syed University - Bank Al-Islami			Residential	Commercial	Open space	Institutional	Commercial + Residential	Recreational
Sir Syed University	Safoora to Jail Chowrangi	University Road				250.88		
alig instit						229.07		
apprt + shops							154.57	
flats			195.47					
shops+ flats							155.35	
shops				133.49				
			2	1.5		4.5	3	

Bank Al-Islami - PIA Garden			Residential	Commercial	Open space	Institutional	Commercial + Residential	Recreational
ground	Safoora to Jail Chowrangi	University Road			223.48			
park								154.16
ground					102.18			
mosque						174.48		
shops banks				205.19				
shops				114.53				
shops				106.46				
PIA								218.32
				4	3	2		3.5

PIA Garden - Babar Hospital			Residential	Commercial	Open space	Institutional	Commercial + Residential	Recreational
expo	Safoora to Jail Chowrangi	University Road						186.37
civic center				161.52				
district corporate east				99.81				
petrol pump				41.34				
hosp						34.59		
				3		0.5		2

Congestion Map of the Selected Arterial

University Road

Direction: Jail Chowrangi to Safoora

Landmark (~ 1 km apart)	Land use	7:00	8:00	9:00	10:00	11:00	12:00	13:00	14:00	15:00	16:00	17:00	18:00	19:00	20:00	21:00	22:00	Congestion Slots
Jail Chowrangi																		
	commercial																	2
Wildlife Aquarium																		
	commercial																	3
Babar Hospital																		
	commercial																	3
PIA Garden																		
	commercial																	7
Bank Al-Islami (Left)																		
1	institutional																	
Sir Syed University																		
	commercial + residential																	
Lalazar Banquet																		
	commercial																	
Usman Institute of Technology																		

No data available after University Lawn Banquet Hall

- Light
- Saturated
- Heavy
- Very Heavy

1 Heavy congestion till Urdu college stop from 4 to 9, thins out steeply onward

University Road

Direction: Safoora to Jail Chowrangi

Landmark (~ 1 km apart)	land use	7:00	8:00	9:00	10:00	11:00	12:00	13:00	14:00	15:00	16:00	17:00	18:00	19:00	20:00	21:00	22:00	congestion slots
Usman Institute of Technology																		
	recreational +																	
Lalazar Banquet																		
1	institutional																	
Sir Syed University																		
2	recreational																	1
Bank Al-Islami (Left)																		
	commercial																	1
PIA Garden																		
3	commercial																	2
Babar Hospital																		
	commercial																	
Wildlife Aquarium																		
	commercial																	
Jail Chowrangi																		

- Light
- Saturated
- Heavy
- Very Heavy

1 Heavy congestion under bridge at 1 and 5

2 Most early congestion under bridge

3 Saturated conditions before civic center bridge, very heavy congestion under bridge at 3

Figure 2: Congestion map of selected Arterials

Right Side

Table 5: Estimated & map values of the area

LOCATION	NUMBER OF CELLS	ESTIMATED VALUE (m)	MAP VALUE (m)	% DIFFERENCE
Saba Apartments	3	75	77.91	3.88
Food Finder	2	50	58.42	16.84
Jamia Masjid	1	25	32.88	31.52
Residential (Sherton Square)	9	225	236.6	5.16
Open space before SZIC	6	150	164.83	9.89
Commercial	3	75	73.72	-1.71
Samama	10	250	231.43	-7.43
Open space after Usman Inst.	6	150	152.22	1.48
Ibn-sina	1	25	25.85	3.40
IIEE	4	100	96.92	-3.08
Sadequain	4	100	100.39	0.39
Commercial + residential	6	150	165.18	10.12
Usmania Resturant	3	75	78.95	5.27
Mujadid Alf Sani	5	125	124.23	-0.62
Askari Park	13	325	321.78	-0.99
Wild Life Park	6	150	152.34	1.56

Left Side

LOCATION	NUMBER OF CELLS	ESTIMATED VALUE (m)	MAP VALUE (m)	% DIFFERENCE
Nadra Office	3	75	73.51	-1.99
Bin Hashim	2	50	47.25	-5.50
Open space after JS Bank	5	125	125.38	0.30
Globe Center	4	100	106.52	6.52
Zam Zam Superstore	3	75	65.2	-13.07
Open space before uni lawn	19	475	477.29	0.48
Uni lawn	2	50	44.18	-11.64
Commercial	5	125	121.78	-2.58
Metro	3	75	78.49	4.65
Karachi Apartments	3	75	62.61	-16.52
Chase Up	3	75	65.84	-12.21
Dpt. Of Env. Sci	4	100	96.28	-3.72
Hakeem Playground	5	125	134.92	7.94
Babar Hospital	1	25	29.11	16.44
Residential after babar hosp	4	100	104.7	4.70
Residential after Rehmania	5	125	119.84	-4.13
New Town	2	50	42.62	-14.76
Residential	9	225	237.85	5.71

Type of Land Use	Total Landuse	Percentage
Residential	62	23.94
Commercial	64	24.71
Open space	57	22.01
Institutional	38	14.67
Commercial + Residential	12	4.63
Recreational	26	10.04
	259	

It can be observed that on University Road the commercial type of land use contributes to the highest percentage i.e. 24.7%. And from our collected data analysis the commercial land use shows the highest influence.

Table 5: Relation between Land Use type

	University Road	Rashid Minhas Road	M.A Jinnah Road	Shahrah-e-Pakistan
Residential	2	10.5	0	8
Commercial	19	10	8.5	9
Open space	6.5	6.5	0	0
Institutional	11	0.5	2.5	6
Commercial + Residential	10	3	4.5	13.5
Recreational	10	0.5	0	0

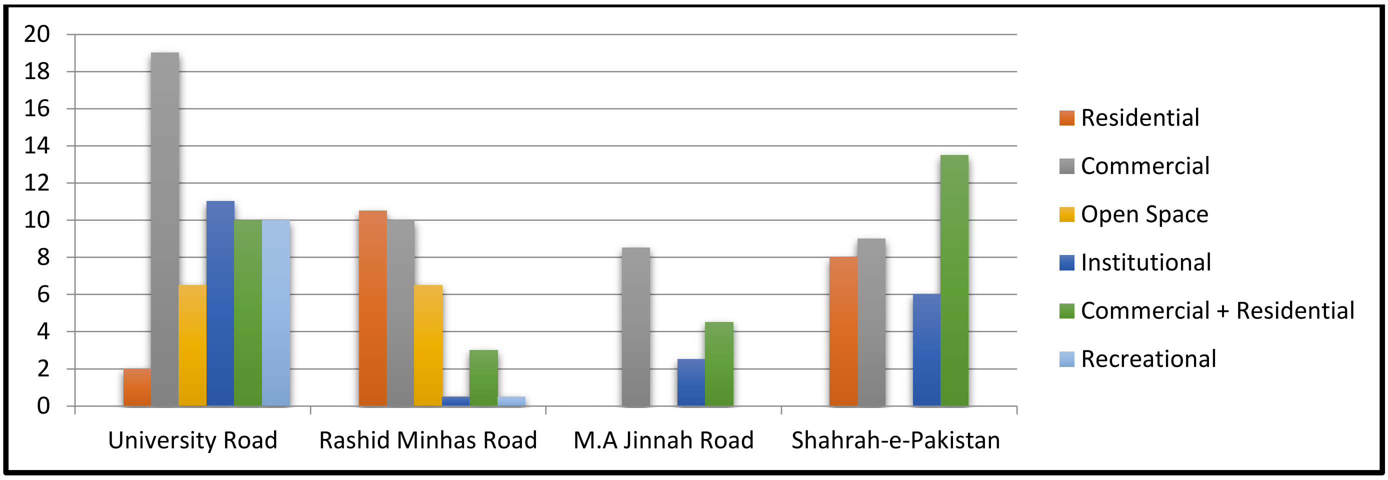

Figure 5: Graph showing the relation between Land Use types

The above graph shows the relation between arterials of Karachi and land uses. On University Road commercial land uses contributes to the highest whereas in Rashid Minhas Road residential area contributes more, on M.A Jinnah Road commercial area contributes highest and on Shahrah-e-Pakistan commercial + residential contributes more which shows the effect of traffic congestion effecting by these land uses throughout the day.

Conclusion

It can be observed that, at higher residential and employment densities, it is likely to reduce vehicle miles traveled (VMT). It suggests that doubling residential density across a metropolitan area might lower household VMT by about 5 to 12 percent, and perhaps by as much as 25 percent, if coupled with higher employment concentrations, significant public transit improvements, mixed uses and other supporting demand management measures.

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Spatio-temporal trend assessment for water quality of Bevern stream, UK

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Abstract
Land use and human activities in a catchment can have considerable effects on the quality of water bodies. This study is an effort to assess spatial and temporal behavior of Bevern stream, England, using trend analysis and consequently, identify sources of contamination. Monthly and seasonal trends are generated for 5 locations along the stream, for NH, nitrate, DO, and PO_4^{-3} using 10 years of data (2003-2013). Linear Regression and Mann-Kendall test have been applied. Spatial results show Spatham Lane, Plumpton Green, and Clapper's Bridge locations with higher nutrient load compared to Swansyard farm and Plumpton mill stream, justified by the presence of numerous farms and agricultural fields including municipal effluent at Plumpton Green and discharge from wastewater treatment plants at SL and Clapper's bridge.

Temporally, Plumpton mill stream show decline in quality with decrease in DO levels providing evidence to the Wales treatment plant as possible source. In contrast, Spatham Lane conditions show improvement with time, in PO_4^{-3} and NH_3. Current trend analyses further provide evidence on vulnerable quality conditions at Clapper's bridge. To conclude, Bevern stream show adequate oxygen levels overall for aquatic life but more attention is needed to control nutrient load coming from waste treatment plants, in order to avoid eutrophication in coming years.

Keywords
trend, nonparametric, Kendall's test, spatial, temporal

1. Introduction

With the growing population there comes modifications in river basins from pressures including agricultural, urban, and industrial sectors. Pollutants from agricultural fields include nutrients, pesticides, and fertilizers, along with manure waste from livestock. These pollutants when eventually discharge in rivers degrade the water quality that is threatening to aqua environment. In USA and Netherlands, 57% and around 60% of the surface waters are effected by nutrients from agriculture, respectively (Min and

Jio, 2002). For the UK, Water Framework Directive (WFD) and Environment Agency (EA) are established to attain and monitor water quality standards for UK coastal and freshwaters (Gardiner and Mance, 1984).

A study done by Bowes et al. (2015) for a river in rural area highlights the concentrations of phosphorous and nitrogen dominated by sewage effluent and groundwater inputs, respectively. To reduce high frequency of concentration, they suggested improvements in sewage treatment works and agriculture.

Many of the researchers have used trend analyses to assess the water quality of water bodies. Chang (2008) tested water quality on 118 sites for a river in South Korea for 8 physio-chemical parameters. He also used Mann Kendall (MK) to test the trends where results showed strong correlation with non-point sources for pollution and he suggested spatial analysis as fundamental part in identifying spatio-temporal distribution. These case studies led to the development of spatio-temporal trend analysis on Bevern stream since the catchment involve wastewater treatment plants and agricultural lands as major threats to water quality.

1.1 Study Area

Bevern stream is a 13.7 km long stream (Geoview, 2015) with 35km^2 of catchment area. The stream joins River Ouse at Barcombe Mills, Sussex. Geographically, it is located at latitude 50°55'36.84" and longitude 0°0'51.84". The land use around Bevern stream is mostly agricultural fields and managed farmlands along with three sewerage treatment plants within the catchment at Ditchling, Wales, and Barcombe villages (Figure 1).

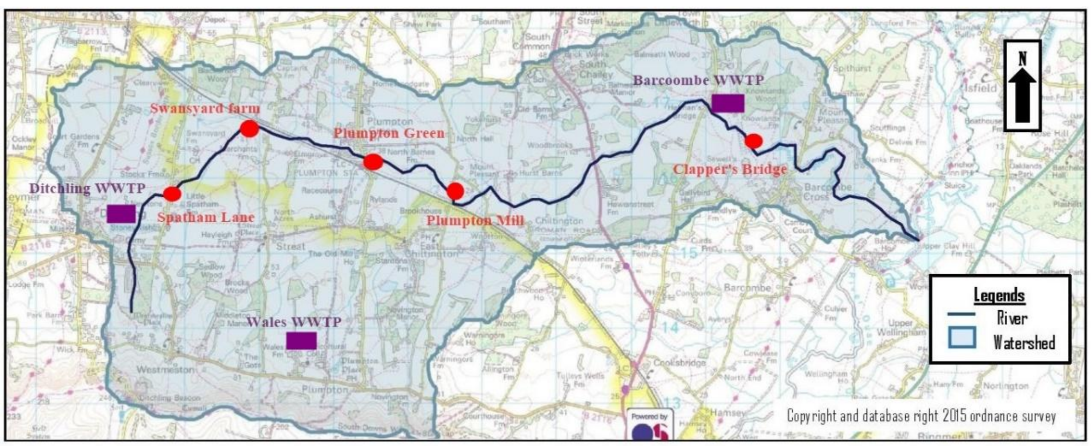

Figure 1: Land use map showing Bevern stream catchment (Source of map: Ordnance survey, 2015).

1.2 Objective

The objective of this research was to perform spatio-temporal analysis of water quality parameters for Bevern stream based on statistics and monthly and seasonal trends.

2. Material and Method

Based on availability of historic data, 4 parameters, namely ammonia (NH_3), phosphate (PO_4^{-3}), Dissolved Oxygen (DO), and nitrate (NO_3) are selected for five stations at Bevern stream. Monthly data from

August 2002 till April 2014 were collected for the parameters which were then assembled for seasonal analysis. The missing values were interpolated using Grubbs' test (Grubbs, 1969).

The secondary data have been obtained from Ouse and Adur Rivers Trust (OART). For statistical analysis, Linear Regression and Mann-Kendall's test are done. Hirsch et al (1982) demonstrates that water quality data are normally non-parametric that has higher efficiency than parametric. Mann-Kendall, henceforth, is recommended for the analysis of trends in water quality data (Lettenmaier, 1976).

Following is the methodology applied throughout the project;

1. Monthly and seasonal spatial and temporal trends of physio-chemical parameters.
2. Statistical tests using linear regression and Mann-Kendall Rank's test.
3. Identification of possible sources of vulnerable pollutants and the most vulnerable pollutant.

3. Results and Discussion

3.1 Spatial Analysis

The sampling site at Spatham Lane (SL) has farms such as Spatham farm, Stocks farm, and Hayleigh farm. Ditchling village is present just at the beginning of the stream before SL sampling site. There is a wastewater treatment plant 0.5 km before from the SL. There is another tributary from the west side that joins in with Bevern stream. Swansyard Farm (SF) sampling site is almost at the junction of the tributaries. From the SF till Plumpton Mill (PM) site, the land use comprises of one village called Plumpton Green (PG) and various farms including Elm Grove farm and North Barnes farm. There is a race course as well in Plumpton which might be noted for manure. PM site that includes water from the south end of the catchment. In the south, there are Wales farm, Plumpton Agricultural college, Stantons farm, and Wales Wastewater Treatment Plant (WWTP). Downstream of PM, a few kilometres before Clapper's Bridge (CB) is Barcombe WWTP.

Figure 2 illustrates monthly trends for all parameters downstream of the river. The x-axis starting from Spatham Lane shows the beginning of stream and CB as the nearest sampling point from the outlet. The graph elucidates that on average, there is an increase in the level of DO up to 8mg/l with a decrease in concentrations of PO_4^{-3}, NO_3, and NH_3 downstream. This trend tells that Spatham Lane has high PO_4^{-3} and nitrate levels than any other sampling site, making it a susceptible site. Figure 2 also depicts PM for having lowest load values for PO_4^{-3}, nitrate, and NH, consequently increasing DO which is highest of all sites.

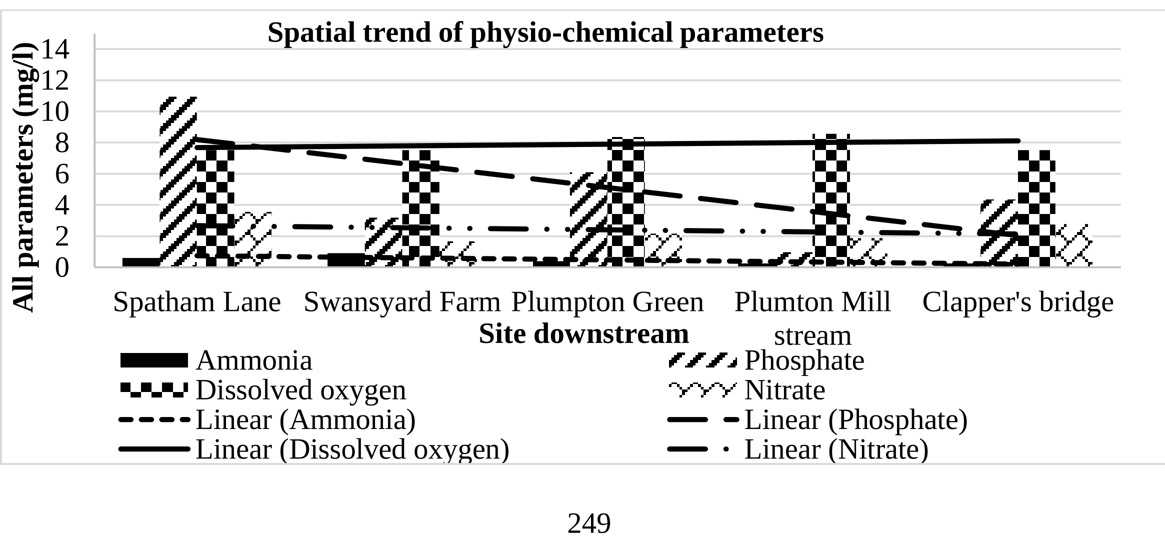

Figure 2: Spatial trend of physio-chemical parameters for Bevern stream.

Figures 3 - 6 illustrate trends for summer, autumn, winter, and spring, respectively. In summer, the level of PO_4^{-3} shows an overall decrease down the stream, starting from the highest load at Spatham Lane followed by abrupt decrease in the next site that is SF. There is an increase in DO from SF to PG.
Nitrate, like PO_4^{-3}, are also soluble. In summer high levels of nitrate are observed at Spatham lane, followed by CB. This compliments the trend of PO_4^{-3} where both parameters are high at SL, PG, and CB. From the land use map it can be justified that SL and PG have more farms than the other sites. Despite the differences of load, DO levels show consistency throughout the stream with a slight overall decreasing trend.

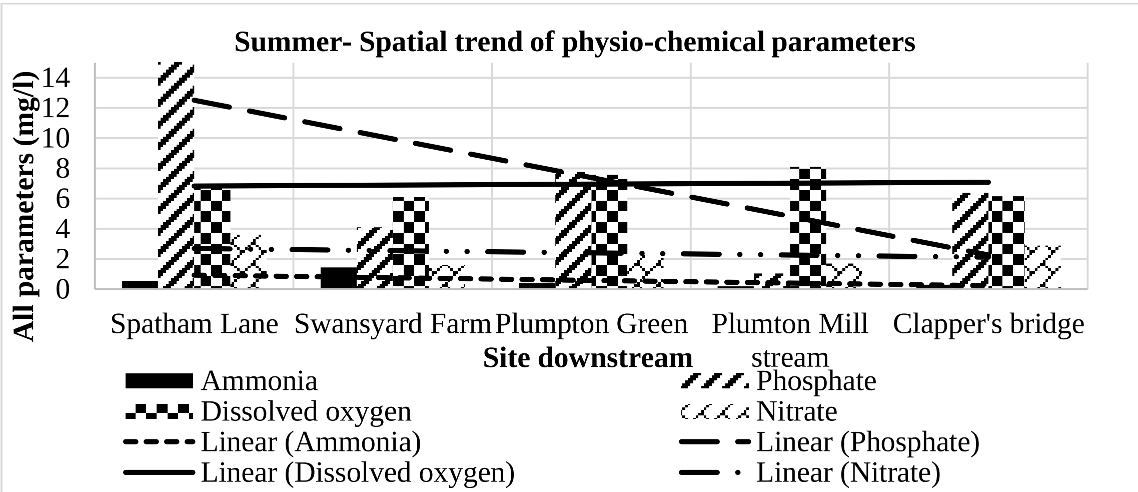

Figure 3: Spatial trend of physio-chemical parameters for Bevern stream in summer.

In autumn, similar sinusoidal pattern and concentration is observed for PO_4^{-3}. For nitrate, the levels decrease downstream from SL to SF, exceeding with increase from PM to CB. This analysis when correlated with land use map can be explained by higher number of farms at SL site and CB. Also, the sewage treatment plants add a lot to the concentration. NH_3 levels show a decrease with the lowest load at PG.

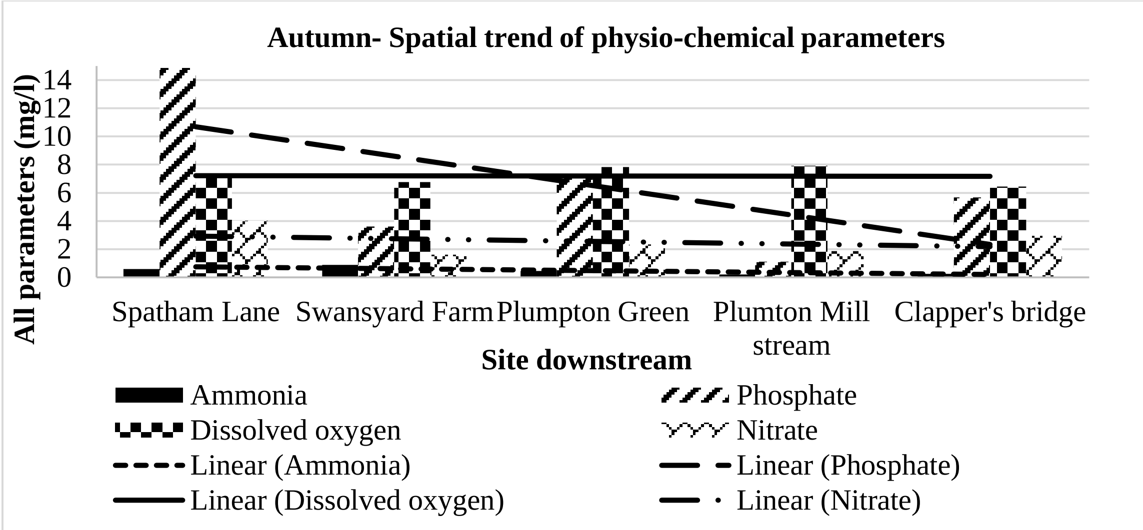

Figure 4: Spatial trend of physio-chemical parameters for Bevern stream in autumn.

For the next two seasons, winter and spring, there is increase in load of DO levels. In winter, overall oxygen rises from 8.3 mg/l to 9.3 mg/l. As for spring, oxygen remain constant at 9 mg/l throughout the stream. In contrast with summer and autumn, PO_4^{-3} show decrease in concentration. However, the sinusoidal pattern of PO_4^{-3} is also followed where load is decreasing at SL. This load, thereafter, shows increase at PG, decrease at PM, and then an increase at CB during winter. This can be explained by referring to WWTP before each of the sites. Conversely, concentration of nitrate in winter show increase at SF and PG. The NH_3 levels in winter show a smooth decrease downstream from 0.33 mg/l till 0.17 mg/l.

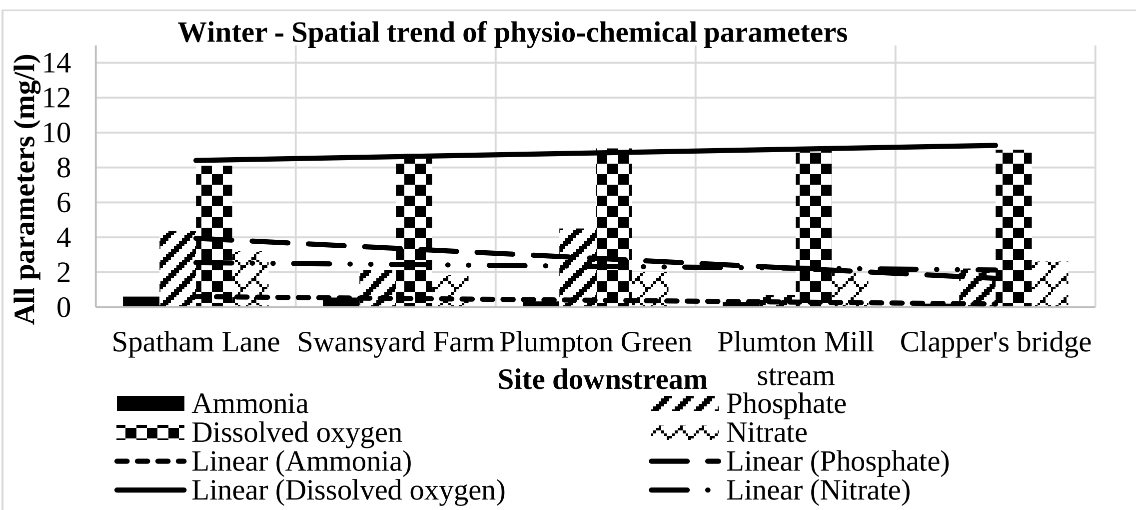

Figure 5: Spatial trend of physio-chemical parameters for Bevern stream in winter.

For spring season, PO_4^{-3} levels increase again from 4.2 mg/l in winter to 7.4 mg/l at SL. Nitrate show an overall increase of 0.2 mg/l downstream. NH_3 follows the same trend for seasons and monthly.

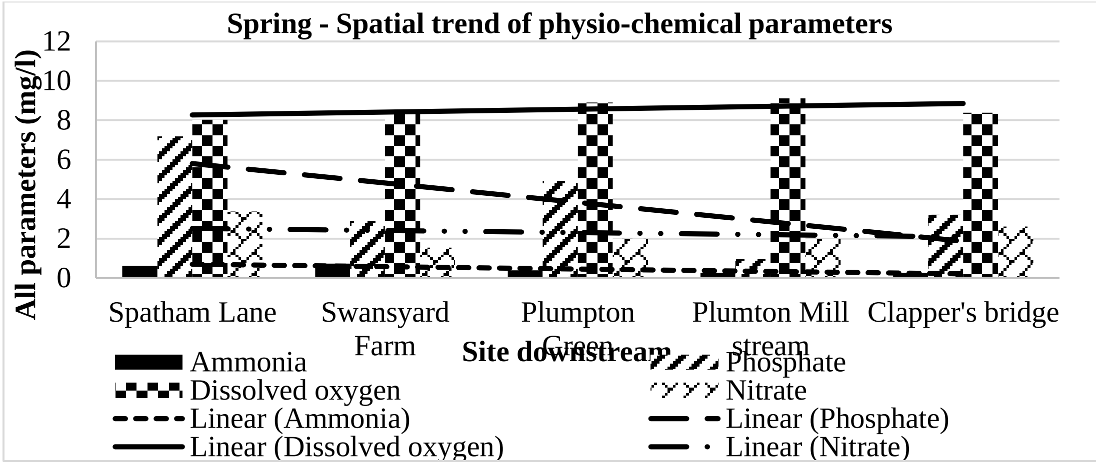

Figure 6: Spatial trend of physio-chemical parameters for Bevern stream in spring.

3.2 Temporal Analysis

The dataset is further analyzed temporally where parameters are assessed for all five sites monthly and for each season. Table 1 illustrates MK's test results and trend lines are shown as graphs in Figures 7 - 10 for monthly analysis for NH_3, PO_4^{-3}, DO, and nitrate, respectively. From Figure 7 it can be observed that for monthly NH_3 level, there is an overall increase in concentration at sites with SL showing maximum.

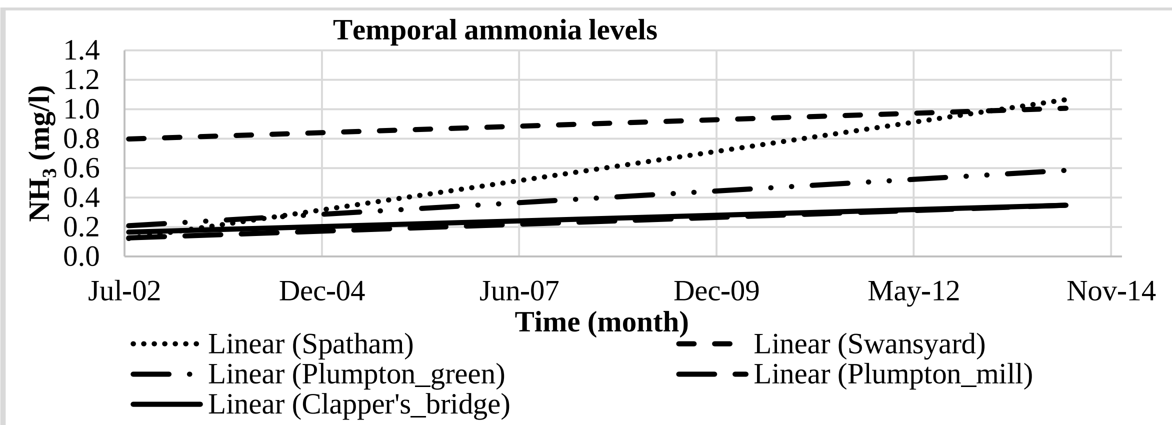

Figure 7: Temporal trend of NH_3 concentration in Bevern stream.

For PO_4^{-3}, Figure 8 illustrates that all sampling sites have increased in concentration with consistency at PG. As of MK, there is no trend in SL and CB's PO_4^{-3} levels. Figure 9 shows DO on the monthly behavior of all sites with a decrease for all sites. For PG, there is no change observed in MK. Figure 10 illustrates that nitrate levels have increased at SL, consistent at CB while decreased at SF, PG, and PM stream.

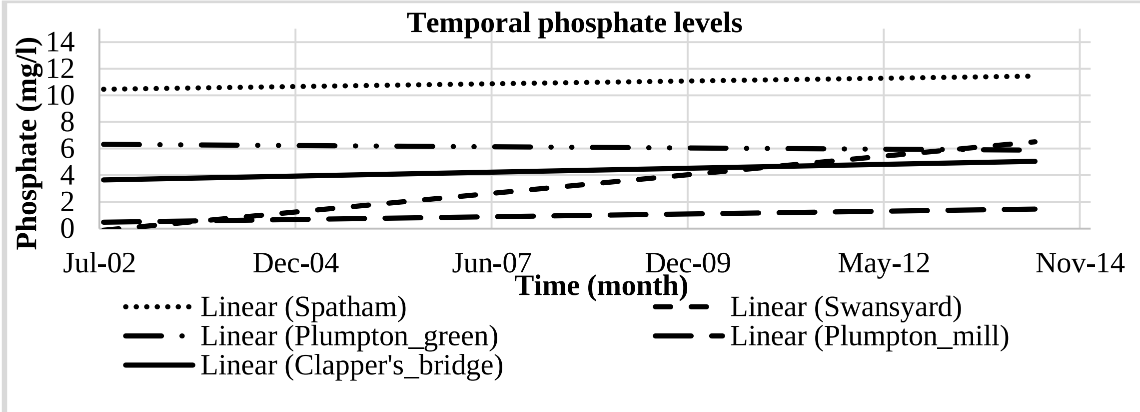

Figure 8: Temporal trend of PO_4^{-3} concentration in Bevern stream.

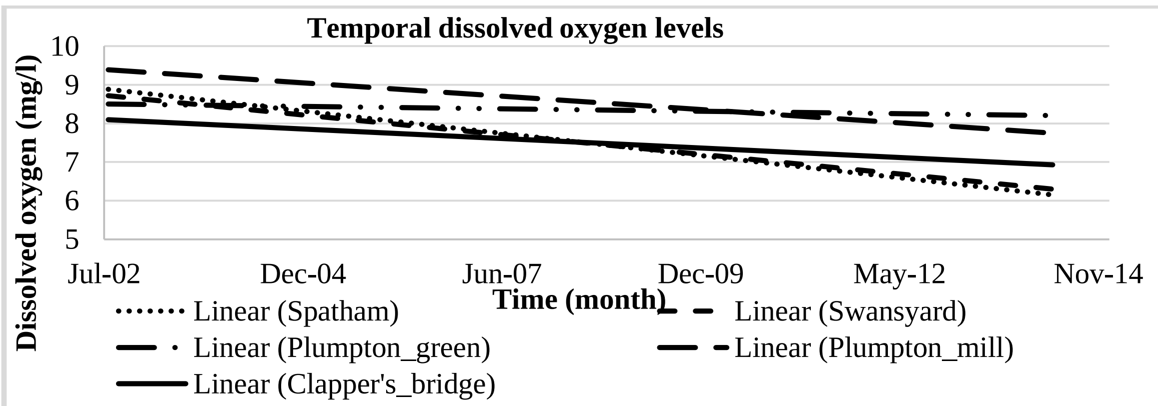

Figure 9: Temporal trend of oxygen concentration in Bevern stream.

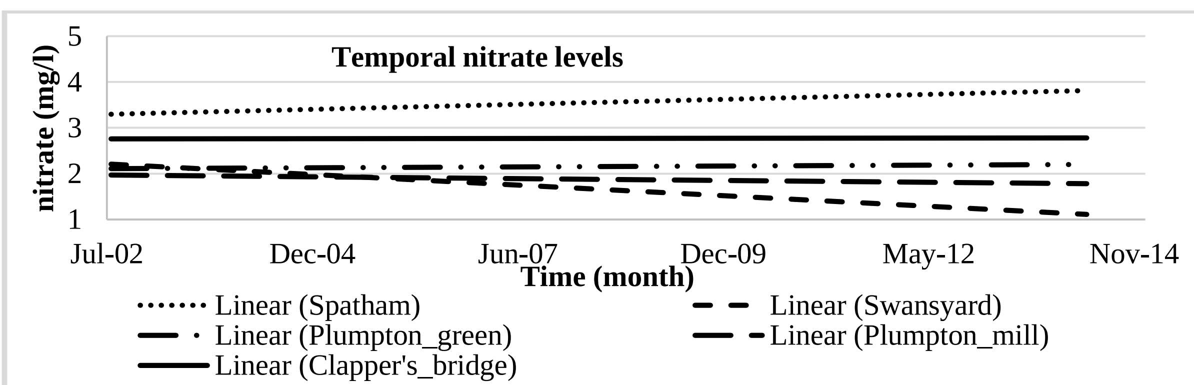

Figure 10: Temporal trend of nitrate concentration in Bevern stream.

Table 1 shows seasonal trends with respect to time. Graphs are not shown in the paper; however, the graphs generated showed increase/decrease of the trend. The analysis depicts that Mann-Kendall recognizes the trend for spring the most. In summer, DO is significantly increasing in PG and decrease in other locations. In autumn again, similar temporal is observed. With respect to time, DO is showing a decrease in winters and spring. This is a matter of concern where the seasonal variation shows the drop of DO level.

Table 1: Temporal trend for monthly and all seasons of Bevern stream using Mann-Kendall tests.

		Trend				
Stations	**Parameters**	**Monthly**	**Summer**	**Autumn**	**Winter**	**Spring**
Spatham Lane	N	Trend	Trend	Trend	Trend	Trend
	PO_4^{-3}	No Trend	No trend	No trend	No trend	Trend
	DO	Trend	Trend	Trend	Trend	Trend
	NO_3	No Trend	No trend	No trend	No trend	No trend
Swansyard Farm	N	No Trend	No trend	No trend	No trend	Trend
	PO_4^{-3}	Trend	Trend	Trend	Trend	Trend
	DO	Trend	No trend	No trend	No trend	Trend
	NO_3	Trend	Trend	Trend	No trend	Trend
Plumpton green	N	Trend	Trend	Trend	Trend	Trend
	PO_4^{-3}	Trend	No trend	No trend	Trend	Trend
	DO	No Trend	Trend	No trend	Trend	No trend
	NO_3	No Trend	No trend	No trend	No trend	No trend
Plumpton mill	N	Trend	Trend	No trend	No trend	Trend
	PO_4^{-3}	Trend	Trend	No trend	Trend	Trend
	DO	Trend	No trend	Trend	Trend	Trend
	NO_3	No Trend	No trend	No trend	No trend	No trend
Clapper's bridge	N	Trend	Trend	No trend	No trend	Trend
	PO_4^{-3}	No Trend	No trend	No trend	No trend	No trend
	DO	Trend	No trend	No trend	Trend	Trend
	NO_3	No Trend	No trend	No trend	No trend	No trend

As for chemical parameters, trends at CB are not of significance in Mann-Kendall. Nitrate trend are also not recognized by MK test other than at SL that shows increase in summer and autumn. Similarly, PO_4^{-3} levels are less at SL but SF, PG, and PM stream for winter and spring. SF shows drastic increase for PO_4^{-3}.

4. Conclusions

1. Temporally, there is decrease in DO levels. The major sources of contamination are present at three sampling points are SL, PG, and CB with SL showing the highest contamination.

2. The sources of contamination are number of farms and agricultural lands (Hayleigh farm, stocks farm, Little Spatham farm, and Garden farm) and a treatment plant near SL. It can be said that monitoring of Barcombe treatment plant at CB site is needed along with the farms nearby CB.
3. Spatial trend analysis show slight increase in DO as the range lies between 7 – 8 mg/l. The decrease in levels of PO_4^{-3} and nitrate down the stream is a justification of contamination sources upstream.

More historic data (such as temperature, rainfall, pH, etc.) can provide further justification and evidence to the results. Following are the possible recommendations to enhance the study.

1. Fencing the river banks where farm animals have easy access to river.
2. Usage of eco-friendly fertilisers (manure) to reduce contaminants in agricultural runoff.
3. Analysis in correlation with rainfall and sediments will provide in-depth evidence to the results. Software are available to simulate the watershed for non-point sources. One of the models is Agricultural Non-Point-Source Pollution Model (AGNPS) developed by Young (1987).

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Assessment of Water Quality in the Peripheral Rivers Along Dhaka City: Spatial and Temporal Variation

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Abstract
Large quantity of untreated waste are being disposed into Buriganga, Turag, Balu and Shitalakshiya rivers around Dhaka city leading to surface water contamination affecting public health, ecosystem, and economic growth. The objective of this study is to monitor and assess the water quality of these peripheral rivers of Dhaka City. Physicochemical study of rivers during pre and post monsoon period between 2015 to 2016 was studied indicating their suitability for intended purpose. Levels of physical parameters, organic matter and nutrients indicated moderate pollution in post-monsoon and heavy pollution in pre-monsoon period. Selected heavy metals were also analyzed in the river water samples. Except in Buriganga, heavy metal contamination was not significant in other locations. To assess the risk of fish, aquatic insects and benethic macroinvertebrates an ecological risk assessment was performed for Turag river following a framework. Certain aquatic species were not found that were present 10 years back. Bacteriological identification revealed that *E. coli* and *S. typhimurium* were the most common bacteria in Turag. Employing sustained monitoring of water quality will lead to improvements in the management and contribute to the rehabilitation of the rivers with positive impact in agricultural sector, avoidance of drainage congestion and abatement of pollution.

Keywords
Water Quality, River, Pollution, Dhaka City

1. Introduction

Industrial and municipal discharge of untreated waste into the peripheral rivers around an urban city has the potential to make water badly polluted that adulterates water quality of river. With a population of over 16 million, Dhaka, one of the densest cities of the world is located on the northern bank of the river Buriganga and surrounded by other rivers, namely, the Turag from the west to north and the Balu and Shitalakshiya to the east. These four peripheral rivers around Dhaka city are the most important watershed and crucial source of water for irrigation as well as for heavy and light industries, aquaculture, animal farming, municipal supply and wastewater dilution.

River plays an important role for carrying municipal and industrial wastewater, run-off from farm land. River is one of the most susceptible water bodies to pollutants. Domestic wastewater, sewage sludge and leachate from solid waste disposal sites are also obvious sources of river water pollution. However, the surface water along these rivers is known to be polluted due to municipal and industrial untreated wastewaters that are discharged into these rivers directly. Water quality of these rivers is deteriorating significantly being a threat to our limited water supply by changing taste and odor, growth of aquatic weeds, aquatic life and wild life.

The main environmental impacts of Peripheral River in Dhaka city are related to the continued discharge of industrial and tannery waste, the release of pollutants into the soil, the application of pesticides and fertilizers, the discharge of animal waste into water bodies, and the reduction and/or extinction of native flora and fauna (Macedo, 2009). Huge quantities of industrial effluents, solid waste from river-side settlements, petroleum products from ships, launches, cargoes, boats, untreated sewage etc. regularly get dumped into the rivers which are already severely polluted (Khan et al., 2007). A clear distinctive variation usually exists while season is changing. During the Pre-monsoon period, the water quality usually reaches its worst condition. The Rivers around Dhaka become deadly for fish and other aquatic organisms. During post monsoon period after heavy rainfall, the river water is diluted to some extent and the quality improves than the pre-monsoon period.

If certain pollution can be reduced under specific guideline that is non-toxic for human consumption, surface water can be a major source of water supply for fulfilling water demand. It is important to have available information on characteristics of water quality for pollution control and to ensure safe consumption of water. Therefore, it is a great prerequisite to evaluate the river water quality. The overall objectives of this study was to evaluate the variation of physico-chemical water quality parameters of the four peripheral rivers around Dhaka City with respect to location and seasonal change which will enable making a comparison judgement as well. On a brief scale, the ecological impact associated with the polluted water was also assessed.

1.1 Impacts on surrounding

River water pollution cause damage to human health and the environment. Toxic waste in water affects the health, quality of soils and plant life. Water pollution affects the aquatic life by blocking out sunlight that killed the aquatic life. In addition, a major environmental impact has been the extensive mortality of fish and marine invertebrates due to the contamination of aquatic systems by pesticides. As a result, amount of dissolved oxygen (DO) is depleted and fish, plant, birds, sea gulls and many other animals' life become vulnerable.

Pollutants interrupt drinking water quality, may contaminate soil and accumulate in plants' and fishs' tissue. Hence, contaminants enter in the natural food cycle and may have the potential to bio-magnify (Majed et al. 2016). Human activities such as industries, steel mills proved to be elevated levels of heavy metals present in fresh water, and among these micro elements lead (Pb), cadmium (Cd), mercury (Hg), chromium (Cr), copper (Cu), zinc (Zn) are the most significant pollutants of the aquatic eco-systems due to their environmental persistence and tendency to be concentrated in aquatic organisms. Moreover, exceeding limit of this trace elements may cause carcinogenic and non-carcinogenic risk on human (Majed et al. 2016). Ecosystem is hampered by unplanned urban and rural development in the near riverside. If contaminated water and aquatic food from the polluted site is consumed, it could lead to some common diseases like allergy & inflammations, gastroenteritis, typhoid & paratyphoid, hepatitis, jaundice and may develop even deadlier cancers.

1.2 Study area

The four rivers are interconnected to each other. The Balu River is a tributary of Shitalakshhya River. Balu River runs mainly through the Beel Belai, joining the Shitalakshiya near demra. It has connection with Tongi Khal with Turag. Turag River passes through the northwestern part of Dhaka city originating from the Bangshi River which is a tributary of the river Dhaleshwari that flows through Gazipur and connected with the Buriganga at Mirpur in Dhaka District.As a preliminary step for initiating the study, a map of the study area showing Dhaka city and the peripheral rivers was developed. The analyzed sites are considered critical to the four rivers. Superficial water samples were collected about 10 cm below the surface. All samples were kept on ice until laboratory analysis. This study was performed during different sampling periods: pre monsoon and post monsoon (October 2015 to October 2016).

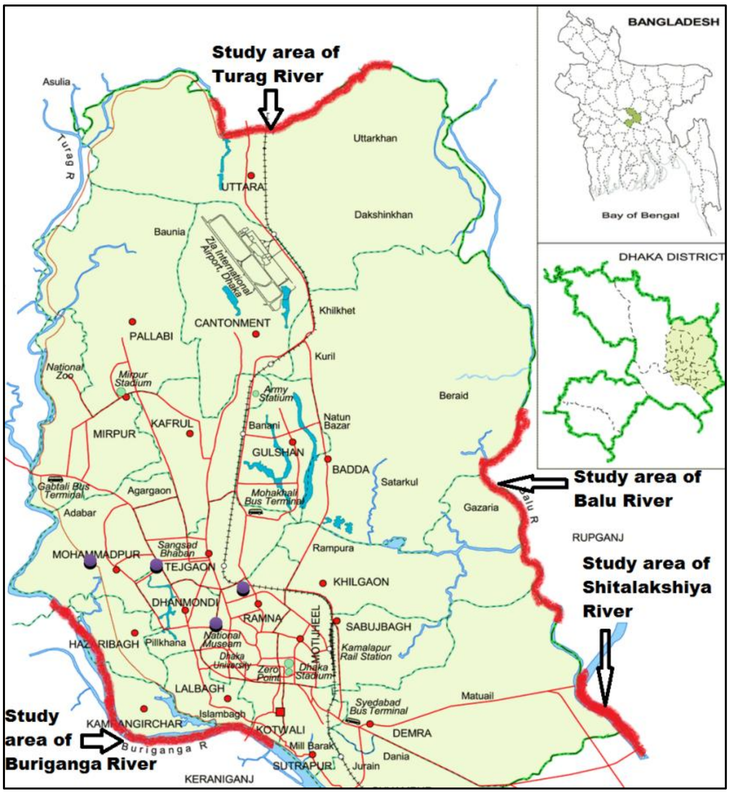

Figure 1: Map of Dhaka city indicating the study area

The study area along with peripheral rivers around Dhaka city is displayed in the map locating the sample collection points to get the demonstrative value of river water quality. Three different sampling points were chosen along the selected reach of each river. Table 1 shows the coordinates of the study area.

Table 1: Coordinates of the sampling location

		Buriganga	Turag	Balu	Shitalakhya
Location 1	Latitude	23°44'37.1"N	23°51'08.6"N	23°48'43.5"N	23°42'09.8"N
	Longitude	90°20'48.1"E	90°31'05.5"E	90°29'04.9"E	90°31'03.4"E
Location 2	Latitude	23°42'33.8"N	23°50'46.2"N	23°44'43.1"N	23°40'33.9"N
	Longitude	90°23'20.4"E	90°29'46.6"E	90°29'23.0"E	90°31'53.3"E
Location 3	Latitude	23°42'35.0"N	23°49'58.2"N	23°43'26.4"N	23°37'58.5"N
	Longitude	90°24'04.5"E	90°29'11.9"E	90°29'55.5"E	90°31'00.1"E

2. Materials and methods

All water samples were collected at each site, early in the morning. For this purpose water samples were collected from four peripheral river sites using laboratory prepared plastic bottles for collection of sample. The samples were bottled carefully so that no air bubble is entrained in the bottle. Water samples from the rivers were analyzed for 5-day biochemical oxygen demand (BOD_5), dissolved oxygen (DO), total dissolve solid (TDS), total suspended solid (TSS), ammonia-nitrogen (NH_3-N), phosphate (PO_4-P), fecal and total coliforms. Qualitative determination of bacteria was also analyzed for the identification of bacteria in Turag River. The primary sampling point is in the surface water layer at the center of the main flow. Water quality parameters (BOD_5, DO, TDS, TSS, NH_3-N, PO_4) and bacteriological analysis were done using standard equipment following standard methods. Heavy metals (cadmium, lead, chromium, copper, zinc and mercury) were also analyzed in our study. Heavy metals were selected on the basis of literature study e.g. Ahmad et al. (2010), Islam et al. (2012), Islam et al. (2014), Islam et al. (2015) and Roy et al. (2014). The metal concentrations were determined by atomic absorption spectrophotometer (Varian spectra AA100). All the parameters except heavy metals were measured in the environment laboratory of University of Asia Pacific (UAP), Bangladesh. Heavy metals were analyzed in soil, water and environment laboratory of Dhaka University.

3 Results & Discussion

3.1 Variation of Water Quality Parameters

Six important water quality parameters of river water were measured at three points of each river. Table 2 represents water quality of four rivers during pre and post monsoon periods based on water quality parameters. Mean concentrations of six water quality parameters are illustrated in figures 2 to 4. The red dotted lines show standards for inland surface water prescribed by DOE, 1997. Comparison with the inland surface water standards is shown in table 2.

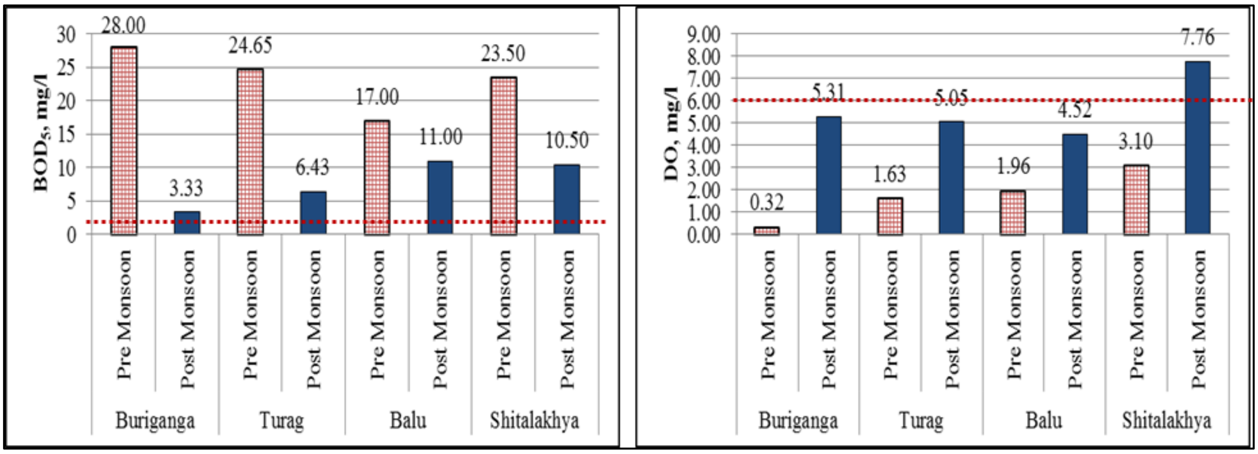

Figure 2: Variation of $BOD_5$and DO between pre and post monsoon period

BOD_5 of water samples of our studied rivers were found higher in pre monsoon period compared to post monsoon period (Table 2). Among the four rivers, the highest average BOD_5 in pre monsoon was found at Buriganga River (36 mg/l), while the lowest value was also found at Buriganga River (2.8 mg/l). The other rivers also show a high BOD_5 level during pre-monsoon period as a result of higher concentrations of organic substances. Unpolluted waters typically have BOD_5 values of 2mg/l or less (Chapman and Kimstach, 1996). The normal range of BOD_5 for good water quality is 5-6 mg/l (Huq and Alam, 2005). High BOD level clearly indicates that the River Buriganga was polluted with the organic chemical and bacterial pollutants that causes unfavorable condition for aquatic life. Seasonal variation of these four rivers is very high between pre monsoon and post monsoon period for BOD_5 and all rivers exceed the standard level (Fig. 2.1).

From figure 2 it can be seen that the mean Dissolve Oxygen (DO) content of the water sample of four rivers varied considerably with an average value of 4.52 to 7.76 mg/l in the post monsoon period and 0.32 to 3.10 mg/l in the pre monsoon period. It was observed that DO values of water bodies under study were higher in postmonsoon than in pre monsoon period (Table 2). Concentrations below 5.0 mg/l adversely affect aquatic life (Murphy, 2007). Low DO values can lead to fish disappearance from water body. The mean concentrations of DO in all of the rivers in pre monsoon period clearly show that the river water is not suitable for use of the aquatic ecosystem during this period.

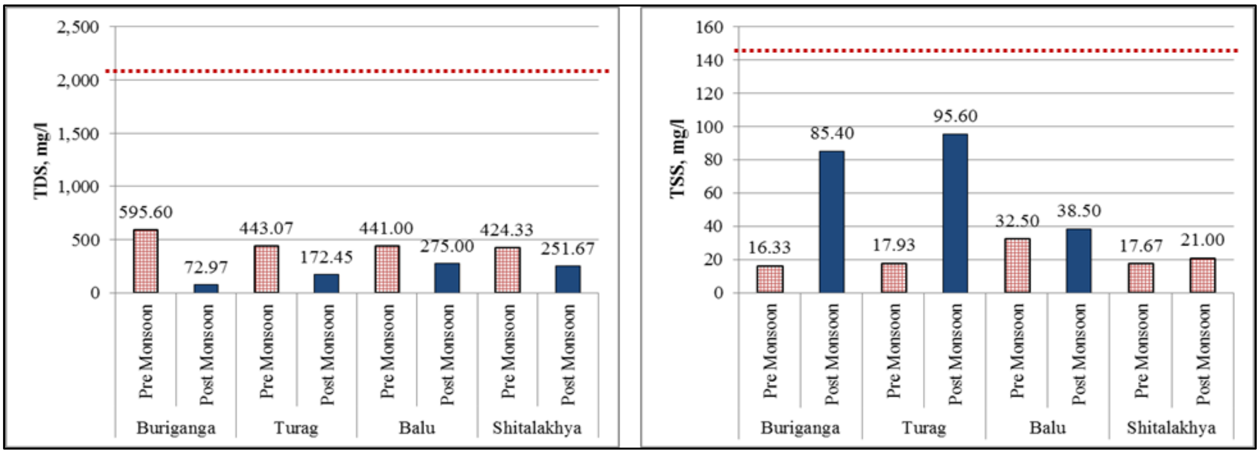

Figure 3: Variation of TDS between pre and post monsoon period

The TDS value is a common indicator of the presence of different minerals and metallic substances in water that are in colloidal and dissolved conditions and also is an important chemical parameter of water (Kabir et al. 2002). Bangladesh Standard for TDS in terms of inland surface water is 2100 mg/L which is indicated by a red dotted line in figure 3. The TDS values of water of the study area ranged from 72.97 to 251.67 mg/l in the post monsoon period and 424.33 to 595.6 mg/l in the pre monsoon period (Table 2). The high TDS value of the water is due to different sewage, domestic waste, industrial and agricultural effluents. Table 2 shows that among the four rivers, the Buriganga had the maximum TDS value of all rivers (708 mg/L). TDS levels were below the standard level during both study periods for four rivers (Fig. 3).

Bangladesh Standard for TSS for inland surface water is 150 mg/L. The mean total suspended solids concentrations in four rivers around Dhaka city were found to be 16.33 to 32.50 mg/L in pre monsoon and 17.67 to 95.60 mg/L in post monsoon period. Extremely high TSS values are stressful to fish and other aquatic organisms. During the post-monsoon period, the average value of both TDS and TSS were lower than the pre-monsoon period. The high value of TDS and TSS during the pre-monsoon season might occur because of the floating fine silt and detritus that was carried by the rainwater from the catchment. Both TDS and TSS values of four rivers do not exceed the DoE limit (Table 2).

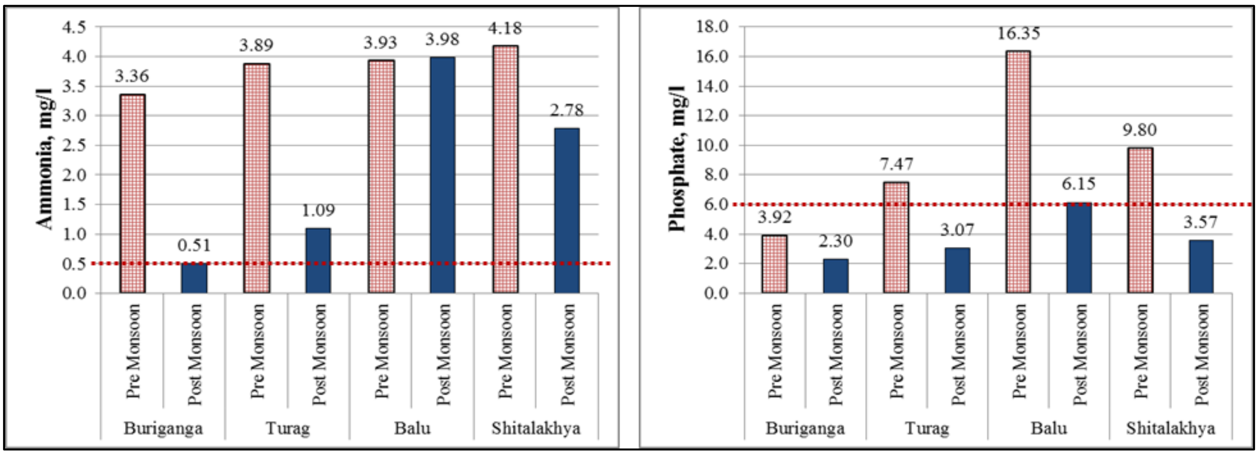

Figure 4: Variation of ammonia and phosphate between pre and post monsoon period

Ammonia and phosphate are two elements that are present in all the fertilizers necessary for plant and animal growth. When it rains, varying amounts of phosphates and ammonia wash out from farm soils into nearby waterways. Ammonia is harmful when present in higher concentrations, and many unexplained production losses have likely been caused by ammonia. Phosphate stimulates the growth of plankton and aquatic plants which provide food for larger organisms in the aquatic food chain. Initially, this increased productivity can cause an increase in the fish population and overall biological diversity of the system. But as the phosphate loading continues, the aging process of the surface water ecosystem can be accelerated. The overproduction of phosphate in lakes or rivers can lead to an imbalance in the nutrient and material cycling process. This leads to a variety of problems ranging from anoxic water (through decomposition) to toxic algal blooms (eutrophication) that eventually decrease aquatic diversity and cause habitat destruction. Excess ammonia increases the load on the nitrifying bacteria in water and even relatively low levels pose a threat to fish health. Figure 4 shows the variation of NH_3-N during pre and post monsoon. Ammonia and phosphate values recorded in pre-monsoon are relatively higher than post-monsoon period for an increased amount of phosphate and ammonia wash from farm soils into those three rivers with storm runoff which is used for cultivation and agriculture production as fertilizer.

Almost all four peripheral rivers of Dhaka city exceed the standard level of ammonia during pre-monsoon and post moonsoon period (Fig. 4). Besides, the recorded value of Phosphates in Turag, Balu and Shitalakshiya River is higher in pre monsoon than post monsoon period and at the same time they go beyond the standard level of phosphates in surface water (Fig. 4). Balu River contains more phosphates than the other rivers.

Table 2:Concentration ranges of WQ parameterin pre and post monsoon period of four rivers

Parameter		BOD_5 (mg/L)	DO (mg/L)	TDS (mg/L)	TSS (mg/L)	Ammonia (mg/L)	Phosphates (mg/L)
Buriganga	Pre-monsoon	6.5-36	0.29-0.35	535-708	68-101	2.15-4.69	2-2.5
	Post Monsoon	2.8-3.6	4.1-6.6	67-82.7	14-19	0.24– 0.89	2.7-6.9
Turag	Pre-monsoon	5.9-35.7	0.34-1.04	311-641	35-155	0.7-8.76	2-4.6
	Post Monsoon	4.9-6.8	1.58-7.39	41.7-314	3-35	0.4-5.65	1.7-14.2
Balu	Pre-monsoon	4.0-17.0	1.42-2.5	439-443	11-66	3.62-4.24	4.4-7.9
	Post Monsoon	6.0-13.0	3.3-5.73	271-279	8-57	3.65-4.3	11.9-20.8
Shitalakshiya	Pre-monsoon	9.0-28.0	1.83-4.67	248-516	5-45	3.9-4.6	0.9-6.5
	Post Monsoon	8.0-11.0	4.92-10.8	142-312	4-38	0.81-3.9	2.5-18.2
National Standard for Inland Surface Water (DOE)		≤ 2	≥ 6	2100	150	0.5	6

3.2 Heavy Metal Analysis

Table 3 shows the heavy metal levels that were analyzed for the samples of the rivers. The collected surface water samples from the Shitalakshiya, Buriganga, Turag and the Balu River contained insignificant amounts of Cd, Pb, Hg, Cu and Zn. However, levels of Cr obtained in Buriganga and Balu rivers were 0.224 and 187.5 ppm respectively. Although Cr level is lower than the discharge standard in Buriganga, it still indicates continuous discharge of this toxic heavy metal from industrial sources. The level of Cr in Balu seems exceptionally high in the river indicating that there might be industrial applications that utilize extensive amount of Cr. Further analysis and monitoring would be required to investigate the alarming scenario. In general, risk seems to be less with respect to the discharge of other heavy metals in the rivers, nevertheless more sampling is essential to confirm the observation.

Table 3:Heavy metal concentration in four peripheral rivers around Dhaka City

Parameter, unit	Buriganga	Turag	Balu	Shitalakhya	Discharge in Inland Water (DoE, 1997)	TRV[a]
Chromium, ppm	0.224	0.007	187.5	<BDL	0.5	0.011
Cadmium, ppm	0.001	0.001	<BDL	<BDL	0.05	0.002
Lead, ppm	0.021	0.009	<BDL	<BDL	0.1	0.003
Mercury, ppm	<BDL	NA[b]	<BDL	<BDL	0.01	0.15
Copper, ppm	<BDL	<BDL	3.5	0.5	Not available	-
Zinc, ppm	0.1067	0.09	<BDL	<BDL	5.0	-

[a] TRV: (toxicity reference value) for fresh water proposed by USEPA (1999).
[b] NA: Not Analyzed

3.3 Bacteriological Analysis

Regarding health aspects and organic pollution, water samples from the Turag River were analyzed for bacteriological contamination. It is obvious that pathogenic contamination is significant in the river water possibly due to sewage pollution. Thus, people should avoid the water for drinking purpose because any king of diseases can be resulted by drinking this water. After conventional treatment this water can be used as it is less than 5000 mg/L. Following bacteria were identified in Turag River at different sampling stations: E. coli, S. typhimurium, Shigella spp. and Salmonella spp. Among all the bacteria E. coli and S. typhimurium were the most common bacteria in every sampling station of Turag River. Escherichia coli bacteria are responsible for diarrhea or illness outside of the intestinal tract (Gorbach 1996). Shigella may cause diarrhea, fever, nausea, vomiting, stomach cramps, and flatulence.

3.4 Ecological Impact

From visual survey and questionnaire survey some fish species and macro invertebrates were identified that are available during pre and post monsoon period. In the post monsoon period availability of fish is more likely than pre-monsoon period. Concerning issue is some of the species of aquatic life were not seen which were available in Turag River about 10-15 years ago. From IUCN (2003), a red list was adopted to identify the threatened fish species in the Turag River. No fish species could be identified recently that were listed in the year 2000 as the endangered and critically endangered species in the Turag river. Among the vulnerable species, only Meni and Puti were identified along with Tengra and Catfish. Macroinvertebrates including dragonfly, crab, snail, aquatic worm and larva were common.

4. Conclusion

These four peripheral rivers are feeding the city in many ways and play important role in the communication system to transport goods and passengers too of the whole region. Like other city belts, the peripheral rivers around Dhaka city are losing its water quality day by day. At present the river is under severe pollution threat. The analysis clearly determine that the water quality of rivers around Dhaka City are not within the standards provided by DOE. The ecological analysis provides evidence that water become unsuitable to life that depends on these water bodies that could be a direct or indirect result of the discharge and flow of waste water. Risk seems to be less with respect to the discharge of other heavy metals in the rivers, nevertheless more sampling is essential to confirm the observation. Bacteriological analysis shows that the surface water is not usable or consumable as water contain bacteria that can lead to serious illness or disease. Though some parameters may not in the deteriorating level but the condition of the river side urbanization and industrialization may cause all kind of water pollution in the near future.

Employing sustained monitoring of water quality will lead to improvements in the management and contribute to the rehabilitation of the rivers with positive impact in agricultural sector, avoidance of drainage congestion and abatement of pollution.

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A Fuzzy Analytic Network Process Model to Treat Critical Environmental Risks in Dams and Hydropower Plants

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Abstract

In the renewable-energy industry, it is extremely important to reduce the likelihood as well as the magnitude of environmental risk events during system's construction and actual operation. Herein, major environmental risks may cause catastrophic damages to individuals or the environment and result in substantial costs. Selection of a suitable strategy for treatment against the risks associated with dams and hydropower plant projects is a very complex and critical task. Hence, the aim of this study is to select the best treatment strategies for major environmental risks in the construction phase of Bakhtiari dam and construction and operation phases of its hydropower plant. To do so, at first, critical environmental risks in the field were identified by a literature review. Then, four options were considered for risk treatment and by combining fuzzy ANP and fuzzy DEMATEL methods the most appropriate treatment strategies were selected. Finally, mitigation, acceptance, mitigation and mitigation strategies were selected for major construction risks, respectively, and avoidance, acceptance, mitigation and mitigation strategies were chosen for critical operation risks, respectively. It was also found that mitigation was the most appropriate risk treatment option for the construction and operation phase of the studied case.

Keywords

Dams and Hydroelectric Power Plants, Risk Treatment, Major Environmental Risks, Fuzzy ANP, Fuzzy DEMATEL

1. Introduction

Dam construction is actually human kind's encroachment in the natural system and mechanism of rivers and also disruption in the flow of water in them; therefore, it causes many problems and leads to high social and environmental costs. This issue forms the negative dimension of dam construction industry and makes it challenging (Bek, 2012). Question of environmentalists and residents in the margins of rivers placed around under construction dams is that whether the benefits of dam construction are significant enough to compensate the losses and damages imposed on the environment and nearby residents (Han et al, 2008). In other words, these great projects are always exposed to environmental risks. These risks are not limited only to the construction phase. The operation phase of dams and hydropower plants is also an important issue that should be investigated in detail. Hence, the aim of this paper is to select the best treatment strategy for critical environmental risks in the operation phase of dams and hydroelectric power plants.

2. Materials and Methods

Firstly, critical environmental risks in the construction phase of Bakhtiari dam and hydropower plant and operation phase of its hydropower plant will be identified based on literature review, and also the previous studies conducted by the authors (i.e. Amiri and Mohammadi, 2016, and Mohammadi, 2016).
In the next step, the best risk treatment strategy will be selected for the identified critical risks. In this step, four risk treatment options based on PMBOK standard (i.e. risk avoidance, transfer, mitigation and acceptance) will be considered. Next, by taking into account the internal relations between the critical risks and using a combination of Fuzzy ANP (FANP) and Fuzzy DEMATEL (FDEMATEL) methods, the most appropriate risk treatment option for each risk will be obtained. Then, the best risk treatment option will be chosen for each risk by a questionnaire survey and gathering experts' opinions. Finally, the best risk treatment option for the construction phase of Bakhtiari dam and power plant and for the operation phase of the plant will be selected as a whole.

2.1 Analytic Network Process

Analytic Hierarchy Process (AHP), developed by Saaty, is essentially the formalization of our intuitive understanding of a complex problem using a hierarchical structure. The AHP enables the decision maker (DM) to structure a complicated problem in the form of a simple hierarchy and to evaluate a large number of quantitative and qualitative factors in a systematic manner with conflicting multiple criteria (Tavakkoli et al., 2011).
Saaty suggested the use of AHP to solve the problem of independence among alternatives or criteria, and the use of ANP to solve the problem of dependence among alternatives or criteria (Dagdeviren et al., 2008).
The ANP is a generalization of the AHP. Whereas AHP represents a framework with a unidirectional hierarchical AHP relationship, ANP allows for complex inter-relationships among decision levels and attributes. The ANP feedback approach replaces hierarchies with networks in which the relationships between levels are not easily represented as higher or lower, dominant or subordinate, direct or indirect (Meade and Sarkis, 1999).

2.2 Decision Making Trial and Evaluation

Decision Making Trial and Evaluation Laboratory (DEMATEL) technique, which has been developed by the Science and Human Affairs Program of the Battelle Memorial Institute of Geneva between 1972 and 1976 and used for research and solving several groups of complicated and interdependent problems. DEMATEL not only can convert the relations between cause and effect of criteria into a visual structural model, but also can be used as a way to handle the inner dependences within a set of criteria. This method has been applied in various fields most recently. DEMATEL uses the knowledge of experts to lay out the structure model of a system. It not only provides a way to visualize causal relationships between criteria through an impact-relationship map but also indicates the degree to which criteria influence each other. As a result, total direct and indirect influences of each factor (indicator) are obtained as each factor's influence given to other factors, but as well influence received from other factors over others. In addition, according to the DEMATEL method, the levels of interdependences of factors do not have reciprocal values, which are closer to the real system (Zhou et al., 2014).

2.3 Pairwise Comparisons Using Language Scale

In network analysis methods and DEMATEL technique, due to the existence of uncertainty in the pairwise comparisons made by experts, Triangular Fuzzy Numbers (TFN) equivalent to experts' opinions will be applied.

In these pairwise comparisons the importance of the element “i” on element “j” for decision-maker “k” will be: (Farrokh et al., 2016).

$$i, j = 1, 2, 3, \ldots, \mathrm{n} \qquad \mathrm{k} = 1, 2, 3, \ldots k$$

If $i = j$ then $\tilde{a}_{ijk} = (1,1,1)$ (1)

If $i \neq j$ then $\tilde{a}_{ijk} = \left(l_{ijk}, m_{ijk}, u_{ijk}\right)$ (2)

Moreover, the importance of element “j” on element “i” for decision maker “k” will be obtained using Equation (4).

$$\tilde{a}_{ijk}^{-1} = \left(1/u_{ijk}, 1/m_{ijk}, 1/l_{ijk}\right) \tag{3}$$

In network analysis methods and DEMATEL technique triangular fuzzy numbers are used to model experts' opinions due to existence of uncertainty and vagueness in the paired comparisons made by them.
In FANP, Table 1 is used for scoring and pairwise comparisons based on Chang's scale (Zheng, 2011). In the same way, Table 2 is used for FDEMATEL method (Attari, 2012; Wu, 2007).
Furthermore, to determine the best treatment option for each risk, priority of treatment options against each risk associated is examined and judged. Base of this judgment is the Chang fuzzy scale. However, in the comparison of options with each sub criteria, the priority of options is discussed nor their importance rate.
To select the appropriate option Lin formula is used as follows:

$$D_i = \sum_{j=1}^{J} w_j E_{ij} \tag{4}$$

where

D_i = Desirability index of the options i

w_j = Relative importance of the sub criteria j

E_{ij} = Preference score of the options i from the sub criteria j

Table 1: Linguistic Variables and Their Equivalent Fuzzy Numbers for FANP Method.

Linguistic Scale for Importance	Linguistic Variable	Triangular Fuzzy Number	Reverse Fuzzy Number
Equally importance	Very low	(1,1,1)	(1,1,1)
Weak importance of one over another	Low	$(1/2, 1, 3/2)$	$(2/3, 1, 2)$
Strong importance	Moderate	$(1, 3/2, 2)$	$(1/2, 2/3, 1)$
Very strong importance	High	$(3/2, 2, 5/2)$	$(2/5, 1/2, 2/3)$
Absolute importance	Very high	$(2, 5/2, 3)$	$(1/3, 2/5, 1/2)$

Table 2: Linguistic Variables and Their Equivalent Fuzzy Numbers for FDEMATEL Method.

Linguistic Variable	Triangular Fuzzy Number
Very high impact	(0.7,0.9,1.0)
High impact	(0.5,0.7,0.9)
Moderate impact	(0.3,0.5,0.7)
Low impact	(0.1,0.3,0.5)
No impact	(0.0,0.1,0.3)

2.3 Testing the Consistency of Questionnaire Data

Furthermore, to check the consistency of gathered data from returned questionnaires, the Gogus and Boucher method, presented in 1997, will be used for calculating the degree of consistency of fuzzy pairwise comparison matrices. In this regard, if the consistency rate is less than 0.1, pairwise comparisons consistency will be acceptable (Gogus and Boucher, 1997).

2.4 Defuzzification of Fuzzy RPNs

Defuzzification is an important step in fuzzy systems. In these systems, the results of an approximate reasoning usually will be obtained in the form of one or more fuzzy numbers. In these cases it is necessary to convert the system's fuzzy outputs to a crisp (non-fuzzy) number. In this step, to perform a better comparison between the results, the calculated Fuzzy RPN will be converted to a crisp RPN.

There are several methods such as Middle of Maximum (MOM), Center of Gravity (COG), Center of Area (COA) etc. In the current study, the average fuzzy defuzzification method will be applied according to equation (5) (Basaran, 2012).

$$FRPN = (a_1, a_2, a_3)$$

$$RPN = \frac{a_1 + 2a_2 + a_3}{4} \tag{2}$$

3. Case Study

The Bakhtiari dam is an arch dam currently under construction on the Bakhtiari River within the Zagros Mountains on the border of Lorestan and Khuzestan Provinces, Iran. At a planned height of 325 metres (1,066 ft) it will be the world's tallest dam once completed and withhold the second largest reservoir in Iran after the Karkheh reservoir. The main purpose of the dam is hydroelectric power production and it will support a 1,500 MW power station. By trapping sediment, the dam is also expected to extend the life of the Dez Dam 50 km (31 mi) downstream (Mohammadi, 2016).

The preliminary studies of Bakhtiari dam began in 1992 and the design phase lasted until 2009 under the supervision of consultants and experts. Currently the project is at the beginning of the construction phase.

Since Bakhtiari power plant is located in a pristine and undeveloped area and also due to problems in approaching sediment surface elevation to the inlet valve of Dez hydropower station in its downstream, valuable environmental issues are present for study (Mohammadi, 2016).


Figure 1: Location of Baktiari Dam and Hydropower Plant in South West of Iran.

4. Results

After conducting a literature review, including the previous studies performed by the authors (i.e. Amiri and Mohammadi, 2016, and Mohammadi, 2016), four critical risks were identified in the construction phase of the Bakhtiari dam and hydropower plant and also four critical risks in the operation phase of its hydropower plant. These risks are listed in the left column of Table 3 and 4.

In the next step, four risk treatment strategies were assigned to the critical risks in the construction and operation phases (eight risks).

Since there are interrelationships among risks, weights of the four critical risks were determined by considering internal relations between them, and by applying the combination of FANP and FDEMATEL methods.

Here, a questionnaire was used to determine the significance and impact of criteria. Table 1 and 2 helped the respondents to answer the questions. Seven experts responded to the designed questionnaire for the operation phase and also seven experts filled the questionnaire of the construction phase. All of these experts were related to the Bakhtiari dam and power plant.

After integrating the opinions of the respondents on the importance of the criteria, CR^m and CR^g values were obtained 0.0416 and 0.0849 for the construction phase risks and 0.0541 and 0.0 987 for the operation phase risks. Hence, since they are less than 0.1, it can be concluded that the geometric average matrix of experts' opinions is consistent.

Then, the combined FANP- FDEMATEL approach was applied to determine criteria weights. The results for both groups of risks are presented in Table 3 and 4.

Table 3: Final Weights for Critical Environmental Risks in the Construction Phase of Bakhtiari Dam and Power Plant.

Criteria	Final Weight
Quality degradation of aquatic habitat	0.24
Cultural changes	0.19
Degradation of soil habitat quality and safety	0.23
Exacerbation of soil erosion	0.33

Table 4: Final Weights for Critical Environmental Risks in the Operation Phase of Bakhtiari Power Plant.

Criteria	Final Weight
Equipment oil and grease leakage to the river	0.20
Development of nearby constructions	0.35
Ingress of dust and animals into the plant	0.25
Decomposition of waste and residual woods behind the dam	0.20

Moreover, to determine the preference of options for each risk and ultimately selecting the most appropriate risk treatment option, pairwise comparisons were performed. The weight of each risk treatment option for each critical risk in the two groups is shown in Table 5. This table also presents the most appropriate risk treatment option for each major risk.

Table 5: The Most Appropriate Risk Treatment Options for Each Risk.

	Risks	Av	Tr	Mi	Ac	Best Strategy
Construction of dam and power plant	Quality degradation of aquatic habitat	0.244	0.236	0.265	0.255	Mitigation
	Cultural changes	0.222	0.229	0.264	0.284	Acceptance
	Degradation of soil habitat quality and safety	0.243	0.240	0.281	0.236	Mitigation
	Exacerbation of soil erosion	0.247	0.222	0.268	0.263	Mitigation
Operation of power plant	Equipment oil and grease leakage to the river	0.281	0.241	0.263	0.216	Avoidance
	Development of nearby constructions	0.217	0.229	0.274	0.280	Acceptance
	Ingress of dust and animals into the plant	0.250	0.246	0.276	0.228	Mitigation
	Decomposition of waste and residual woods behind the dam	0.249	0.253	0.270	0.227	Mitigation

Av: Avoidance, Tr: Transition, Mi: Mitigation, Ac: Acceptance

By combining the results presented in Table 5, the best strategy for each of the phases is chosen and presented in Table 6 (colored in green) according to Equation 4.

Table 6: Best Treatment Option for Risks of Each Group Based on Desirability Index.

Risks	w_j	E_{ij} Av	Tr	Mi	Ac	$w_j \times E_{ij}$ Av	Tr	Mi	Ac
Quality degradation of aquatic habitat	0.24	0.244	0.236	0.265	0.255	0.0586	0.0566	0.0636	0.0612
Cultural changes	0.19	0.222	0.229	0.264	0.284	0.0422	0.0435	0.0502	0.0540
Degradation of soil habitat quality and safety	0.23	0.243	0.240	0.281	0.236	0.0559	0.0552	0.0646	0.0543
Exacerbation of soil erosion	0.33	0.247	0.222	0.268	0.263	0.0815	0.0733	0.0884	0.0868
$D_i = \sum_{j=1}^{J} w_j E_{ij}$						0.2381	0.2286	0.2668	0.2562
Risks	w_j	E_{ij} Av	Tr	Mi	Ac	$w_j \times E_{ij}$ Av	Tr	Mi	Ac
Equipment oil and grease leakage to the river	0.20	0.281	0.241	0.263	0.216	0.0562	0.0482	0.0526	0.0432
Development of nearby constructions	0.35	0.229	0.223	0.272	0.275	0.0802	0.0781	0.0952	0.0963
Ingress of dust and animals into the plant	0.25	0.250	0.246	0.276	0.228	0.0625	0.0615	0.0690	0.0570
Decomposition of waste and residual woods behind the dam	0.20	0.249	0.253	0.270	0.227	0.0498	0.0506	0.0540	0.0454
$D_i = \sum_{j=1}^{J} w_j E_{ij}$						0.2487	0.2384	0.2708	0.2419

Av: Avoidance, Tr: Transition, Mi: Mitigation, Ac: Acceptance

5. Conclusion

In this study, eight critical environmental risks were selected for analysis. Within them, four risks were related to the operation phase of the Bakhtiari hydroelectric power plant and four other risks were related to the construction phase of Bakhtiari dam and power plant. Finally, four risk treatment strategies were allocated to these risks the most appropriate treatment option was selected for each risk.
For critical risks related to the operation of the power plant, i.e. equipment oil and grease leakage to the river, development of nearby constructions, ingress of dust and animals into the plant and decomposition of waste and residual woods behind the dam, the most appropriate treatment options were determined avoidance, acceptance, mitigation and mitigation, respectively. Furthermore, the best treatment strategy for the risks in the construction phase of the Bakhtiari dam and power plant, i.e. quality degradation of aquatic habitat, cultural changes, degradation of soil habitat quality and safety and exacerbation of soil erosion, were determined mitigation, acceptance, mitigation and mitigation, respectively. Finally, by combining the results of the ANP and DEMATEL methods, mitigation was determined as the most appropriate risk treatment option for the operation phase of the hydroelectric power plant and also the construction phase of dam and power plant.

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Assessment of Floods and Hazard Analysis for the City of Karachi Using Simulation Models

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Abstract
Karachi is the largest metropolitan city of Pakistan with population over 20 million that is expected to reach 27 million by the year 2020. Heavy rainfall events of high intensity have been experienced by the city on various occasions resulting in damage to the infrastructure and loss of human lives. Therefore, impacts of floods in the city are worth studying for betterment of the inhabitants. The main objective of the study was to develop flood inundation maps for Karachi and identifying most vulnerable areas using computer simulation models. Surface water assessment for the city has been done using Hydrologic Simulation Program – Fortran (HSPF) and then flood inundations maps were developed using HEC-River Analysis Simulation (HEC-RAS). After this identification of existing prone areas was done using Arc-GIS. The most vulnerable areas in terms of flood damage that was simulated for various return period rainfall events are the lower parts of Malir watershed, and towns of Korangi, Gadap, and Gulberg. To reduce the impacts of flash floods in the city, improved drainage system for storm water flow needs to be ensured along with the implementation of methods to reduce the volume of surface runoff like Low impact development (LID).

Keywords
Climate Variability, Flood Inundation Modeling, Vulnerability Analysis, Hazard Assessment, Adaptation measures

1. Introduction

Rapid population growth and urbanization are making Karachi city vulnerable to the impacts of flash floods. High intensity rainfall events have been experienced by the city on various occasions causing colossal damage to human lives and paralyzed communication and transportation for multiple hours (Bakhsh et al., 2011). During the Monsoon season from July to September risk of urban flooding is very high. In 1977 severe flood occurred in the flood plains of the Malir River and the Lyari River which killed 267 people, more than 30,000 were homeless and 100,000 were temporarily dislocated. Houses were destroyed and roads were damaged. The total loss was estimated about 5 billion rupees (KDA, 1981).

The effect of climate change can result in the change in the frequency and severity of extreme weather events including short duration/high intensity rainfalls and temporal distribution of precipitation. United States Environmental Protection Agency (USEPA) (2013) report states that since 1901, the global average surface temperature has risen at an average rate of 0.13°F per decade (or 1.3°F per century), and global precipitation has increased at an average rate of 1.9% per century. A study by Chaudary et al. (2009) indicated that Pakistan experienced a rise of 0.76°C in average temperature during the last 40 years. Analysis of Variations (ANOVA) was used in a study by Salma et al. (2012) to analyze rainfall trends for 30 years (1976 – 2005) for Pakistan, from 30 observatories through the country, concluding an average decrease of 1.18mm in rainfall per decade, across the country. A recent study conducted by Mahar et al. (2010) on Karachi's climate identifying climate change by using climatic data of 50 years and found that the trend is surprisingly different for Karachi as compared to Badin and Hyderabad showing exponential increase of and decrease in annual precipitation. Quantifying flooding with hydrologic/hydraulic distributed simulation models can help developing flood inundation mapping and planning for disaster risk reduction for Karachi. This can help in preparing efficient mitigation and preventive measures to reduce the impacts of flooding.

Multiple international and local studies have shown that hydrological models, specifically HSPF is capable of simulating various hydrological components at the watershed scale at very high accuracy making it useful in hydrological estimations (Ahmed et al. 2012, Tong et al., 2012). Conclusion of multiple studies has showed that HEC-RAS is user-friendly, floodplain model and provide reasonable assessment of hydrologic and hydraulic behavior of a watershed (Maidment & Tate 1999, Siddiqui et al., 2012). The main objective of the study was to develop flood inundation maps and suggest mitigation measures for disaster risk reduction for Karachi using computer simulation Hydrologic/hydraulic modeling.

2. Materials and Methods

2.1. Data requirement and description

Minimum meteorological data requirement for this study includes the datasets like daily minimum (min) and maximum (max) temperature, and precipitation (pcp). These parameters have primarily been acquired from the National Oceanic and Atmospheric Administration (NOAA) website. Hydrological modeling has been done using met data from year 1978 to 2014. Data before 1978 has a lot of missing slots making it hard to be used for the modeling exercise, therefore that data has been avoided. Daily flow data are required to calibrate the hydrological model as well as for development of hydrographs. Daily flow data for Malir River were acquired from University of Engineering and Technology (UET), Lahore (UET Lahore acquired these data from PMD). However, the data available is on volume basis and not rate of flow. Time line for daily flow data is 1994 to 2003. Landuse data used in this study is custom generated and based on surveys conducted by the research team. Soil type data have been acquired from the Food and Agriculture Organization (FAO) website. The DEM is acquired from the ASTER website. This DEM is based on 90m × 90m resolution. Intensity duration frequency (IDF) curves used to develop flood scenarios are taken from National Engineering Services Pakistan (NESPAK). These IDF curves are used to model future scenarios and also for obtaining some extreme rainfall events for the analysis. IDF curves for Karachi were developed around 1970s. It is believed that these curves are relatively obsolete now and new IDF curves need to be developed with the help of better dataset. However, due to the limitation of precipitation data required for developing a new IDF curve, the existing one has been used in this study.

2.2. Hydrologic and Hydraulic Modeling

Before proceeding to the hazard analysis for any watershed, it is essential to assess the amount of water present in the watershed. Basic Methodology of the process includes surface water assessment by using hydrologic and hydraulic models, flood plain mapping, and identification of existing prone areas.

For the purpose of this study, HSPF has been selected as the hydrologic model, and HEC-RAS is selected as hydraulic model. This selection is made based on the suitability to the purpose of the study. The hydrologic model (HSPF) has been simulated and calibrated and the output of the model are imported as input to the hydraulic model (HEC-RAS), which then simulated the river flow. Output of the model HEC-RAS illustrates the flooding in case of overflow of streams which is further used for the hazard analysis.
HSPF is a semi-distributed and continuous model that simulates hydrologic and associated water quantity/quality processes on pervious and impervious land surfaces and in streams. It is a complex model simulating much more than other simple models do. Simulation results include a time history of the surface and subsurface runoff flow, sediment, nutrient, and pesticide concentrations, along with a time history of water quantity and quality at any point in a watershed.

Climatic data (temperature and precipitation) are prepared in model format acceptable to HSPF using the tool Watershed Data Management (WDM) utility, Better Assessment Science Integrating Point and Nonpoint Sources (BASINS) has been used for preparing GIS related data. Watershed delineation done using BASINS reveal that Karachi is divided into three sub watersheds, namely Lyari, Malir, and Hub watersheds. Figure 1 is an illustration of the sub watersheds highlighting the division of the areas. Hub River watershed crosses the western boundary of Karachi city and does not contribute to the city its self; and, therefore, it is ignored in the flood analysis. The hydrologic model has been simulated for 36 years (1978-2014). Hydrological modeling simulation results are presented as the water balance and simulated stream flow.

2.3. Calibration and Validation

Parameters in HSPF have been adjusted in order to calibrate the simulated stream flow according to the observed flow. Table 2 shows the list of parameters of HSPF, along with their respective ranges and adjusted values, and Table 3 shows the calibrated water balance.

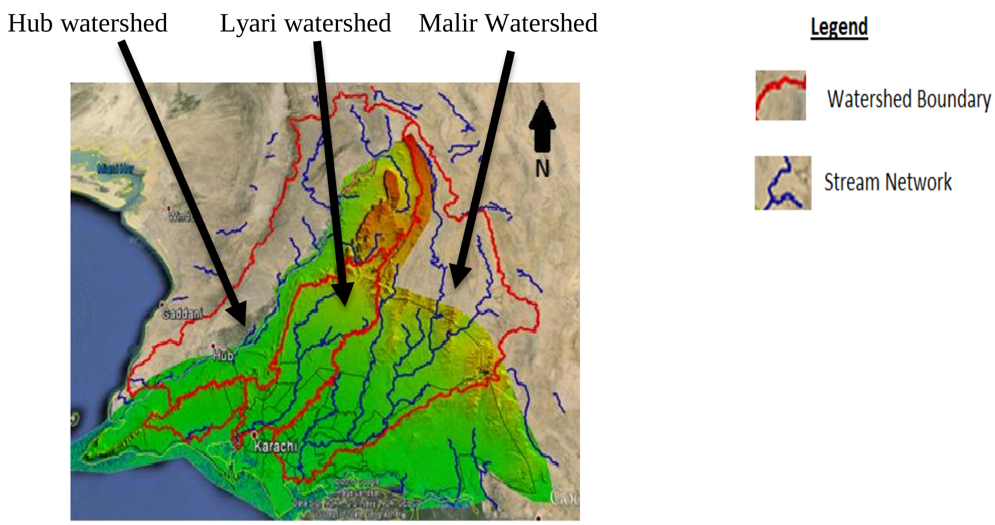

Figure 1: Delineated sub watersheds of Karachi.

Table 2: HSPF parameters for calibration for all areas

Parameter	Unit	Function	Possible Range of Values	Adjusted values Malir	Adjusted values Lyari
FOREST	Fraction	Fraction forest cover	0.0-0.95	0	0
LZSN	Inches	Lower zone nominal moisture storage	2.0-15.0	0.25	0.25
INFILT	In/hr	Index to mean soil infiltration rate	0.001- 10	0.15	0.1
LSUR	Feet	Length of overland flow plane	100-700	250	250
SLSUR	Ft/ft	Slope of overland flow plane	0.001-0.30	0.0737	0.0737
KVARY	1/inches	Ground water recession flow meter	0.0-5.0	0.7	0.7
AGWRC	None	Ground water recession rate	0.85-0.999	0.6	0.6
DEEPFR	None	Fraction of water lost to deep percolation	0.0-0.50	0.085	0.085
BASETP	None	ET due to riparian vegetation	0.0-0.20	0.025	0.025
AGWETP	None	ET directly from ground water	0.0-0.20	0.998	0.998
CEPSC	Inches	Fraction of precipitation retained by vegetation	0.01-0.40	0.00001	0.00001
UZSN	Inches	Upper zone nominal moisture storage	0.05-2.0	0.12	0.12
NSUR	none	Manning's 'n' for overland flow	0.05-0.50	0.2	0.2
INTFW	None	Water infiltrated for interflow	1.0-10.0	0.5	0.5
IRC	None	Interflow recession coefficient	0.3-0.85	0.8	0.5
LZETP	None	ET directly from the lower zone	0.1-0.9	0.001	0.001

Table 3: Calibration of HSPF generated water balance for Malir and Lyari watersheds.

Water Balance	Malir	Lyari
Component	**Percentages**	
Rainfall	100	100
Runoff		
Surface	11.44	16.92
Interflow	6.11	6.42
Baseflow	24.29	21.31
Total	**41.84**	**44.51**
Deep Percolation	**2.50**	**2.35**
Evapotranspiration	**56.26**	**53.76**
Intercept Storage	32.60	26.64
Upper Zone	5.95	7.36
Lower Zone	15.04	16.14
Ground Water	2.03	3.13
Baseflow	0.62	0.62
Total	**56.24**	**53.89**
GRAND TOTAL	**100.6**	**100.62**

From Table 3, it can be concluded that around 42% of the water is being released from Malir, and because the stream network is wide for Lyari, the percentage has increased up to 44.5%. Since the area is arid, there is more evapotranspiration (ET) observed which the model justifies. Based on the satisfactory water balance for both the sub watersheds, flows are also simulated. Calibration and validation of HSPF is only done for Malir River since only Malir River observed flow data are available. Calibration is done over monthly flow data of years 1994-2003; whereas validation is done for year 2003. Coefficient of Determination (R^2) obtained for calibration of monthly flows is 0.74, and for validation is 0.98 as shown in Table 4. It is important to mention that validation results show very healthy comparison between observed and simulated values. Daily observed and simulated flow graph is shown in Figure 2 and monthly calibration and validation is shown in Figures 3 and 4.

It can be observed in Figures 2 that there are multiple points at which observed flow is very low; whereas simulated flow is high. This simulated flow corresponds to the precipitation data provided to the model; therefore it is evident that there is an issue in the value of observed data and not with the simulated data. Most of the observed data show no stream flow which is not possible in reality. Figure 5 shows the averaged monthly observed and simulated flow versus precipitation for Malir River and Table 4 shows that R^2 obtained for Averaged monthly flow is 0.84.

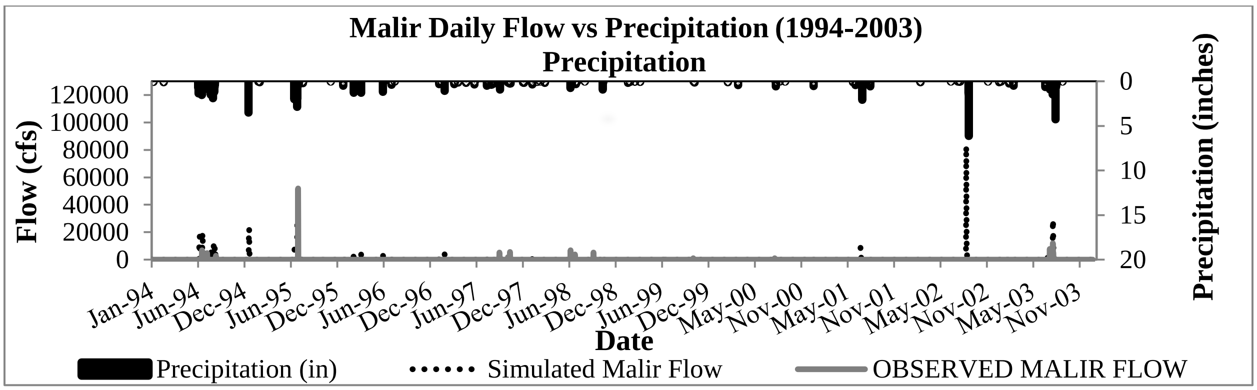

Figure 2: Daily flow Calibration graph versus precipitation for Malir watershed (1994-2003).

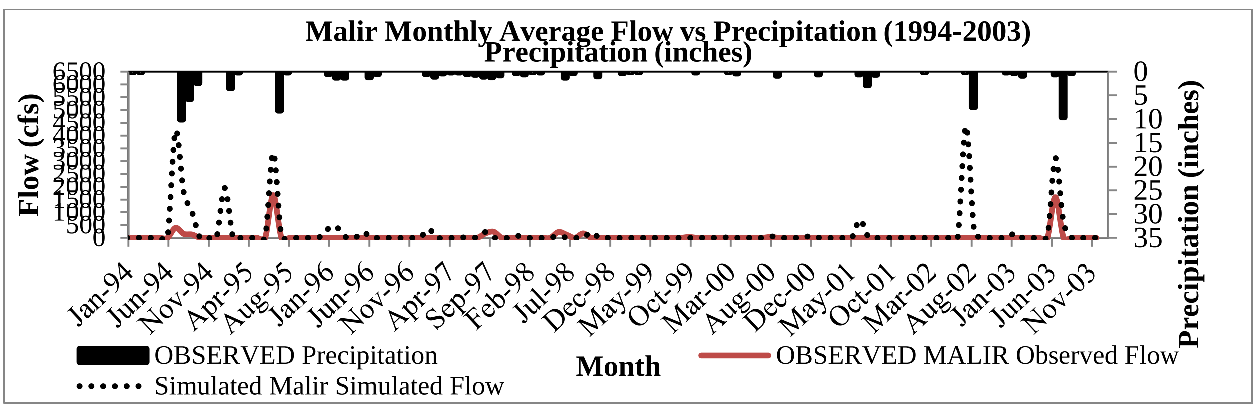

Figure 3: Monthly Average flow Calibration graph versus precipitation for Malir watershed.

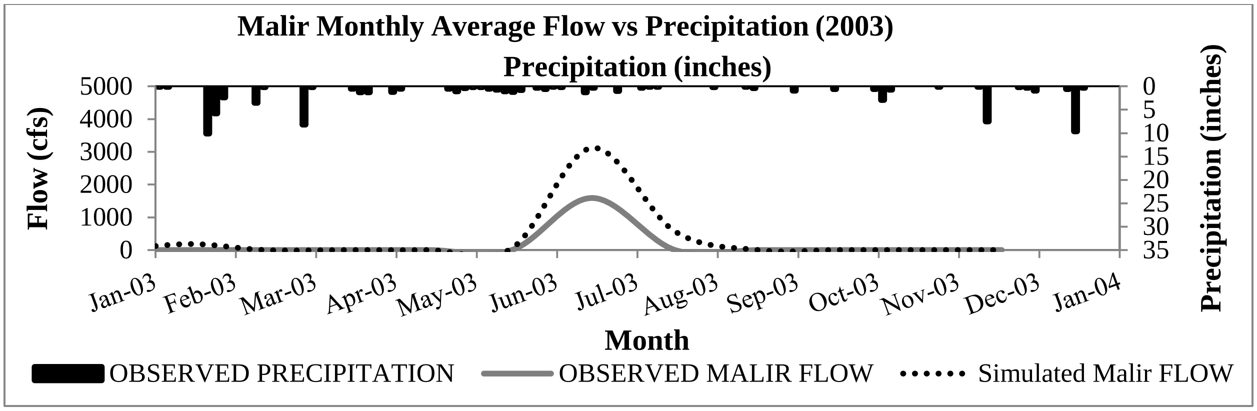

Figure 4: Monthly Average flow Validation graph versus precipitation for Malir watershed.

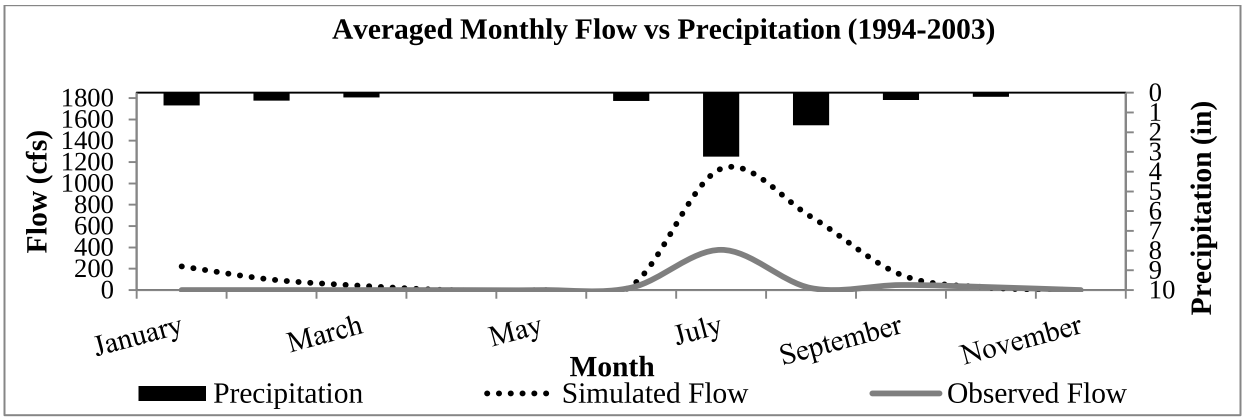

Figure 5: Averaged Monthly flow and precipitation graph for existing situation for Malir watershed.

Table 4: Statistical analysis (R^2) for daily and monthly Calibration and Validation periods.

Calibration Results	
Duration	R^2
Daily (2003)	0.72
Monthly (1994-1995)	0.70
Averaged Monthly(1994-2003)	
Validation Results	
Duration	R^2
Monthly (2003)	0.98

HEC-RAS model is capable of channel flow analysis and flood plain determination. It includes numerous data entry capabilities, hydraulic analysis components, data storage and management capabilities, and graphing capabilities. It models the hydraulics of water flow through natural rivers and other channels.

HEC-GeoRAS is set of ArcGIS tools designed to process geospatial data for use with HEC-RAS. HEC-GeoRAS creates a file of geometric data for import into HEC-RAS and enables viewing of exported results from RAS. The import file is created from the data extracted from ArcGIS layers and Digital Elevation Model (DEM). The layers and DEM are referred to collectively as RAS layers. Geometric data are developed based on intersection of these RAS layers. This model is capable of modeling subcritical, supercritical, or mixed flow regimes. Hydraulic calculations are performed at each cross section to compute water surface elevation, critical depth, energy grade elevation, and velocities.

Relevant GIS layers of the watershed are imported to HEC-GeoRAS. HEC-GeoRAS works as the interface to develop GIS based HEC-RAS readable input dataset. Cross sections and river banks are marked and channel properties are defined to the software for each cross section.

3. Results and Discussions

Once HSPF model was calibrated, flows obtained from the model were used to create inundation maps for different probabilistic rainfall intensities, derived using the IDF curves for Karachi. Details of the probabilistic analysis done are given in Table 5. The intensities are taken for 30-min and 60-min time of concentration (T_c) at 5-, 20-, 100-, and 200-yr return periods. These intensities are replaced by hourly data on one specific date at a time to obtain respective inundation maps. In addition to the probabilistic analysis, few extreme rainfall events that occurred in the past have also been regenerated for current urbanization status in Karachi to depict what would be the situation if one of those events occurred again today.

Flood inundation maps produced using HEC-RAS are overlaid on shape files of different landuse such as hospitals, schools, and building count. Quantification of possible damage to the important facilities due to different probabilistic rainfall events as well as two real precipitation events is presented in Table 6. Table 6 also gives the total area in acres and road length in miles that may get inundated for both the watershed, for every single flood event.

Table 5: Details of probabilistic flood analyses.

Flood scenarios		**30 min**		**Malir**	**Lyari**
Return period (yr)	**Date**	**Intensity (in/hr)**	**Intensity (mm/hr)**	**Flow (ft^3/s)**	**Flow (ft^3/s)**
5	Aug/4/1988	2.2	55	23100	8720
20	Jul/24/1989	3.0	76	39300	13800
100	Aug/7/1990	4.1	104	44200	18500
200	Feb/10/1991	5.3	134	52900	15600
Flood scenarios		**60 min**		**Malir**	**Lyari**
Return period (yr)	**Date**	**Intensity (in/hr)**	**Intensity (mm/hr)**	**Flow (ft^3/s)**	**Flow (ft^3/s)**
5	Jul/30/2008	1.2	30	16500	5720
20	Aug/30/2009	2.2	57	45900	16600
100	Aug/8/2010	3.1	78	42700	17900
200	Sep/7/2011	3.9	99	79900	29900
Flood scenarios		**Per day**		**Malir**	**Lyari**
Other events	**Date**	**Intensity (in/day)**	**Intensity (mm/day)**	**Flow (ft^3/s)**	**Flow (ft^3/s)**
Huge:7th August 1953	Jul/31/2004	11.06	281	152000	55500
high:18th July 2009	Jul/18/2009	9.65	245	50900	21700

Table 6: Results of quantification of possible Flood damages for Malir and Lyari Watersheds.

	Event		**Area (ac)**	**Hospitals**	**Schools**	**Road Length (mile)**
	Return Period (yr)	**T_c (min)**				
Malir	5	30	15506	5	4	48
	5	60	48696	19	12	68
	20	30	89025	30	24	98

	20	60	79641	33	25	104
	100	30	84170	32	24	107
	100	60	72543	32	24	96
	200	30	89931	34	28	115
	200	60	103492	39	30	104
Lyari	5	30	55791	2	13	77
	5	60	1282	4	3	41
	20	30	18207	10	6	57
	20	60	20142	12	6	62
	100	30	21538	13	5	66
	100	60	2131	12	5	65
	200	30	19931	12	6	61
	200	60	26373	16	9	79

Real Time Events

	Year	Area (ac)	Hospitals	Schools	Road Length (mile)
Lyari	1953	34291	22	10	101
	2009	23242	14	6	70
Malir	1953	135306	47	39	168
	2009	90462	34	28	116

For same probabilistic flood scenarios, a lot more stream flow is being generated by Malir watershed than Lyari watershed. Less area being inundated and less number of facilities are being affected in Lyari watershed as compared to Malir watershed. Only exception is the 5-year return period with 30-minute time of concentration rainfall event for which Lyari watershed shows more area being affected as compared to area for Malir watershed.

Malir River watershed shows a lot more area being inundated for both 1953 and 2009 rainfall. Similarly a lot more road length, hospitals, and schools are under threat of possible flood damage in Malir watershed than Lyari watershed if a similar rainfall event occurs in existing level of urbanization in Karachi. One major reason of greater inundation in Malir watershed is its steeper slope and bigger drainage area. Therefore, more water runs off in the watershed creating more inundation and possibly more destruction. Moreover, most of this destruction is in the lower part of Malir watershed where all the water travel down due to topographic conditions. Most of the area around the vicinity of the streams gets inundated; however, modeling results also showed that water travelled a lot farther than the streams and inundated the other areas too. For all possible flood scenarios based on 5-, 20-, 100-, and 200-year return period rainfall intensities road length being affected ranges from 41 miles to115 miles. Minimum damage is being caused in the shortest (5-year) return period with 60-min time of concentration rainfall for Lyari watershed; whereas the maximum damage is being caused in the longest (200-year) return period with 30-min time of concentration rainfall in Malir Watershed.

As the magnitude of return periods increase the probable flood damage also increases. T_c of 30-min gives more intense rainfall as compared to 60 minute for all return periods. Another hazard quantification done is in the form of number of buildings that would possibly inundate if there is a 200-year return period flood in Karachi city. Results of this analysis are presented in Figure 8. The graph also represents the results of 1953, and 2009 rainfall events. It can be seen that the most vulnerable towns in terms of building damage due to flood are Korangi, Gadap, and Gulberg. On the other hand, 1953 rainfall event poses the worst threat to Karachi city if it occurs today, based on the existing landuse and population conditions. Almost 32, 26, and 21 thousand buildings are under threat of getting inundated in Korangi, Gadap, and Gulberg town, respectively, if 1953 flood event occurs again.

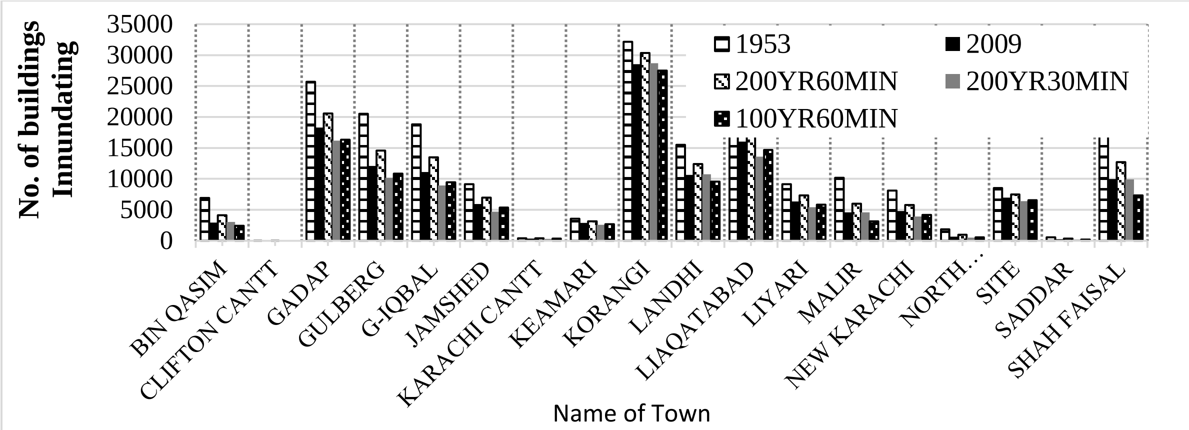

Figure 8: Graphical representation of number of buildings inundated during 200 year return period, 1953, and 2009 rainfall events.

Table 7: Total possible damage in Karachi city due to different flood scenarios.

Event		Area(ac)	Hospitals	Schools	Road Length (mi)
Return Period (yr)	**T_c (min)**				
5	30	71297	7	17	125
5	60	49978	23	15	109
20	30	107232	40	30	155
20	60	99783	45	31	166
100	30	105708	45	29	173
100	60	74674	44	29	161
200	30	109862	46	34	176
200	60	129865	55	39	183
1953		169597	69	49	269
2009		113704	48	34	186

4. Conclusions and Recommendations

The modeling results of the study showed that simulated flood inundation maps were helpful in assessment of anticipated damage for the vulnerable areas due to flooding for Karachi. As the number of years of return period increase the probability of the amount of surface flow increases; therefore, the simulation of range of various rainfall return periods produced possible amount of water volume which could be helpful in planning and management for disaster risk reduction. More specifically, the most vulnerable areas in terms of flood damage, marked for various return period rainfall events are lower part of Malir watershed, and towns of Korangi, Gadap, and Gulberg. The study also concludes the importance of water modeling for rainfall-runoff analysis for prior knowledge of amount of water that could damage various structures. Therefore, it is necessary to be prepared for the worst case scenario. Moreover, the modeling of the effect of historical rainfall events for existing landuse has depicted the extent of possible damage due to the increased urbanization.

Karachi city is highly urbanized, with little land available for infiltration of rain. Whenever a long duration or short duration high intensity rainfall occurs there is a chance of high inundation and heavy damage in the city. This damage can be reduced by increasing the infiltration rate of water which would reduce the amount and rate of runoff. This can be done by promoting low impact development (LID). Low impact development is the use of small grassy patches at multiple intervals with in the city that increases the infiltration and breaks down the power of runoff. Being a metropolitan city spread out in all directions with congested series of buildings, planned and unplanned, Karachi needs a well-developed sewerage and storm water drainage system. In addition, the existing drainage systems are not able to fulfill the requirement due to excessive urbanization. This has been observed in previous rainfall events when most of the water that poured down went nowhere but stayed on the surface or ran over it creating all sorts of damage. Storm drainage system should be reevaluated and improved accordingly specifically in areas that are most vulnerable to flood damages and generally all over the city. Seasonal maintenance of storm water drains should be done to reduce the risk of heavy flood inundation and loss of infrastructure and economic loss.

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A Comparative Study of Ground Improvement by Lime and Granular Columns

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Abstract
The rapid increase in construction activities over the past few decades have undeniably increased the importance of ground improvement techniques. Ground improvement techniques are numerous but all aim to rectify the engineering behavior of problematic soils so as to achieve desired properties. According to theoretical and experimental research, reinforcing soil by inclusion of granular/lime columns is considered to be economical, feasible and effective in soft soils. This research aims at investigating the effects of floating columns in clayey soil with silty deposits by developing small scale laboratory models. A comparison is made among lime and granular columns whereby the results of the treated ground are compared to the untreated ground. The effects of granular/lime columns on soils of different shear strengths (low-medium- high) and slenderness ratio (L/D) of columns are investigated. Based on the results, it is concluded that the granular columns gave higher strength than lime columns in the soil of low shear strength, whereas the lime columns gave more strength than granular columns in soil of higher shear strength.

Key Words
Ground Improvement, Granular Columns, Lime Columns, Shear Strength, Slenderness ratio

1. Introduction

The growing trend in construction industry and increasing value of land has encouraged engineers to develop certain techniques to improve the engineering properties of weak soil deposits. There are many ground improvement techniques which are subject to site specific conditions, nature and intended purpose of the structure, design specifications and material available, loading conditions, etc. Soft soils are usually associated with low shear strength and low permeability which decreases their load carrying capacity and compromise the serviceability of overlying structures. Such soils need proper time to dissipate the excess pore pressures accumulated due to increasing load on top of it which when not allowed will cause failure of the soil. Stone columns as a ground improvement technique was first used in Bayonne, France in 1830 by Moreau et al. to support the loads coming from iron works *(Hughes and Withers, 1974).* In-situ and

laboratory tests indicated significant increase in load carrying capacity of sand column treated ground *(Aiban, 2002; Ashford et al., 2000; Mitchell, 1981). Wong P.K* Carried out theoretical studies on lime and stone columns and proved that both ground improvement techniques are able to provide increase in shear strength and stiffness of soil mass resulting in increased undrained shear strength after treatment. They are installed as floating columns which carry the overlying load through skin friction developed along the length or as end bearing columns which transmits the load to harder strata along with skin resistance. Stone columns act similar to vertical drains except that they are stiffer than vertical drains and differ than pile foundations in that they carry the overlying loads by mutual sharing with the surrounding soil. They have been used to increase the bearing capacity and reduce settlement of soil under storage tanks, earthen embankments, raft foundations etc. *(Ambily and Gandhi, 2005).* Stone columns can be installed by either replacement method (dry) or displacement method (wet) depending upon the nature of soil and site conditions *(Munfakh et al. 1987, Hayward Baker Inc. 1996).*
Stone columns fail in three different ways as observed by different researchers. The modes of failure, after loaded in compression, were identified as bulging *(Hughes and Withers 1974, Hughes et al. 1976),* general shear failure *(Madhav and Vitkar 1978)* and sliding *(Aboshi et al. 1979).* The bulging of stone columns usually occurred in the top regions where confining stresses were comparatively lesser *(Hughes and Withers 1974, Balaam and Booker 1981, Greenwood 1970).* The column failure mechanism was controlled by critical column length which is the shortest length of the column which can carry ultimate load without failure *(Hughes and Withers 1975).* A critical column length of 4 to 5 times of diameter of column (D) was identified by different researchers beyond which no significant increase in load carrying capacity was observed *(Hughes and Withers 1974; Mithra and Chattopadhyay 1999; Samadhiya et al. 2008).*

Different numerical, analytical, theoretical and experimental models have been developed over the course of years to analyze the behavior of granular and lime columns in soft soils. *Ambily and Gandhi (2007)* performed an analytical as well parametric study over single as well as group of seven stone columns considering drained behavior of soil surrounding the column. A detailed experimental observation was carried out and the results were compared with numerical models developed using finite element packages PLAXIS 2D. *Balaam et al. (1978)* performed finite element analysis and considered the effects of stiffness of stone columns on load-deformation behavior of improved ground. *Sivakumar et al. (2007)* performed laboratory tests on single floating sand column to examine the load-deformation behavior of improved soft clay with varying length of the column. The method of installation varied as well which included: wet compacted columns and previously frozen columns. *Najjar (2010)* compared results of different numerical, laboratory and full scale field tests performed on granular columns i.e. stone/sand columns in order to relate them to the bearing capacity and settlement calculations. *Shivashankar et al. (2011)* performed plate load tests on stone columns in layered soils comprising of top weak layer. It was found out that the stiffness and load carrying capacity was affected due to increasing depth of weak top layer. *Poorooshasb and Meyerhof (1997)* studied the efficiency of end bearing lime and stone columns supporting a rigid mat foundation in reducing the settlement.

This research is aimed at developing small scale models that can be used for laboratory testing to study the effects of treatment of soft soil deposits with floating granular (stone, sand) and lime columns and compare the results with that of untreated ground. These columns are installed in weak soil of varying shear strength to define a limit for the type of soil most responsive to treatment by columns for a selected column material. The effects of depth of the columns on the load-deformation behavior of improved soil will also be investigated by increasing the depth of columns.

2. Experimental Setup

2.1 Test Program

A series of model tests of treated soil with sand, stone and lime column was performed in laboratory to study the efficiency of these columns in improving the load carrying capacity of soil. The steel moulds used for experimentation had the following dimensions; height = 360 mm, internal diameter = 300 mm, wall thickness = 6 mm and were locally assembled. Load tests were carried out in two steps; first on untreated soil and in the second stage on treated soil in a Compression Testing Machine.

(a) (b)

Figure 1: Steel moulds: (a) Dimensions (b) Mould loaded in Compression Testing Machine

2.2 Properties of Materials

2.2.1 Soil

Soil was collected from Jahangira District Sawabi, KPK. The soil was pulverized in laboratory to conduct different tests to obtain various engineering properties like moisture content, grain size distribution, Atterberg's limits, unconfined compressive strength and proctor compaction tests. The results are summarized in table 1.

Table 1: Physical Properties of Soil

Property	Results
Optimum Moisture Content (%)	24
Liquid Limit (%)	51
Plastic Limit (%)	28
Plasticity Index	23
Shrinkage Limits (%)	26.43
Specific Gravity	2.56
Passing No. 200 Sieve (%)	100
Silt Contents (%)	40
Clay Contents (%)	60

Maximum Dry Density (lb/ft^3)	100
Classification according to USCS	CH

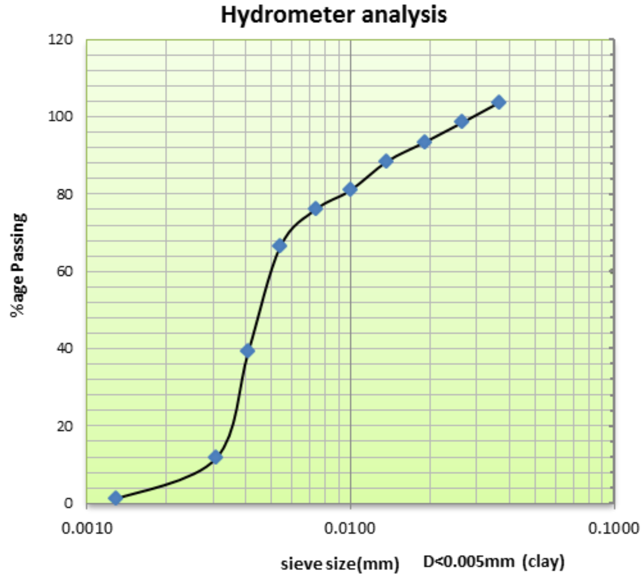

Figure 2: Hydrometer Analysis of Soil

2.2.2 Sand

Locally available sand from Lawrencepur was used as backfill material in sand columns. The sand was thoroughly washed and oven dried, sieve analysis was performed and the angle of internal friction was found using direct shear test. The maximum and minimum dry density was found out to be 94 lb/ft^3 and 89 lb/ft^3.

2.2.3 Stone

Crushed marble stones ($CaCO_3$) were collected from Rashakai, District Mardan, KPK. The stones were washed and passed through 10 mm sieve and retained on 2 mm sieve were deemed suitable for constructing columns. Direct shear test was performed on the crushed stones to find its angle of internal friction "ϕ". The γ_{max} and γ_{min} of the crushed stones are 105.94 lb/ft^3 and 86 lb/ft^3. A light compacting effort was adopted to achieve a density of 95.5 lb/ft^3 which was used for direct shear test as well.

2.2.4 Lime

Limestone ($CaCO_3$) was obtained from Barakahu area of Islamabad and pulverized. The pulverized lime was passed through sieve no.4 (4.75 mm) and oven dried for further use as a column material.

Table 2: Engineering Properties of Column Material

S. No.	Properties	Stone	Sand	Lime
1.	G_s	2.94	2.7	2.61
2.	Φ_s	41°	31°	-
3.	γ_{max}	105.94 lb/ft^3	94 lb/ft^3	90 lb/ft^3
4.	γ_{min}	89 lb/ft^3	89 lb/ft^3	73.4 lb/ft^3
5.	Fineness Modulus	-	2.018	2.75
6.	Water absorption	0.23 %	0.51 %	0.98 %

2.3 Preparation of Clay Bed

All the tests were performed on soft clay bed at three different shear strengths of 54 kPa, 32 kPa and 14 kPa. Before the preparation of clay bed, unconfined compression tests were performed in cylindrical sample of 40 mm diameter and 80 mm height. A relationship was developed between moisture content and unconfined compressive strength. Moisture contents corresponding to desired shear strength were found to be 31 %, 35 % & 39 %. For preparation of each clay bed, oven dried clay sample was used and then required amount of water was added to soil to obtain desired shear strength and thoroughly mixed to form a uniform paste. A thin coat of oil was applied along the inner surface of steel container to reduce friction between clay and container wall. The soil was filled in the container in six layers, each layer 50 mm thick after compaction. The soil was filled in the container up to a total height of 300 mm. The surface of each layer was provided with compaction energy of 12375 ft-lb/ft^3 as in the case of Standard Proctor Test. Each layer was compacted with a tamper of 10 kg dropped from a height of 300 mm and given 70 blows. Care was taken to ensure that no significant air voids were left out in the test bed.

2.4 Construction of Granular and Lime Columns

After the preparation of soil bed, columns were constructed by a replacement method. A thin open-ended seamless steel pipe of 37 mm outer diameter and wall thickness 1 mm was pushed down into the clay at the center of the steel container up to the desired depth. Slight grease was applied on the outer surface of the pipe for easy penetration and withdrawal without any significant disturbance to the surrounding soil. To avoid suction, a maximum height of 50 mm of soil was removed at a time. After removing the soil the column material were fed into the hole from top in layers of 50 mm each. To achieve a uniform density, compaction was given with a 1.25 kg circular steel tamper with 15 blows of 100 mm drop to each layer. This light compaction effort was adopted to ensure that it did not create any disturbance in the surrounding soft clay by bulging laterally. The procedure was repeated until the column is completed to the required heights of L/D = 4, 5.5 and 7. After the installation of column, the top surface of the container was covered with plastic sheet for 4 days as curing period to ensure uniform moisture. After 4 days compression load test was carried out on each model.

2.5 Test Procedure

After preparation of model, the load-deformation behavior was studied by applying vertical load on surface of untreated as well as treated soil in a compression chamber. A 100 mm thick steel plate with 200 mm diameter was placed at the center of steel container to transfer the uniform load on soil. Strain gauges were attached to the upper plate to constantly monitor the settlement. The load was applied at a constant loading rate of 0.025 MN/min. Load was applied continuously until a settlement of 25 mm was achieved. The sample was extracted from the mould and soil surrounding the column was removed carefully to observe the failure pattern of the column.


(a)

(b)

(c)


(d)

(e)

(f)

Figure 3: (a) Mixing and compacting soil (b) Borehole for column (c) Pouring column material (d) Model wrapped for curing (e) Model testing (f) Failure pattern of composite model

3. Results and Discussions

3.1. Effect of Column Material

The comparison of untreated and soil treated with columns at relatively high shear strength (S_u = 54 kPa) is shown in figure 4 by comparing the load-settlement behavior. These results show that there is a significant increase in load carrying capacity of treated soil, i.e, 33% to 47% increase as compared to untreated soil at a settlement value of 25 mm. Furthermore, it is noted that soil treated with lime column has a higher load carrying capacity than granular columns. The lime column treated soil gives 47% increased strength whereas the sand column

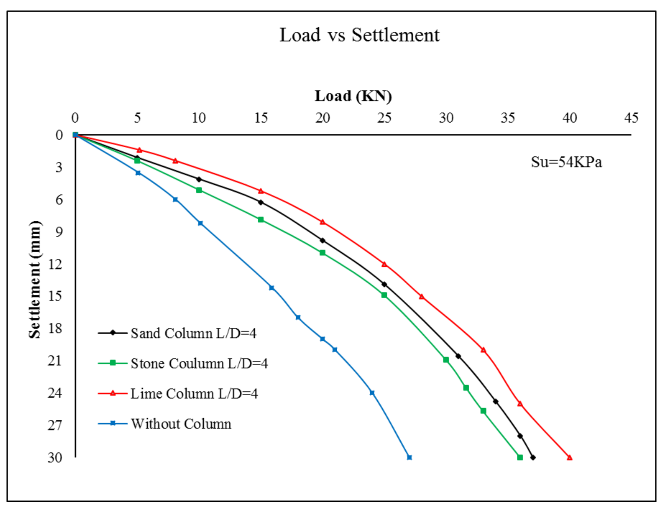

Figure 4: Load-settlement graph of untreated and treated soil at S_u = 54 kPa

and stone column treated soil gives a 39% and 33% increase in strength respectively. It is inferred from these results that at low moisture content (higher shear strength), lime columns are more effective in this soil as compared to granular columns.

Figure 5 shows a typical load-settlement behavior of lime and granular columns treated soil and untreated soil at relatively lower strength (S_u= 32 kPa). The increments in strength are observed to be 16% for sand columns treated soil, 28% for lime column treated soil and 37% for stone columns treated soils as compared to the untreated soil. Stone columns are considered to be more effective at relatively lower shear strength values.

Similarly in figure 6, the results for a soil of low shear strength are shown i.e. S_u= 14 kPa. As literature shows that in soft soils i.e. soil of low shear strength (S_u=15 kPa - 25 kPa), the effects of stone columns are more pronounced *(Kempfertn 2003)* and also preferred in soils with S_u < 10 kPa *(Raju 1997).* The same trend is observed as the shear strength is reduced from 32 kPa to 14 kPa. Stone column treated soil shows 10% increase in load carrying capacity compared to untreated soil. While no significant improvement is shown by sand column and lime column treated soil due to very low shear strength of the soil.

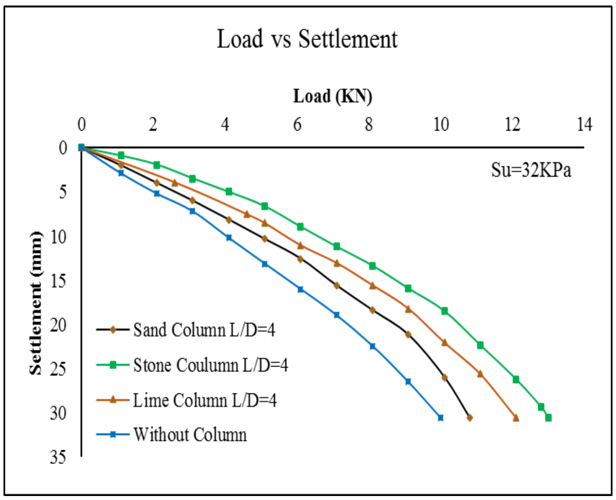

Figure 5: Load-settlement graph of untreated and treated soil at S_u = 32 kPa

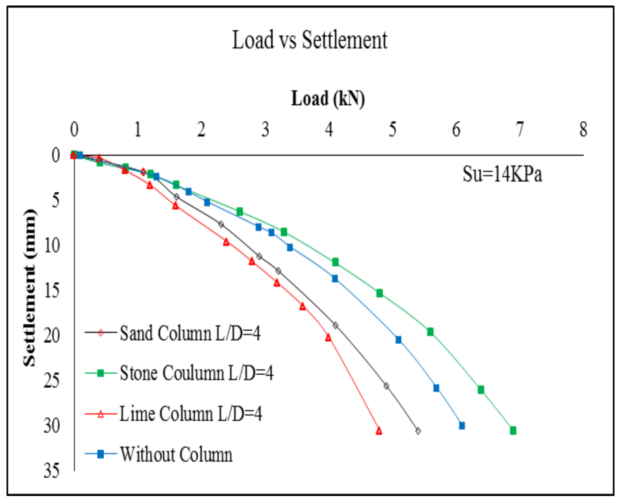

Figure 6: Load-settlement graph of untreated and treated soil at S_u = 14 kPa

3.2 Effects of L/D Ratio on Load Carrying Capacity

Figure 7 illustrates the relationship between loading intensity and the corresponding settlement of stone column treated ground at different length to diameter ratios i.e. L/D = 4, 5.5 and 7. As the slenderness ratio of column increases the load carrying capacity decreases. The load carrying capacity is higher at L/D = 4 (the column is short). Previous studies show that column length beyond critical limit does not show any increase in load carrying capacity due to failure by bulging *(Hughes and Withers 1974; Mithra and Chattopadhyay 1999; Samadhiya et al. 2008).*

A similar trend is observed in figure 8 and figure 9 whereby increasing the L/D ratios for sand column and lime column in soft soil gives reduction in load carrying capacity.

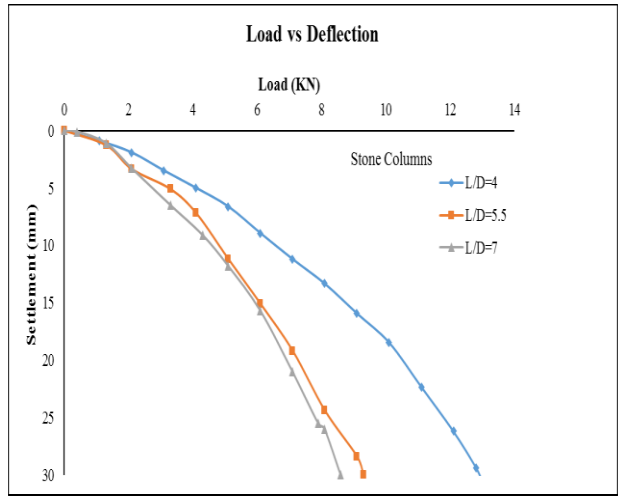

Figure 7: Load-settlement graph of stone column treated ground at different L/D ratio

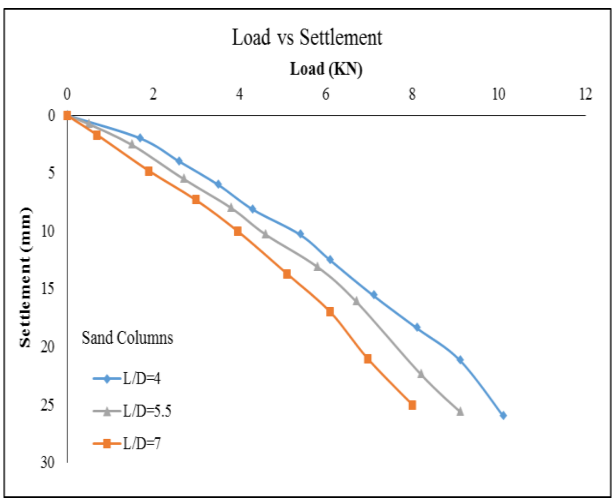

Figure 8: Load-Settlement graph of sand column treated soil at different L/D ratio

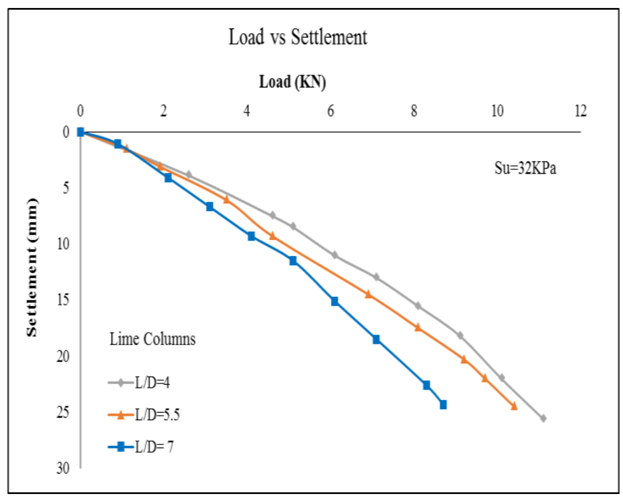

Figure 9: Load-Settlement graph of lime column treated soil at different L/D ratio

4. Conclusions

The composite models containing column in the center represented the behavior of interior column among a group of columns in practice. The experimental work confirmed the use of granular and lime columns as an effective ground improvement technique. The variance in shear strength of soil showed the suitability of column material to corresponding shear strength. The conclusions based on the results and discussions are presented as under;

- The soil treated with granular and lime columns exhibited a stronger response to applied load. Being a stiffer material than the untreated native soil, the composite material has greater load carrying capacity and causes reduction in settlement at same loading intensity as compared to untreated soil.

- When the slenderness ratio (L/D) of the columns is increased, a reduction in load carrying capacity is observed. This reduction shows that increasing length beyond the critical column length bears no enhancing effect on the load carrying capacity of the treated soil.
- At higher shear strength, the soil treated with lime columns has greater load carrying capacity than granular columns owing to greater absorption of moisture from the soil as well as expansion of the column due to slaking of lime.
- At low shear strength, granular columns are observed to have relatively higher load carrying capacity than lime columns owing to higher stiffness of the granular material. Lime gets saturated at high moisture content and it has less stiffness than the granular material so its load carrying capacity decreases.

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CORRELATION OF CBR WITH INDEX PROPERTIES OF SHALE

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Abstract

Shale is a type of soil which is problematic due to laminated and fissionable in nature so extra care and time is needed for a Geotechnical Engineer to test Stiffness and other important parameters of shale type of soil. Since Geotechnical CBR (ASTM D1883-16) test is carried out to determine stiffness modulus of soil. Beside reliable test CBR became even more time consuming and expensive for Shale type soil, which cause delay in the completion of construction projects on such type of soil. For Geotechnical Engineer correlations are always useful tool to tackle such problems. Work on Correlation of CBR with different parameters like Liquid Limit (L.L), Plasticity index (P.I), OMC, Maximum dry density, %fines are done by different researchers for different type of soil These correlation are comparatively easy, cheap, hasten and effortless to determine the CBR. This research is based on regression model of CBR with other simplest parameters like Liquid Limit (L.L), Plasticity index (P.I), MDD, OMC and %Finer for shale type of soil.

Keywords

Correlation, Shale, Atterberg Limits, Soaked CBR, Un-soaked CBR

1. Introduction

Since the soil parameters varies from site to site or location to location thus selecting the reliable properties of soil is always a challenge for the Geotechnical Engineers. One of the options is intense soil investigations. However, there are various soil properties whose determination is time consuming and expensive, CBR (California Bearing Ratio) is one of those. The intensive soil investigation for such properties will be cumbersome and may cause delay in the completions of the civil engineering projects. In order to facilitate the local construction industry, many researchers Agarwal and Ghanekar, Dharamveer singh (2013) [1], K.S Reddy, Laxmiikant Yadu (2011), T.Datta, B.C Chottopadhyay (2011) [2], Ramasubbarao, G.V, Siva Sankar, G (2013) [3]- have developed the empirical relationships between CBR and index properties of their local soils.

This paper gives an overview to obtain a correlation between CBR value with soil index properties that is suited for Shale subgrade soil. Previous researches and investigations on Shale soil indicates that weathered and hard rock, and much more swelling then the normal soil exists in it. Hence, this paper deeply focuses on developing the intended correlation for different distributions of suitable subgrade soils, specifically non-expansive fine grained soils, which represents the study area.

Jamshoro is a major educational city of Pakistan, which is causing rapid development throughout the vicinity. The soil that is obtained in that vicinity is almost shale in nature. Currently various projects of building and road are under execution on Jamshoro soil that also include Hyderabad-Karachi Motorway. This paper gives an overview to obtain a correlation between CBR value with soil index properties that is suited for Shale subgrade. Previous researches and investigations on Jamshoro soil indicates that weathered and hard rock-on other side, the soil of Jamshoro is composed of clay layers at shallow depth, which gives low strength and high compressibility; Aneel Kumar and Ghous Bux Khaskheli (2012)[1].

In the light of above discussion construction in the vicinity of shale soil is very complicated and unstable. In order to have a sustainable construction in the vicinity of shale type soil, the intensive soil investigation is always required. Which becomes cumbersome when time consuming tests like CBR is required. The output of the predicted correlation using SLRA and MLRA will provide Local authority, consultants and contractors preliminary background information on the value of CBR, for a localized sub-grade material, from simple soil index properties like Liquid Limit (LL), plastic Limit (PL), %Finer, OMC, MDD, with saving of time saving and without performing laborious and costly laboratory CBR test.

2. Experimental Work

For this research Five (5) soil samples were collected from different locations of Jamshoro, at a depth of 2-2.5 ft from the ground surface as shown in **Figure 01**. These samples were tested at Geotechnical Engineering laboratory in Civil Engineering Department of MUET for water content, specific gravity, plastic limit, liquid limit, plasticity index, particle size distribution, maximum dry density and optimum moisture content, soaked and unsoaked CBR values. The modified proctor compaction test was used to find MDD and OMC. Each CBR mould was compacted in five layers of soil at predetermined OMC. The compaction effort used to prepare CBR moulds was generated by 56 blows per layer in CBR mould of diameter 6 inches.

Figure 1; Soil Sample collection

Figure 2; Soaked CBR Test

All the tests were performed according to AASHTO and ASTM specifications. Table.1 shows the summary of laboratory and field 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 (%)	CBR (%)		AASHTO Soil Group
							Unsoaked	Soaked	
1	55	28	27	78.3	1.85	17.1	41.15	13.72	A-7-6
2	56	30	26	72.4	1.81	14.37	20.16	12.17	A-7-6
3	50	29	21	68.5	1.91	10.6	49.8	10.21	A-7-6
4	34	17	18	75.8	2.05	10.38	33.07	19.51	A-6
5	57	33	25	82.6	1.77	12.6	42.32	22.65	A-7-5

3. Results And Discussions

Above practical data confirms that, shale type soil generally belongs to A-6, A-7-5 or A-7-6, which is our main concern in this research work. Using Single regression model (SLRA) and Multiple regression model (MLRA) we generated different correlation with varying R^2 value.

3.1 Empirical Correlation of Soaked CBR with Liquid Limit, Plasticity index, %Finer, MDD and OMC using SLRA:

Different forms of correlations between soaked CBR and other simplest parameters like Liquid Limit (LL), Plasticity Index (PI), % Finer at 200 sieve, OMC and MDD were generated using SLRA, of which best possible correlation is as shown in **Figure 03**

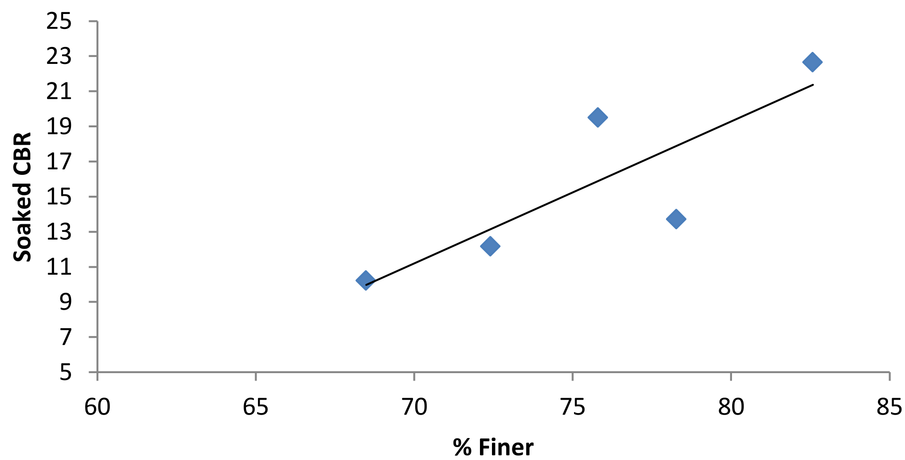

Figure3; Scatter plot of Soaked CBR vs %Finer

The correlation found is

$$\textbf{Soaked CBR=0.81 (\%Finer) -45.36}$$
$$(R^2=0.7)$$

3.2 Empirical Correlation between Soaked CBR and Atterberg Limits using MLRA :

Using Multiple Linear Regression Model between Soaked CBR and other parameters suitable correlations were obtained which are discussed as below.

$$\textbf{Soaked CBR} = 0.88\ (\%\text{Finer}) - 0.22\ (\text{LL}) - 40.05$$
$$(R^2 = 0.84)$$

$$\textbf{Soaked CBR} = 0.92\ (\%\text{Finer}) + 16.25\ \text{MDD} - 84.1$$
$$(R^2 = 0.80)$$

$$\textbf{Soaked CBR} = 0.98\ (\%\text{Finer}) - 1.03\ \text{OMC} - 45.52$$
$$(R^2 = 0.9722)$$

$$\textbf{Soaked CBR} = 0.98\ (\%\text{Finer}) - 0.71\ (\text{P.I}) - 41.84$$
$$(R^2 = 0.93)$$

$$\textbf{Soaked CBR} = 488.18 - 189.89\ (\text{MDD}) - 2.29\ (\text{LL})$$
$$(R^2 = 0.76)$$

Where;
LL= Liquid Limit, MDD= Maximum Dry Density, OMC= Optimum Moisture content,
P.I= Plasticity index

3.3 Empirical Correlation between Un Soaked CBR and Atterberg Limits using SLRA :

No reliable empirical correlation was found between Un soaked CBR and other simplest parameters

3.4 Empirical Correlation between Un Soaked CBR and Atterberg Limits using MLRA :

No dependable correlation was found between Un soaked CBR with other simplest parameters using MRLA

4. Conclusions

The recommended correlation developed to predict Soaked CBR (SCBR) from other simplest parameters is

$$\textbf{Soaked CBR} = 0.98\ (\%\text{Finer}) - 1.03\ \text{OMC} - 45.52$$
$$\text{With } (R^2 = 0.97)$$

No suitable correlation of Uncloaked CBR was found with other simplest parameters either by using SLRA or MLRA

% Finer would be reliable parameter because of its consistency in the values to generate the value of CBR.

5. Recommendations

To improve reliability of the correlation, it is recommended to utilize more number of soil samples.

The CBR relations proposed in this research are developed by conducting field and laboratory tests on limited locations of Jamshoro (i.e. within the premises of Jamshoro). Hence, it is recommended that its bearing for soils of similar type in other parts of Sindh where shale is found- should be studied.

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Subsurface Exploration for Road Sinking at Sorchen and Counter Measures

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Abstract
The vulnerability of the valley to the road sinking are increased relatively due to various physical properties of the areas. Failure of land masses including the sub-grade were attributed to several factors. It may be manmade or natural. The morphology of the slopes and consequent to the slope failures are complex and controlled by many factors, such as lithology, rock mass strength and other physical properties. This study is useful on the way in locating the vulnerable to sub-grade sinking pockets along the highway within the study area is a prior concern. In this paper the members propose to use site investigation techniques using trial pits and geophysical test using ground resistivity test to prepare physical characteristics maps where decision makers would take sufficient care to avoid from this possible risk of road sinking which otherwise would result into various pitfalls. The current site investigation is primarily based on routine sampling in exploratory trial pits, with transport and testing of samples at distant laboratory. In order to validate the procedure, further Wenner's 4 Spikes method ground resistivity test were advanced and checked for matches and mismatches for: zone I, zone II, zone III and zone IV and confirmed positive result.

Keywords
Causes identification, Engineering solution, Geophysical Investigation, Geophysical Site Characterization, Pile Foundation, Road sinking.

1. Introduction

Sorchen lies within the stretch of Thimphu-Phuentsholing highway having geographically, the terrain features within the range of steep to very steep slope and its elevation ranges from 230 meters to 2055 meters approximately from sea level. It starts from 0+00Km (Phuentsholing) and ends at 21+00Km (Sorchen) along the stretch from Thimphu–Phuentsholing Highway. The area is sparsely populated mainly due to the tough geographically condition. Risks posed by the road sinking is usually common in the monsoon. Sorchen lies within the stretch of Thimphu-Phuentsholing highway with a fragile terrain and is geologically an unstable region. Being seated on an active earth quake zone V, Sorchen is afflicted with other severe natural calamities such as landslides and the problems of road sinking. Sorchen has posed problems to Bhutanese people since the first landslide in the area occurred in 1983 (The Bhutan society newsletter, July 2002).

Figure 1 : Road section in Sorchen showing the sinking profile

Table 1: Details on Study Stretch

Zone Designation	*Approx. Distance from P/ling(km)*
Zone 1	21.2
Zone 2	17.01
Zone 3	16.02
Zone 4	11.5

2. Methodology

2.1 Preliminary Geological Survey

Land topographical survey using total station was conducted. The preliminary survey for the assessment of sinking area along the Phuentsholing–Thimphu highway at Sorchen have shown four critical pavement sinking zones. These zones are seen to be temporarily maintained through normal practice of re-surfacing, however, the problem of sinking remains unsolved for the past last decades. The zones were identified as Zone I, II, III and IV.

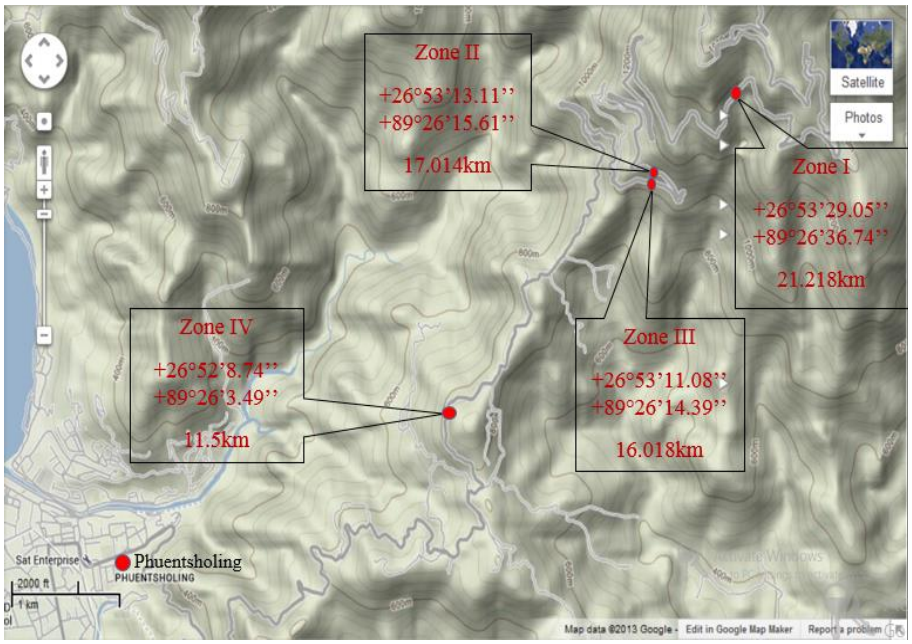


Figure 2: Site demarcation

2.2 Geotechnical Site Investigation

To further assess the site characteristics of the critical sinking zones, the geotechnical site investigation were carried out through physical observation at the site, laboratory test and field exploration in each zones. The site investigation of the study area was achieved through trial pitting method. Three trial pits of dimension 1 m X 1 m X 1.5 m was dug at the three zones in the study area and owing to difficult terrain features of the ground, digging of pit at zone III was not feasible. Site investigation from the trial pits under this project were obtained by two methods:

- By Physical observation / visual investigation of the soil strata in the field.
- By conducting laboratory tests

2.3 Subsurface Soil Exploration

Electrical resistivity test is found to be quick and reliable tool to classify and predict physical properties of materials based on the ease at which electric current can pass. Open test pits were also conducted to visualize and verify the soil conditions reported by using the electrical resistivity test. The test requires inserting four test rods into the test area, in a straight line, equally spaced and all at a depth of 229 mm

(9 in.). A constant current is injected into the earth from the earth resistance tester through the two outer test rods. The voltage drop resulting from the current flow through the earth is then measured across the inner two test rods. Most testers are designed to provide a direct reading in ohms. This value is then used in one of the following formulas to calculate the soil resistivity (ρ) of the tested area. $\rho = 2\pi\pi \times A \times R$

Where: ρ = soil resistivity in Ω-m

A = Distance between test rods (m)

R = Resistance obtained from tester (in ohms)

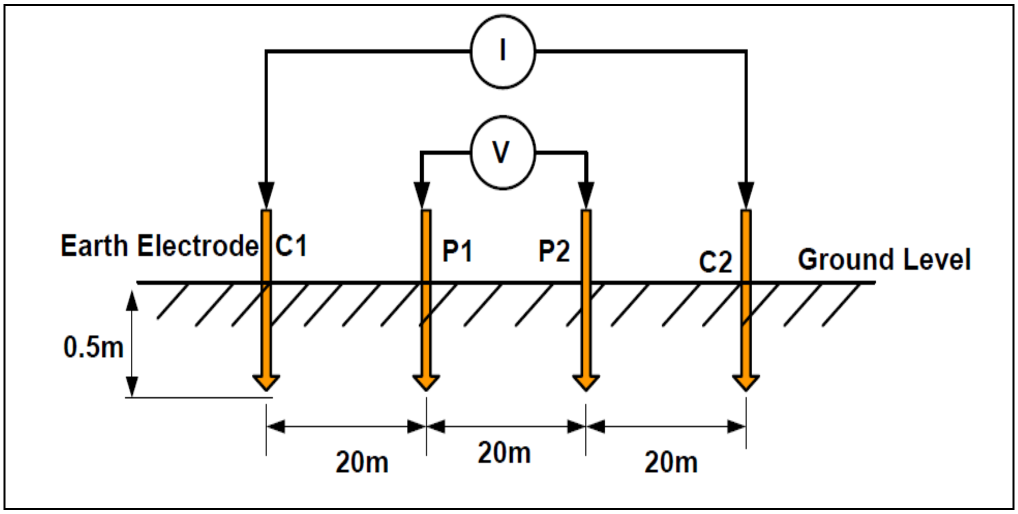


Figure 3: Wenner's four pin method

3. Results and Discussions

The topographic survey outcomes indicated settlement of 0.54m, 0.81m, 0.79m and 0.74m for Zone I, II, III and IV respectively caused due to sinking effect.

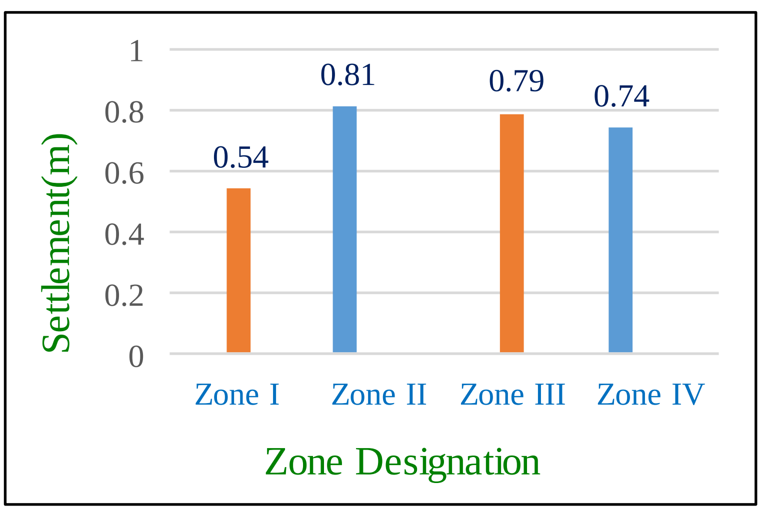


Figure 4: Settlements at different Zones

Based on the test results, the shallow soil profiling and classification were carried out. The predominant soil type in all the zones were identified as coarse grained soil with little fine content.

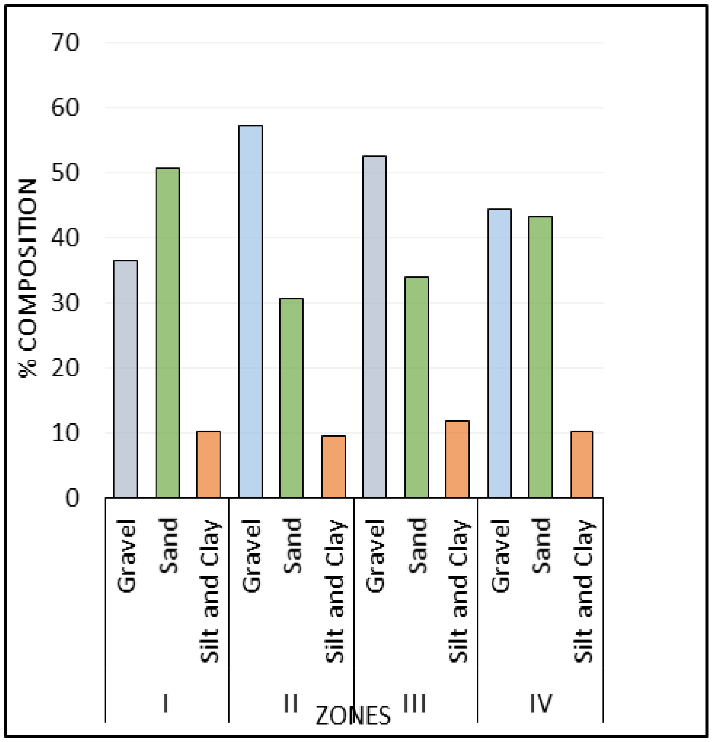
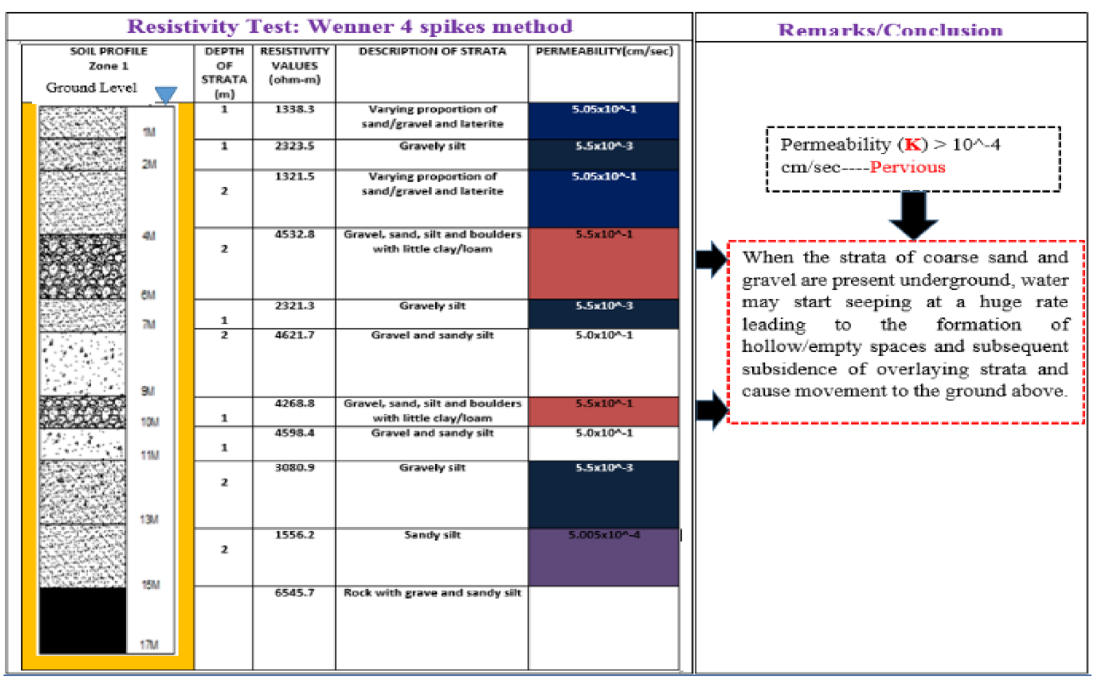


Figure 5: Division of Soil Fraction on the basis of Grain Size

Although the conventional counter measures such as slope drainage, shotcrete techniques, or jet grouting, retaining structures, etc., the realization of permanent solution to this sinking problem was felt necessary. The need to conduct deeper sub-surface exploration was necessary. Sub-surface exploration using Wenners 4 Spike Method, a resistivity test, was conducted in all the zones.



Resistivity Test: Wenner 4 spikes method

DEPTH OF STRATA (m)	RESISTIVITY VALUES (ohm-m)	DESCRIPTION OF STRATA	PERMEABILITY(cm/sec)
1	1338.3	Varying proportion of sand/gravel and laterite	5.05x10^-1
1	2323.5	Gravely silt	5.5x10^-3
2	1321.5	Varying proportion of sand/gravel and laterite	5.05x10^-1
2	4532.8	Gravel, sand, silt and boulders with little clay/loam	5.5x10^-1
1	2321.3	Gravely silt	5.5x10^-3
2	4621.7	Gravel and sandy silt	5.0x10^-1
1	4268.8	Gravel, sand, silt and boulders with little clay/loam	5.5x10^-1
1	4598.4	Gravel and sandy silt	5.0x10^-1
2	3080.9	Gravely silt	5.5x10^-3
2	1556.2	Sandy silt	5.005x10^-4
	6545.7	Rock with grave and sandy silt	

Figure 6: Soil Profiling from Resistivity test for Zone I

The resistivity value obtained in each layers were evaluated to correlate to the sub-surface soil characteristics. The soil profile indicated high content of gravel, coarse sand and silt of high permeability value which contributed to sinking of the pavement in all the critical zones.

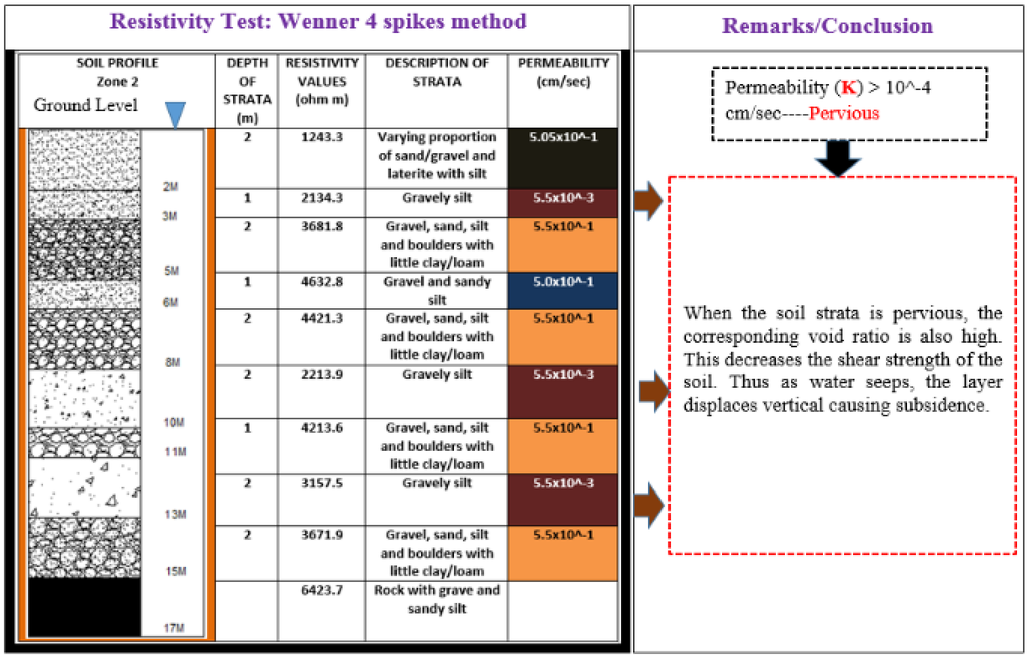


Figure 7: Soil Profiling from Resistivity test for Zone II

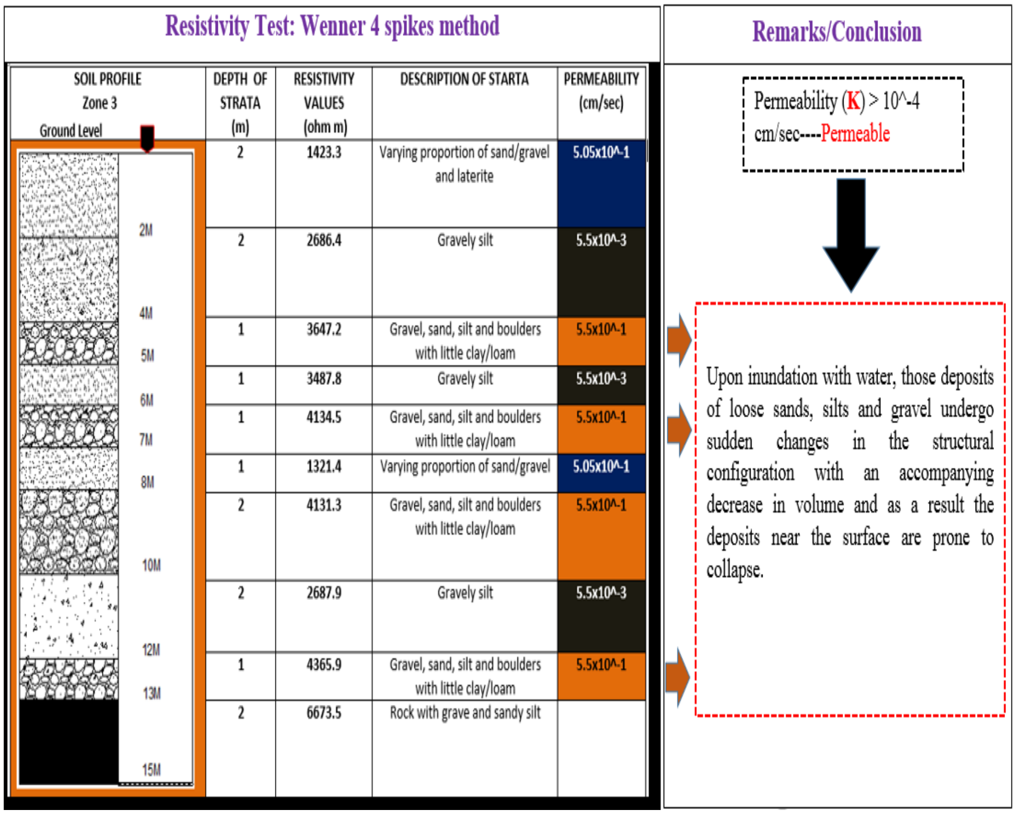


Figure 8: Soil Profiling from Resistivity test for Zone III

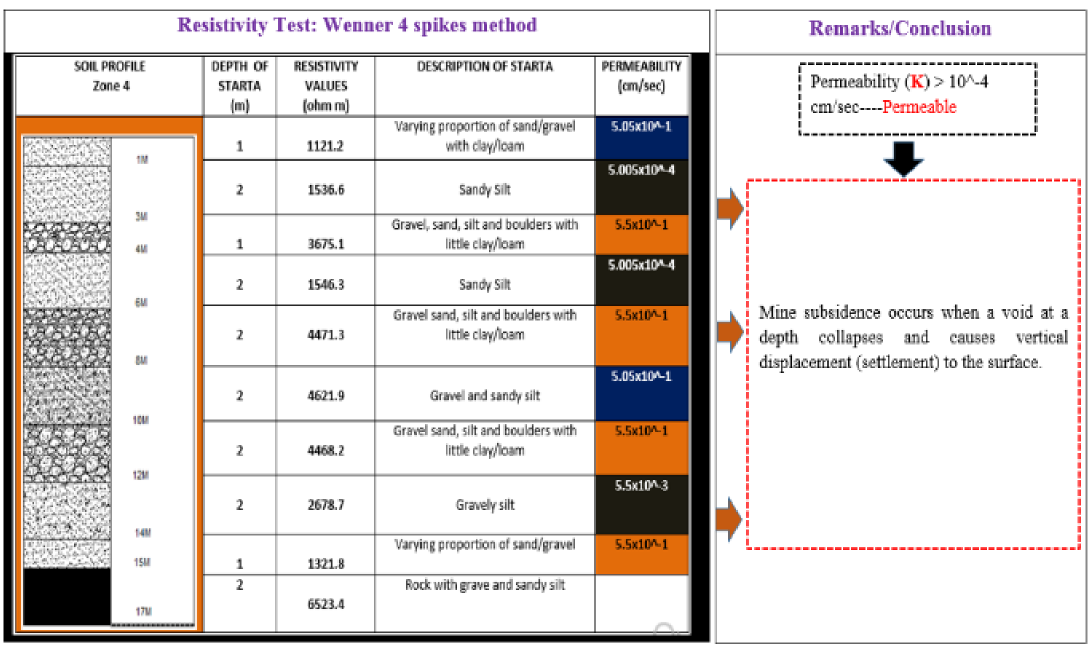


Figure 9: Soil Profiling from Resistivity test for Zone IV

The permeability value ranged from 5.5 x 10-4 cm/s to 5.05 x 10-1 cm/s. Based on the soil profile and the high permeability value, the alternative solution of pile foundation with rigid pavement are proposed. This solution is realized as permanent solution as it derives load bearing capacity from skin frictional resistance and the end bearing resistance from the rigid stratum approximately 18.0m from the ground surface.

4. Conclusion

The preliminary survey for the assessment of sinking area along the Phuentsholing–Thimphu highway at Sorchen have shown four critical pavement sinking zones. The zones were identified as Zone I, II, III and IV. To understand the profile and extend of the sinking of the pavement, topography survey were carried out in each zones. The outcomes indicated settlement of 0.54m, 0.81m, 0.79m and 0.74m for Zone I, II, III and IV respectively caused due to sinking effect. To assess the site characteristics of the critical sinking zones, the geotechnical site investigation were carried out through physical observation at the site, laboratory test and field exploration in each zones. Based on those tests, the shallow soil profiling and classification were carried out. The predominant type of soil identified in all the zones are coarse grained soil with little fine content.

Sub-surface exploration using Wenners 4 Spike Method, a resistivity test, was conducted in all the zones. The resistivity value obtained in each layers were evaluated to correlate to the sub-surface soil characteristics. The soil profile indicated high content of gravel, coarse sand and silt of high permeability value which contributed to sinking of the pavement in all the critical zones. The permeability value ranged from 5.5 x 10-4 cm/s to 5.05 x 10-1 cm/s. Based on the soil profile and the high permeability value, the alternative solution of pile foundation with rigid pavement are proposed. This solution is realized as permanent solution as it derives load bearing capacity from skin frictional resistance and the end bearing resistance from the rigid stratum approximately 18.0m from the ground surface.

5. Recommendations

The future researchers at Sorchen can use the data obtained (particularly the depth of settlement) as a reliable reference material to calculate the annual settlement rate of the road stretches. Future researchers can also work on collecting the soil samples from the sites where no settlement has taken place for laboratory test and analysis and field test to be conducted for further exploration. It is recommended to adopt Refraction test for subsurface exploration over Electric Resistivity Test for better results. The cause of road settlement in this project has been explained as non-homogeneity of soil stratum for different layers and its permeability. Other factors such as ground water table, seismic activity, and rainfall may be considered for future researchers. The cause of landslide in Sorchen could be carried out through the same methodology adopted in this research considering more factors. Through the soil exploration tests carried out at Sorchen in this project, the principal cause of road settlement identified was the appreciable permeability of the soil strata determined in all the four zones. However there could be other contributing factors causing the settlement which the future researchers can work on. Hence, the need for slope stability analysis in Sorchen is felt necessary.

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To Develop the Correlation between California Bearing Ratio (CBR) and Dynamic Cone Penetration Test (DCPT) for Shale Soil

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Abstract

This research paper aims at developing the correlations between DCP and Soaked CBR, DCP and Unsoaked CBR that best suit the type of soil in Jamshoro. California Bearing Ratio (CBR) test is a commonly used indirect method to evaluate the strength of sub-grade soil and the materials used in sub-base and base course in flexible pavement design works. The problems associated with conventional CBR testing are 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 sub-grade soil strength evaluation device currently available. Therefore, an appreciable alternative is to predict the CBR values from direct soil field tests such as Dynamic Cone Penetration test (DCPT). In the present study, several field and laboratory tests were conducted on soil samples from different locations. From the tests, the Atterberg limits (PI, LL, and PL), Natural Moisture Content, Specific Gravity, Gradation Analysis, Maximum Dry Density (MDD) and Optimum Moisture Content (OMC), Soaked and Unsoaked CBR values and Dynamic Cone Penetration Index (DCPI) values were acquired. Based on these results, correlations have been proposed in the study to predict CBR values for Jamshoro soil from DCPT Results.

Keywords

Dynamic Cone Penetrometer (DCP), Soaked CBR, Unsoaked CBR, Jamshoro soil.

1. Introduction

In civil engineering, it is essential to improve structural safety and economic aspects during the site investigation of sub-grade materials in pavement design works. One of the activities involved is the assessment of sub-grade material strength with different in-situ and laboratory tests such as the Dynamic Cone Penetrometer (DCP) test and the California Bearing Ratio (CBR) test.

California Bearing Ratio (CBR) is a parameter which evaluates the strength of road soils and used as an inherent part of pavement design. This test measures the soil resistance indirectly and involves various phases i.e. sampling, transporting, preparing, compacting, soaking, and penetrating with a plunger of CBR machine. As it consumes a lot of time and cannot be easily determined in the field, civil engineers always face problems in obtaining representative CBR values and repeatability of results is also doubtful. To obtain soaked CBR value of a soil sample, it takes about four days. In addition, conventional laboratory CBR test requires a large number of soil samples which is arduous. This would result in serious amount of delay in the progress of any construction project. In order to overcome these troubles, Dynamic Cone Penetrometer is used. DCP is a multi-advantageous device, being light and portable which offers an attractive means of determining in-situ soil strength at a comparative speed and ease of operation. Its repeatability is considerably higher from CBR. Moreover, it takes a very short time to conduct DCP test and carrying out its analysis. Various correlations have been developed by different researchers from soil samples of their locality. Therefore, predicting CBR value from DCP during the evaluation of pavement performance makes better choice than using expensive and time consuming procedures.

Mainly, this research paper is focused on Jamshoro soil. Jamshoro (Co-ordinates: 25.4169° N, 68.2743° E) is a city and capital of Jamshoro District, Sindh, Pakistan. It is located on the right bank of Indus River, approximately 18 km Northwest of Hyderabad and 150 km Northeast from the provincial Capital (Karachi) of Sindh. The soil in Jamshoro is mostly shale in nature along with limestone mixed in it. Shale soils are difficult to deal with as they created various problems during laboratory tests i.e. it was difficult to transfer the compaction energy to the soil specimen as with the increase in moisture content, the soil exhibited irregular increase in plasticity and stuck to the rammer. During Soaked CBR test, Jamshoro soil showed varying swelling potential. Swelling in most of the soil specimens was high and sometimes, it showed low swelling potential which led to the repetition of CBR test to clear any doubts. Misleading results were obtained when piston tip rested on small stone particle or pebbles in the loading machine.

By taking into account the afore mentioned troubles with CBR test and Jamshoro soil, an attempt has been made in this research to predict the CBR values from DCPT results by establishing a suitable correlation between them.

2. Experimental Work

Six (6) soil samples were collected for this research from different locations of Mehran University of Engineering and Technology (MUET) (Co-ordinates: 25.4084° N, 68.2605° E), Jamshoro, at a depth of 2-2.5 ft from the ground surface. These samples were tested at Geotechnical Engineering laboratory in Civil Engineering Department of MUET for water content, specific gravity, plastic limit, liquid limit, plasticity index, particle size distribution, maximum dry density and optimum moisture content, soaked and unsoaked CBR values. The modified proctor compaction test was used to find MDD and OMC. Each CBR mould was compacted in five layers of soil at predetermined OMC. The compaction effort used to prepare CBR moulds was generated by 56 blows per layer. The diameter of CBR mould was approximately 6 inches.

DCP test was performed on field at each and every sample location. A pit of about 2ft was excavated which was leveled at the surface for accommodation of DCP apparatus. The required depth of penetration was 100mm. From DCPT results, graphs were plotted and Dynamic Cone Penetration Index (DCPI) values were worked out corresponding to 100mm penetration depth.

All the tests were performed according to AASHTO and ASTM specifications. Table.1 shows the summary of laboratory and field 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 (%)	CBR (%)		DCPI (mm/blow) at 100mm penetration depth	AASHTO Soil Group
							Unsoaked	Soaked		
1	55	28	27	78.27	1.85	17.1	41.15	13.72	42.5	A-7-6
2	56	30	26	72.41	1.81	14.37	20.16	12.17	70	A-7-6
3	22	13	9	73.41	2.01	8	112.55	17.07	36	A-4
4	50	29	21	68.48	1.91	10.6	49.8	10.21	47.33	A-7-6
5	34	16	18	75.8	2.05	10.38	33.07	19.51	29.2	A-6
6	57	32	25	82.58	1.77	12.6	42.32	22.65	14	A-7-5

3. Results And Discussions

It should be noted that Sample No.3 i.e. A-4 soil sample was excluded because it was different in nature from other soil samples. Also, Jamshoro type of soil is mostly shale (clayey soil) having group of A-6, A-7-5 or A-7-6 which was our main concern in this research. Development of correlation between CBR (soaked & unsoaked) and DCP is done in the following way.

3.1 Simple Linear Regression Analysis (SLRA):

Using Single Linear Reggration Model (SLRA) we tried to develop reliable correlation.

3.1.1 Empirical Correlation between DCP and Soaked CBR:

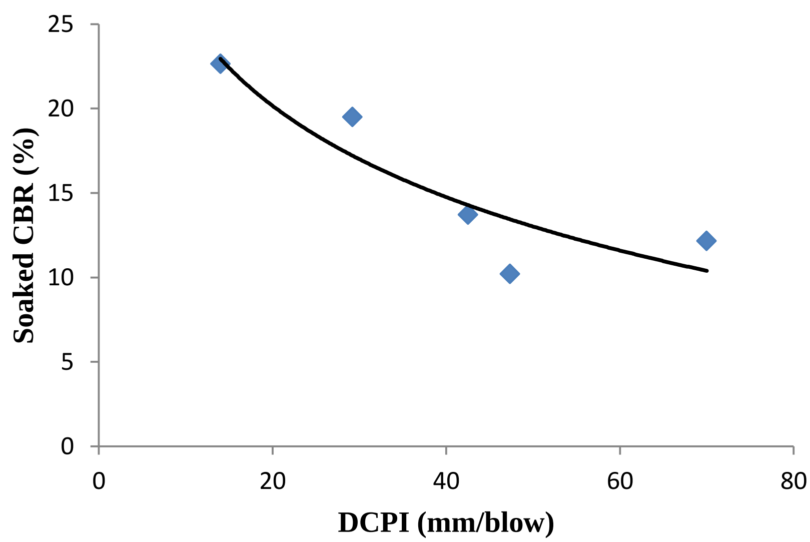

Figure 1; Scatter plot OF DCP Index vs. Soaked CBR (Logarithm)

The good correlation between DCPI and Soaked CBR is logarithmic which is expressed as:

$$\mathbf{SCBR = -7.799\ ln(DCPI) + 43.523}$$

With the reliability of $\mathbf{R^2 = 0.8234}$

Where; SCBR = Soaked California Bearing Ratio value in %.

DCPI = Dynamic Cone Penetration Index in mm/blow at 100mm penetration depth.

3.1.2 Empirical Correlation between DCP and Unsoaked CBR:

No such suitable correlation was found between Un-soaked CBR and DCPI

3.2 Multiple Linear Regression Analysis (MLRA):

3.2.1 Prediction of Soaked CBR from DCPI, Percentage finer and OMC:

Using MLRA, the best correlation obtained was the one which relates DCPI, % Finer and Optimum with Soaked CBR expressed as:

$$\mathbf{SCBR = 0.0929(DCPI) + 1.3224(\%F) - 1.4675(OMC) - 68.8789}$$

With $\mathbf{R^2 = 0.9922}$, **Adj.** $\mathbf{R^2 = 0.9687}$

Where; SCBR = Soaked California Bearing Ratio value in %.

DCPI = Dynamic Cone Penetration Index in mm/blow at 100mm penetration depth.

%F = Percentage finer, OMC = Optimum Moisture Content.

Fig.2 shows the graph between predicted and experimental soaked CBR values(R^2 = 0.9922). The model predictions show a good validation with the experimental CBR values.

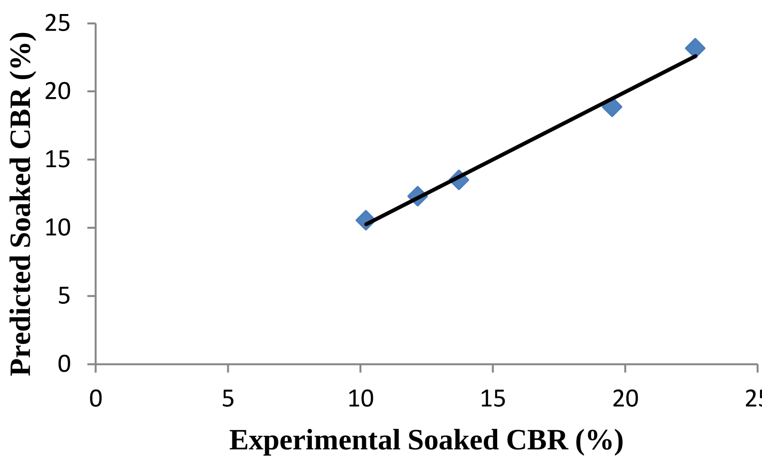

Figure 2; Predicted Soaked CBR vs. Experimental Soaked CBR

3.2.2 Prediction of Unsoaked CBR from DCPI, Percentage finer and OMC:

Using MLRA, the best correlation obtained was the one which relates DCPI, % Finer and Optimum with Unsoaked CBR expressed as:

$$\mathbf{UCBR = 4.9473(OMC) - 4.4081(\%F) - 1.3062(DCPI) + 358.8192}$$

With $\mathbf{R^2 = 0.9132}$, **Adj.** $\mathbf{R^2 = 0.6528}$

Where; UCBR = Unsoaked California Bearing Ratio value in %.

DCPI = Dynamic Cone Penetration Index in mm/blow at 100mm penetration depth.

%F = Percentage finer, OMC = Optimum Moisture Content.

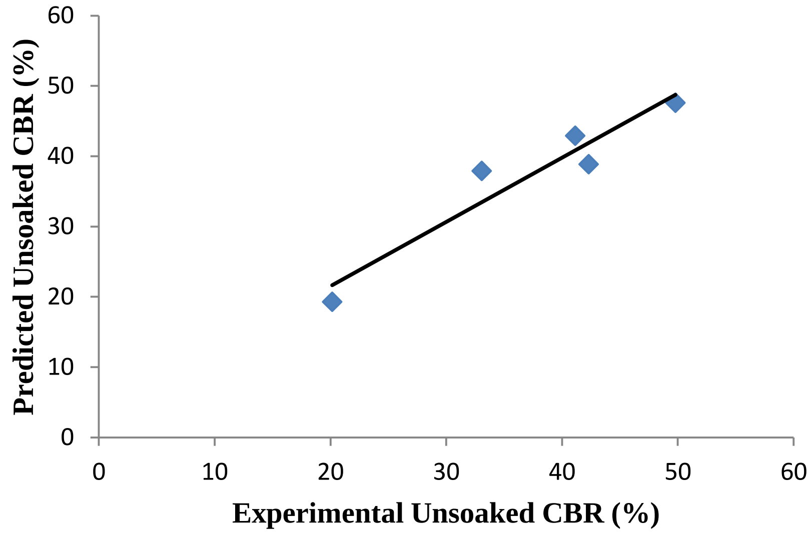

Figure 3; Predicted Unsoaked CBR vs. Experimental Unsoaked CBR

Fig.3 shows the graph between predicted and experimental unsoaked CBR values (R^2 = 0.9132). The model predictions show a good validation with the experimental CBR values.

4. Conclusions

The correlation developed to predict Soaked CBR (SCBR) from DCPI value by SLRA for Jamshoro soil is:

$$\mathbf{SCBR = -7.799\ ln(DCPI) + 43.523}$$
$$\mathbf{(R^2 = 0.8234)}$$

The correlation developed to predict Soaked CBR (SCBR) by MLRA for Jamshoro soil is:

$$\mathbf{SCBR = 0.0929(DCPI) + 1.3224(\%F) - 1.4675(OMC) - 68.8789}$$
$$\text{With } \mathbf{R^2 = 0.9922,\ Adj.\ R^2 = 0.9687}$$

The correlation developed to predict Unsoaked CBR (UCBR) by MLRA for Jamshoro soil is:

$$\mathbf{UCBR = 4.9473(OMC) - 4.4081(\%F) - 1.3062(DCPI) + 358.8192}$$
$$\text{With } \mathbf{R^2 = 0.9132,\ Adj.\ R^2 = 0.6528}$$

From SLRA, the correlation developed for Soaked CBR has the reliability level (R^2) greater than 0.8 (80%), respectively, which is acceptable while the correlation developed for Unsoaked CBR has the reliability level (R^2) unsatisfactory.

From MLRA, the correlations developed for both Soaked and unsoaked are valuable having reliability level (R^2) more than 0.9 (90%) which is quite exceptional. MLRA proves to be better than SLRA.

Looking at the overall working conditions, it can be summoned that DCP test is more effective and efficient than CBR for road sub-grade strength evaluation during soil investigation.

Finally, the results of this research suggest that both the correlations for Soaked and Unsoaked CBR obtained from MLRA can be reliably and extensively used to predict CBR values from Field Dynamic Cone Penetration Test (DCPT) for Jamshoro soil. These correlations will serve as a base for the advancement of this research and its bearing for soils in various other parts of Jamshoro.

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Investigation on the Implementation of Waste Management and Minimization (WMM) Plan for Non-Residential Project in Southern Region of Peninsular Malaysia

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Abstract
Tremendous growth of the construction industry in Malaysia leads to excessive construction waste generation in the country. The unnecessary construction waste generation needs to be minimized using a proper waste management plan. Therefore, this paper investigates the implementation of the Waste Management and Minimization (WMM) plan for a non-residential project in the Southern Region of Peninsular Malaysia. The methodology employed in the study is using site visit approach. The site visit was conducted 32 times for two months' time period. The findings indicate three types of construction waste are applied WMM plan, namely concrete & masonry waste, timber waste and metal waste. These construction wastes are using prevention, reuse and recycle strategy. This paper can be an exemplar for other contractors to implement the similar WMM practices for their construction project. Future research is recommended to investigate the WMM plan in different types of construction projects.

Keywords
Waste Management and Minimization (WMM) Plan; Non-Residential Project; Construction Waste; Southern Region; Peninsular Malaysia.

1. Introduction

Construction industry is essential for economic growth due to its significant impact in creating job opportunity and enhancing living standards. However, rapid development of this industry leads to surplus materials such as generation of construction waste (Saadi et.,al, 2016; Nagapan et.,al, 2013). Globally, the estimation of waste generated in Europe was about 820 million tons; for United States of America was approximately 450 million tons and in India is about 530 million tons (European Commission, 2013; Townsend et al., 2014; Centre for Science and Environment, 2014). In Malaysia, production of construction waste in central and southern region estimated around 29% (Mei and Fujiwara, 2016). However, until now, there is no any total record of construction waste generated for all states in Malaysia. Most of the study in Malaysia collected data from limited construction project sites and no generalization (Begum et al, 2006; Nagapan et al, 2013). However, the generation of construction waste getting higher; therefore each construction project needs to be implemented Waste Management and Minimization (WMM) plan. Waste Management Plan is the basis for successful waste management practices in construction projects (Nagapan et al, 2013). For this study, investigation visits were carried out to identify whether WMM plan were implemented in the selected project.

2. Waste Management and Minimization Plan

Waste Management and Minimization (WMM) Plan is aim to deliver the best material resource efficiency and minimize the amount of waste required to send to landfill (WRAP, 2011). One of the good practices in WMM plan is implementation of the waste hierarchy at construction projects. Figure 1 showed the waste hierarchy approach and the criteria, namely reduce, reuse, recycle, recovery and disposal. By applying the WMM plan in construction projects, it can upsurge efficiency of waste management, reduce generated waste, reduction in material usage, cost savings and sustainable environmental benefits (Gunalan, 2015; WRAP, 2011).

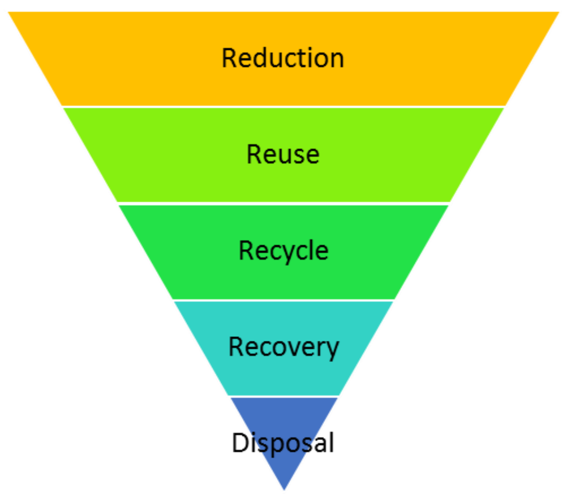

Figure 1: Waste Hierarchy (Gunalan, 2015)

Besides carrying out waste hierarchy approach at site, possibility for minimizing and managing waste should be measured throughout the overall project phases. A necessity for good practice of WMM needs to be introduced at the project. The key aspects for waste management and minimization throughout projects are design solutions, tendering procurement, logistic, construction technology implementation, logistics, material procurement and material packaging. (WRAP, 2011).

3. Study Location

One of the projects located in the Southern Region of Peninsular Malaysia was chosen as a research location for this paper. This project site is selected based on permission granted by the developer and its accessibility. This project is a non-residential project with a contract value of RM 60 million. This project started in the year 2016 and expected to be finished before mid of 2017. For confidentiality purposes, this project labeled as Project A.

Figure 2: Project A

4. Methodology

The method used in this study is by site visit. Site visit technique is a vital step in determining real scenario of ongoing construction projects, identification of main construction waste generated at a site, identification of current waste management and minimization plan applied at site (Sim, 2005). For this study, site visit was conducted four times on a weekly basis for two months' time period. Figure 3 illustrates work flow from accessibility to site until reporting the WMM practices.

Accessibility to site (Seeking Permission) → Conduct site visit → Checking of WMM plan implementation → Report the current WMM practices at site

Figure 3: Site Work Flow

5. Results of the WMM Plan Implementation

Based on the several site visits, the study found that, there are only three (3) types of waste executed WMM plan at Project A, namely concrete & masonry waste, timber waste and metal waste. The detail of the execution is explained as follows:

i. Concrete and Masonry Waste

The study finds out WMM plan that implemented for this type of waste is using reuse strategy. The scattered concrete waste was transported to lorry to be reused as aggregates as a sub base for road construction as shown in Figure 4. Meanwhile, some of the concrete waste was recovered from waste bin and used as topping up for drainage level base level. Besides that, bricks which were not similar size and damaged were reused for building a small security room inside the building. Extra or surplus concrete was used as beam lintels for future brick wall construction. Cement and plaster waste were recovered and utilized as mass concrete for landscape area.

Figure 4: Concrete Waste Transported to Lorry

ii. Timber Waste

The WMM plan execution for timber waste is using reused and recycled approach. For the drainage construction phase, the site used a huge number of plywood as indicates in Figure 5, the respective support was installed to reduce damage and repair to plywood. This timber waste (plywood) was reused as protection for completing pebble wash stones for walkways. In addition, the plywood used for side beam and slab formwork was properly arranged in order to reduce damage to materials when hoisting up for upper floor usages. Finally, the used formwork also recycled and was sold to secondhand formwork supplier.

Figure 5: Timber Waste Stockpiled at Site

iii. Metal Waste

The WMM plan which is used for metal waste in this project is prevention and recycles approach. For avoiding/preventing wastages purposes, reinforcement bar was ordered cut to size 8m length.

This is because to avoid wrong cutting on the site which be end up as waste. This can save reinforcement lapping length when installation for column. For recycling purpose, the extra reinforcement bar was gathered in roll off bin (presented in Figure 6) and sold to scrap metal collector.

Figure 6: Metal Waste Gathered in Roll Off Bin

6. Conclusion

The study confirms that the Project A is implementing Waste Management and Minimization (WMM) plan for three types of waste through prevention, reuse and recycle. The three (3) types of waste are concrete & masonry waste, timber waste and metal waste. Thus, it can be concluded that the practice adopted by the Project A is in line with the sustainable construction approach.

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Assessing Integrated Role of Project Management in relation to Quality Management within Construction Projects of Pakistan Focusing "The Iron Triangle"

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Abstract:
This piece of research work shows the trends and techniques for success criteria of project management and development of life-cycle of a project. Focusing on time, cost & quality The Iron Triangle), over the last 50 years have become indivisibly linked with measuring the success of project management. Research in this area so far has mainly been devoted to identifying causes of time, cost and quality overruns. There is limited research geared at studying factors inhibiting the quality management and ability of practitioners to effectively control their projects. To fill this gap, research has been conducted on construction project within Pakistan, which was followed by face-to-face interviews with experienced construction project stakeholders. A brief interview with all stakeholders has evaluated that the contractor has lack of knowledge of project management & project delivery method skills. From the evaluation of the project several deficiencies are assessed within project. It is suggested that project participants can use the information from this thesis to identify deficiencies in their project-related activities and therefore, take the appropriate action to improve their management practices in future projects.

Keywords:
Project Management, Life Cycle, the Iron-Triangle, Stakeholders, Overruns

1. Introduction:

Project Management in construction project of Wassay Tower expedites some tools and techniques for planning, monitoring and execution while staying within the limits of "The Iron Triangle". This project executes multi-story residential building with specified "Time, cost and quality. For achieving the quality management with in construction projects there is essential need of well-defined aims, objectives and goals. For the successful completion of project following fundamental constituents plays vital role.

- Scope: The scope of the project involves that after developing the project what will be its features & what will be functionality of the project

- Schedule: Schedule indicates the time, which refers that right time should be allocated to right activity so that project could be completed within the dead line
- Resources: Resources are needed for project tasks & they can be people, equipment, budget or any other things required activity for project.
- Quality: Quality means excellence; it is degree of satisfaction that fulfills the requirement of the project.

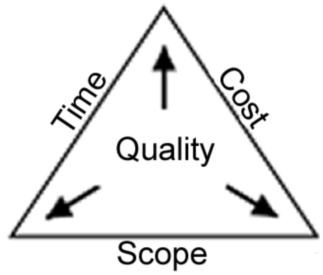

Figure 1. Time, Cost, scope and Quality Triangle

(Pinto & Slevin, 1988) illustrated in their research that the following basic characteristics are vital for the project success:

a) "A definite beginning and end (specified time to completion)
b) A specific, preordained goal or set of goals (performance expectations)
c) A series of complex or interrelated activities
d) A limited budget"

There has been a lot of the research carried for defining a project; in simple words *"A project is an organization of people dedicated to a specific purpose or objective. In construction projects, activities are typically divided into functional areas, which are performed by different disciplines" (e.g. architects, engineers, and contractors).*(Munns & Bjeirmi, 1996) also stated that project contains specific activities associated with time, cost and defined quality. According to (Atkinson, 1999)identified that a project is a series of different activities involving human resources, financial, and organizing aspects to obtain an integrated product. (Kerzner, 2003) stated in his research that the project and its management is the process of *"planning, organizing, controlling &managing resources"* to achieve set targets and successful project.

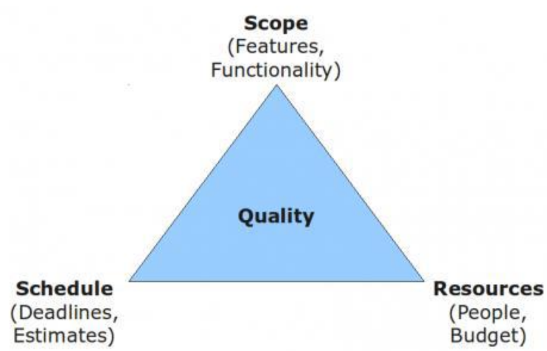

Figure 2. Three Major aspects of the project

2. Integrated Role of Project Management

The integrated role of project management has been widely used and applied in various projects especially in construction project this technique plays vital role to achieve “the iron triangle”. The project management role is one of the most exclusive and most challenging roles to control overall projects and its resources. Project Manager is the one who handles all the duties related to project management. From very beginning to the end project manager is sole responsible for all activates. As the project moves forward and achieves success steps through its life cycle phase’s then project manager should adapt himself to the new challenges and obstacles in order to convert threats into opportunities.

3. Project Management Life-Cycle:

Westland (2006) mentioned that there are five major PM Phases:
1) Initiating
2) Planning
3) Executive
4) Controlling
5) Closing

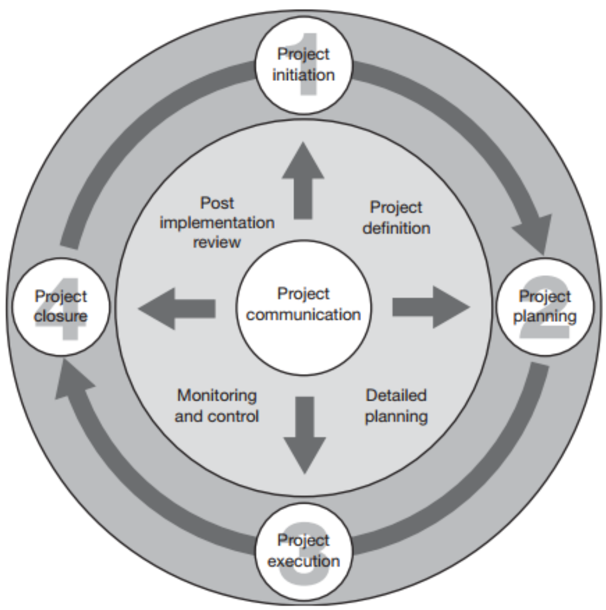

Figure 3. Project Life Cycle four phases Westland (2006)

4. Quality Management:

(Ashokkumar, 2007) illustrated that Quality ISO *“defines quality as the degree of excellence in a competitive sense, such as reliability, serviceability, maintainability or even individual characteristics”*. In general, people think quality as a best product or output of any service that comes up to the expectations. Indeed, all the expectations are lying on the basis of use and its cost. (Ashokkumar, 2007) mentioned that according to (Dale Bester field, 2004), the Quality can be identified as:

$$Q = P / E$$

Where: Q = Quality P = Performance E = Expectation

Whereas, if the Q>1 the client achieves great satisfaction. However, if the contractor demonstrated best performance and delivery of the product or service then it will increase the ration of demand. (Ashokkumar, 2007)has also enlightened the Quality systems *"refer to the organizational structure, process, resource and procedure needed to implement quality management".*(Cooke-Davies, 2002) mentioned that the strategies are the best possible processes and decisions that have been translated through corporate strategic development with projects.

5. Focused approach towards Quality Management (Wassay Tower):

The word quality refers to the excellence, most of the time two objects or materials are usually judged by their qualities. According to International Organization for Standardization (ISO 9000:2000) defines quality as *"the degree to which a set of inherent characteristics fulfills requirements".* Quality in construction industry is the major issue all around the world. In same way it is major challenge for the construction industry of the Pakistan as well. When we are talking about the quality of building project of Wassay Tower thenit looks quite satisfactory. (Dvir et al.,2003)&(Khan, & Khoso 2014) identified that the quality of any construction project is judged by the material used at the construction site, technical persons, engineers, labors & methods of construction work employed in the project. The quality of material is the key source for the quality of overall project. If the material used in any construction project is good enough than its overall quality of the project will be good, in same way if the quality of the material is poor than overall quality of project will be poor. Most frequently used materials at construction of Wassay Tower were Cement, fine aggregate (sand) coarse aggregate (stone / crush), reinforcement (steel), bricks, tiles & water. The quality of these materials which was noticed at project is discussed below:

5.1 Cement:

In construction industry cement is considered as versatile material & it is extensively used as binding material. The type of cement which was used at Wassay Tower project was ordinary Portland cement (O.P.C) of lucky cement company was used, but that cement was 3 months old than manufacturing date. But ISO recommends that cement should be used as fresh as much possible because with the passage of the time strength of cement decreases, decrease in strength as relative to fresh cement is shown in Table 1 below.

Table1. Showing the ISO Standards for Storage of Cement

Storage Period of Cement	3 (Months)	6 (Months)	1 (Year)	5 (Years)
Decrease in strength	20%	30%	40%	50%

When this idea was discussed to engineer Wassay that with storage the strength of cement decreases, he replied that builders always remains in the fear of increasing rate of materials so they try to keep themselves safe from any loss. We keep at least 6 months in advance and that reduces 30% of its strength.

5.2 Fine Aggregate (sand)

Fine aggregate is used as filler material in construction work. It makes the structural elements like beams, columns, slabs etc. impermeable. Fine aggregate which was used on the construction site was well graded, hard, strong, & durable. It was free from silt, Clay, salt& any organic matter that may attack the reinforcement.

5.3 Coarse Aggregate (Stone/Crush):

Strength of any structural element like beams, column, and slabs depends upon the quality of coarse aggregate. The coarse aggregate which was used at the construction site were strong, durable, with round shape & well graded & with angular sharp corners, & non- porous.

5.4 Reinforcement (Steel)

The steel used at the construction of Wassay Tower was AAMRILI mild steel, the specification of which are shown in Table 2. It was used to increase the tensile strength of the structural elements & AAMRILI steel has much flexibility than others. Besides that, usage of steel was done in proper way according to needs & specifications of the project. Steel used at the site was free from rust & corrosion.

Table 2. Showing the Specification of steel used in project

For raft	For columns	For beams	For slab
(1) Main bar dia #6 (2) Distribution bars #6	(1) Main bar #6 (2) Ties #3	(1) Main bar #5 (2) Tie bar #3	(1) Main bars #5 (2) Distribution bars #4

5.5 Bricks

Brick are block of tempered clay molded to suitable shape & size. These bricks were used for project of Wassay Tower for partitioning of wall. According to Engineer Wassay he told that there are four categories of 1st class category which is considered as the best quality brick. Second class category which also a good category brick but its strength is somewhat less than the 1st class, another category is 3rd class brick which cannot be for such project. (Khoso et al., 2014) mentioned that these bricks can be used for temporary construction & another type of brick is over burnt brick which has irregular shape & size but that brick is strongest brick than the 1st category brick. But 1st class category of brick was used over the construction site. this category of bricks has uniform size, smooth surface, sharp edges & uniform color, besides these qualities it has high strength compared to other category bricks. The size of the brick used on the project was 9×6×6.

5.6 Tiles

Tiles are thin slabs which are used for covering the roofs, floors & for drains. Tiles are usually used for cleanliness purpose. But yet tile was not used on the project, but was intended to be using only tile for flooring. According to engineer of Wassay Tower he told that there are two types of tiles:
(1) Standard tile (expensive, a good quality, strong & durable)
(2) Commercial tile (cheap, fair quality, not much strong & durable) a standard tile was used & size of tile brick was 1‘×1‘.

5.7 Quality of Concreting Proportions in Wassay Tower:

Concrete is a composite material which is made with combination of binding material, fine aggregate (sand), Coarse Aggregate (stone / crush) & water forms a concrete. All materials which are used for the preparation of the concrete at the project were of good quality besides these materials water also plays a great role for the preparation of concrete. (Salim Khoso et al.,2016)suggested that it is always recommended that water used in the concrete should be drinkable water but in same water used for the concrete at project was drinkable water& its PH was seven. Different ratios of concrete which were used at construction of project of Wassay Tower were the following

1) Lean concrete at base 1:3:6
2) Raft concrete at base 1:1.5:3
3) Columns concrete ratio 1:2:4
4) Beams concrete ratio 1:2:4
5) Slab concrete ratio 1:2:4
6) Topping concrete ratio 1:2:4

6. Research Analysis:

This Research analysis is based on the case study of an actual project called Wassay Tower. It consists of a secondary analysis of documents and project plans on the building project and for which interviews conducted with several peoples involved in project at which observational study was carried out regarding this project by conducting interviews & visiting construction site.
The key barriers in construction projects of Pakistan are following:

- ✓ Lack of Knowledge and awareness to the stakeholders within the construction projects of the Pakistan
- ✓ Week regulations and control in construction projects
- ✓ Lack of proper training & experience of project management
- ✓ Project fraud & corruption
- ✓ Government Regulations and Socio-Political pressures

Therefore, it is recommended that Pakistan needs aapplied and integrated approach toward the path of success in management of construction project within shorter time and limited budget. In comparison to developed countries, Pakistan construction industry needs a management approach and use of modern tools and techniques to get maximum out with minimum resources utilization. The interviews were mainly based on discussing the key issues of the construction project management like controlling on (Time, Cost & Quality). Following figure provides more information about all the interviews. The Following Graph is a mix of all stakeholders with varying job description but mostly similar kind of projects.

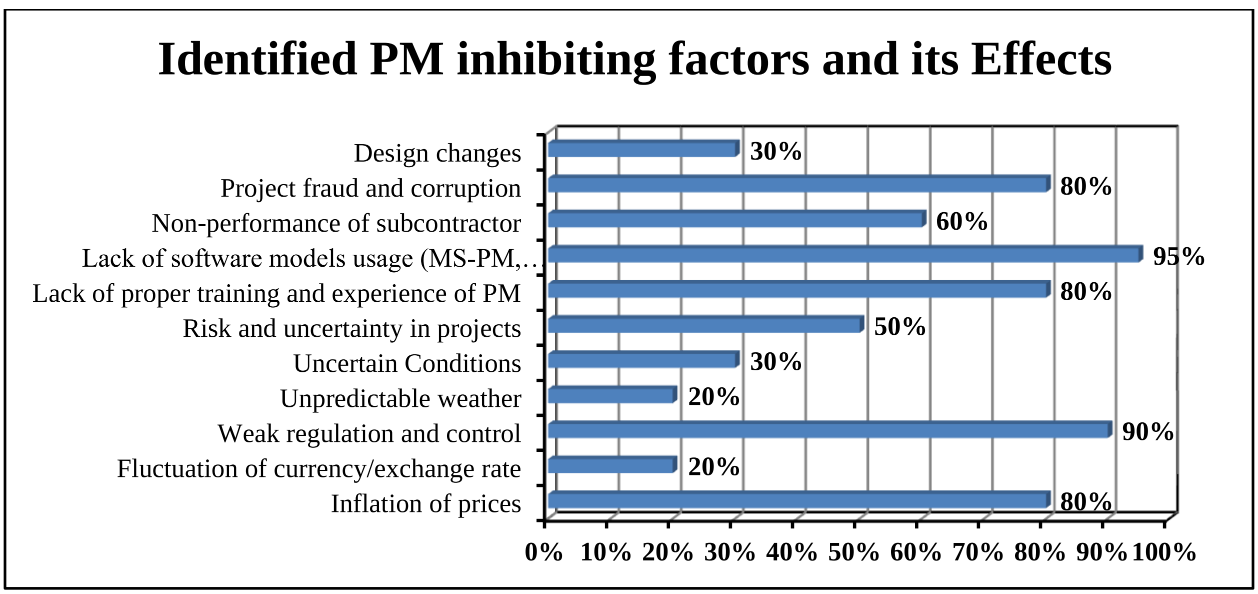

Figure 5. Inhibiting factors and its Effects on Project's "Iron Triangle"

7. Conclusion:

The conclusion of this research is depicts the findings of a building project. This case study is based on the opinions of project team participants, contractor & engineers about the importance of integrated role of project management and relationship between times, cost and quality management. This project has a great time management where some of activates are found before the scheduled time and completed near about 70% within approximate cost of 3 million PKR in construction work that is in control limits. From findings the quality was unsatisfactory within the standard of ISO limitations. Especially the construction projects in Pakistan are lacking the quality management tools and techniques. In Order to achieve Project Quality Management (PQM) Pakistan Construction Industry (PCI) has to set goals and strategies to equip project managers and other team members with best training and up to date knowledge of PQM systems. Research identified that the major problems faced by all the stakeholders are:

- Unskilled and Untrained Project Managers and other team members
- Lack of Knowledge and Training
- Shortages of good quality (ISO certified) material
- Skilled labor
- Mechanical plant
- Government Regulations
- Political influence
- Project fraud & corruption
- Overall control system

If these major problems are addressed, then construction industry will get better endeavors and attract foreign investments with in Pakistan. There are great challenges yet to be faced by the construction industry of Pakistan but if the leader PM is well educated and equipped with the Project Management knowledge, skills and education then good quality can be delivered with the help of “The Iron Triangle”.

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Labor Productivity In The Construction Industry of Pakistan

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Abstract
One of the dominating industries; 'the construction industry' that aids a lot in the economy and GDP growth of Pakistan, is mainly dependent on the workmen/labor force. As any of the construction project has to be completed considering few constraints such as cost, quality and time, so every aspect of project has to be taken care of in construction industry like Pakistan's. Labor productivity is one of the main factors which affects project outcome. Therefore, the aim of this research is to highlight critical factors affecting labor productivity in the building construction sector. After detailed factor identification through literature review, several unstructured interviews are conducted to get the point of view of industry experts. Through the enhanced factor list, the questionnaire was designed and floated amongst the respective stakeholders to rate the significant factors affecting the labor productivity. This paper presents the discussion related to analyzed factors in the context of the Pakistan's construction industry. The results will help the stakeholders in improvement of Labor Productivity in the building construction sector of Pakistan.

Keywords
Significant Factors, Improvement, Labor Productivity, Building Sector, Construction Industry of Pakistan.

1. Introduction

In the construction industry, productivity is the dominating aspect, and enhancing productivity helps to prosper project's value. Numerous research works have been done on the overall productivity concerns of

the project and there are several factors that affect it, out of which one of the usual important declared factor which affects the physical progress of a construction work is the labor productivity (Shashank et al., 2014).
Pakistan's construction industry specifically being a developing one and this much labor intensive; faces the issue of low labor productivity which necessitates the need to tackle labor carefully to get the utmost possible productivity. The building construction sector has achieved a high growth rate; increasing the job opportunities for both skilled and unskilled labor (PBS 2013) thus improving the economic and social development of the area.

As the workers constitute a dominant part of this business and according to Guhathakurta and Yates, 1993, the labor costs will approximately be 30-50% of total cost of the project. The requirement to maintain a competitive personnel has become a challenge to the construction industry, for getting best performances which determines the competitiveness and sustainability of construction companies (Marzuki et al., 2012). Usually the site workers are the most neglected part of the construction industry and not much attention is given to them, even though they are the basic tool of the industry to boost up the company's performance and country's GDP & economic growth (Abdullah et al., 2011).

The labor performance is affected by many factors like time schedule, assigned budget, and required quality. Identification and impact level of factors affecting labor productivity are serious issues which are faced by the project managers during the execution of project (Motwani et al. 1995). Evaluation of factors affecting labor productivity at construction have been done in the last decade across world, yet in Pakistan there is no proper documentation of labor productivity so, a lot more effort is required to be put in, in order to ascertain the true factors which are more specific to our own construction environment. Only by identifying these factors, ranking them and further study of high ranking factors in detail will make this effort justified. Hence this research work aims to address one of the important issue of construction industry in Pakistan.

Several times, in our local construction industry, workers create issues regarding their salaries or wages, improper timings, unavailability of required facilities etc., causing hindrances in the progress of work and ultimately the productivity. In relation to it, another objective of this research besides finding the critical factors affecting labor productivity is to examine from these analyzed factors such aspects which affect the job satisfaction of a laborer. So that the concerned issues be taken care of in order to improve the productivity of labor as an ultimate result.

2. Literature Review

Productivity is an average measure of the production's efficiency or measure of firm's performance. Construction Project of any nature, is dependent mainly on the performance of man and machine over the site. Labor productivity in particular is of great importance and has always been a matter of concern to the project managers on site because it directly is related to the project's cost, time and quality.

A study conducted in Pakistan's construction industry in 2015 related to the factors affecting productivity of labor used the tool of questionnaire and the method of relative importance index. The analysis reveals five critical factors that Drawings and specifications alteration during execution of project, Poor relations between labor and supervisors, Low amount of pay, Lack of Laborer experience and Working seven days per week without taking a holiday (Tahir, 2015).

In the Jordanian construction industry, the research conducted by Bekr, 2016, revealed that the labor productivity is negatively affected by the factors such as material shortage at project site, lack of skilled labor, rework due to construction errors, slow response of consulting staff inspecting the work, poor planning and scheduling, lack of supervisor's experience etc.

An investigation was carried out in Bahrain; with its primary objective of assessing the critical factors that affect the site productivity concerning labor. It was done through a questionnaire survey comprising 37 factors and implied that lack of labor supervision, stringent inspection by the engineer, labor skills, delay in responding to requests for information, lack of incentive scheme, errors and omissions in design drawings and working overtime are the few important factors in the construction industry of Bahrain (Jarkas, 2015).
While a study carried out in the state of Kerala, India on the same topic, mentioned the unavailability of materials as the critical most factor affecting productivity of labor on site. The study underlined several others factors such as improper supervision, improper drawing management, tool and equipment issues, project management incompetency, lack of meetings, poor labor motivation, poor material planning and lack of communication. (Thomas & Sudhakumar, 2013)

One of the research carried out by Soekiman et al., 2011 with this concern revealed 113 variables affecting construction labor productivity. These factors were then grouped according to their characteristics: labor, material, equipment, health and safety, design, execution plan, supervision, leadership & coordination, project factor, quality, working time, consultant, financial, organization and external.

In 2007, Enshassi et al., 2007 classified similar kind of factors into 10 different groups, namely: factors related to supervision, factors associated with the internal workforce, factors related to security, factors associated with work motivation, factors associated with leadership, factors associated with materials and equipment, factors related to quality, external factors, factors associated with time and factors related to project characteristics. While Jang et al., 2011, grouped the identified 25 critical variables into 4 groups, namely work characteristic, work management, worker component and work technique.

The thesis work done by Abo Mostafa, 2003, at post-graduation level in the region of Palestine, gave the overall ranking of factors negatively affecting labor productivity and depicted Misunderstanding between labor and superintendents at 4th, Payment delay at 6^{th}, Working for 7 days of week without taking a holiday at 9^{th}, Misuse of time schedule at 12^{th}, Accidents at 13^{th}, Labor dissatisfaction at 14^{th} rank, Violation of safety precautions at 23^{rd}, Insufficient Lighting at 27^{th} rank respectively.

The Turkish construction industry analysis was done by (Durdyev et al., 2012) on the issue of productivity; and 24 factors were identified. The first important factor identified was Lack of experience of labor. Several others authors such as Durdyev & Mbachu, 2011 and Jarkas & Bitar, have categorized various productivity factors of labor in groups. As under the headings of human, technological, external and management for Kuwait and the groups such as project characteristics, unforeseen events, workforce process, project finance, project management etc., to classify the factors in a better pictorial view for proper management.

Precising the works of Kazaz & Ulubeyli, 2004 and Kazaz et al., 2008, illustrates that the main reason for cost overrun and time overrun is the poor productivity and its improvement enhances the project outcomes, respectively. That’s why the contractors, engineers and researchers have been continuously researching to know factors and reasons affecting the labor productivity.

In the construction companies of Indonesia, several job facets such as reward, fulfillment of higher order needs, relations with coworkers and with supervisors were found to have influence on the workers’ job satisfaction (Marzuki et al., 2012). As stated by Cox et al., 2005, that a contended worker is a productive worker, so the job satisfaction’s concern with the productivity of labor is also of importance here.

3. Methodology

3.1 Measures:

An extensive literature review is carried out initially to find the factors affecting directly or indirectly the Labor Productivity in construction projects. In addition to this, unstructured interviews are conducted from industry experts to enhance the factor list.

3.2 Development of questionnaire and data collection:

The factor list reduced to 45 in number were transformed into the questionnaire to gather the data from different stakeholders such as client, consultant and contractors, especially which were in direct contact with labor such as site engineers, supervisors etc. were targeted, to have a better consent of the productivity of labor in the building projects of Pakistan.

The respondents were asked to encircle the appropriate item according to their experience. Quantitative data analysis of 50 responses of the professionals engaged in construction projects for 45 different factors were analyzed on Likert's scale. "1" described as not significant, whereas "5" extremely significant.

The factors were assessed with Statistical Package for the Social Sciences (SPSS) using frequency and Average Index (AI) method calculated with formula adopted from Hussin, et al. AI is calculated by using the following formula.

$$AI = \frac{\sum(1X1 + 2X2 + 3X3 + 4X4 + 5X5)}{\sum (X1 + X2 + X3 + X4 + X5)}$$

Where;

X1 = Number of respondents for scale 1
X2 = Number of respondents for scale 2
X3 = Number of respondents for scale 3
X4 = Number of respondents for scale 4
X5 = Number of respondents for scale 5

4. Results and Discussions

4.1 Factors affecting Labor Productivity

After the analysis, the mean values of the factors are shown in Table 1.1.

Table 1.1

S.No	Factors Affecting Labor Productivity	Mean	Rank
1	Material shortages in the site stock	4.4000	1
2	Tool and equipment shortages during work	4.1667	2
3	Improper selection of sub-contractors	4.1333	3
4	Improper distribution of work & not assigning right work to the right person	4.1000	4
5	Skill/Expertise level of Labor	4.1000	4
6	Unavailability of skilled & unskilled labor	3.8667	5
7	Unclear instruction to laborer before start of work	3.8333	6
8	Ineffective site planning and scheduling leading to low labor productivity	3.7667	7
9	Poor health of the workers	3.6667	8

10	Payment delay by contractor to technical staff & labor	3.6333	9
11	Low wages	3.6000	10
12	Labor absenteeism	3.5667	11
13	Low quality of raw materials & resulting in rework	3.5667	11
14	Unsuitable material storage location	3.5667	11
15	Drawings and specifications changes during execution resulting in rework	3.5333	12
16	Working in hazardous situations/conditions	3.5333	12
17	High level of work quality requirements resulting slow progress	3.5333	12
18	Lack of adoption in advanced construction methods & equipment	3.5333	12
19	Site congestion & layout complexity, leading to errors and then rework	3.4667	13
20	Unnecessary interference by supervision staff due to poor relations among workers and supervisors	3.4333	14
21	Workers fear while working at heights	3.4333	14
22	Old age of labor	3.4333	14
23	Project design complexity/constructability issues	3.4000	15
24	Working 7 days per week without taking a holiday	3.4000	15
25	Bad weather condition	3.3667	16
26	Inadequate lighting	3.3333	17
27	Lack of cooperation and communication between labor and management team	3.3000	18
28	Unhealthy/polluted working environment	3.2667	19
29	Lack of training & awareness schemes for skilled & unskilled workers	3.2667	19
30	Stress due to domestic/personal issues	3.2667	19
31	Labor strikes	3.2333	20
32	Drug use	3.1000	21
33	On site accidents causing stoppage of work	3.1000	21
34	Poor response to labor grievances	3.1000	21

35	Difficult access to project location	3.0333	22
36	Overcrowding on site	3.0000	23
37	Lack of incentive and reward schemes	2.8667	24
38	Lack of periodic meeting with labor	2.8667	24
39	Lack of motivation of labour	2.8333	25
40	Lack of job security	2.8000	26
41	Improper arrangements for dinning and rest during break	2.8000	26
42	Work in overtime hours	2.8000	26
43	Unavailability of personal protective equipment	2.7667	27
44	Personal clashes between labor	2.7333	28
45	Unavailability of safety engineer on site	2.5667	29

Few of the top factors identified are discussed here in view of the construction building sector of Pakistan.
Materials and equipment shortages at the site are the most significant cases in our local industry leading to the low productivity of labor as due to its unavailability the schedule is disturbed too.

Furthermore, assigning the right work and proper amount of work to the right person is very important; otherwise the lack in productivity of workforce will be observed and may be the quality is also not up to the mark. The selection of the sub-contractors is also one of the main criteria to maintain the standard quality of work keeping in the view the project triangle. If sub-contractors aren't selected well; the labor will also be not working up to the standards and as per the requirements of the project thus again affecting the overall performance of the project because of the labor productivity.

The expertise level or the skill of labor does affect the output level. If the person working is expert at his work, the work accomplished will be productive. It should be noted that while assigning work component to a labor person, proper instructions are really important for better understanding otherwise the worker may not be that much productive.

4.2 Issues regarding Job Satisfaction

The listed factors if examined with the perspective of job satisfaction; it will be evident that there are several issues which seem to be affecting the job satisfaction of a laborer. So, with this regard, the Table 1.2 identifies such factors which in some way directly and indirectly relate to the job satisfaction of an individual. This signifies the importance of job satisfaction in relation to the labor productivity, if such factors properly assessed in relation to the labor productivity can increase/improve the same on the sites where such problems exists.

Table 1.2

S.No	Factors
1	Improper distribution of work & not assigning right work to the right person
2	Unclear instruction to laborer before start of work
3	Payment delay by contractor to technical staff & labor

4	Low wages
5	Working in hazardous situations/conditions
6	Unnecessary interference by supervision staff due to poor relations among workers and supervisors
7	Workers fear while working at heights
8	Working 7 days per week without taking a holiday
9	Unhealthy/polluted working environment
10	Lack of training & awareness schemes for skilled & unskilled workers
11	Stress due to domestic/personal issues
12	On site accidents causing stoppage of work
13	Poor response to labor grievances
14	Lack of incentive and reward schemes
15	Lack of job security
16	Improper arrangements for dinning and rest during break
17	Work in overtime hours
18	Unavailability of personal protective equipment
19	Personal clashes between labor

5. Conclusions and Future Recommendations

In today's world, the construction industry is rated as one of the strategic industry especially in developing countries like Pakistan. Study and knowledge of construction labor productivity are very important because they influence the economics of the industry. Therefore, prior knowledge of labor productivity during construction can save time and money. The study fills a gap in knowledge of factors affecting labor productivity in Pakistan, which can be used by industry practitioners to develop a wider and deeper perspective of the factors influencing the efficiency of workforce of the building construction projects; and provide guidance to construction managers for efficient utilization of the labor force, thereby assisting in materializing a reasonable level of competitiveness and cost effective operation.
The construction projects undertaken in the research were related to the buildings only; highway, water resource and railway projects can be assessed for the mentioned objective of the research. The study should be conducted further in detail to remedy the identified problems, provide measures and suggestions to recover the lost productivity of the workforce and bring it back to the standards.
Further, as mentioned that several identified factors showed a relationship to the job satisfaction, so this aspect can be assessed in relation to the productivity to find the effect of job satisfaction on the labor productivity, ultimately which leads to the improvement of the workforce efficiency on site.

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Factors Affecting Material Procurement in Oil and Gas Sector Construction Projects in Sindh

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Abstract
Oil and gas is one of the very important sectors of this country. Construction projects in oil and gas sector are usually fast-track projects. The availability of materials significantly affects the schedule of fast track projects. Therefore material procurement is very vital part of any project due to its profound effects on the project success. This research study comprises two phases. In the first phase the factors affecting material procurement in construction projects at global level were identified through literature review and their applicability with oil and gas sector construction projects of Sindh was explored through unstructured interviews with experienced professionals. In the second phase, the identified factors were presented in the questionnaire survey to get the importance level. Results were analyzed with SPSS and Average Index (AI) method and critical factors were finalized. This study addresses an important issue of oil and gas sector construction projects in Sindh. The findings of the study will provide the knowledge about the factors affecting material procurement in oil and gas sector construction projects in Sindh. It will help to workout necessary measures for effective planning and management at early stage, so that the projects can be completed as per schedule.

Keywords
Material procurement, Oil and gas sector, Construction management, Sindh, Pakistan

1. Introduction

N.B. Kasim suggests that in the materials management the objective of procurement is the provision of the materials in the agreed time, place, quality and cost (N.B. Kasim et al., 2005). Construction projects require active movement of materials from the suppliers to the production areas (Pheng, LS et al. 2001). Varghese suggested that the routing of materials is one of the main points which affect time and cost (Varghese et al., 1995). According to Williams duration of fast track projects is less than 70% of the duration of traditional ones (Williams, GV 1995). When duration is lessened it impacts on cost of the project.

Oil and gas sector construction projects are mostly at remote locations therefore need better material management especially material procurement to meet the targets on time. Therefore comprehensive understanding of required material, best possible sources, and effective time to start the procurement process to ensure timely delivery and management at site from its arrival till its usage is obligatory to reduce the construction time.

2. Literature Review

Procurement systems have evolved globally with innovations in process of improvement and service delivery (E. Kohilambal et al., 2016). Unfortunately developing countries have not followed the trends. According to a research 87% of the construction participants sampled believe that good procurement is synonymous with successful project performance (CIOB, 2010). Studies have been conducted globally to analyze the factors affecting material procurement and rank factors according to their limitations and specific areas. Different studies like; (Eriksson et al. 2012), (Hashim, MB. 1999), (Rasid et al. 2006), (Tarun et al. 2016), (E. Kohilambal et al. 2016) suggest different factors related to material procurement that can affect project performance. Many studies have analyzed projects and tried to identify factors affecting project performance. Though the factors found are abundant, a lot of the studies point out that procurement related factors have significant effects on construction project performance (E. Kohilambal et al. 2016), (Peter Davis et al. 2006), Mathonsi, M. D et al. 2011), (Bima Abu bakar et al. 2015).

Therefore, this is an area of importance as highlighted above and this study will provide an understanding of the factors affecting material procurement in oil and gas sector construction projects of Sindh. Material Procurement is the act of finding, acquiring and buying material from an external source. According to (PMBOK, 2008), Project procurement management processes include Plan procurements, Conduct procurements, Administer procurements, Close procurements.

3. Methodology

The factors affecting material procurement in construction projects at global level were identified in four criteria; Plan, Conduct, Administer & Close through literature review. The relevance and applicability of the above factors with oil and gas sector construction projects of Sindh was explored through unstructured interviews with experienced professionals and 62 factors were filtered as relevant & applicable. The identified factors were presented in the questionnaire survey to get the importance level. Likert scale was used in the questionnaire as; 1 = Not Significant, 2 = Slightly Significant, 3 = Moderately Significant, 4 = Very Significant, 5 = Extremely Significant. The importance level was obtained using SPSS statistical software package and Average Index (AI) method (Abd. Majid, 1997),

$$\text{Average Index} = \left(\frac{\sum_{i=1}^{5} aiXi}{5\sum_{i=1}^{5} Xi} \right) \text{ for five scale rating}$$

Where,

ai = Constant expressing weight for i,
Xi = Variable expressing for frequency of response for;
i = 1, 2, 3, 4, 5 are shown as follows:
X1 = No. of respondents for scale 1
X2 = No. of respondents for scale 2
X3 = No. of respondents for scale 3
X4 = No. of respondents for scale 4
X5 = No. of respondents for scale 5

Table 1: The Evaluation and level of importance for average index method analysis

Level of Importance	Average Index
Least Important	$1.00 \leq$ Mean Score < 1.50
Less Important	$1.50 \leq$ Mean Score < 2.50
Important	$2.50 \leq$ Mean Score < 3.50
Very Important	$3.50 \leq$ Mean Score < 4.50
Most Important	$4.50 \leq$ Mean Score < 5.00

Reliability was checked through Cronbach's alpha using SPSS and found reliable with the value of Cronbach's alpha as 0.96. Cronbach's alpha value ranges from 0 to 1, with 0.70 being considered the minimum desired value (Pallant, 2005).

Table 2: Average Index Value and Ranking of Factors

No.	Factors	Mean	Rank
Plan			
1	Procurement without planning	4.36	1
2	Schedule unavailability before starting the project	3.88	2
3	Incomplete drawings / data	3.74	3
4	Flexibility for changes	3.56	4
5	Insufficient time to work out a good structure	3.54	5
6	Revision of design	3.54	5
7	Fast track projects / Speed	3.26	7
8	Inappropriate type of procurement system	3.26	7
9	Scarcity of materials in market	3.24	9
10	Undefined scope	3.24	9
11	Demanding quality management plan	3.18	11
12	Time certainty or constrains of client	3.18	11
13	Inaccurate cost estimation	3.18	11
14	Client's specific requirement	3.10	14
15	Inadequate identification and representation of requirements during the development process	3.08	15

16	Difference between plans & specifications / Supplier gets wrong data of requirements	3.06	16
17	Unstructured approach	2.98	17
18	Inexperienced clients	2.90	18
19	Project complexity	2.78	19
20	Temporary demand for technical staff	2.60	20
Conduct			
1	Bad law & order situation	3.80	1
2	Unavailability of delivery time schedule	3.70	2
3	Poor accessibility of material to the site	3.62	3
4	Incompetent material supplier selected	3.54	4
5	Materials transportation extra cost due to distant / difficult site location	3.54	4
6	Local issues	3.52	6
7	Delay in issue of purchase order	3.22	7
8	Late or incorrect submittals of suppliers	3.22	7
9	Unrealistic delivery dates	3.20	9
10	Delay in approval	3.18	10
11	Incomplete proposals / Required documents not provided by the suppliers	3.18	10
12	Price competition / Price reduction to match competitor's price	3.00	12
13	Political issue/Transport strike	3.00	12
14	Time spent in investigating non-qualified suppliers	2.92	14
15	Risk allocation / avoidance	2.84	15
16	Weather problems	2.62	16
17	Over ordering of construction materials	2.48	17
18	Socio cultural unsuitability	2.46	18
Administer			
1	Communication gap / Lack of communication between parties	3.52	1
2	Changing sequences in construction movement	3.52	1
3	Re handling of materials	3.50	3
4	Lesser use of Information Technology	3.50	3
5	Staff incompetency	3.50	3
6	Compliance with safety procedure / Demanding safety plan	3.16	6
7	Late deliveries	3.16	6
8	Improper tracking and reporting system of construction materials	3.16	6
9	Delay in work permissions	3.10	9
10	Poor organization Structure	3.02	10
11	Incomplete or ineffective meetings / Issues not resolved in meetings	3.00	11

12	Loss of material / wastage	2.92	12
13	Disputes on site	2.84	13
14	Material theft cases at site	2.68	14
15	Material storage space shortage	2.66	15
16	Limited working hours	2.66	15
17	Site entrance make delivery of materials difficult	2.62	17
Close			
1	Too many variations / Variation between contract sum and final account	3.60	1
2	Inappropriate Payment method of project	3.58	2
3	Additional work / change in quantities of work	3.56	3
4	Lack of conformance to requirements / non-compliance to the specification	3.54	4
5	Complaints of other parties to the contract	3.16	5
6	Disputes between parties to the contract / arbitration	2.76	6
7	Prompt honouring of payment certificates	2.60	7

4. Results and Discussion

A total of 70 questionnaires were distributed in professionals experienced in oil and gas sector construction projects in Sindh. Questionnaire received were 50 and all were filled complete and valid for analysis.

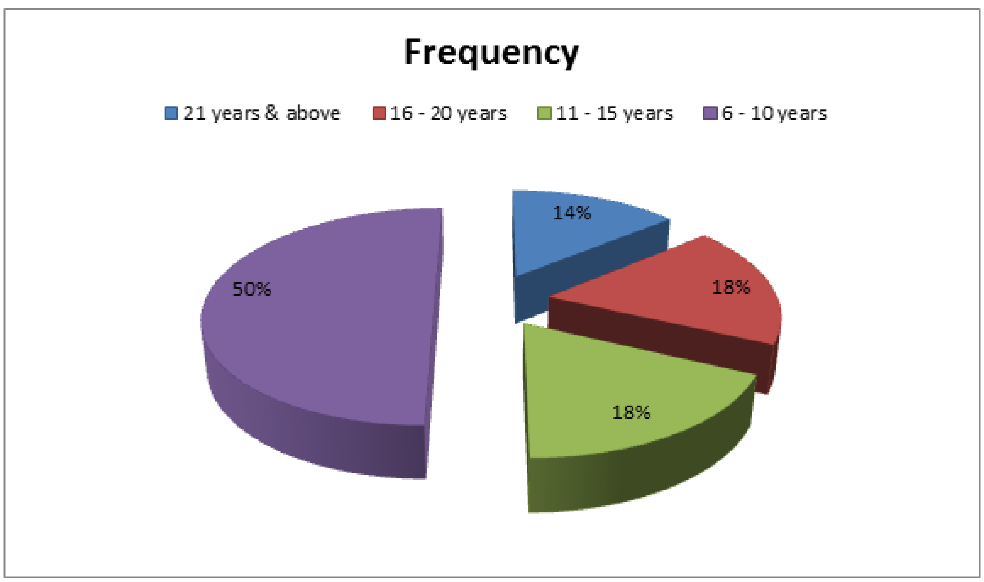

Figure 1: Work experience of the respondents

Top factors were finalized as critical factors affecting material procurement in oil and gas sector construction projects in Sindh using 3.5 as cut-off value from table 1. Top factors in criteria plan, conduct, administer and close are as in Table 3.

Table 3: Top factors

No.	Factors	Mean	Rank
Plan			
1	Procurement without planning	4.36	1
2	Schedule unavailability before starting the project	3.88	2
3	Incomplete drawings / data	3.74	3
4	Flexibility for changes	3.56	4
5	Insufficient time to work out a good structure	3.54	5
6	Revision of design	3.54	5
Conduct			
1	Bad law & order situation	3.80	1
2	Unavailability of delivery time schedule	3.70	2
3	Poor accessibility of material to the site	3.62	3
4	Incompetent material supplier selected	3.54	4
5	Materials transportation extra cost due to distant / difficult site location	3.54	4
6	Local issues	3.52	6
Administer			
1	Communication gap / Lack of communication between parties	3.52	1
2	Changing sequences in construction movement	3.52	1
3	Re handling of materials	3.50	3
4	Lesser use of Information Technology	3.50	3
5	Staff incompetency	3.50	3
Close			
1	Too many variations / Variation between contract sum and final account	3.60	1
2	Inappropriate Payment method of project	3.58	2
3	Additional work / change in quantities of work	3.56	3
4	Lack of conformance to requirements / non-compliance to the specification	3.54	4

The results show that, “Procurement without planning, Bad law & order situation, Communication gap / Lack of communication between parties, Changing sequences in construction movement and Too many variations / Variation between contract sum and final account” are the top most factors affecting material procurement with average index 4.36, 3.80, 3.52, 3.52 & 3.6 respectively.

The findings of this study indicate that proper planning is very essential for material procurement. The administering by qualified and experienced professionals will help in proper planning of procurement process. Client should keep the provision of site security in contract document/conditions and maximum use of day light should be ensured. Communication between all stakeholders should be improved. Viability of planning and scheduling should be ensured to minimize the changes in sequences in construction movement and to avoid variations.

5. Conclusions

Material procurement is very vital part of any project due to its profound effects on the project success. Therefore proper understanding regarding the factors affecting material procurement is necessary for effective planning and management. This study highlights the top factors which affect material procurement in oil and gas sector construction projects of Sindh in four criteria; Plan: Procurement without planning, Schedule unavailability before starting the project. Conduct: Bad law & order situation, Unavailability of delivery time schedule. Administer: Communication gap / Lack of communication between parties, changing sequences in construction movement. Close: Too many variations / Variation between contract sum and final account, Inappropriate Payment method of project. In context of the findings it is necessary to take appropriate measures for ensuring proper procurement of materials in oil and gas sector construction projects in Sindh.

6. Suggestions

Based on the findings, it is suggested that procurement process should be administered by qualified and experienced professionals and viable schedules should be worked out. Changes are inevitable in construction projects therefore appropriate change management plan should be implemented. Further in depth analysis should be carried out based on case studies.

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