1st Conference on Sustainability in Civil Engineering (CSCE'19)

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FOREWORD

We would like to welcome you all to the 1st Conference on Sustainability in Civil Engineering (CSCE'19) held in Department of Civil Engineering, Capital University of Science and Technology, Islamabad, Pakistan. The main focus of CSCE'19 is to highlight sustainability related to the field of civil engineering. It is a platform for civil engineers from academia and industry to share their experiences and different research findings in relevant specializations. Th format of this conference is to have several parallel sessions of different specialties, where we (the researchers and engineers) can interact and improve our understanding of sustainability in the field of civil engineering.

We are lucky to have six wonderful and renowned keynote speakers for our opening edition of CSCE. This year, we have received 132 manuscripts for our conference from China, Pakistan, Saudi Arabia, Germany, Italy and UK. After the screening and review process, there are 59 papers to be presented in Conference. All papers under gone double-blind review process. The review committee has comprised of 35 PhDs serving in industry and academia of Australia, Malaysia, Hong Kong, China, Pakistan, Saudi Arabia, UAE, UK and Chili.

With this opportunity, we would like to express many thanks to everyone, especially all the faculty and staff at the Capital University of Science and Technology for their great support and participation. We are also grateful to all the reviewers and keynote speakers who have dedicated their time to share their expertise and experience in this conference.

We give our greatest appreciation to all the participants of CSCE'19, as authors, presenters and audience, without whom this conference will have no positive interactive atmosphere. Last but not least, the greatest honor is given to our organizing committee whose hard work has made this day a success.

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Ex-Vice Chancellor - University of Engineering & Technology, Taxila - Pakistan
Talk Title: Time Management for Engineers

Dr. Attaullah Shah
Vice Chancellor, KaraKoram International University Gilgit - Pakistan
Talk Title: Use of Supplementary Cementitious Material (SCM in Producing Suitable Concrete

Dr. Qazi Umar Farooq
Head of Civil Engineering Department, Islamic University, Madinah - Saudia Arabia
Talk Title: A Dry Future: Prospective Water Scarcity in Islamabad Region and Its Probable Solution

Dr. Khurrum Iqbal
C.E.O. SHANON & Consultant, Islamabad – Pakistan
Talk Title: Sustainability through Materials and Design using Digital Concepts

Dr. Hassan Abbas
Head of Civil Engineering Department, SCET, Wah Cantt – Pakistan
Talk Title: Role of Civil Engineers in Sustainable Water Economy

Dr. Zia U. R. Hashmi
Head: Water Resources, GCISC, Islamabad - Pakistan
Talk Title: Civil Engineering in the Age of Climate Emergency

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Concrete Materials

Paper ID: 101

Strength Properties of Multi-Scale Hybrid Fiber Reinforced Concrete

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Abstract

Recently, hybrid fiber reinforced concrete (HFRC) has gained popularity for its superior mechanical properties. The fiber hybridization in HFRC means the addition of two or more than two fibers in a suitable way to take full benefits from each fiber. The growth of cracks in concrete is multi-scale process from micro to macro scale. Also, the restriction of cracks with one dimension and length of fibers is limited at their particular scale and have no or little effects at other scales. Therefore, it is logical to combine various types and sizes of fibers in concrete for achieving optimized strength properties. In this study, the compressive and flexural strength of concrete with incorporation of calcium carbonate whisker, basalt fiber and steel fiber are evaluated. The mix design ratio of PC and HFRC is 1:2:1.5 (cement: sand: aggregate) with water cement ratio of 0.4. The HFRC, HFRC1, HFRC2 and HFRC3 were prepared with 5% steel fiber and 5% calcium carbonate whisker having basalt fibers content of 0%, 2%, 4% and 6%, respectively. The compressive and flexural strength tests are performed in accordance with the relevant ASTM standard. It is revealed that the compressive and flexural strength of HFRC are improved by 14% and 46%, respectively when compared with that of plain concrete. It is recommended to optimize the length and content of basalt fiber in hybrid fiber reinforced concrete.

Keywords: Multi-scale fibers, steel fiber, basalt fiber, calcium carbonate whisker, strength.

1. INTRODUCTION:

The concrete brittleness and poor crack resistance can be controlled up to some extent by reinforcement of randomly distributed fibers. The cracks from micro- to macro level can be arrested by use of fibers reinforcement. The fibers help to resist the initiation and crack growth from micro- to macro level and provide bridging effect which ultimately enhances the strength and toughness (Banthia and Sappakittipakorn 2007). The single type of fiber in concrete as a reinforcement can only be effective up to limited extent. Nearly 45 years ago, Walton and Majumdar (1975) studied the use of combining organic and inorganic fibers to achieve higher toughness and strength. After that different type of hybrid fiber reinforced composites were developed using fiber hybridization. It has been observed that the performance of multiscale hybrid fibers reinforced concrete mixtures is superior to that of concrete mixtures reinforced with single-type of fibers due to positive interaction between them and this phenomenon is commonly known as “fiber synergy”. The fibers in hybrid fiber reinforced concrete (HFRC) can be classified by their geometrical size i.e. micro and macro fibers (Sorelli et al. 2015). The HFRC having two or more types of fibers has been studied and shows higher compressive strength, tensile strength and energy dissipation capacity (Li et al. 2017). Also, the strength can be enhanced by using optimized content of different types of fibers.

Sivakumar and Santhanam (2007) combined the different hybrid fiber combination, i.e. steel-glass, steel-polyester and steel-polypropylene fibers to study the mechanical properties of HFRC. It was reported that addition of steel fiber improves the energy absorption mechanism, i.e. bridging effect while the glass, polyester and polypropylene fibers results in delaying the development of micro-cracks. The reason for improved mechanical properties was due to increased hybrid fibers which provide the bridging effect. Steel fibers can bridge macro cracks and restrict the crack propagation at large scale ultimately enhances the mechanical properties of concrete. The basalt fibers restrict the formation of cracks at meso level. Meanwhile, $CaCO_3$ whiskers can bridge micro cracks and prevent further crack propagation at micro scale (Cao et al. 2018). Yoo et al. (2017) reported that the restriction of cracks with one dimension and length of fibers is limited at their particular scale, but have no or little effects at other scales. Therefore, it is necessary to combine different types of fiber and still the research is needed form macro-scale to micro scale hybrid fibers at multi-level cracking. In this study, the strength parameters of HFRC with steel fibers and calcium carbonate whiskers having different contents of basalt fibers are investigated.

2.EXPERIMENTAL PROCEDURES:

2.1 Materials:

The raw materials include cement, fine aggregate, coarse aggregate, super plasticizer, calcium carbonate whisker, basalt fibers (BF) and steel fibers. Table 1 shows the chemical composition of calcium carbonate whisker. The steel, basalt fibers and $CaCO_3$ whisker are shown in Figure 1. The properties of different types of fibers are presented in Table 2.

Table 1: Chemical composition of $CaCO_3$ whisker (wt.%)

Chemical Composition	**CaO**	**SiO_2**	**Al_2O_3**	**Fe_2O_3**	**CO_2**	**MgO**	**SO_3**
$CaCO_3$ Whisker	54.93	0.29	0.11	0.07	42.07	2.14	0.31

a.

b.

c.

Figure 1: Raw materials, a. $CaCO_3$ whiskers, b. basalt fibers, and c. steel fibers

Table 2: Properties of different types of fibers

Raw Ingredients	Size		Tensile strength
	Length	Diameter	
$CaCO_3$ whisker	20–30μm	0.5–2μm	3000-6000 MPa
Basalt fiber	12 mm	7~15 μm	3000-48000 MPa
Steel fiber	35 +10% mm	0.55 +10% mm	1345 +15% MPa

2.2 Mixing Design:

The mix design ratio of plain concrete (PC) and HFRC is 1:2:1.5 (cement: sand: aggregate) with a water cement ratio of 0.4. The super plasticizer content of 1%, by cement mass, is added to all HFRCs. The mix design ratios of all concrete mixes are shown in Table 3. The mix design ratio is selected from the previous study (Khan et al. 2018). A layer procedure for the mixing of fiber reinforced concrete was adopted for the HFRC mix. Khan and Ali (2016) and Khan and Ali (2018) also reported this method for uniform mixing and to avoid balling effect.

Table 3: Mix design ratio of all concrete mixes

Mix Type	Steel fiber	$CaCO_3$ whisker	Basalt fiber	Super plasticizer
PC	-	-	-	-
HFRC	5%	5%	0%	1%
HFRC1	5%	5%	2%	1%
HFRC2	5%	5%	4%	1%
HFRC3	5%	5%	6%	1%

Note: The mix design ratio was 1:2:1.5:0.4 (cement: sand: aggregate: water).
All percentages are by cement mass.

2.3 Test Specimens:

After uniform mix, three cylinders of size 100 mm diameter and 200 mm height and three beam specimens of size 100 mm width, 100 mm depth and 400 mm length were cast from each batch. The fresh concrete mix was poured into the plastic moulds and then compaction was performed on vibrating table. After 24 hours, the cylinders and beams were demoulded and kept for 28 days in to the curing room. The ASTM standard C 192 was followed for making and preparation of specimens.

2.4 Testing:

The compressive strength (CS) and flexural strength (FS) tests were conducted in accordance with the ASTM standard C39 and C1609, respectively.

3. RESULTS:

3.1 Compressive strength:

Figure 2 shows the CS of PC and all HFRCs with standard deviation values of up to 10%. The CS of HFRC is increased to 22.4 MPa and then reduces up to 20 MPa at 6% BF content. For HFRC, the CS is enhanced with incorporation of 2% BF content and then reduced with increase in BF content to 6%. The reason for enhanced CS is the filler effect of calcium carbonate whisker and also the crack bridging effect of basalt and steel fibers. Also, the addition of higher content of fibers results in creation of air voids which ultimately reduces strength. The CS of HFRC is increased up to 14% than that of PC. Kizilkanat et al. (2015) also reported the increase CS with addition of basalt fiber. The calcium carbonate whisker improved the CS of cement based composites (Cao et al. 2014). The incorporation of steel fibers results in enhanced CS of concrete and is also reported by Song and Hwang (2004). In comparison with PC, the CS of all HFRCs is higher. The increasing trend in CS is observed up to the HFRC1.

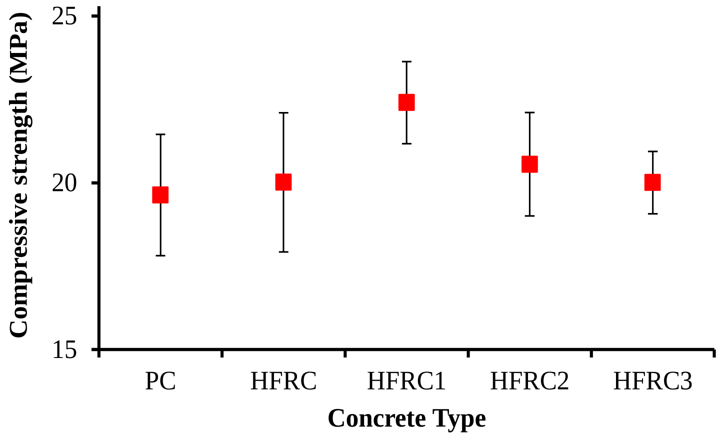


Figure 2: Compressive strength of all concrete types

3.2 Flexural strength:

The flexural strength with standard deviation values of PC and all HFRCs are shown in Figure 2. The FS is improved up to 46% with incorporation of up to 6% basalt fiber content. The multi-scale hybrid fibers crack arresting mechanism results in improved FS.

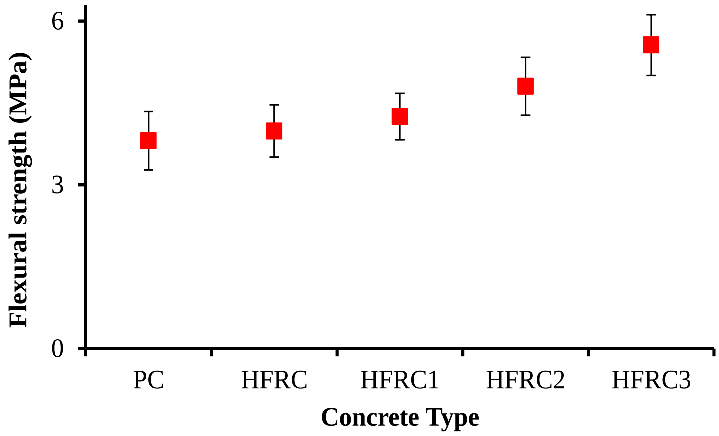
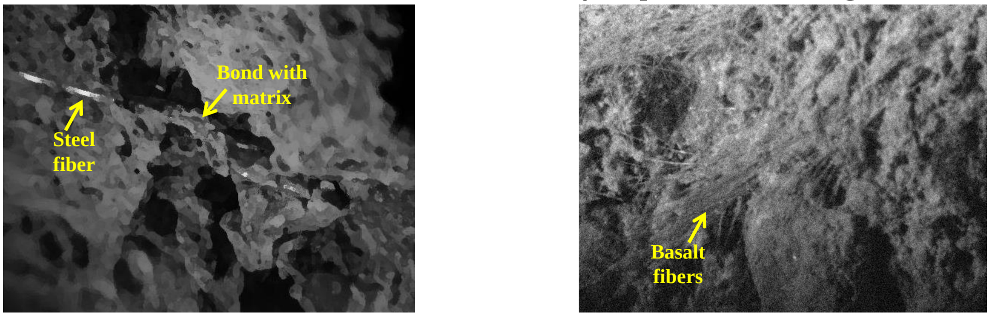


Figure 3: Flexural strength of all concrete types

The steel fibers, BF and calcium carbonate whiskers offered crack resistance at macro-, meso- and micro-level, respectively. Song and Hwang (2004) stated that the incorporation of steel fibers improves the flexural strength. The improvement in FS of concrete with inclusion of BF is also observed by Kizilkanat et al. (2015). The FS of all HFRCs are higher as compared to that of PC. It may be noted that as the content of basalt fiber increases the FS of HFRC increases. However, the HFRC3 showed the highest FS than that of PC and all other HFRCs. The standard deviation values of all concrete types are up to 14%.

4. DISCUSSION:

The fibers are surrounded by adhesive contact of paste which improves the strength of concrete from stress transfer mechanism between matrix and reinforced fibers as shown in Figure 4. Moreover, a part of tensile stresses are taken by fibers generated due to applied loading, thus resist crack growth and help to retain compact microstructure. Also, the applied load is transferred to the fibers cement-matrix which also keeps the fibers together. The competence of a HFRC is dependent on the fiber-matrix interface and the capability to transfer stress from the matrix to the fiber. The tensile cracking of concrete is controlled and delayed through discontinuous randomly distributed multi-scale fibers throughout the cement paste. A slow controlled crack growth of the inherent unstable tensile cracks is due to the incorporation of multi-scale fibers during crack propagation at different levels. This slow crack growth property of fibers delays the initiation of shears and flexural cracks and ultimately improves the strength.



a. b.

Figure 4: Fiber bridging during crack propagation, a. steel fiber, b. basalt fiber

5. CONCLUSIONS:

Following conclusions are made:

- The compressive strength of hybrid fiber reinforced concrete having 2% basalt fiber content is enhanced up to 6%, when compared with that of plain concrete.
- Compared to plain concrete, an increment of 46% is observed in flexural strength of hybrid fiber reinforced concrete with 6% basalt fiber content.
- The positive synergy of hybrid fibers can be observed from improved strength properties of concrete.
- The increased strength shows that the multi-scale hybrid fiber can resist the cracking and control the initiation of cracks at multi-level.

Based on above results, the multi-scale hybrid fibers showed the satisfactory performance. Therefore, further studies should be carried out on optimization of basalt fiber length and content in HFRC.

ACKNOWLEDGEMENTS:

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Paper ID: 102

Incorporation of Bagasse Ash and Stone Dust in Cement Concrete

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Abstract

Nowadays, one of the main concerns of the researchers, is to control the increasing rate of pollution. Several studies are conducted to overcome the burden of environmental pollutants. The Sugarcane bagasse ash (SBA) being pollution needs proper disposal. In addition to SBA, the stone dust (SD) being remaining of the stone processing plants, also requires proper disposal. SBA due to cementitious nature and SD being inert nature can be used as a partial substitute to the cement and sand, respectively. Hence, the suitability of the partial replacement of the cement with SBA and sand with SD, needs to be explored. In this pilot study, the slump, compressive strength (CS) and splitting tensile strength (TS) of the normal strength concrete for partial replacement of cement with SBA and of sand with SD are examined. 9% of cement weight is replaced with the same amount of SBA. And 40% and 50% of sand weight is replaced with equal weight of the SD. The specimens are tested according to the ASTM standards. The highest slump is noticed for the normal concrete having zero percent of SBA and SD. The samples comprising of replacement of 9% SBA and 40% SD showed the highest CS as compared to other samples. The incorporation of 9% SBA and 50% SD gave maximum TS. Hence, the partial replacement of cement with SBA and sand with SD, can be employed for strength improvement of normal strength.

Keywords: Normal strength concrete, Sugarcane bagasse ash, stone dust, strength improvement.

1. INTRODUCTION:

In the modern era, the researchers are trying to utilize the industrial wastes in some beneficial products to overcome the burden of pollution. The industries have significant role for the financial stability of the world but on other hand the by-products and wastes generated in the industries during the manufacturing process also grounds a huge burden of pollution on the society. One of the wastes produced by Sugarcane industry is bagasse ash. Bagasse ash being waste of the sugar mills is damped out in valuable lands because of which various ecological issues have been accounted for (Cokca et al. 2009). Janjaturaphan and Wansom 2010 reported that the Sugarcane bagasse ash (SBA) is a cementitious material which carries good binding properties and could be utilized for improving the engineering properties of the soil. Sugarcane bagasse could be used as a substitute for the manufacture of particleboards and it could also be utilized for various applications of civil engineering like construction of floors etc. (Battistelle et al. 2016; dos et al. 2014). Castaldelli et al. (2013) stated that the SBA could be employed for preparing concrete and other building material, such as blocks and bricks.

Like the SBA the stone dust (SD) being waste material is also available in large quantity. SD can reduce the cost of concrete by replacing the sand partially. The dust produced during the processing of stone in the stone processing plant, normally surpasses the limit specified by the ASTM C778-17 and the sand needs to be washed. This process shows a significant economic loss in the available natural resources, and an increase of waste that makes dust disposal problem at quarries (Malhotra & Carette 1985; Kalcheff 1977), since, only slight amounts are used as a filler material in asphalt concrete. Balamurugan et al. (2013) reported that significant improvement was reported in mechanical strengths of M20 and M25 grade concrete when sand was replaced by SD. Safiuddin et al. (2007) investigated the effectiveness of the sand replacement with quarry dust for cement concrete. It was stated that the sand replacement with quarry dust, resulted minute rise in the dynamic modulus of elasticity with reduced compressive strength as compared to that of the plane concrete.

In the current research program, the behaviour of the normal strength concrete for the partial replacement of cement with Sugarcane bagasse ash and sand with stone dust are evaluated. The general aim is to examine the effectiveness of the usage of Sugarcane bagasse ash and stone dust in the same mix of the concrete. In this study, the compressive and splitting tensile strengths along with the slump of the concrete are evaluated for replacement of the 9% cement weight with same amount of Sugarcane bagasse ash and 40% and 50% of sand weight with same quantity of the stone dust.

2. MATERIALS AND CASTING, PROPORTIONING, AND DESIGNATION:

2.1 Materials

Ordinary Portland cement (OPC) complying with the ASTM C150 Type I, is utilized in this experimental study. The maximum size of coarse aggregates is taken as 3/4 inch (19 mm). Sand graded between No.4 (4.75mm) sieve and No.100 (150 µm) is used in making of all samples. The stone dust used complies with the requirements of ASTM C33/33M-18. Locally available Sugarcane bagasse ash of Mardan Sugar Mill is used in the current investigation. Sugarcane bagasse ash obtained is sieved through No. 200 sieve.

2.2 Samples preparation and properties

The samples for each test are cast and cured according to the ASTM standard specified for corresponding test. The mix design of the 1:2:4 is used for the concrete with water to cement ratio of 0.68. The samples used in the study are designated as NC, 9B:40S, and 9B:50S. The NC designates the sample, which have zero percent of Sugarcane bagasse ash (SBA) and stone dust (SD). In each of the 9B:40S, and 9B:50S, the 9B presents the percentage of cement weight replaced by same quantity of SBA while 40S presents the percentage of the sand replaced by same amount of SD. Hence 9B:40S is the sample having 9% cement weight replacement with SBA and 40% of sand with SD. Designations and mixing properties of the samples are demonstrated in the Table 1. Three samples are tested for each property and mean of the three results is considered as a final value for the corresponding property.

Table 1. Sample designation and mixing properties

Sample	Mix design	W/C	Binder		Fine Aggregate		Coarse Aggregate
			OPC	SBA	River Sand	Stone Dust	
NC	1:2:4	0.68	100%	0%	100%	0%	100%
9B:40S			91%	9%	60%	40%	100%
9B:50S			91%	9%	50%	50%	100%

OPC = Ordinary Portland cement, NC = Normal concrete (0% SBA and 0% SD)
SBA = Sugarcane bagasse ash

3. EXPERIMENTAL PROCEDURES

The standard procedure of the ASTM C143 / C143M-15a is used for the slump test. ASTM standard C39 / C39M-17 is used to perform the compressive strength test of the standard size cylindrical specimens in 2000 kN compression testing machine. The cylindrical standard size specimens are tested according to the ASTM standard C496 / C496M-17 for determining the splitting tensile strength.

4 RESULTS AND DISCUSSIONS

4.1 Slump

Slump test results of the normal concrete (NC) specimens and Sugarcane bagasse ash (SBA) and stone dust (SD) incorporating specimens (i.e. 9B:40S and 9B:50S) are displayed in the Table 2. The slumps of 3.62 inch, 2.87 inch, and 2.56 inch are noticed for NC, 9B:40S, and 9B:50S, respectively. The NC having zero percent of SBA and zero percent of SD gave the highest slump for the same W/C ratio. The slumps of the 9B:40S and 9B:50S are 0.75 inch and 1.06 inch, repectively, less than that of the NC.

Table 2. Results of slump tests

Samples	W/C ratio	Slump (Inch)	Slump (%)	Slump difference (%)
(1)	(2)	(3)	(4)	(5)
NC	0.68	3.62	100	0
9B:40S	0.68	2.87	79	21
9B:50S	0.68	2.56	71	29

The percentage comparsion of the slumps and their percent differences with respect to NC, are given in the fourth column and fifth column, respectively, of the Table 2. The slump of the 9B:40S and 9B:50S, reduced by 21% and 29%, respectively as compared to that of the NC. Hence, it can be deduced that the addition of SBA and SD into concrete may reduce its workability. The possible reason for the decrease in the slumps may be the high tendency of the SD for water absorption due to its clayey nature.

4.2 Compressive Strength:

28 days compressive strength (CS) test results of the normal concrete (NC) specimens and Sugarcane bagasse ash (SBA) and stone dust (SD) incorporating specimens (i.e. 9B:40S and 9B:50S) are revealed in the second column of Table 3. The CS of 3009.9 psi, 3221.1 psi, and 3194.7 psi, are observed for NC, 9B:40S, and 9B:50S, respectively. The CS of the 9B:40S and 9B:50S are 211.2 psi and 184.8 psi, respectively, greater

than that of the NC. The maximum CS of 3221.1 psi is noted for 9B:40S.

Table 3. Results of compressive strength (CS) and splitting tensile strength (TS) tests

Samples (1)	**28 days CS (psi)** (2)	**28 days TS (psi)** (3)
NC	3009.9	273.46
9B:40S	3221.1	251.58
9B:50S	3194.7	274.45

The percentage comparison of the CS is demonstrated in the Figure 1. The CS of the 9B:40S and 9B:50S are found 7% and 6%, respectively, more than that of the NC. It can be concluded that the collective role of both SBA and SD may resulted in the improved CS. The Sugarcane bagasse ash may be helpful in improving the CS by virtue of its cementitious nature, which can bond excess quantity of the inert ingredients (aggregates) of the mix. Similarly, the SD may also be helpful in strengthening the bond between the inert materials due to its clayey nature. Hence, the usage of optimized amount of the SBA and SD may be helpful in upgrading the CS of the concrete.

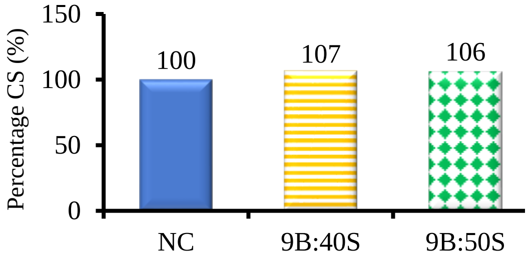

Figure 1. Percentage comparison of the compressive strengths (CS)

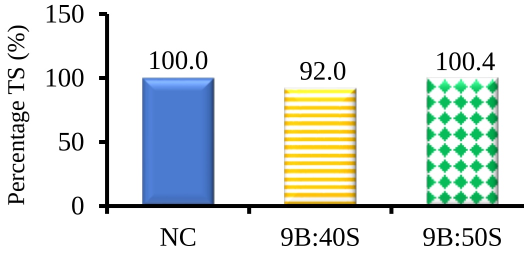

Figure 2. Percentage comparison of the splitting tensile strengths (TS)

4.3 Splitting tensile Strength:

28 days splitting tensile strength (TS) of the normal concrete (NC) specimens and Sugarcane bagasse ash (SBA) and stone dust (SD) incorporating specimens (i.e. 9B:40S and 9B:50S) are demonstrated in the third column of Table 3. The TS of 273.46 psi, 251.58 psi, and 274.45 psi are noticed for NC, 9B:40S, and 9B:50S, respectively. The TS of the 9B:50S is 1 psi greater than that of the NC. While the TS of the 9B:40S is 21.88 psi less than that of the NC. The 9B:50S outperformed the other SD samples in upgrading TS.

The percent comparison of the TS is presented in the Figure 2. The TS of the 9B:40S is 8% less than that of the NC. And the TS of 9B:50S exceeded the TS of the NC by minute amount of 0.4%. A significant variation in the TS of the concrete is noticed by varying the amount of partial replacement of the sand with SD. As compared to sand samples with 0% SD, increase in the TS of the samples having partial replacement of the sand with SD is observed. This can be associated with the improvement in the packing and binding among the SD particles and other surrounding aggregates due to presence small size particles. But this increase in clinging can be limited only to a specific percent for any unique mix design. So, use of the optimized percentages of the partial replacement are very important for attaining the maximum possible TS of any mix design of concrete. The percentage of the SBA and SD need more in-depth optimization for the TS.

5. CONCLUSIONS:

Following conclusions are made from the present investigation:

- The slumps of the 9B:40S and 9B:50S, reduced by 21% and 29%, respectively, as compared to that of the normal concrete (NC).
- The compressive strength of the 9B:40S and 9B:50S, increased by 7% and 6%, respectively, as compared to that of the NC.
- Splitting tensile strength (TS) of the 9B:40S is 8% less than that of the NC. And the TS of 9B:50S increased by 0.4% than that of the NC.

The experimental outcomes showed significant improvement in the considered strength properties of the concrete by the partial replacement of the cement with Sugarcane bagasse ash and sand with stone dust. The optimization of the percentages of the partial replacements with Sugarcane bagasse ash and with stone dust in NC is under consideration in the parallel study.

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Production of Low Cost Concrete using Waste Foundry Sand and Recycled Aggregate Concrete

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Abstract

Foundry sand consists primarily of clean, uniformly sized, high-quality silica sand or lake sand that is bonded to form moulds for ferrous (iron and steel) and nonferrous (copper, aluminium, brass) metal castings used by foundries. This sand can be recycled and reused three to five times before disposal. The disposed sand is known as Waste Foundry Sand (WFS). The opportunity to replace the natural fine aggregate with industrial by products embodies various technical, economic and environmental advantages resulting into a more sustainable construction sector. Different experiments have been carried out to assess the strength and characteristics of concrete using WFS. Proposed work is an effort to determine the strength and economic feasibility of using WFS and recycled aggregates an alternative to fine and coarse aggregate respectively in preparation of structural concrete. Concrete mixes were prepared with 0%, 10% and 15% replacement (by weight) of fine aggregate by waste foundry sand and 20% replacement (by weight) of coarse aggregate by recycled aggregate. Mechanical and physical properties of the materials were evaluated using various tests. It includes Fineness Modulus, Bulk Specific Gravity and Water Absorption test. These materials were used to cast the sample in a cylinder of dimensions 6″ × 12″ for compression testing. Testing was carried out at 7th, 14th and 28th day. From the tests it was found that WFS based concrete has acceptable working strength and can be used in civil structures.

Keywords: WFS: Waste Foundry Sand, CA: Coarse Aggregate, RA: Recycled Aggregate, F.M: Fineness Modulus

1. INTRODUCTION:

The word concrete originated from the Latin word "Concretus" which means compact or condensed [1]. It is a very strong construction material and consists of cement, sand and coarse aggregate mixed with water. It is the most used material after water and its utilization is about a metric ton per annum per capita. Its remarkable properties in fresh and hardened state have raised its usage to 12 billion tons per year globally. It is a cheaper material and performs better than aluminium and steel

Waste materials like waste foundry sand and recycled aggregate concrete which are burden on environment can be used as a new technique in construction industry that is exploring rapidly on a large scale Waste foundry sand is obtained by burning sand after the casting process of metal is reuse for many times but when it cannot be longer used it is removed from foundry as a waste for disposal. Use of waste foundry sand as a partial replacement or total replacement by fine aggregate in concrete results in production of economic, light weight and high strength concrete[2]..

Recycled aggregate can be generated from demolished construction structure which comprises of broken members or components likes the slab, beam, brick wall and others. Since the quality data of these broken materials are often unknown, such as water cement ratio, kind of admixtures, aggregate origins and gradations, as well as the differentiation of its properties during the performance time, thus it should refer to historical data of the components, physical characteristics, mechanical characteristics and environmental characteristics [3].

Due to rapid growth in population the demand of building construction and hence the demand of construction materials like concrete is also rising. So other than using natural sources we have to find other alternative sources of concrete constituents to produce concrete of adequate strength. Waste foundry sand from metal industry causes various environmental problems. Such waste material which is harmful for the environment can be used for the development of low cost and eco-friendly building materials. Concrete is a material which is composed of coarse aggregate, fine aggregate, cement, admixtures and water, all of these each material in concrete contributes towards its strength. So, by partial replacing of material affects different properties of concrete. The purpose of this research work is to study the effect of partial replacement of waste foundry sand (WFS) with natural sand on mechanical properties of concrete using recycled aggregate concrete. And to produce low cost and eco-friendly concrete using waste foundry sand and recycled aggregate concrete.

2. EXPERIMENTAL PROCEDURES:

2.1 Materials

The Ordinary Portland Cement (OPC) ASTM Type l of Grade: 42.5 with 32 % consistency was used. The initial and final setting time of OPC was 31 minutes and 130 minutes respectively. The Lawerancepur sand was used as fine aggregates. The physical characteristics of fine and coarse aggregates are summarized in table 1.

Table 1: physical characteristics of fine and coarse aggregates

Properties	Fine Aggregates	Coarse Aggregates	Non-Ferrous waste foundry sand
Bulk Specific Gravity (Oven Dry)	2.52	2.63	2.14
Bulk Specific Gravity (SSD Condition)	2.58	2.64	2.27
Apparent Specific Gravity	2.69	2.66	2.48
Water Absorption (%)	2.48	0.546	6.40
Fineness modulus	2.64	7.96	0.93
Bulk Density (Compacted) lbs/ft^3	-	96.76	-
Bulk Density (Loose) lbs/ft^3	-	88.16	-
Flakines index (%)	-	5.10	-
Elongation index (%)	-	13.72	-

2.2 Concrete Mix Design

Table 2: Mix Design Ratio

Cement (C)	Sand (S)	Coarse aggregate (CA)
364	673	1088
1	673/364	1088/3644
1	1.94	2.98

- *Ratio for concrete mix design is 1:2:3*

Table 3: Mix Proportion

Sr.No	Mix ID	No of Specimen (6″× 12″ Cylinder)	Cement	Sand	W.F.S	NA	RA	Water
					Kg/m^3			Lit/m^3
1	Conventional Concrete	09	19.5	39	0	58.50	0	11.70
2	10 % W.F.S	09	19.5	35.10	3.90	117	0	11.70
3	15 % W.F.S	09	19.5	33.15	5.85	58.50	0	11.70
4	0 % W.F.S + 20 % RA	09	19.5	39	0	46.80	11.70	11.70
5	10 % W.F.S + 20 % RA	09	19.5	35.10	3.90	46.80	11.70	11.70
6	15 % W.F.S + 20 % RA	09	19.5	33.15	5.85	46.80	11.70	11.70
	Total no of specimen	54						

a. b. c.

Figure 1: Recycled Aggregates and Waste Foundry Sand

2.3 Casting of Concrete Specimens

Cylindrical concrete specimen with 6” diameter and 12” high were made for compressive strength. The concrete constituents were mixed in a revolving drum type mixer for approximately three to six minutes to obtain uniform consistency. Additional mixing time of about two minutes was provided for the waste foundry sand mixed concrete mixtures to ensure homogeneity. After mixing, the cylindrical moulds were filled in three layers and fully consolidated on a vibrating table to remove any entrapped air.

3. COMPRESSION TEST

The compressive strength of the sample was calculated on 7th and 28th day by dividing the maximum load achieved during the test on the transverse region. The test was made following the standard procedure described in ASTM C39. Cylindrical specimens 6" x 12" were used. The specimens were cured in lab curing tank until the age of testing. Each value of the compressive strength represents an average of three. Cylindrical specimens were capped with plaster of Paris and tested in saturated state. Rate of loading was kept at 200 to 400 lbs/sec according to ASTM standard and peak loading was kept 23 to 68 lbs. Testing procedure was followed as described in ASTM C 39.

4. RESULTS AND DISCUSSION

4.1 Slump Test

Workability of concrete mixture is measured by slump test: The slump test was conducted in accordance with the ASTM C-143 guidelines. In this test the slump cone was used. Three equal layers of concrete were filled in the sliding cone and compressed using 25 strokes of crimping rod. The rod was tempered having a diameter of 5/8in and length of 24 in. The slump test provides a good estimate of expected operability.

Figure 2: Slump test

Table 2: Slump test values of various mixes of WFSC

Sr.#	Mix ID	Percentage replacement of WFS + RA	Slump Value (inch)
1	NRAWFS 0	Traditional concrete 100 % natural sand 0 % W.F.S	3
2	NRAWFS 10	10% replacement by WFS	4
3	NRAWFS 15	15% replacement by WFS	6
4	RAWFS 0	0 % W.F.S + 20 % RA	3
5	RAWFS 10	10 % W.F.S + 20 % RA	5
6	RAWFS 15	15 % W.F.S + 20 % RA	6

4.2 Results of Compressive Strength Test:

At each age average of three cylinders was taken for determination of average compressive strength. Compressive strength is significantly increased. The behaviour can be seen through table 3.

Table 3: Results of compressive strength test of WFSC

Sr.#	Mix ID	Percentage Replacement Of WFS	Compressive Strength Psi		
			7 Days	14 Days	28 Days
1	NRAWFS 0	Traditional concrete 100 % natural sand 0 % W.F.S	2980	3240	3465
2	NRAWFS 10	10 % replacement by W.F.S	2900	2980	3155
3	NRAWFS 15	15 % replacement by W.F.S	2740	2750	2835
4	RAWFS 0	0 % W.F.S + 20 % RA	3370	3125	3220
5	RAWFS 10	10 % W.F.S + 20 % RA	2840	2840	3080
6	RAWFS 15	15 % W.F.S + 20 % RA	2690	2670	2970

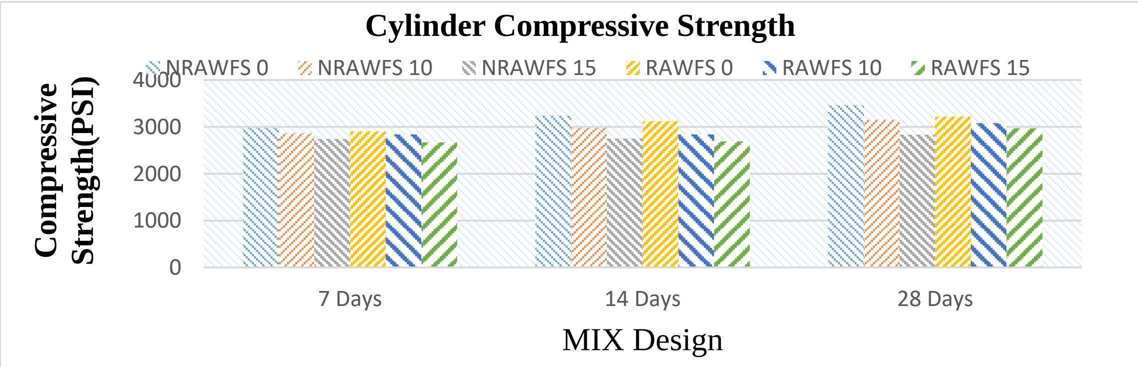

Figure 3: Compressive strength of N-WFSC at all ages

5. CONCLUSIONS:

The reuse of non-ferrous waste foundry sand and recycled coarse aggregate as a substitute for natural sand and coarse aggregate respectively in concrete production was evaluated based on the mechanical properties of the resulting concrete. Following conclusions are drawn from the finding of literature survey and results of the detailed experimental work of this project:

- Concrete prepared with non-ferrous waste foundry sand concrete has various benefit characteristics such as reduced the cost and environmental problem from the foundry waste disposal.
- The workability of fresh concrete increased with increase in the percentages dosage of non-ferrous waste foundry sand content.
- The compressive strength values for concrete with 10% and 15% regular sand replacements with Non-ferrous waste foundry sand are lower than the concrete with no replacement.

- Compressive strength of non-ferrous waste foundry sand concrete also decreases with increase in content of non-ferrous waste foundry sand.
- Concrete containing 0% replacement of fine aggregate with Non-ferrous waste foundry sand and 20% recycled coarse aggregate showed flexural strength higher to that of the control mix at 14 days.
- Compressive and flexure strength of non-ferrous waste foundry sand concrete is observed to decrease with increase in dosages.
- The analysis of non-ferrous waste foundry sand indicated that non-ferrous foundry sand can be a very suitable material for concrete production. However, the fineness and high water absorption of this sand increases the water demand of the concrete, and by increase in the workability of the concrete due to its fineness and composition and chemical reaction.
- Recycled coarse aggregate increases bond strength and hence effective to use as partial replacement without sacrificing strength at all and economy can also be achieved.
- This sand and recycled coarse aggregate can be used in those projects where low or medium strength is required because these will contribute towards economy.
- Cost of non-ferrous waste foundry sand is not more due to their free availability, however this sand is cheaper than natural fine aggregate resulted in economy.

6. RECOMMENDATIONS:

In this research WFS and recycled aggregate used and recommend that it is safe to use partial replacement of fine aggregate with WFS up to 10% replacement. And coarse aggregates partially replaced with recycled aggregates for 20% obtained from demolished concrete structures to produce normal strength concrete.

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Suitability of Local Wood Ash for Concrete as a Partial Replacement of Cement

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Abstract

Natural resources of limestone, coal, and oil are depleting day by day due to its high usage in the production of cement. Researchers are searching for easily obtainable and economical materials, which can be used as a cement replacer in concrete. Bagasse ash, wood ash (WA), and rice husk ash are pozzolanic nature materials obtained as byproducts from agriculture and industry. These are pollutants for surrounding and utilizing them as cement substitution materials will lessen the contamination as well as expense of the cement. The overall aim of this study is to evaluate the performance of the concrete for cement replacement with WA. In the current research, the effect of replacement of cement with local WA on workability and compressive strength of concrete as well as chemical composition of ashes and strength activity index of WA samples were examined experimentally. ASTM C39/C39M-17, was adopted to cast and test concrete cylinders for evaluation of the compressive strength at the age of 7 days, 28 days, and 56 days. Wood ash of three different local sources i.e. boiler of Rado 80 textile mill, kiln of the Liaquat Hall mess, and Doce bakery was used. The chemical composition of each type of the WA was determined by using wet analysis method. The control mix consisted of cement, sand and aggregates in the proportions of 1, 2, and 4, respectively, with water to cement ratio of 0.60. The test specimens were also cast in the same proportion with 10% replacement of the cement by same amount of the WA. The workability of the test mix got reduced as compared to that of the control mix. The results of compression test showed that concrete containing WA of boiler of Rado 80 textile mill, was comparatively good as compared to that of other types of the WA samples used in the investigation. The incorporation of the WA showed the potential to achieve the required strength of the concrete with low cost wood ash as replacement for cement. But detailed optimization of the percentage of the replacement of the local wood ash with cement is required.

Keywords: Normal strength concrete, local wood ash, cement replacement, properties improvement.

1. INTRODUCTION

Cement is an important and expansive ingredient of the concrete, which forms 10% to 20% of concrete's mass. The cement has the biggest part of ozone harming substance in environment. The utilization of waste material as a substitution for cement has turned out to be increasingly latest trend to save atmosphere. Number of industrial wastes like fly ash, bagasse ash, wood ash, and rice husk ash are produced during different processes in industries. Recently, some of the locally produced wastes like lime stone quarry dust, electric arc furnace slag, industrial granite sludge, bagasse ash, and glass waste sludge were investigated as a possible cement replacer (Khan et al. 2019; Amin et al. 2017; Amin et al. 2017; Amin 2017). It was reported by different researchers that some of the pozzolanic industrial wastes could be utilized as cement substitute in various types of the cement composites (Khan et al. 2019; Rukzon & Chindaprasirt 2012; Naik et al. 2003; Ganesan et al. 2007).

The use of timber processing waste and forestry biomass as fuel in various industries has caused a key problem, linked to the production of significant quantities of ash as a by-product from the burning of such biomasses. A local method of land-filling is used for disposal of large portion (about 70%) of produced wood waste ash (Etiegni and Campbell 1991; Campbell 1990). The seepage of rainwater or leakage of the heavy metal contents may arise numerous issues like contamination of ground water (Udoeyo et al. 2006). Hence, the long-term impacts of wood ash disposal through landfilling is not a safe and proper solution. These issues need suitable way of wood ash disposal as a solution. Several types of factories in Pakistan such as Gourmet foods, Doce foods, Rado 80 textile mill and many other are using wood as a fuel. Moreover, timber industries have developed boiler units at small size, which use timber wastes as fuel. In this way, wood wastes obtained from the same industry are used as fuel for boilers. Recent researches had indicated that wood waste ash was found feasible as a cement substitute material in production of concrete with satisfactory amount of durability and strength (Cheah & Ramli 2011). Naik (1999) reported that all the samples of mortar mixes obtained by 10% replacement of cement showed the maximum compressive strength. Rajamma et al. (2009) also analyzed that wood waste fly ash from wood biomass fired power plant when used as a 10% replacement with cement in mortar mix gave higher 28 days compressive strength. As the wood ash showed the potential to be used as a cement replacer. Therefore, the suitability of the locally available ash also needs to be checked as a cement replacement in concrete production. For this purpose, in the current research three different types of locally accessible ashes were analyzed as a cement replacer in the concrete mix.

In the present investigation, the wood ashes local sources of boiler of textile mill of Rado 80, Doce bakery, and kiln of the mess of Liaquat Hall were incorporated as a cement replacer in the cement mortar. The outcomes of the ash wet analysis test, strength activity index test, workability, and compressive strength tests of concrete were examined experimentally. In this study, 10% of cement weight was replaced in concrete mix by same amount of locally available wood ash.

2. SAMPLES CASTING, PROPORTIONING, AND DESIGNATION

2.1 Materials

The locally available wooden ash (WA) was used in this work. The ashes of the three different sources were used for this purpose i.e. i. ashes from boiler of Rado 80 textile mill ii. Ashes from Doce bakery situated in Lahore and iii. Ashes of kiln of mess of the Liaquat Hall located in Government College of Technology, Rasul. The wood ash was passed through 0.074 mm sieve (No. 200) to bring it within the specified size of the cement and to make it free of dust and other impurities.

2.2 Samples preparation and designation

The mix proportion of 1:2:4 (cement:sand:aggregates) was used for preparation of all concrete samples with a consistent water to cement ratio (W/C) of 0.60. Slump cone test was used for determining the workability of the fresh concrete. The standard size (150 mm diameter and 300 mm height) cylinders were used. Normal concrete was used as a control concrete and three specimens were cast. Twenty-seven cylindrical specimens having 10% of cement replacement with same amount of WA (nine for each type of wood ash) were cast for determining the 7 days, 28 days, and 56 days compressive strength of the concrete. Sample designation and mix proportion for each type of the mix are shown in the Table 1. The CC represents the control mix (0% of WA), BWA represents the sample had WA of boiler of Rado 80 textile mill, DWA is the sample which had the WA of Doce bakery Lahore, and MWA is the sample had WA of kiln of the Liaquat Hall mess. The mortar cubes of 50 mm side were cast for performing the strength activity test. Total of nine mortar cubes (three for each type of WA) were cast. Average of three results was considered the final value for each property.

Table 1. Sample designation and Mix proportion

Sample	**Mix Design**	**W/C**	**Binder Content**		**Fine Aggregate**	**Coarse Aggregate**
			OPC %	**WA** %	%	%
CC	1:2:4	0.6	100	0	100	100
BWA	1:2:4	0.6	90	10	100	100
DWA	1:2:4	0.6	90	10	100	100
MWA	1:2:4	0.6	90	10	100	100

OPC = Ordinary Portland cement, WA = Wood ash

3. EXPERIMENTAL PROCEDURES

Slump test was performed as per the standard method of the ASTM C143/C143M-15a. The chemical composition of all the wood ash samples and cement were determined by wet analysis to check the criteria of ASTM C618-19. The compressive strength test of mortar cubes for strength activity index and concrete cylinders were carried out according to ASTM C109/C109M-16a and ASTM C39/C39M-17, respectively.

4. RESULTS AND DISCUSSIONS

4.1 Chemical content of wood ashes

Chemical composition of all the wood ash (WA) samples and cement were determined by wet analysis to check the criteria of ASTM C618-19 (Mineral Admixture Class C). Wet analysis test results are demonstrated in Table 2.

Table 2. Results of wood ash chemical analysis

Constituents	**Cement** %	**Boiler WA** %	**Doce WA** %	**Mess Kiln WA** %
SiO_2	13.81	55.52	13.50	20.66
Al_2O_3	6.85	3.11	4.21	3.663
Fe_2O_3	0.01	0.401	1.37	1.2
CaO	60.52	9.92	25.76	17.92

From the chemical analysis of wood ashes, it was found that the amount of CaO was less in every type of the WA as comapred to that of the cement. The summation of the total amount of the silicon dioxide (SiO_2), aluminium oxide (Al_2O_3), and Iron oxide

(Fe_2O_3) was found 26%, 18%, and 59%, for Boiler WA, Doce WA, and Mess Kiln WA, respectively. The Boiler WA was able to achieve the higher value of the sum of three types contents as compared to other two types of the ashes but still it did not achieve the minimum requirement for natural pozzolana for using as a mineral admixture in cement concrete according to ASTM C 618-94.

4.2 Strength activity index

ASTM C311/C311M-18 was followed for determinng the strength activity index (SAI) for wood ash samples. The SAI of the BWA, DWA, and MWA mortar specimens were 91%, 97%, and 91%, respectively. Hence the SAI of each of the sample was more than 75%, confirming the pozolanic activeness of the ashes as well as their suitablity for use in concrete.

4.3 Slump Test

Workability of concrete was determined by slump cone test in accordance with ASTM C143. The slump of 50 mm, 40 mm, 40 mm, and 40 mm was noticed for control mix having zero percent of wood ash (CC), Boiler wood ash samples (BWA), Doce wood ash samples (DWA), and Mess Kiln wood ash samples (MWA), respectively. The slump of BWA, DWA, and MWA reduced by 20% as compared to that of the CC. Slump results showed that for increase in the contents of the wood ash caused significant decrease in the slump for the same W/C ratio.

4.3 Compressive Strength:

The compressive strength (CS) of concrete specimens at an age of 7 days, 28 days, and 56 days were determined in accordance with ASTM C39 / C39M-17. The compressive test results are presented in Table 4. The 7 days CS of the control mix having zero percent of wood ash (CC), Boiler wood ash samples (BWA), Doce wood ash samples (DWA), and Mess Kiln wood ash samples (MWA) were 17.8 MPa, 17.1 MPa, 11.4 MPa, and 12.6 MPa, respectively. The 7 days CS of 17.1 MPa of BWA was the nearest to the 7 days CS of control mix with the slight reduction of 0.07 MPa. The lowest 7 days CS of 11.4 MPa was noticed for DWA. The 28 days CS of the CC, BWA, DWA, and MWA were 23.5 MPa, 21.2 MPa, 16.4 MPa, and 15.3 MPa, respectively.

Table 3. Compressive strength test results

Mix	**Compressive strength 7-days (MPa)**	**Compressive strength 28-days (MPa)**	**Compressive strength 56-days (MPa)**
(1)	(2)	(3)	(4)
CC	17.8	23.5	30.0
BWA	17.1	21.2	29.5
DWA	11.4	16.4	16.8
MWA	12.6	15.3	15.5

By comparing the 28 days CS of the wood ash samples, the highest 28 days CS of 21.2 MPa was noticed for BWA while the lowest 28 days CS was observed for MWA. The 28 days CS of the BWA was the nearest to that of the control mix with zero percent wood ash. The 56 days CS of the CC, BWA, DWA, and MWA was 30.0 MPa, 29.5

MPa, 16.8 MPa, and 15.5 MPa, respectively. By comparing the 56 days CS of wood ash samples, the BWA showed the highest CS of all. The 56 days CS of the BWA was 12.7 MPa and 14 MPa greater than that of the DWA and MWA, respectively. The 56 days CS of the BWA was slightly decreased by 0.5 MPa than that of the control mix.
The percentage comparisons of the 7 days, 28 days and 56 days compressive strengths are demonstrated in Figure 1, 2 and 3, respectively. The 7 days CS of the CC was 4%, 36%, and 29%, more than that of the BWA, DWA, and MWA, respectively. The lowest decrease in the 7 days CS was noticed for BWA incorporated samples. The 28 days CS of the BWA, DWA, and MWA were less than that of the CC by 10%, 30%, and 35%, respectively. The minimum decline of 10% was noticed in the 28 days CS of BWA than that of CC. As compared to the 56 days CS of control mix, the 56 days CS of BWA, DWA, and MWA was decreased by 2%, 44%, and 48%, respectively. A slight decrease of 2% was noted in 56 days CS of the BWA sample as compared to 56 days CS of the control mix. The 28 days strength results showed that by replacing 10% cement with BWA, drop in strength was 10% and this drop in strength reduced to 2% after 56 days probably due to delayed hydration in comparison with control concrete. Hence, the samples having wood ash of the boiler of Rado 80 textile mill, showed the highest compressive strength as compared to samples with wood ash of Doce bakery Lahore and wood ash of kiln of the mess of Liaquat Hall.

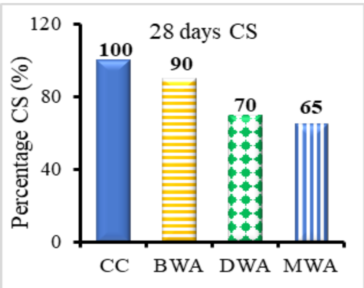

Figure 2: 7 days compressive strength

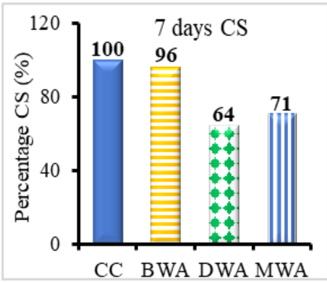

Figure 3: 28 days compressive strength

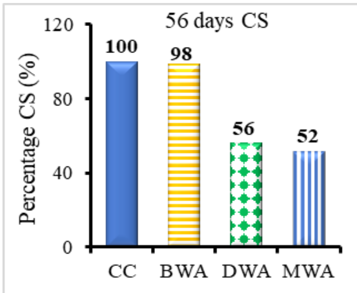

Figure 4: 56 days compressive strength

It can be concluded that the test results indicated that by addition of wood ash in concrete the compressive strength decreased, but this reduction in compressive strength was less prominent after long time curing. This may be due to the late pozzolanic action of the wood ashes. BWA performed better out of three ashes in improving the compressive strength of the samples.

5. CONCLUSIONS

Following conclusions were made from the current study:

- The strength activity index of the Boiler WA, Doce WA, and Mess Kiln WA mortar specimens was 91%, 97%, and 91%, respectively.
- The slump of BWA (Boiler WA samples), DWA (Doce bakery WA samples), and MWA (Mess Kiln WA samples) reduced by 20% as compared to that of the control mix (CC) “0% wood ash”.
- As compared to the 7 days, 28 days, and 56 days compressive strength (CS) of the CC, the minimum reduction of 4%, 10%, and 2%, respectively, was noticed in CS of the BWA as compared to CS of other companions.
- The lowest 28 days, and 56 days CS was observed for the MWA that was 35%

and 48%, respectively, less than that of the CC. While the lowest 7 days strength was noted for DWA that is 36% less than that of CC.

The experimental outcomes showed significant impact on the considered properties of concrete by addition of the wood ash as a partial replacement for cement. Further investigation is required to evaluate the optimized content of the wood ash for better strength properties of the concrete as well as cement mortars.

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Effect of PVA on Rubberized Concrete

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Abstract

Use of alternative aggregates has become a dire need of today's modern civilization, as they have significantly reduced the socio-economic stresses which the construction industry is facing nowadays. This research is dedicated to studying the behavior of concrete incorporating crumb rubber as a partial replacement of fine aggregate and polyvinyl alcohol fibers as the addition of cement. PVA dosages of 1% and 2% by mass of cement and rubber dosages of 5% and 10% by weight of fine aggregate were incorporated into concrete. The parameters of the study were slump test, the fresh density of concrete, water absorption and compressive strength. Test results show that the density, workability, and strength of concrete took a nosedive as the rubber content increases which is attributed to the fact that rubber is lighter in weight, has a rough texture, increases the viscosity and form a weak bond with cement. On the flip side, PVA has shown a positive influence on the engineering properties of concrete. Hence, PVA can be used to overcome the issues associated with the use of crumb rubber in concrete.

Keywords: Alternative aggregates, rubberized concrete, fiber reinforced concrete, polyvinyl alcohol fibers, crumb rubber, mechanical strength parameters, fresh properties of concrete.

1. INTRODUCTION:

Concrete is the second most widely used material in the world (Gagg, 2014). The annual production of concrete is estimated to be 30 billion tons(Monteiro et al. 2017). In concrete ingredients, aggregates proportion is the highest. It is a fact that these virgin resources are limited and needs to be preserved in order to maintain balance in the ecosystem. Therefore, it has become a dire need of today's world to use alternative aggregates which has dual advantages; that is the reduction of cost and removal of waste. Which benefits the environment and enable us to conserve natural resources(Mannan, April 2004). The investigated alternative aggregates (AAs) includes; recycled concrete aggregate (RCA)(Jin et al. 2015), building rubbles (Khalaf and DeVenny, 2004), etc.

In recent years, preservation of the environment seeks undivided attention of engineers and scientists. Keeping that in mind crumb rubber can be added into concrete to aid preservation of the environment and to conserve natural resources. Crumb rubber is recycled rubber produced from automotive and truck scrap tires. The demand of tire production has increased drastically over the years, but their disposal has been an alarming concern, as these tires cannot be recycled and are dumped which has caused severe threats to the society from environmental impact to health concerns. Among all the methods suggested for the disposal of rubber, its incorporation into concrete has been proved most effective. The only problem associated with the use of crumb rubber as an alternative aggregate is that it causes a reduction of mechanical strength of concrete (Liu et al. 2016). Likewise, they are found to cause an increase in the workability of concrete but had a negative influence on its compressive and tensile strength(Mercy and Ramarao, 2016). However, the aforementioned shortcomings can be regained by using different additives/polymers like re-dispersible polymer powders, liquid resins, monomers, and water-soluble polymers (Eren et al. 2017). PVA (chemically known as polyvinyl alcohol) are high-performance fibers and are the first synthetic colloid prepared by Herrmann and Haehnel in the year 1924(Finch, 1973). It is water soluble polymer and is always used in concrete for enhancing the mechanical strength and durability of concrete. PVA being a chemically reactive polymer on reaction with cement forms calcium complexes that fills the pores, thus, densifies the structure and improves the properties of the cement paste(Singh and Rai, 2001). PVA fibers increase the ductility, toughness, tensile strength and flexural strength of reinforced concrete and they help in bridging the cracks thus enhance the crack resistance ability of concrete (Noushine et al. 2013). The significance of this research work is that it would curb the issues or problems associated with tire disposal. Moreover, the rubberized concrete would strike the pay dirt in the construction industry for being comparatively cheaper and readily available. The rubberized concrete is recommended for non-structural applications like sidewalks, etc. In addition, rubberized concrete improves thermal protection and is therefore recommended to be used as general insulation of walls and heat insulation on roofs. Moreover, it can be used as a noise insulator in theatres, cinema halls, noise proof rooms, etc.

2. EXPERIMENTAL PROGRAM

2.1 Materials:

During the research work, Bestway Cement was used i-e type-1 cement. The fine aggregate having a maximum size of 2mm and fineness modulus of 2.64 was used. While the locally available crush with a maximum size of 19 mm and fineness modulus of 2.65 was used as coarse aggregate. A tire rubber, used as an alternative aggregate, was obtained from a recycling industry Swat Tyre & Rubber Co Pvt Ltd Hayatabad, Peshawar. It has a fineness modulus of 2.76. PVA fibers were imported from China and its properties are shown in table 1.

Table 1: Properties of PVA

Colour	Off white
Length	6mm
Diameter	25μm
Density	1.29g/cm^3
Elongation	≤ 40%
Tensile Strength	425MPa

2.2 Specimen designation:

While conducting experiments, two varying abbreviations were used i-e CM and PR. Where CM stands for control mix while in PR, P stands for PVA and R stands for crumb rubber. In PR, the digits 0, 1 & 2 comes before "P" and 0, 5 & 10 comes before "R" that refers to the percentages in which they have replaced the respective ingredients of concrete. For instance, 2P5R states the specimen in which 2% PVA was added for cement and 5% fine aggregate was replaced by crumb rubber.

2.3 Mix proportion:

A total of nine (9) concrete mixes have been used with the ratio of 1:1.5:3 (1 part of cement, 1.5 parts of sand and 3 parts of coarse aggregate). They include one CM (control mix), while the rest of the eight consists of different proportion mixes of PVA and Crumb rubber. The water-cement(w/c) ratio was kept constant i-e 0.45 whereas, the design strength was assumed to be 21 MPa. PVA was used in a proportion of 1 & 2 % while crumb rubber was used in a proportion of 5 & 10 %. The complete mix design is summarized in table 2.

Table 2: Mix Design

Mix	Cement (kg/m^3)	PVA (kg/m^3)	w/c ratio	Water (kg/m^3)	Fine Aggregate (kg/m^3)	Crumb Rubber (kg/m^3)	Coarse Aggregate (kg/m^3)
CM	411.11	0	0.45	185	709.49	0	1049.4
0P5R	411.11	0	0.45	185	674.02	35.48	1049.4
0P10R	411.11	0	0.45	185	638.54	70.95	1049.4
1P0R	406.99	4.11	0.45	185	709.49	0	1049.4
1P5R	406.99	4.11	0.45	185	674.02	35.48	1049.4
1P10R	406.99	4.11	0.45	185	638.54	70.95	1049.4
2P0R	402.88	8.22	0.45	185	709.49	0	1049.4
2P5R	402.88	8.22	0.45	185	674.02	35.48	1049.4
2P10R	402.88	8.22	0.45	185	638.54	70.95	1049.4

2.4 Testing

Concrete testing was divided into two phases; the first phase was concerned with fresh properties of concrete that include slump test and fresh density of concrete while the second phase was concerned with hardened properties of concrete that includes water absorption test and compressive strength testing.
The slump test was performed in accordance with ASTM C 143. The densities of concrete were determined in their fresh state by weighing them and subsequently dividing them by their volume. However, the water absorption test was performed using ASTM C 642 – 97. While for assessment of compressive strength of concrete ASTM C 39 was used.

3. RESULTS AND DISCUSSIONS

3.1 Workability

Slump test was performed for evaluating the workability of concrete. Workability reflects the fresh properties of concrete. The results are shown in figure 1, which illustrates that rubber has a negative influence on the workability of concrete i-e upon

5% of rubber replacement there was a reduction of 17% in concrete's workability while for 10% of rubber replacement the reduction was almost 27%. This reduction in concrete's workability was attributed to the rough texture of rubber and to the increase in viscosity of the concrete with the addition of rubber. Unlike rubber, PVA has shown a positive influence on the workability of concrete. Where it can be seen that upon adding 1% and 2% of PVA the workability was increased by 14% and 45% respectively. Likewise, their combined effect has shown an increase in workability i-e upon comparing with 0P5R the increase for 1P5R was 15% while for 2P5R the increase was 25% respectively. The increase in the concrete's workability was attributed to the ball-bearing effect of PVA (Allahverdi et al. 2010).

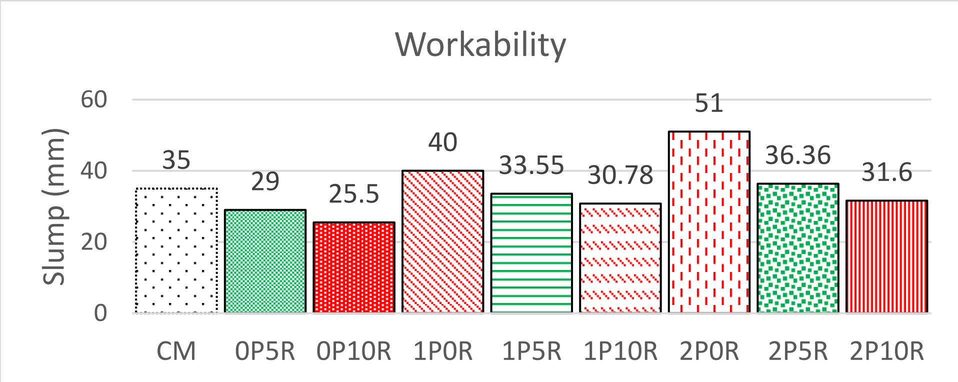

Figure:1 Average slump test of the fresh mixes versus PVA dosages

3.2 Density of fresh concrete

The fresh density of concrete is of extreme importance for its effect on the strength parameters and durability etc. It is the measurement of concrete's solidity. The results of this test are illustrated in figure: 2.

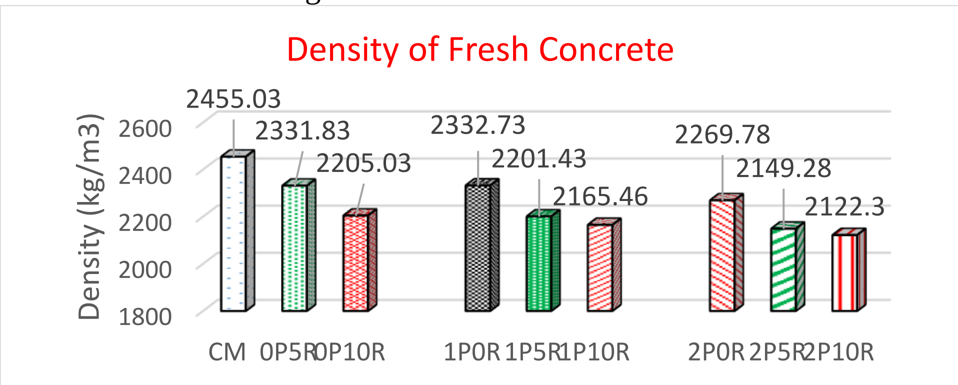

Figure:2 Densities of the fresh mixes at different PVA dosages

Rubber, being lighter in weight, tends to cause reduction of concrete's density. Higher the replacement, lower is the density and vice versa. Likewise, the addition of PVA also tends to cause a reduction of concrete's density.

3.3 Total water absorption

The total water absorption of concrete helps in working out the durability of concrete. The concrete with higher water absorption suggests that it has high porosity and lower durability or vice versa. In this research, the water absorption was evaluated at 7th and 28th day respectively. The results are illustrated in figure 3.

It is clear from the graph that with the addition of rubber the water absorption of concrete increases, this is due to the weak bonding between rubber and concrete material that lead to cracks in concrete, thus making it susceptible to water penetration. Whereas the behavior of PVA was found out to be a bit ambiguous that is at 7 days of curing it doesn't show any significant influence on the water absorption properties of concrete, while at the 28th day it causes a significant reduction in the water absorption of concrete. At the early days of concrete, the PVA hasn't undergone any sorts of chemical interaction, therefore, the voids remain in the concrete and thus the water absorption is high whereas with time it forms calcium complexes that fill the voids, densifies the structure and results in a reduction of water absorption (Singh and Rai, 2001).

From figure 3, upon adding 1% of PVA into rubberized concrete, the reduction in total water absorption was 14% while for 2% of PVA the reduction was 26% respectively. Whereas, when both rubber and PVA is added it tends to cause a significant reduction in the water absorption of concrete i-e when 1% PVA was added to 5R it causes a reduction of 11% while when 2% PVA was added to 5R it causes a reduction of 17%.

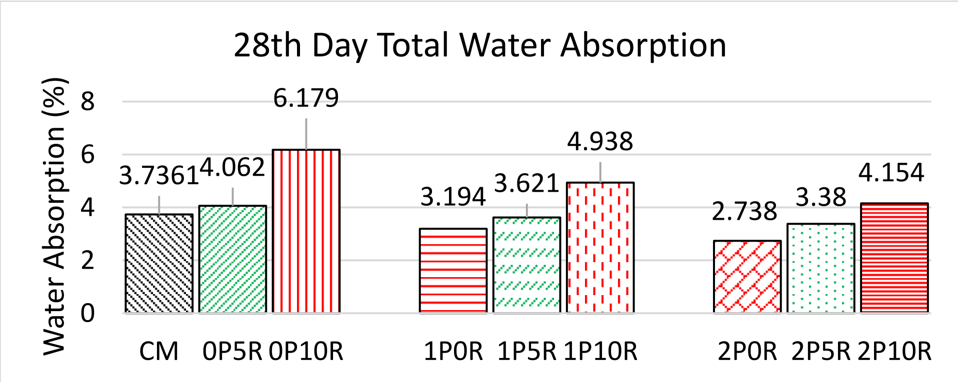

Figure:3 Total water absorption of mixes at different PVA dosages

3.4 Compressive strength

For concrete, compressive strength is of utmost importance. In the following study, compressive strength was evaluated at 7th and 28th day of curing. The experimental results are shown in figure 4. It was witnessed that rubber due to weak bonding tends to decrease the compressive strength of concrete. For 5% and 10% replacement of rubber, there was a reduction of 17% and 22% respectively.

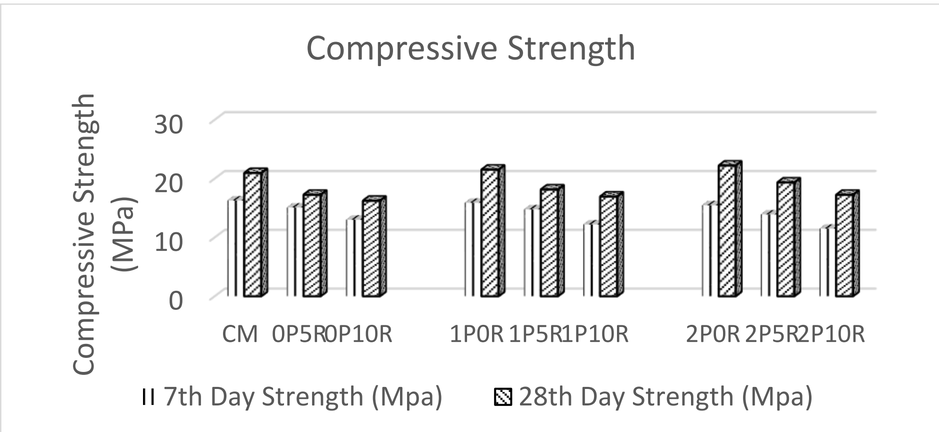

Figure:4 Compressive Strength versus PVA dosages

While the addition of PVA has struck the pay dirt by overcoming the reduction in strength that results from the addition of crumb rubber. Though at the 7th day the effect was negligible as initially PVA hasn't interacted chemically with cement while at the 28th day, after proper chemical interaction PVA enhance the strength significantly. The increase in compressive strength for 1% and 2% of PVA was estimated to be 3% and 6% respectively. Moreover, their combined effect has shown to cause an increase in the compressive strength of concrete. An increase of 5% for 1P5R and 12% for 2P5R was observed upon comparison with 0P5R.

4. CONCLUSIONS:

The overall result indicates that incorporation of crumb rubber causes a decrease in concrete's workability up to 27%, a reduction in concrete's compressive strength of almost 22% and an excessive increase in water-absorption of 64% respectively. Whereas, PVA, a synthetic water-soluble polymer, has overcome these aforementioned problems and has significantly improved the concrete engineering properties i-e an increase of 45% in concrete's workability, a reduction of 26% in water absorption and a tad increase in concrete's compressive strength was observed upon adding PVA into the concrete. Whereas, rubber and PVA both have shown to cause a steady decrease in fresh density of concrete.

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Recycled Aggregate Concrete Filled Steel Tube (CFST) and Concrete Filled Plastic Tube (CFPT)

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Abstract

Recycled aggregates are used worldwide as the replacement of the natural aggregate in different ratios causing significant reduction on concrete strength and other properties. In this research recycled aggregate concrete filled tubes were used along with lumps of recycled aggregate taken from demolished waste. These lumps (50mm to 90 mm) of recycled aggregate were used as a replacement of coarse aggregate in different proportions (0%,10%,20% and 30%) for casting control specimen, CFST (concrete filled steel tube) and CFPT (concrete filled plastic tube) cylinders. Various tests were conducted such as slump test, water absorption test, fresh concrete density test, compressive and indirect tensile tests. Upon 30% replacement of the recycled aggregate reduction in concrete strength for recycled aggregate CFST, CFPT and simple cylinders (without any confinement) was to be found 9.22% ,43.2% and 54.14%, respectively when compared with control specimen.

Keywords: Recycled aggregate CFST, Recycled Aggregate CFPT

1. INTRODUCTION:

Concrete is utilized more generally than some other substance after water, on account of its numerous points of interest. The global material extraction is 48.5 billion tons/year, out of which the share of construction material is 16.2 billion tons/year (Steinberger et al., 2010). The Building demolition rate is always expanding, making it fundamental to successfully reuse destruction waste to save the non-renewable natural resources. Nowadays, a large proportion of demolition waste and useable construction material is discarded in landfill destinations, making natural issues because of the shortage of such sites, unplanned transfer of disposal, and the ecological expense of transporting demolition waste. In a concrete mixture, aggregate represent about 80% of concrete. Therefore, the replacement of NCA (natural coarse aggregate) in various percentages with the RCA (recycled coarse aggregate) can be really helpful to make a traditional concrete as a sustainable material (Safiuddin et al., 2011). Panda et al studied that up to 30% replacement of aggregate in SCC (self-compacting concrete) there is no noticeable decrease in strength and other properties were found. Increment in RCA above 30% will inversely affect concrete properties(K C Pandaa 2013).

Recycled aggregates additionally reduces the amount of virgin aggregates to be made, consequently less evacuation of natural resources. While being smashed into smaller particles a lots of carbon dioxide is absorbed. This diminishes the amount of CO_2 is the air. The utilization of reused aggregates isn't easy but difficult to utilize on the grounds because their properties are not quite the same as natural coarse aggregates. That's the reason the nature of RCA can vary when gathered from various sources. The qualities of RCA ought to be low density, low mechanical strength, and high water absorption, more noteworthy porosity when compared with NCA (P. Saravana Kumar and G. Dhinakaran, April 1, 2012, Etxeberria et al., 2007). However, the cost of crushing RCA is still expensive. The CFST (concrete-filled steel tubular) structures have many structural benefits, which includes high load bearing capacity and fire resistances, large energy absorption and ductility capacities. It also reduces the construction cost and time required for shuttering because of no need for shuttering (Han et al., 2014).Several researchers come up with the result that most of the mechanical properties of recycled aggregate CFST are similar to that of the ordinary concrete CFST; however, reduction in its strength and modulus was found. Steel tabular columns with concrete filled are vulnerable to degradation due to corrosion, which results in the reduction of strength. For the GFRP (Glass Fibre Reinforce Plastic), the brittle failure of hoop break led to the failure of GFRP confined concrete(Xiao et al., 2012). The exceptional properties which includes higher resistance to environmental attacks and electromagnetic transparency make the plastic attractive for various structural applications. Compressive strength of CFPT (concrete filled plastic tube) increased between 1.18 to 3.65 times the unconfined strength(Gathimba Naftary K, 2014).

In this study, recycled aggregate CFST and CFPT confined concrete were investigated for mechanical and durability assessment by using various percentages of Recycled aggregate (0%,10%,20% and 30%) as a replacement of coarse aggregate. Various tests were performed and results were compared with the control specimen.

2. MATERIALS AND METHODOLOGY:

The materials that were used were: Coarse Aggregate, Fine Aggregate, Water, Cement (OPC), Recycle Coarse Aggregate (Lumps), Steel Cylinders/tubes and Plastic Tube.

2.1 Coarse Aggregate:

Coarse Aggregates are obtained from the ware house located near CUI, Abbottabad Campus, Pakistan. Specific gravity, fineness modulus and water absorption of CA was found to be 2.68, 2.9 and 3.07% respectively.

2.2 Fine Aggregate:

Fine Aggregates are obtained from the shop located near CUI, Abbottabad Campus, Pakistan. Specific gravity, water absorption and fineness modulus of FA was found to be 2.43, 2.04% and 2.65 respectively.

2.3 Cement:

Ordinary Portland cement (OPC) ASTM C150 Type-I was used throughout the research. Density, Initial and the Final setting time was found to be 3.15g/cm^3, 66 min and 335min respectively. Fineness and surface area of Cement was 97.76% and 2827 cm^2 /g.

2.4 Recycled Aggregate:

Demolished building waste was taken which includes (slab, beams and columns) opposite to Daewoo Bus stand Abbottabad and then with the help of crushing plant crushed the demolished waste into lumps (having size 50mm to 90mm). These lumps were used as recycled aggregate and replaced with natural aggregate different proportion (0%,10%, 20% and 30%). Water absorption and Specific gravity of RCA was found to be 8.34% and 2.50 respectively.

2.5 Steel Tube and Plastic Tube:

Diameter (inner to inner) and the wall thickness of steel and plastic tubes were 6in x0.0662in and 6in x 0.19in respectively, while the height of both tubes were 12in.

2.6 Testing Procedure:

Concrete cylinders used had height and diameter (12in x 6in) and therefore volume (339.29in^3) and concrete mix was M20 (Mix whose compressive strength after 28 days curing is 20N/mm^2) while mix proportion was 1:1.5:3. The Steel tube and plastic tube were used in which concrete was cast and properties of the recycled aggregate concrete filled tubes (plastic and steel) were observed. Recycled aggregate lumps (50mm to 90mm) were replaced with natural aggregate in different proportion (0%,10%, 20% and 30%). After the selection of material, we had casted 3cylindres for each replacement of recycled aggregate for each test. Different tests, such as the slump test for workability, compressive and indirect tensile tests for strength were conducted for checking structural performance and mechanical properties. Recycled aggregate simple cylinder, CFST and CFPT results were compared with the control specimen (without any recycled aggregate used).

3. RESULTS AND DISCUSSION:

3.1 Workability Test:

This test was conducted as per ASTM C 143. Reduction in slump value was noticed with the increment of recycled aggregate proportion that’s may be due to high water absorption of recycle aggregate. R0, R10, R20 and R30 shows recycled aggregate replacement percentages with natural coarse aggregate.

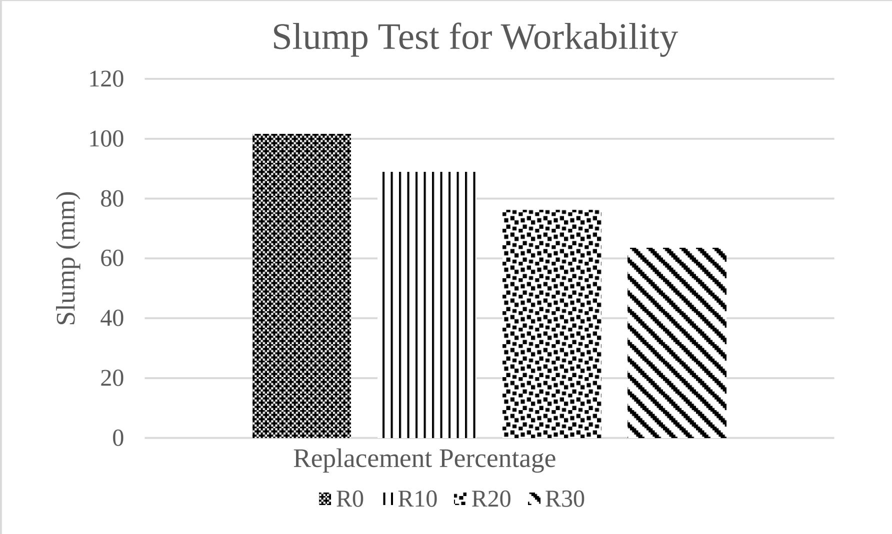

Figure-1: Workability Test by Slump method

3.2 Fresh Concrete Density:

An increase in a percentage of recycled aggregate reduces the fresh concrete density. By the replacement of the recycled aggregate up to 30%, fresh concrete density decreased by 7.84% as compared to natural aggregate concrete. Values ranges from 2269.77 kg/m^3 to 2447.836 kg/m^3.

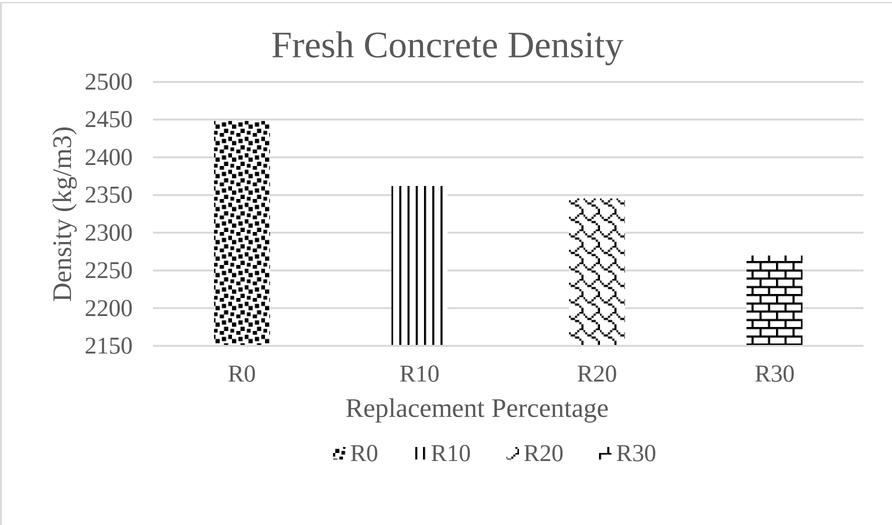

Figure-2: Fresh Concrete Density

3.3 Water Absorption Test:

The rate of water absorption was increasing with an increase in recycled aggregate proportion that's may due to the quantity of mortar attached (because it has porous structure) with it, and also recycled aggregate initial water absorption was 2.74 times (174%) higher than natural coarse aggregate.

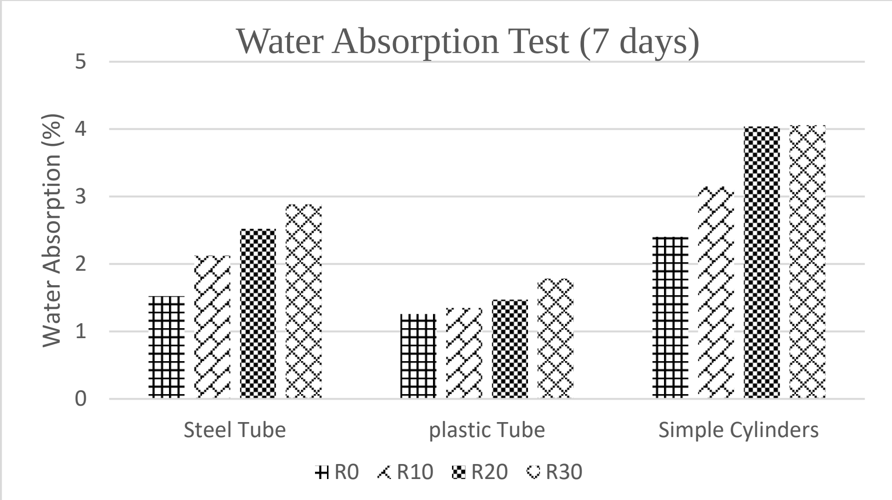

Figure-3: Water Absorption Test (7 days)

3.4 Compressive Strength Test:

This test was determined as per ASTM C39. 7-days compressive test shows decrease in concrete strength with increment in recycled aggregate proportion. But recycled aggregate CFST shows strength which is more than double of the no confinement concrete (simple cylinders). Upon 30% replacement of the recycled aggregate reduction in concrete strength for recycled aggregate CFST, CFPT and the simple cylinders (without any confinement) was to be found 9.22% ,43.2% and 54.14% respectively.

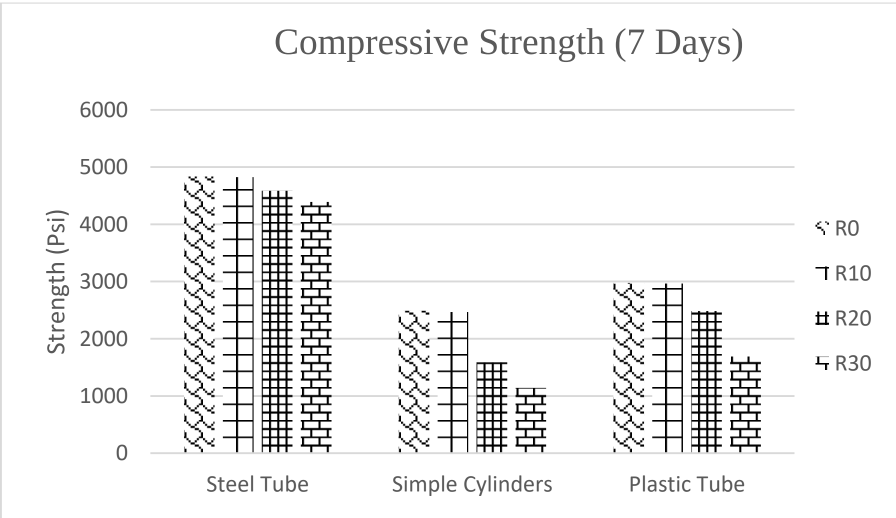

Figure-4: Compressive Test 7days

3.5 Indirect Tensile Test (Plastic Tube & Simple Cylinders):

An indirect tensile test was conducted on recycled aggregate CFPT and the simple cylinders because steel is good in tension so, we did not perform on it. It is found that upon 30% replacement of the recycled coarse aggregate reduction in CFPT and the simple cylinders, reduction in tensile strength was 32.2% and 45.68% respectively.

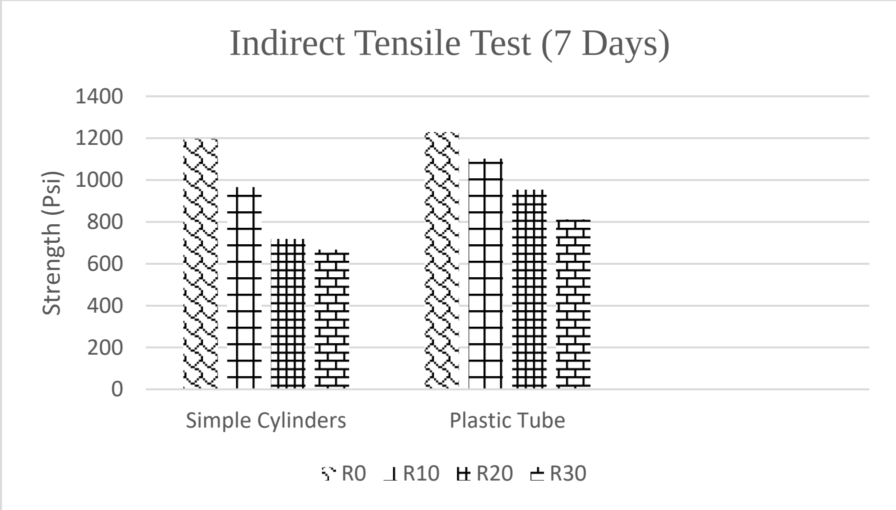

Figure-5: Indirect Tensile Test (Plastic Tube & Simple Cylinders)

4. CONCLUSIONS:

From the conducted study following conclusions can be drawn:

- Recycled Aggregate in CFST and CFPT reduces the amount of demolished waste of construction and reduces the use of virgin materials, which makes it a sustainable step towards the eco-friendly environment.
- 37.5% reduction in slump value was observed upon 30% replacement of the natural aggregate with recycled aggregate.
- Upon 30% replacement of recycled aggregate reduction in concrete strength for recycled aggregate CFST, CFPT and simple cylinders (without any confinement) was found 9.22% ,43.2% and 54.14% respectively.
- It is found that upon 30% replacement of the recycled coarse aggregate in CFPT and simple cylinders, reduction in tensile strength was 32.2% and 45.68% respectively.

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Two Stage Concrete using Recycled Coarse Aggregate and Bagasse Ash

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Abstract

In this experimental research, the effect of different ratios of bagasse ash and recycled coarse aggregates in Two-stage concrete (TSC) was evaluated. TSC is not quite the same as normal concrete. In TSC, coarse aggregates are set in formwork and after that grout or mortar is infused through a pipe with high pressure. Four mixes of TSC concrete were prepared. Control Mix-I was made with 100% natural coarse aggregates. Control Mix-II was prepared with 100% recycled coarse aggregates (RCA). The third mix was made with 10% bagasse ash (BA) as a fractional substitution of cement and 100% RCA. Fourth mix was prepared with 20% bagasse ash as a fractional substitution of cement and 100% RCA. 1% super plasticizer by the weight of cement was added in concrete mixes with 10% and 20% bagasse ash. Water to cement ratio (w/c) was 0.5 and used for all mixes. Different tests like compressive strength test and split tensile strength test were performed on samples made from all four mixes. Compressive strength and tensile strength of Control Mix-I was highest among all mixes. Results indicate that tensile strength and compressive strength was increased with the addition of bagasse ash in mixes having RCA. The maximum increase in compressive strength and tensile strength was in 20% BA mix.

Keywords: Two- stage concrete (TSC), Recycled coarse aggregates (RCA), Natural coarse aggregates, Bagasse ash (BA)

1. INTRODUCTION:

The most extensively used building material around the globe is concrete (Meyer, 2004). In TSC, coarse aggregates are set in formwork and after that grout or mortar is infused through a pipe with high pressure. TSC has a number of applications, it is mainly used in concrete and masonry repair in under water construction, places where placing conventional concrete is difficult, in mass concreting where a low heat of hydration is required (Stubbs, 1959). TSC need more coarse aggregates than required in normal concrete (Abdelgader and Elgalhud, 2008). Shrinkage in TSC is lower due to point to point contact of coarse aggregates (Abdelgader and Górski, 2003). TSC costs 25% to 40% less than traditional concrete (Abdelgader, 1995).

Around 10 billion tons of concrete is produced annually, making it the largest consumer of Earth's natural resources, that are water, natural aggregates (gravel and crushed rock) and sand. Around 12.6 billion tons of natural aggregate is used annually. Cement industry releases around 7% of the total Carbon dioxide (CO_2) (Mehta, 2002). To protect our environment from depleting virgin aggregate resources, recycled aggregates has been used to produce concrete. Recycled aggregates consists of natural aggregates and adhered mortar. Concrete obtained from demolished buildings is crushed to obtain recycled aggregates. It has more absorption capacity. Due to increased absorption capacity, 5% more water is required for concrete made with recycled aggregates to acquire similar workability as that of normal concrete (Etxeberria et al., 2007).

Therefore, recycled aggregates can be used in TSC as an alternative of natural aggregate because there is no issue of workability in TSC, as coarse aggregates are placed in formwork. It will also help in conservation of natural resources of coarse aggregates. But concrete with recycled aggregates need more cement than typical concrete to achieve higher strength (Hansen, 1986).

Wastes obtained from agricultural and some other industries can be used as replacement materials in concrete (Hansen, 1986). Sugarcane contains about 25% bagasse. Bagasse is also used in paper industry. When bagasse is burnt for energy purpose, it produces 3% of ash, which is dumped in landfills (Amin, 2010). Pozzolan's silica reacts with $Ca(OH)_2$ and forms calcium silicate hydrate, which enhance the strength of concrete (Martirena-Hernández et al., 2001).

TSC is used in the foundation of an 18 storey building in Gdansk, Poland, refacing of Baker dam, Colorado, USA, piers of Mackinac Bridge, USA and repair of water dam in Czchow on the Dunajec river, Poland (Nowek et al., 2007). This significance of this research work is to tackle the issue of pollution caused by bagasse ash and concrete waste. It will help in creating sustainable development and preserve the sources of natural aggregates. Moreover, TSC will play a vital role in underwater construction, repair and mass concrete. TSC with bagasse ash and recycled aggregate will be economical and will have strength almost equal to conventional concrete.

2. EXPERIMENTAL DETAILS AND METHODOLOGY:

2.1 Materials:

2.1.1 Cementitious materials:

Ordinary Portland cement (ASTM Type-I) was used for the preparation of TSC. Fineness of cement was 93.15%. The surface area of cement was 2137 cm^2/gm. Bagasse ash was brought from Premier sugar mill, Mardan. It was grinded/crushed in PCSIR, Peshawar. It was passed through sieve#200. Specific gravity of bagasse ash was 1.35. The surface area of bagasse ash was 2840.7 cm^2/gm.

2.1.2 Aggregates:

Coarse aggregates were brought from a quarry near COMSATS University Islamabad, Abbottabad Campus. Recycled coarse aggregates were brought from an empty plot near Daewoo terminal, Abbottabad. The demolished concrete waste of a building was crushed with the help of a crusher to obtain recycled coarse aggregates. Cost of crushing concrete waste with the help of a crusher was less than purchasing natural aggregate.

Twenty-five millimeter was the maximum size of both natural and recycled aggregates. Table 1 shows the physical properties of both natural and recycled aggregates.

Table 1. Physical properties of natural and recycled aggregates

Physical Properties	Natural aggregates	Recycled aggregates
Water absorption	1.85%	7.59%
Specific gravity	2.75	2.62
Impact value	14.72%	22.23%
Fineness Modulus	2.04	2.19
Density	1532.3 kg/m^3	1399.2 kg/m^3

Fine aggregates were brought from a quarry near COMSATS University Islamabad, Abbottabad Campus. Fineness modulus of fine aggregate was 2.96. Fine aggregates had water absorption of 1.1% and specific gravity 2.35.

2.1.3 Admixture:

Ultra Super Plast 470 was used throughout the casting of TSC. It was procured from Ultra Chemicals, Peshawar.

2.2 Mixture Proportions:

Four different mixes of TSC were made with ratio 1:1:2.7 (Cement: Fine aggregate: Coarse aggregate). Control mix- I was made with 100% natural coarse aggregates. Control mix- II had 100% RCA. Third mix was prepared with 10% BA as a fractional substitution of cement and 100% RCA. Fourth mix had 20% BA as a fractional substitution of cement and 100% RCA. 1% super plasticizer by the weight of cement was used in mixes with 10% and 20% bagasse ash. Water to cement ratio (W/C) used in this experimental research was 0.5. It was used for all four mixes.

Table. 2 Mix types with identification based on replacement ratio

Mix Types	Concrete Mix Proportion
CM-I	Control Mix (100% natural coarse aggregates and 100% cement)
CM-II	Control Mix (100% recycled coarse aggregates and 100% cement)
10% BA	100% recycled coarse aggregates and 10% cement replaced by Bagasse ash
20% BA	100% recycled coarse aggregates and 20% cement replaced by Bagasse ash

2.3 Specimens casting and curing:

Cylindrical moulds of 6 inches diameter and 12 inches height were used for casting of TSC. The inner surface of the mould was oiled, so that concrete should not adhere to its inner surface. A pipe of 1-inch diameter and 2-meter height was placed in the middle of a mould. In the second step, a mould was filled with coarse aggregates. In the third step, grout was injected from the top via pipe. The grout was poured under gravity pressure, which was created with the help of 2- meter pipe. This pressure was sufficient for filling the voids between coarse aggregates with grout. After the appearance of grout at the top of a mould, the pipe was removed from the mould. This procedure was used for all specimens. After 24 hours, specimens were taken out from the moulds and kept in a water tank. 72 specimens were prepared in total, each mix had eighteen specimens.

2.4 Test methods:

In this research, following tests were carried out on hardened concrete:
a) Compressive strength test was carried out according to ASTM C39/C39M-03. Tests on cylindrical specimens were done at 7,28 and 56 days.
b) Split tensile strength test was carried out according to ASTM C496-96. Tests on cylindrical specimens were done at 7,28 and 56 days.

3. RESULTS:

3.1 Hardened properties:

The results of compressive strength test of concrete cylinders at a given curing age are shown in Fig. 1. Each compressive strength is an average of three measurements. This figure shows that compressive strength at 56 days was higher than 28 and 7 days for all concrete mixes. This is because of an increase amount of hydration due to longer curing age. Nine concrete cylinders were casted for each concrete mix proportion. The compressive strength of CM-I was found to be highest for 7,28 and 56 days of curing. CM-II compressive strength is decreased by 22% when compared with CM-I at 7 days. This reduction of strength is due to recycled aggregates. Recycled coarse aggregates have inferior quality due to a porous surface caused by adhered mortar and high water absorption. The compressive strength of 10% BA and 20% BA at 7 days increases by 14% and 19% with respect to CM-II. This is due to the effect of pozzolanic reaction between $Ca(OH)_2$ and BA, which forms calcium silicate hydrate (C-S-H), enhancing the compressive strength as reported by (Martirena-Hernández et al., 2001). There was no issue of workability due to recycled aggregate because recycled aggregates were preplaced in formwork. The same pattern is followed at 28 days and 56 days. The increase in compressive strength was not expressive from 28 to 56 days. The normal compressive strength at 7 days is about 60-80% of 28 days in case of normal curing (Neville, 1996).
The results of split tensile strength of concrete cylinders at a given curing age are given in Fig. 2.The tensile strength of CM-I was highest at 7,28 and 56 days. The maximum increase in tensile strength at 56 days was 44.82% in comparison with 7 days of the same mix. This increase was in 20% BA. This was mainly due to the effect of pozzolanic reactions. The results follow the same pattern as that of compressive strength. This is because of similar reasons discussed in the case of compressive strength.

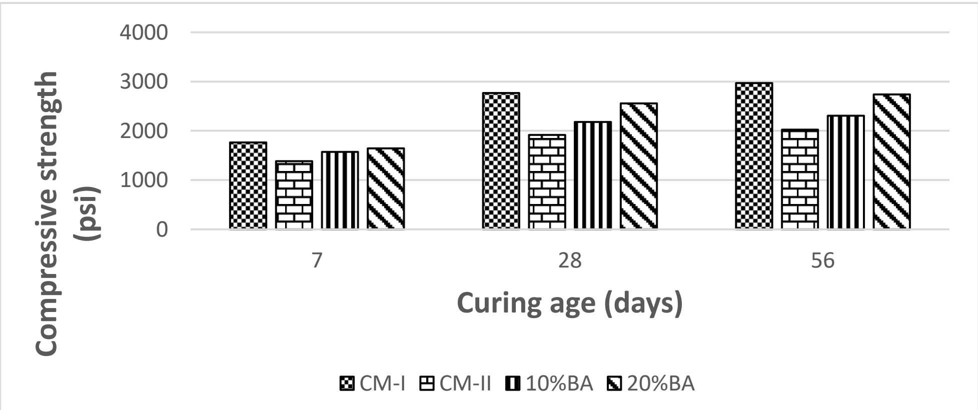

Figure 1. Curing age vs Compressive strength

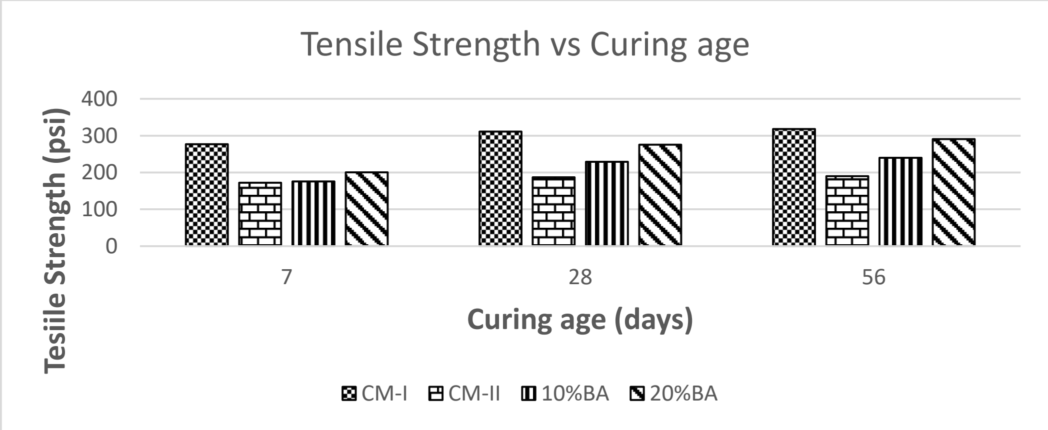

Figure 2. Curing age vs Tensile strength

4. CONCLUSIONS:

The following conclusions are drawn from results:

- Compressive strength of CM-II decreased by 22% when compared to CM-I at 7 days.
- Tensile strength of CM-II decreased by 38% when compared to CM-I at 7 days.
- Compressive strength of 20%BA increased by 35.35% when compared to CM-II at 56 days.
- Tensile strength of 20%BA increased by 52.77% when compared to CM-II at 56 days.
- Compressive strength and tensile strength of TSC decreased with recycled aggregates.
- Compressive strength and tensile strength increased with bagasse ash as fractional substitution of cement.

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Effect of Hybrid Fiber Reinforced Concrete on Strength of Concrete

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Abstract

The major chunk of construction is covered by concrete construction. Concrete is a mixture of cement, sand, crush and water in appropriate ratio. It is good in compression but weak in tension. In order to improve these strengths of concrete, this research was conducted to evaluate the effect of Hybrid Fiber Reinforced Concrete (HFRC) on strength of concrete. Steel reinforced fibers were used to improve the compressive and tensile strength of concrete and also to control the progress of cracks in concrete. Steel reinforced fibers concrete (SFRC) was produced having 5000 Psi as target strength. Two types of steel wires having 25mm and 18mm fibers were used. 1.25% of concrete volume was replaced with steel fiber having aspect ratio for 25mm fiber was 60 and for 18mm fiber it was 40. Different proportions of steel fibers from 25% to 100% for both types of fibers were used to evaluate their effect on strength of concrete. The main purpose of introducing fiber steel concrete is to eliminate the traditional shears stirrups in concrete members. Cubes and prisms were casted to test for compressive strength and tensile strength on concrete. It was observed that there is no significance effect of steel fibers on compressive strength of concrete while around 58.33% of tensile strength was improved because short length steel fibers controlled the propagation of cracks in concrete. The maximum results were achieved at 100% replacement of 18mm fibers. The results are helpful for building stakeholders to improve strength of concrete by using steel fibers in concrete. Further studies can be carried out to find out other properties of steel fiber reinforced concrete.

Key words: Hybrid steel fibers, compressive strength, flexural strength, Mechanical properties, Aspect ratio

1. INTRODUCTION

A stone like material is known as concrete which is attained by a warily balanced mixture of cement, sand, gravel and water. While in fresh state, concrete is a plastic which can be molded into any desired shape but with time it becomes hardened. Concrete develops micro cracks during curing. Cracks propagate in the fibers that are right under the load and these hybrid steel fibers block crack propagation. Due to dry shrinkage problem in concrete, formation of cracks also occurs and by elapsing of time increase in size and magnitude of cracks take place resulting in failure of concrete. (Maruthachalam et al, 2013). To minimize this phenomenon fibers are introduced as a new technique which helps to increase the tensile strength of concrete.
Concrete which contain fibrous material is known as Fiber Reinforced Concrete (FRC) which improves its structural strength. It incorporates quick isolated fibers which are

equally allotted and haphazardly oriented. Fiber reinforced concrete consisting of, cement, water, fine and coarse aggregate, along with discontinuous fibers. The small Piece of reinforcing material which own certain properties and minimize the propagation of cracks are called Fibers which are equivalently disseminated and arbitrarily arranged. This concrete is named as fibers reinforced concrete. (Grijaet al, 2016). The addition of fibers greatly reduced the post cracking behavior in concrete which improves structural integrity and cohesiveness of material.(Vandewalle, 2007)
Typical aspect ratio of fibers ranges from 20 to 100 while length dimensions ranges from 6.4 to 76mm (ACI 544.1R-96). The volume fraction (V_f) is a term, used to represent the amount of fibers added in the concrete mix as a total volume of concrete. The steel fiber volume fraction used in concrete typically ranges from 0.1% to 3.0% (Global Research Analysis). More than 1.0% of volume fraction generally decreases workability and fiber dispersion and need a special mix design or concrete placement techniques (Portland Cement Association).
Figure 1 shows the effect of long and short fibers on concrete. Short length fibers bridges micro cracks, control the growth of cracks and also delay the coalescence in fiber reinforced concrete. Long length fibers prevent the propagation of micro cracks, control the macro cracks also and then improve the fracture toughness of composites.

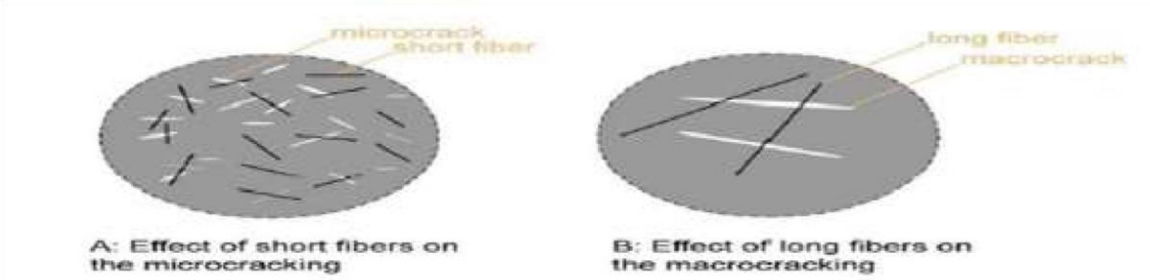

Figure 1: Effects of short and long fibers

An attempt was made to find out the effect of steel fibers in concrete. Steel fibers of length 18mm and 25mm was used in this research at different percentages to evaluate their effects on mechanical properties of concrete like compressive strength, flexural strength, ultimate load carrying capacity and ductility. Some positive effects were observed in concrete after adding hybrid steel fiber in concrete as compare to Plain concrete. The results of this study will helpful in selection of concrete with improved compressive and flexural strength and also with improved ductility. The greater compressive and flexural strength co concrete will help the design engineer to make their design more economical and safer.

2. MATERIALS USED

Ordinary Portland cement having Type-I manufactured by DG cement with fairly high CS content for good early strength development was used. Usually include natural aggregate with passing through a 9.5mm sieve. Source of Fine Aggregates was Lawrencepur having Specific gravity 4.81, Fineness Modulus 3.37 and 0.80% water absorption capacity. Similarly, source of Coarse Aggregates was Margalla having Specific gravity 2.53, aggregate size 12mm and 1.37% water absorption capacity.
Hybrid mild steel fibers having diameter 0.43mm were used in concrete having length 25mm and 18mm with 60 and 40 aspect ratios respectively. Portable water was used for this research and Chemrite 520 BA was used as water reducing and set retarding concrete admixture at the rate of 0.5 lit/50 kg cement. These fibers were added in

concrete at a volume fraction of 1.25% whereas no fibers were added in control mix (CM) specimen.

Table 1: Different Ratios of Fibers used

Mix specimens	***Steel fibers by volume of concrete (%)***
CM	Normal Concrete
Sample-I	100% (25mm)
Sample-II	100% (18mm)
Sample-III	50% (25mm), 50% (18mm)
Sample-IV	75% (25mm), 25% (18mm)
Sample-V	*25% (25mm), 75% (18mm)*

The concrete mixture design was carried out to find out values of ingredients. Trial mixture design was carried out first to find out the mixture ratios for required strength of concrete. For this research concrete mix 1:1.6:1.8 was used with 0.4 water to cement ratio.
Deformed high strength steel of 13mm & 16mm bars were used to provide longitudinal reinforcement in beams. 16mm bars provided at bottom of beams and 13mm bars provided at top of beams. For stirrups used 10mm bars for all the beams. In plain reinforced concrete beams stirrups are provided throughout the beam but in fiber reinforced concrete beams two stirrups are provided at its one end two stirrups are provided at its other end. Stirrups are provided to hold the top and bottom bar (Singh et al, 2016).

3. EXPERIMENTAL METHODOLOGY

3.1 Compressive Strength Test

This test was performed according to ASTM C39. To find out compressive strength of concrete the cylinder of size 300x150mm were used. Specimen were placed on bearing surface of UTM, of capacity 100 tones deprived of eccentricity and uniform rate of loading of 0.25 MPa per second was applied till the failure of cylinder. Machine gives compressive strength direct in MPa so no need to convert the compressive strength value (Ohitha et al, 2016).

3.2 Flexure Strength Test

Plain and SFRC beams of size 100x150x1200mm were tested using a universal testing machine. The loading scheme was two point. The beam was simply supported over a span of 970mm and a two-point loading system was adopted having an end bearing of 115mm from each support. A load was applied to stiff steel beam that distributed into two points and then from the two points load is transferred to beam specimen. The rate of loading applied on beam was 0.5 MPa per second. The load is applied till the failure of specimen. The first crack load and ultimate load are noted and deflection was measured using the dial gauge (Cho et al, 2009).

4. EXPERIMENTAL RESULTS

After the preliminary tests, various samples were casted included cylinders and beams to evaluate the properties of concrete with and without use of steel fiber in concrete. Cylinders were casted for the compressive strength test and beams were casted for the flexural strength test. These samples have various percentages of steel fiber in concrete.

The below figure-2 indicates the comparison of plain concrete and steel fiber reinforced concrete with all described percentages. It is clear from figure-2 that there is a significant improvement in compressive strength of concrete by adding steel reinforced fiber in concrete as compare to plain concrete. The addition of 25%, 75% and 100% steel fiber of 18mm and 25mm lengths have not produced large difference at 28 days. While among the steel fiber reinforced concrete, the addition of 25mm and 18mm long steel fibers at 50% yielded the maximum results as compare to all other options. Therefore, it is clear that this is the optimum percentage of steel fiber reinforced in concrete for the improvement of compressive strength.

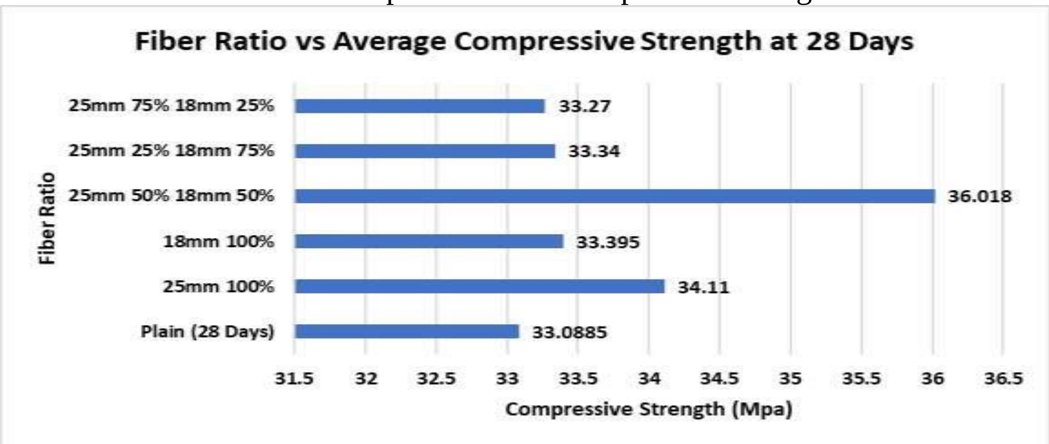

Figure 2: comparison of compressive strength for plain and fibrous cylinders

Similar trend was also found in concrete at age of 14 days. Where the addition of 25mm and 18mm long steel fibers at 50% yielded the maximum results as compare to all other options.

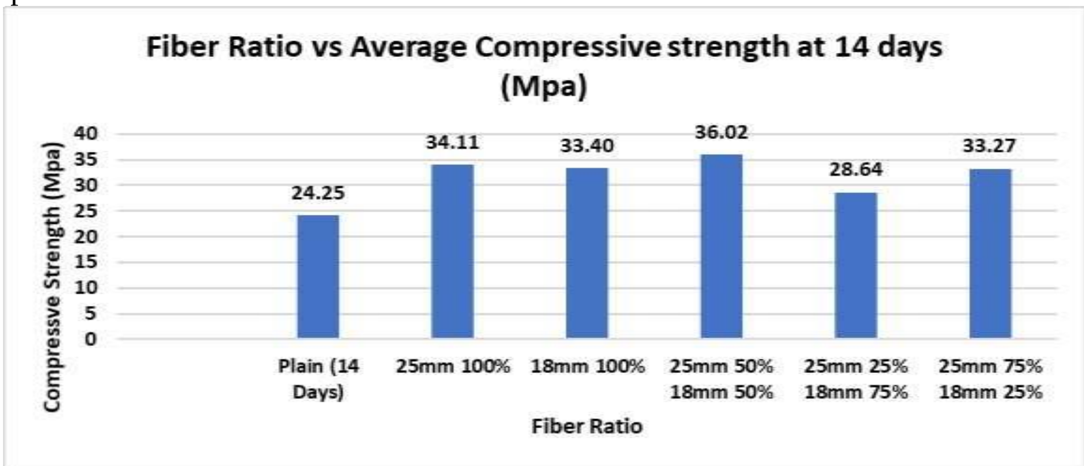

Figure 3: Compressive strength at 14 days

Similar trend was also found in concrete at age of 7 days. Where among the steel fiber reinforced concrete, the addition of 25mm and 18mm long steel fibers at 50% yielded the maximum results as compare to all other options. But the rate of gain of compressive with rest to time was not found here in steel fiber reinforced concrete. The compressive strength of steel fiber reinforced concrete remains constant at around 36.02 Mpa in different ages.

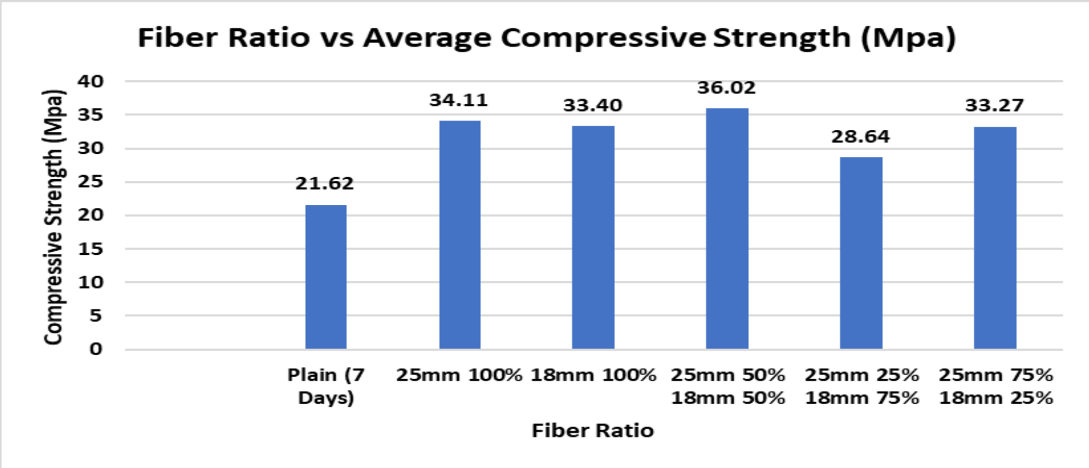

Figure 4: Compressive strength at 7 days

Deflection tests were performed on beams to find out the ductility of steel fiber reinforced concrete. It is clear from below graph that steel fiber reinforced concrete with 25mm long steel fiber with 100% replacement yielded maximum deflection as compare to other samples. It is also clear that there is significant improvement in deflection of steel fiber reinforced concrete as compare to plain concrete which shows that addition of steel fiber reinforcement improves the ductility of concrete and hence improve the warning before failure of steel fiber reinforced concrete structures.

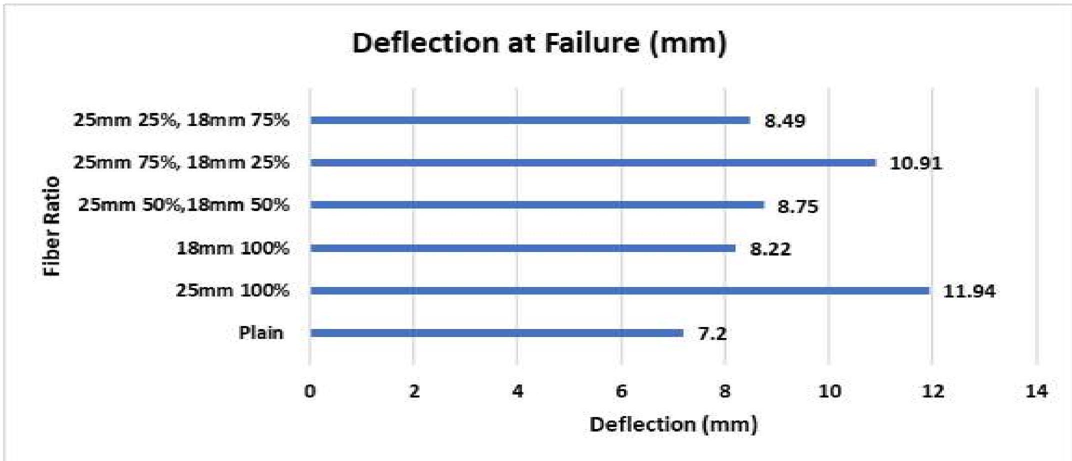

Figure 5: comparison of deflection at failure for plain and fibrous beams

5. CONCLUSION

Concrete is a good building material and widely used in construction all over the world. It has good compressive strength but it is very weak in tension which make it lesser durable against tension loading. In order to improve the tensile strength of concrete steel fibers used in this research has significant effects on all properties of concrete when compared to conventional concrete. Following are the major findings of this study.

a. Compressive strength of plain cylinder is 33.09 MPa. There is 8.13% improvement in compressive strength of concrete by adding steel fiber ratio (25mm 50% +18mm 50%) in concrete. Because long fibers have greater pull out resistance which improves the post cracking tensile strength of concrete and short fibers provides micro crack control at early stage so both long and short fibers combined to achieve the maximum compressive strength.

b. Flexural strength at first crack load in plain beam is 7.5 KN. As compare to plain concrete, around 58.33% flexural strength of concrete was improved by adding steel fiber ratio (18mm 100%). It is because short length fibers control the micro cracks in early stage of loading which leads to higher tensile strength of concrete so the first crack appear later on 18mm 100% beam sample.

c. Flexural strength at ultimate load in plain beam is 85 KN (Average of two samples). Around 8.11% improvement was found by adding steel fiber ratio (25mm 50%+18mm 50%) because when the beam was subjected to flexural loading the long fibers bridge the micro cracks and prevent the expansion of cracks and when long fibers fail than short fiber bridging cracks until its fail so combination of both long and short length fibers combine to achieve maximum flexural strength at ultimate load.

d. The variation in results of Load Deflection Curve is due to hand compaction of concrete and there is no use of vibrator for compaction concrete, due to this deflection is more in 18mm fibers rather than 25mm fiber but according to previous studies deflection should be more in long fibers as compared to short fibers.

e. The energy absorption in plain beam is 445 KN-mm. Around 36.49% improvement was found by adding steel fiber ratio 18mm 100% because deflection in case of this ratio also maximum so energy absorption also maximum. More the deflection more is energy absorbed by the beam sample.

6. RECOMMENDATIONS

1. Improvement in tensile strength and also in ductility of concrete is helpful in choosing steel fiber reinforced concrete structures specially in earthquake areas where tension and ductility is the basic requirement of structures.
2. This improvement inf tensile strength and ductility due to steel fiber reinforced concrete also helpful for the designers to reduce the member size and make the structure more economical as compare to traditional concrete.
3. Future research can be carried out by changing the aspect ratios and volume of fractions of fibers (such as 1.5% and 1.75%) and check the compressive and flexural behavior.
4. Analytical modeling of fiber reinforced concrete beam can be done.
5. Stress strain curve can be plotted and their behavior can be studied.

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Effect of Locally Available Water and Admixture on Compressive Strength of Concrete

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Abstract

Construction industry is considered as one of the biggest industries in the world. It mainly covers construction of roads, dams, highways, bridges, residential and commercial and high-rise buildings. Some useful construction materials including steel, concrete and wood are used for the construction of all the structures. Concrete is widely used construction material all over the world for the construction of different structures. Concrete usually contains cement, sand, crush and water which collectively regulate its strength and other properties. Water is the most important ingredient of concrete which takes most of the part in the functional properties that is why it is necessary to evaluate the quality and availability of water. In this study, the effect of quality of locally available water on the properties of concrete has been identified. The locally available water from Tap, Spring, Nula and Marble waste sources were used for the sampling. In first run the effect of water quality on strength of concrete was determined. In second step of sampling, the sample having maximum compressive strength in stage one was considered as benchmark and other samples were made by adding admixture to get the benchmark strength. In the first stage 120 concrete samples including 60 cubes and 60 cylinders were casted to check the effect water quality on compressive strength of concrete at different stages like 7 days, 14 days and 28 days. In this stage concrete made up of spring water yielded the maximum strength and concrete made up of Nula water yielded minimum strength. In the second stage fly ash was added as an admixture in concrete and casted same number of samples to check the effect of fly ash on concrete strength. It was observed that with addition of admixtures, concrete with Nula water and marble waste water yielded maximum strength as compare to that of tap water and spring water. Therefore, it is recommended that at the site where water is not easily available these types water can be used to prepare concrete by using some suitable admixture.

Keywords: Concrete Ingredients, Water sources, Water Quality, Compressive Strength and Admixture.

1. INTRODUCTION

Construction industry consists of five major sectors including Environmental Structures, Infrastructures, Residential Buildings, Commercial Building and Industrial Buildings. Concrete is mostly used construction material all over the world. The properties of concrete highly depend upon the properties of its ingredients like water, cement, crush and sand. An attempt is made here to investigate the effect of water quality on the compressive strength of concrete prepared by using locally available water in the studied area (Gilchrist, 1972).

For proceeding the work for different qualities of water that is Tap water, Spring water, Nula water, and industrial waste water are the best options chosen for practice because in our country various sources of water are available in different regions of the country but above-mentioned qualities of water are most feasible and easy to be collected for mixing the concrete. The water which is fit for drinking will be good for concrete but this criterion is not applicable in all condition like if water contains small amount of sugar or salt even then it would be fit for drinking but it will not be fit for concrete (Ofori, 1993).

The construction industry is a tool through which society accomplishes its objective of urban and provincial development (Horner, Marenjak, & El - Haram, 2002). It greatly affects the economy of all nations however it is at or close to the best in the yearly rate of business disappointment and coming about liabilities contrasted with different industries. This is a result of the complexities of the development procedure itself and the expansive number of parties involved in the development procedure i.e. customers, clients, originators, controllers, temporary workers, providers, subcontractors and experts. Construction industry is one of the greatest employers of the working population (Hooi & Leong, 2017).

Concrete is used in construction, which is made up of Coarse Aggregate, fine aggregate, binding material like Ordinary Portland Cement (OPC) and water. Concrete have good ability to bear Compressive stresses, which comes from the Structure self-weight and live loads but it Is weak in tension, Steel is use in concrete to resist the tensile stresses and it is also known as Reinforced Concrete cement. The coarse aggregate must be retained on sieve no (4.75mm) and fine Aggregate should be passing from sieve no (4.75mm) (Papadakis, Fardis, & Vayenas, 1992). Concrete is the oldest and common material which is use in construction. Concrete is mainly use due to it is in low cost and material which is use in concrete it is easily available. Steel is use in concrete to handle the tensile stresses. Volume of concrete is made up of around 75% of Aggregate and 15 % of cement. The Ph. Value of water that is going to be used in concrete must be in the range of 6-8. Water should not contain salt contents because it cannot make proper bonding between cement and Aggregate (Page & Vennesland, 1983).

Concrete have different properties at different state like slump value, Initial and final setting times, bleeding, segregation, consistency, pore pressure and freeze and thaw are the basic properties of fresh concrete in plastic stage while compressive strength, tensile strength flexural strength, shrinkage, creep elastic behavior and thermal behavior are the basic properties of hardened concrete. Admixture are used in concrete to controls the quality of concrete (Neville, 1995). Addition of admixture in concrete improves the properties of concrete. The addition of fly ash in concrete in the replacement of cement enhance the mechanical properties, chemical properties and also durability of concrete (Flatt, 2004). Similarly, the addition of super plasticizers in the concrete will also improve the compressive strength, workability, flexural strength, permeability of concrete (Criado, Palomo, Fernández-Jiménez, & Banfill, 2009).

The water in concrete is very important role in making of concrete, it is used in concrete to complete the hydration process. The quality of water is important factor in concrete. The water ph. value must 7 to 7.5 (Raki, Beaudoin, Alizadeh, Makar, & Sato, 2010)

Water functions as the single most important and critical factor influencing the workability or ease of mixing and placing the concrete. Moreover, water also controls the fresh and harden properties in concrete. These properties include compressive

strength of concrete, durability, cracking, permeability and workability (Flower & Sanjayan, 2007).
Concrete is well-defined as a combined tough material that is attained by the setting of a mixture of cement, aggregates, and water in standard detailed proportions. The discrete materials when mixed together forms a plastic mixture, which can be molded to any shape. Later, within a specified time period into hardens. Concrete is a numerous material with variable assets. The mixing ratio of concrete ingredients is changeable and depending upon the properties of constituent and mix design (Palacios, Puertas, Bowen, & Houst, 2009).
Water is the most important ingredient of concrete which takes most of the part in the bond formation that is why it is necessary to be studied regarding its availability and preference. Concrete is composed of coarse aggregate, fine aggregate and water. Cement is used as a binding material which forms a strong bond between all its ingredients. This binding power of cement is only being activated through the application of water in short the hydration of cement is only possible in its presence (Siddique & Chahal, 2011).
So many properties of concrete of concrete i.e. setting time, hardening time and strength can be affected by the quality of a mixing water. As the strength and durability of structures depend more upon the properties of concrete which is used in the construction of that structure (Wongpa, Kiattikomol, Jaturapitakkul, & Chindaprasirt, 2010).
Sea water usually comprehends 3.5salinity but faintly hurries the setting time of cement. This comprises approximately 78% sodium chloride and 15% chloride sand sulphates of magnesium and these chlorides which are present in concrete comprehending surrounded steel to steel corrosion. The chemical composition of different types of water are analyzed and it is concluded that the sea water and rain water had less strength due to their chlorides and some other constituents while the fresh water showed better results and achieved high strength concrete (Pangdaeng, Phoo-ngernkham, Sata, & Chindaprasirt, 2014)
As the world is suffering from the water scarcity that is why it is the need of the time to reuse the water from different sources like wash basin, kitchen floor wash and other sources. But the properties of concrete like setting time of concrete, compacting factors, slump, compressive and tensile strength properties are highly depending upon the quality of water. These properties are highly affected by the impurities in the mixing water. That is why pure and drinkable water must be used in concrete to achieve maximum (Le et al., 2012).

2. RESEARCH METHODOLOGY

The research is based on the experimental study. In the initial stage different preliminary tests were performed like dry density, bulk density, surface saturated dry density, water absorption, crushing value, initial and final setting time of cement, sieve analysis of cement, fine and coarse aggregates and consistency of cement. After performing the preliminary tests on ingredients of concrete, several trials were carried out to find the best suitable concrete mixture design for the required compressive strength of concrete. From each trial mixture cubes, cylinder and beams were casted to perform compressive strength, tensile strength and flexural strength tests respectively. Below table indicates the results of preliminary tests for the concrete mixture design.

Table 1: Properties of Material for Mix Design

Properties of Material for Mix Design	**Values with Units**	**Properties of Material for Mix Design**	**Values with Units**
Grade of concrete	M15	1 Gallon water	8.34 lbs
Nominal maximum aggregate size	19mm	Slump value	75 mm
Specific gravity of cement	3.15	Dry rodded weight of aggregate	109
Specific gravity of fine aggregate	2.69	Coarse aggregate moisture content	1.20 %
Specific gravity of coarse aggregate	2.63	Coarse aggregate water absorption	0.52 %
Density of cement	195 Pcf	Fine aggregate moisture content	5.20 %
density of water	62.4 Pcf	Fine aggregate water absorption	0.71 %
1 cubic foot water	7.48 Gal	Fine aggregate fineness modulus	3.71

After that procurement of materials, preliminary tests on material as per ASTM D-7332, mix design procedure as per ASTM C-94, sampling of concrete as per ASTM C-172 and curing of samples as per ASTM C-31was carried out. After the final concrete mixture design, ingredients of concrete were mixed as per the ASTM C-94 standards and then cubes and cylinders were casted and cured in the ponds for different time like 7-days, 14-days and 28-days. Total 120 samples comprised of 60 cylinders and 60 cubes were casted with and without addition of admixture for the compressive strength of concrete. Tests on fresh concrete included slump test and density & void content test were performed.

After prescribed time period, compressive strength tests were performed to find out the rate of gain of compressive strength at different age.

3. RESULTS AND DISCUSSION

After the detail and preliminary tests, 120 samples without admixtures and 120 samples with admixtures were casted to find out the effect of locally available water quality and admixture of compressive strength of concrete. It was observed that in initial samples where admixture was not added in concrete, spring water yielded the maximum strength as compere to other sample whereas concrete prepared with marble waste water and Nula water yielded the minimum strength. The ultimate strength that achieved from the concrete sample prepared y using marble waste water and Nula water was lesser than the target strength. These results are shown in the below figure-I.

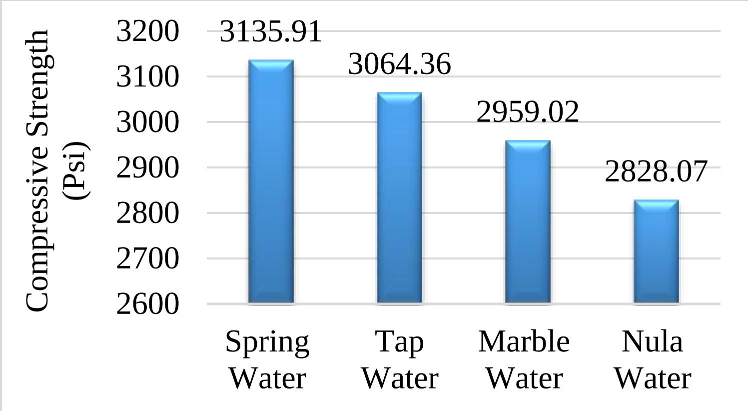

Figure 1: Analysis of Results of Cylinders of All Water Qualities at 28 Days

It is clear from the above figure that there is a clear difference in the compressive strength of concrete samples prepared with spring water and tap water as compare to marble waste water and Nula water. It is also clear from the above graph that compressive strength of concrete prepared with marble waste and Nula water is lesser than the target strength and hence it is not recommended to use these types of water to prepare concrete at any site without use of proper admixtures.

As concrete samples prepared with Nula water and marble waste water failed to provide required strength, therefore, an admixture was added while casting the samples of Nula water, Marble Waste Water and also tap water. Admixture which was used to improve the strength was Superplasticizers. Selection of this type of admixture was based on its availability and literature. This type of admixture is used to improve the compressive strength of concrete by reducing the water to cement ratio in concrete. Amount of admixture was calculated according to ASTM C-494 and also from the specifications provided by the supplier. Twelve cubes and twelve cylinders samples were prepared for each water quality and they were kept for curing for different time period. All of these samples were tested at time period of 7-days,14-days and 28-days to check the rate of gain of compressive strength at these intervals of time. Below figure-2 indicates the results of these samples.

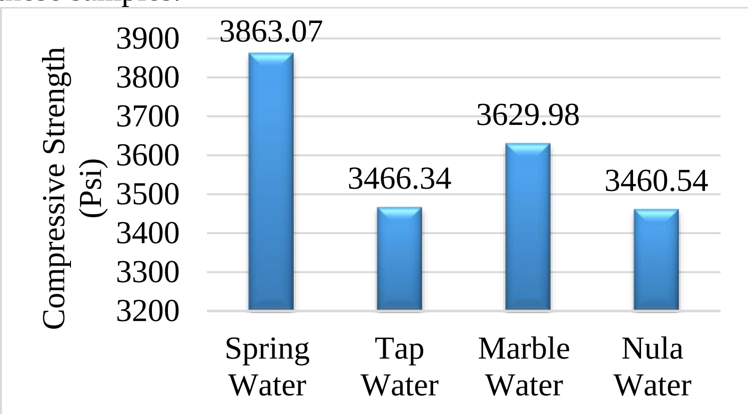

Figure 2: Analysis of Results of Cubes of All Water Qualities at 28 Days

It is clear from above figure that the strength of all the samples with addition od admixture has been improved but samples prepared with marble waste water has gained more strength as compare to tap and Nula water. There is an improvement of 18.54% in compressive strength of concrete prepared with marble waste water by adding admixture in it. Therefore, marble waste water can be used in concrete mixture by adding suitable admixture to achieve the target strength.

Below figure-3 indicates the rate of gain of compressive strength of concrete samples at different ages like 7-days, 14-days and 28 days with the addition of admixture in the concrete samples.

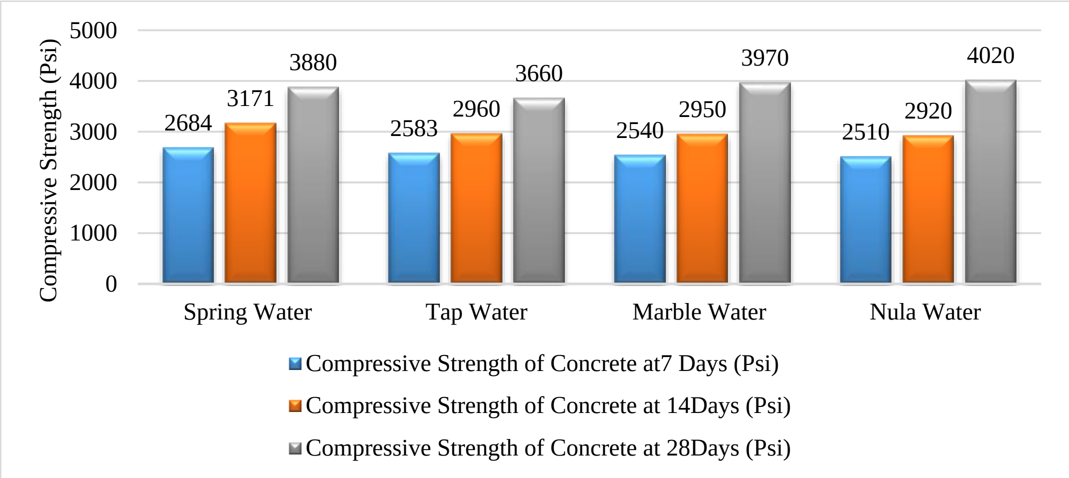

Figure 3: Compressive Strength of Cubes of All Water qualities with Admixture

It is clear from above figure that the maximum improvement in compressive strength of concrete is in concrete prepared with Nula water where concrete achieved maximum strength as compare to all other samples. But these cubes sample tests result also indicates the reasonable improvement of compressive strength of concrete with admixture.

Similarly, cylinders were casted, cured and then tested at different ages to check the improvement in the rate of gain of compressive strength of concrete prepared with these qualities of waters. The results are given below in figute-4.

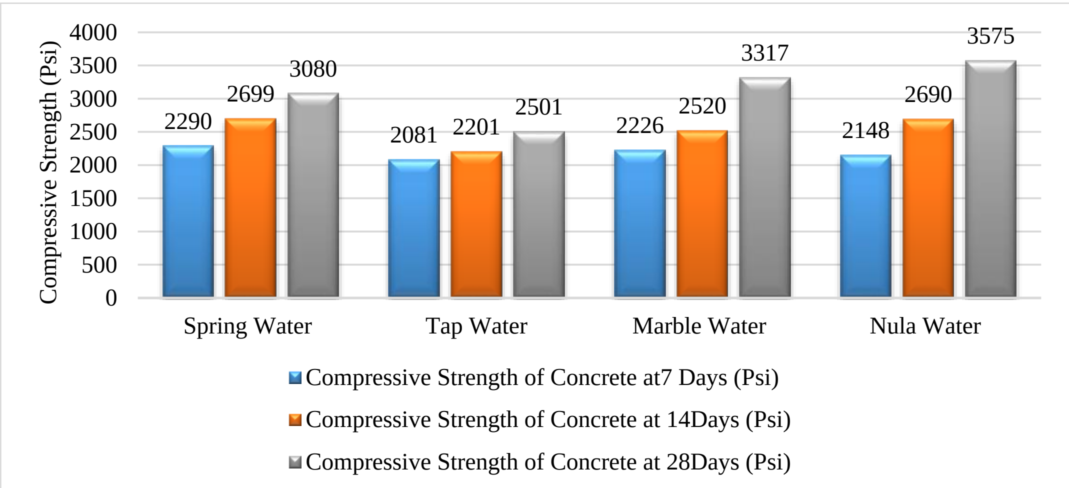

Figure 4: Compressive Strength of cylinders of All Water qualities with Admixture

It is also clear from the above graph that again the improvement in compressive strength of concrete samples prepared with Nula water yielded maximum strength as compare to all other samples. But it is also cleared that the rate of gain of compressive strength and also improvement in compressive strength of all samples is quite good. That is why it can be concluded here that at the construction sites where tap water or drinkable water is not available for concrete work any type of water can be used to prepared the concrete but with the addition of some suitable admixture.

4. CONCLUSIONS

Conclusions are made after the sampling and testing of concrete prepared with locally available water including spring water, tap water, Nula water and marble waste water. Sample were tested at 7-days, 24-days and 28-days to check the rate of gain of compressive strength also to check the effect of quality of water on concrete strength. It is concluded that spring water yielded maximum strength as spring water is the purest form of water that is why it provided suitable results. Compressive strength of concrete prepared with tap water, Marble waste water and Nula water yielded relatively lesser strength as compare to spring water respectively. It is also concluded that concrete having Nula and marble waste water yielded lesser than the target strength. Therefore, it is highly recommended that at any construction site concrete should not be prepared with these types of water. After the addition of admixture in concrete mix, Nula water yielded maximum strength as compare to all other samples. There was a significant increase in the compressive strength of concrete prepare with Nula and marble waste water with the addition of admixture. The rate of gain of compressive strength of concrete having Nula water and marble waste water was also improved as compare to spring and tap water. The results of both cylinder and cubes tests indicated that with the addition of admixture in concrete having Nula and marble waste water yielded maximum strength as compare to traditional tap water concrete sample. Therefore, at construction sites where tap water is not easily accessible, these types of studies helpful to provide the suitable replacement of water for concreting.
After analyzing the results of concrete samples of all water qualities with admixture and without admixture it was concluded that Compressive Strength of concrete which is one of the most important property of concrete and it is affected significantly by the water quality which is to be used in the casting of samples. Therefore, it is necessary to study water quality for its chemical properties regarding its suitability for making the concrete because it does not only have an effect on the strength of concrete but it also affects concrete quality after construction.

5. RECOMMENDATIONS

From the results and conclusions, the recommendations are made that spring water should be used in concrete if it is easily available to achieve better compressive strength as compared to tap water and other types of water. Marble waste water and Nula water can also be used in the concrete mixing by using suitable admixture. Marble Waste Water can also be used in preparation of concrete if Compressive Strength is to be achieved regardless its other effects on concrete after preparation as this is waste water and if it will be used in concreting then its effect on environment will overcome. Future research can be made on evaluation of chemical properties of water and their effect on the compressive strength of concrete. In future, the research can be done by exploring other qualities of water and check their effect on properties of concrete such as Tensile Strength, Flexural Strength etc.

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Effect of Hybrid Carbon Nanotubes/Graphite Nano Platelets on Mechanical Properties of Cementations Composite

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Abstract

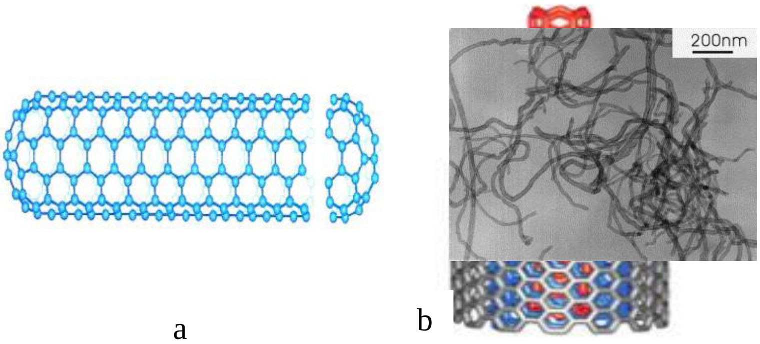
Nanomaterials and its application in construction industry attracted researchers to explore their effect due to exceptional properties in term of mechanical and have the potential of reinforcing within cementious matrix. Among them carbon based nanomaterial exhibit tremendous advantages in the construction industry. In this study hybrid intrusion of carbon nanotubes with graphite nanomaterials were added with small dosage/concentration ranging from 0-0.08% to surfactant ratio of 1:1 in the cement matrix, to explore mechanical properties in term of flexural and compressive strength is explored. The result reveals that using a small percentage of nanomaterials enhances the flexural strength and compressive strength up to 185% and 70% respectively.

Keywords: carbon nanotubes, graphite nanoplatelets, flexural strength, compressive strength

1. INTRODUCTION:

Cementitious materials are widely used construction materials, due to having high exceptional compressive strength. However these conventional cementitious material offer good compressive values, but still limits the tensile capacity, which makes them vulnerable to cracking. These cracks propagate form micro level, which then conjoin to microcrack leading to failure of cementitious material. To tackle the mentioned issues various materials and techniques have been explored by several researchers which include the incorporation of SRMs, well-engineered steel fibres, Carbon nanofibers, Carbon nanotubes, Graphene oxide and carbonaceous nano/micro inert, to alter the traditional properties of composites and achieve the required milestones. These nanoscale fibers arrest the propagation of crack at the nanoscale, causing enhancement in mechanical properties. These nanomaterials have distant and fruitful properties when used in cementious material. Among various nanomaterials, carbon-based nanomaterials have the most enlightened properties. Small concentration increases the mechanical response of cementitious composite. Konsta-Gdoutous et al. used carbon nanotubes with small concentration of 0.08% in cement paste and reported 35% increase in flexural properties (Konsta-Gdoutos et al. n.d.). Wang et. al. used MWCNTs in cement paste and reported increase flexural and compressive strength up to 10% and 50% respectively (Wang et al. n.d.). Gong et.al incorporated GO sheets with small

dosage of 0.03% in cementitious composite and reported 46% and 25% enhancement in compressive and flexural response. This is due to the refinement of pore size (Gong et al. 2015).
The properties of Carbon nanotubes can be influenced by its aspect ratio, diameter as well as from its chirality. Historically Carbon nanotubes were discovered in 1991 by sumio Lijima (Japanese Researcher) while investigating the surface of graphite electrodes as sown in Figure 1 (nature and 1991 n.d.) . This discovery of CNTs leads to a new dimension which opened a path towards utilization of CNTs in numerous fields of engineering. Based on its characteristics, properties and its production CNTs is used in numerous viable applications like rechargeable batteries, sporting goods and automotive parts (Volder et al. n.d.).



a b

Figure 1: (a) Carbon nanotube (Breuer and Sundararaj 2004) (b) Schematics image and TEM image of MWCNTs (Breuer and Sundararaj 2004)
(Li et al. n.d.)

Graphite nanoplatelet (GNP) is another form of graphite nanomaterials (GNMs) mainly carbon-based conductive nano-particles which is produced from graphite. Normal graphite consists of stratified layers including series of two dimensional (2D) graphene layers stacked together in the parallel form (Pierson 1993) as shown in Figure 2. However, graphite nanoplatelets can be obtained via exfoliation and intercalation of graphene layers.

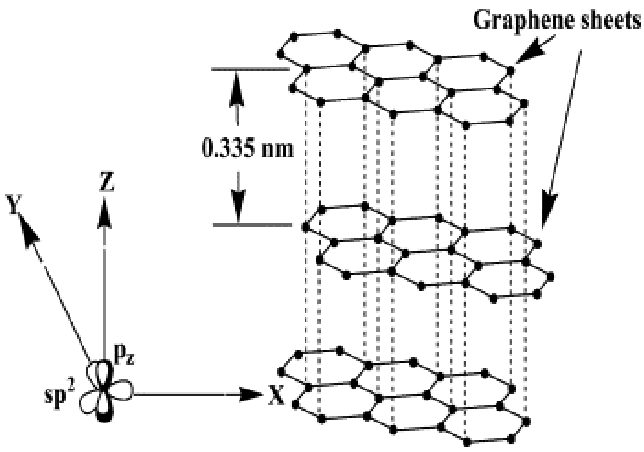


Figure 2: Structure of Graphite showing the sp2 carbon atoms bounded in hexagonal rings (Sengupta et al. n.d.)

Graphite nanoplatelets (GNPs) consist of many graphene layers bonded together with a Van Der Vaal forces, having a thickness in nano-meter and diameter ranges in microns. There nanoscale size, when used in cementitious composite, enhances the mechanical properties.

In order to improve the mechanical properties of composites with nanomaterial addition, it is mandatory to properly disperse it inside the matrix. The problem normally faced with improper dispersion of nanomaterial is the agglomeration and bundling of tubes inside the matrix for which different surfactants need to employ to make the nanomaterial homogenously disperse through cement matrix. In the continuation of mentioned topic an investigation on the dispersion of nanomaterial and its effects on the mechanical response of cementitious matrices (Sobolkina et al. n.d.), concludes that proper dispersion leads to 40% enhancement in compressive resistance. In most of the research investigations, it has been emphasized those nanomaterials agglomerates within the cement matrix without giving proper attention to dispersion (Luo, Duan, and Li 2009; Konsta-Gdoutos et al. n.d.). In this research dispersion of nanomaterials was done using acacia gum as surfactant with nanomaterial to surfactant ratio of (1:1), as it yields maximum dispersion.

In this paper, hybrid intrusion carbon nanotubes with graphite nanoplatelets in cement mortar, with dispersion and exploration of mechanical properties in term of flexural and compression has been reported.

2. MATERIALS AND EXPERIMENTS

Type 1, Grade 53 Cement in line with ASTM C150 was used as binding material. The features properties of cement/binder can be seen in Table 1. Sand was obtained from lawrenecpur, with fineness modulus of 2.4 and superplasticizer obtained from BASF chemical for making cementitious mortar. The Multi-wall carbon nanotubes and graphite nanoplateletes used in this research were purchased from US Research Nanomaterials and Deijung Company and their properties are listed in Table 2. Acacia gum used as surfactant for proper dispersal of nanomaterials in water. The features properties of superplasticizer are listed in Table 3.

Table 1: Properties of OPC

Elemental Composition	Content (%)
CaO	65.11
SiO_2	19.17
$Al2O_3$	4.96
Fe_3O_4	3.21
MgO	2.23
MnO + K_2O	0.55
TiO2	0.28
P_2O_5 + Na_2O	0.64

Table 2: Properties of MWCNTs and GNPs

MWCNTs Properties	External diameter (nm)	Internal diameter (nm)	Length (μm)	Purity (%)	Specific surface area (m^2/g)	Ash content (wt.%)	Density (g/cm^3)
	20-30	5-10	10-30	>97	110	<1.5	2.1

GNPs Properties	Specific surface area (cm^2/g)	Particle size analysis (µm)	Specific gravity
	154	6.78	1.62

Table 3: Properties of third generation superplasticizer

Master Glenium ®51	Aspect	Relative Density	pH	Chloride ion content
	Light Brown Liquid	1.08 ± 0.01 at 25°C	≥6	<0.2%

2.1 Mixing regime of nanocomposite cement mortar

The homogenous solutions of graphite nanomaterials (GNM's) after dispersion were used to make nanocomposite cement mortar. The water to binder ratio of 0.38 was selected for all formulation and binder to sand ratio of 1:1.5 was used for casting of cement mortar. The mixing of all formulation was done by using Hobart mixer of 5 litre capacity. After mixing, and for the evaluation of mechanical properties, prisms mould having dimensions of $160x40x40mm^3$ were casted. The details of formulation and the mixing time taken by each formulation has been listed in Table 4 and Table 5

Table 4: Formulation regime

S.No.	Formulation	CNT (%)	GNP (%)
1	CS	0	0
2	C	0.08	0
3	CG	0.04	0.04
4	G	0.	0.08

* CS control sample, C carbon nanotubes, CG hybrid carbon nanotubes/graphite nanoplatelets, G graphite nanoplatelets

Table 5: Mixing regime

Mixing regime	Time of mix
Dry mix	Half minute (gentle mix)
Dry mix + Dispersed GNM's	1 minute (gentle mix)
Dry mix + Dispersed GNM's	2 minutes (fast mix)

3. RESULT AND DISCUSSION

3.1 Dispersion

The dispersion of nanomaterials was checked by using Uv-spectroscopy. The dispersed solution of nanomaterials was diluted by using lambert beer law, before using spectroscopy. The wavelength was kept between 200-1100 nm and 500 nm wavelength was kept to check the dispersed solution of nanomaterials, as it is unaffected at ambient conditions (Baloch et al. n.d.). It can be seen that graphite nanomaterials (CG) having small concentration with GNMs to surfactant ratio of 1:1 yield maximum absorbance. This is due to the synergistic effect of nanomaterials as shown in Figure 3.

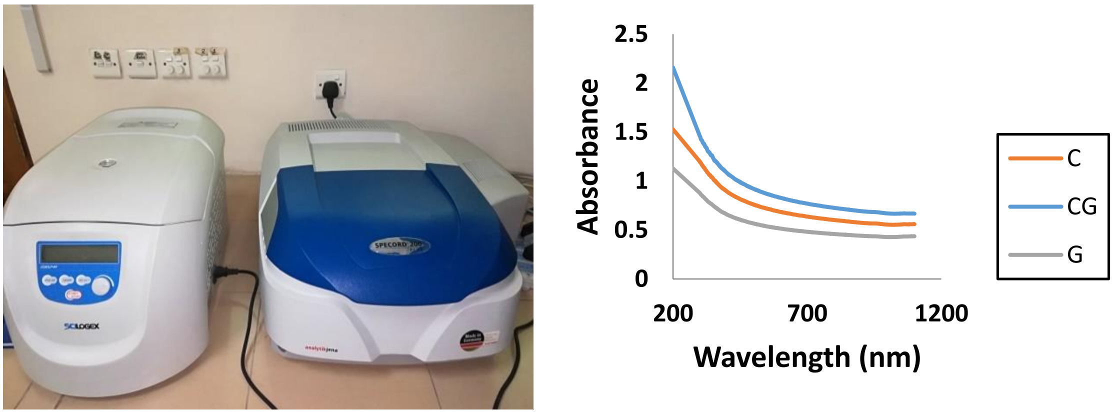


Figure 3: UV-Spectroscopy of nanocomposite

3.2 Flexural Strength

The flexural strength of all prisms was performed according to ASTM C348, under bending setup on load controlled machine as shown in Figure 4. The flexural test was performed after 28 days of curing. It can be seen in Figure 5 that the flexural strength of nanocomposite increases as compared to the control sample. Also, concentration (CG) increases the flexural strength up to 185% compared to control specimen. This is due to the synergistic effect, which enhances the microstructure causing an increase in load carrying capacity

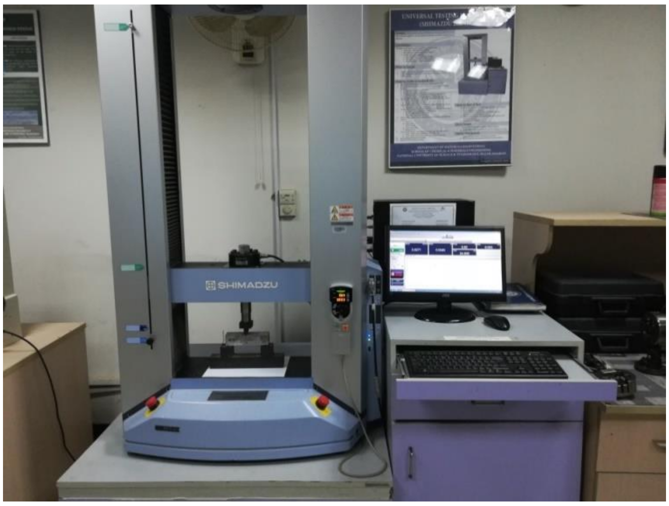


Figure 4: Bending test assembly

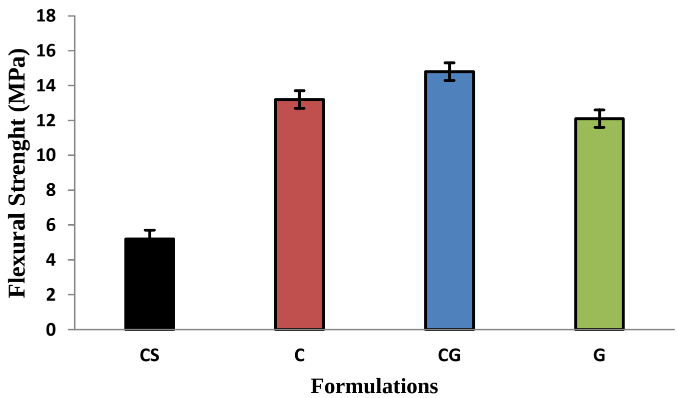


Figure 5: Flexural response of nanocomposite at 28days

3.3 Compression Strength

The compression test was performed according to ASTM C349-14 on two broken pieces of prism after the flexural test as shown in Figure 6. It can be seen in Figure 6 that the compressive strength of nanocomposite (CG) increases up to 70% as compared to control sample, as these fiber enhances the load carrying capacity between fiber to fiber and with the adjacent matrix.

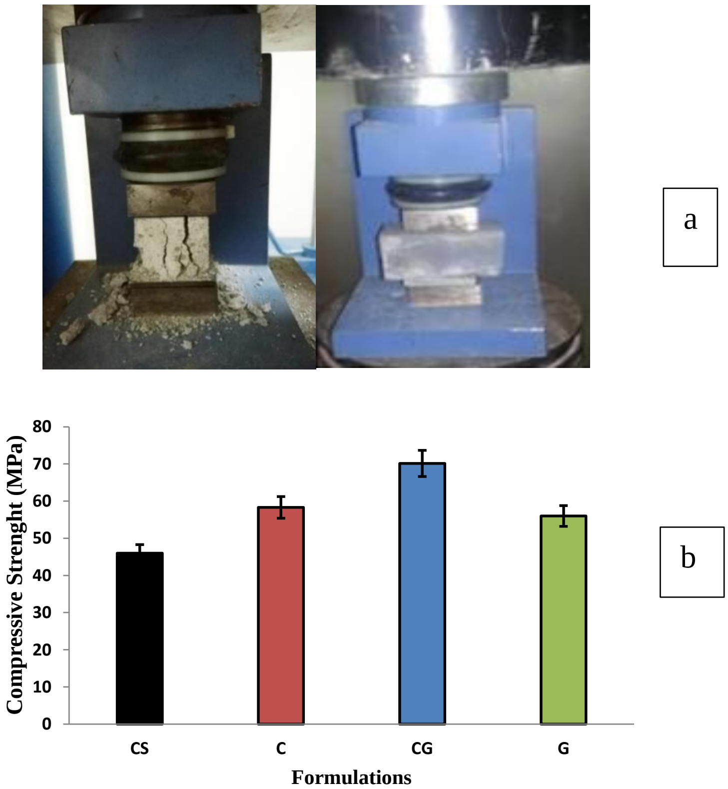


Figure 6 (a) Assembly of compression test, (b) Compressive strength of nanocomposite at 28days

4. CONCLUSIONS

From the study and research regarding nanomaterials, following conclusion has been drawn by using small concentration of nanomaterials in cementitious mortar.

1. Dispersion of Hybrid CNTs/GNPs (CG) nanomaterials with nanomaterials to surfactant ratio of (1:1) yield maximum dispersal. This is due to the synergistic effect, as these both fibers belong to the family of graphite nanomaterials
2. Intrusion of hybrid CNTs/GNPs (CG) by small dosage of 0.04/0.04% in cementitious mortar increases the compressive strength up to 70% compared to control specimen. This is due to enhancement in load carrying capacity
3. Intrusion of hybrid CNTs/GNPs (CG) with small dosage increases the flexural response of nanocomposite of hardened mortar sample. The bending test elucidated that using small concentration increases the flexural response up to 185% as compared to the control specimen.

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Effect of Water to Cement Ratio and Curing Condition on Compressive Strength of Recycled Aggregate Concrete

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Abstract

The waste material produced from the demolition of concrete structures every day throughout the world. This concrete waste includes the recycled aggregates and the best way of use of this waste is to use as coarse aggregates in the production of fresh concrete. The millions of tons of coarse aggregates is use for the production of concrete and the rocks are the source of aggregates (whether natural or broken)..In the new production of concrete the recycled aggregates is used as the coarse aggregates. In this research work from the experimental work performed on the recycled aggregates concrete and the compressive strength of the recycled aggregates was determined by using the different water to cement ratios. For to determine the compressive strength of the recycled aggregate concrete the Lawrancepur sand is used as a fine aggregate, from a demolished concrete the recycled aggregates is collected and used as coarse aggregates and the DG Cement is used as a binding material. The different water to cement ratios has a effect on the recycled aggregates concrete compressive strength. W/C 0.50, 0.55, 0.60 was examined in our research work and the results found that W/C 0.50 gave greater compressive strength. The results show that submerged treatment gives greater compressive strength compared to the coagulant because in the case of submerged treatment, there is no loss of moisture from concrete samples and enhances water reaction. Therefore, to achieve high pressure is recommended submerged treatment. The properties of recycled aggregates was determined and compared with the properties of natural aggregates. After the experimental work it was determined that the water absorption capacity of the recycled aggregates was more as compare to fresh aggregates due to the attachment of cement particles.

Keywords: Recycled concrete, Water cement ratio, Aggregate, Compressive strength.

1. INTRODUCTION:

The concrete which is produced by the use of natural aggregates has better quality as compare to the concrete which is produced by the use of recycled aggregates as coarse aggregates. Because the recycled aggregates have greater water absorption capacity due to the attachment of cement particles with the aggregates and have a porous mortar matrix around the surface of aggregates this make the lower bond in concrete. The quality of the recycled aggregates is low as compare to natural aggregates, however the recycled aggregates have been used in different construction works for the cost benefit analysis and to reduce the waste material produced from demolished of concrete structures.

The properties of recycled aggregates are low so the use of recycled aggregates is limited and used for only low strength concrete and low grade concrete. And if the recycled aggregates have good properties then can be used in the production of high quality concrete such as structural concrete. The research and the experimental work provided good guidance on quality control of recycled aggregates and then use in high quality concrete.

2. EXPERIMENTAL PROCEDURES:

2.1 Test Standards

All the experimental work performed by the used of ASTM standard.

2.2 Test Performed on Concrete

- Slump test
- Temperature test
- Compressive strength test

a)

b)

Figure 1: View of a) RCA in Boulder and b) Crushed form

2.2 Research Methodology

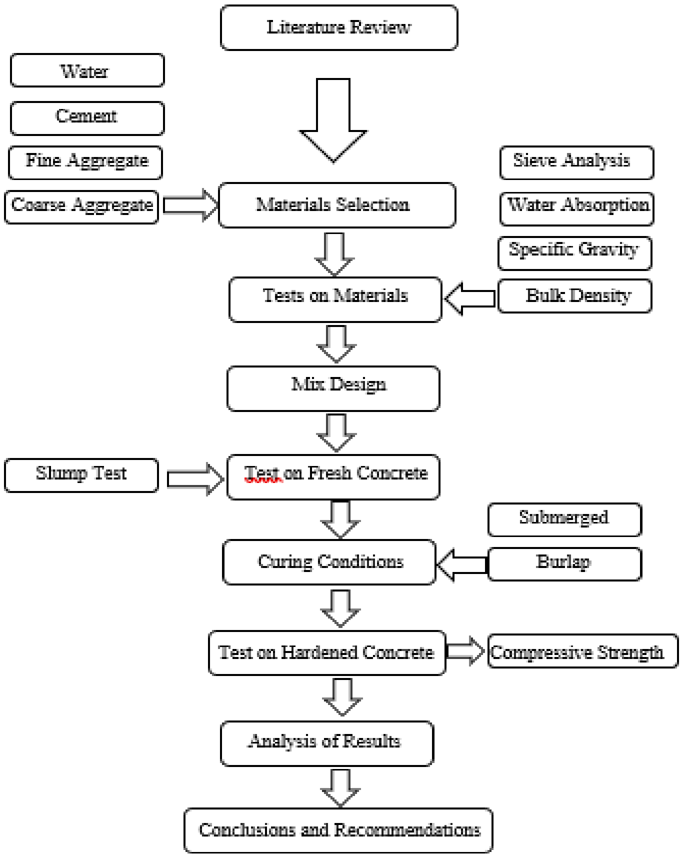

Figure 2: Flow Diagram of Research methodology

2.3 Slump Test

On the fresh concrete the slump test was performed according to ASTM C-143 procedure for or to check the slump value of fresh concrete the slump cone apparatus was used. The slump cone was filled in three equal layer with considering height of the slump cone and was compressed with 25 strokes using the steel tamping rod after each layer. The diameter of the temping rod is 5/8in and have 24 inches long. The slump value gives the indication about the water content and ease in working on concrete.

Figure 3: Slump test

2.3.1 Procedure

- The slump cone base plate is provided on flat surface, the base plate is lubrication is provided and the base plate is made of steel.

- The slump cone is filled up to 1/3 level and then provided the 25 equal strokes by the used of tamping rod.
- Then in second layer the slump cone is filled to 2/3 and the 25 strokes equally provided by the used of tamping rod
- Then the last layer upto the top level of the slump cone was filled and then removed the concrete outer on the top of the slump cone and provided the 25 strokes equally on top layer of the concrete.
- Then the slump cone was removed carefully in the vertical direction (it takes about five seconds).
- Then the slump cone is placed near to the sloping concrete, then place the temping rod horizontally across the slump cone and the sloping concrete, and measured the slump value in inches by the used of scale. Figure 3.8 above is shown that the slump test of fresh concrete in inches.

2.3 Compression Test

Due to the high compressive strength of concrete it has commonly used to carry compressive loads. ASTM C 39 specifies is the standard test method for to check the concrete compressive strength. The cylinder size of 6”×12’ was used to check the compressive strength. The uniformly load applied on the specimen. The compressive strength of the specimen was determined at 7 and 28 days.

Figure 4: Compression Testing setup

2.3.1 Procedure

- The bearing plates provided on the platen of the machine with hard face up and wipe clean the upper and lower plate of the machine.
- The specimen with accurate alignment was placed in the compressive strength apparatus and then at the centre of the specimen between the plated the load was applied.
- The apparatus applied the uniform load on the specimen with the constant loading value of 35$\pm$ 7 psi/s.
- The apparatus was applied compressive load on the specimen until the load was reduced steadily and specimen displays a well define load fracture pattern.

3. RESULTS:

3.1 Strength at different Curing conditions

On 7 days curing in different conditions the results are being compared as shown in figure above. In submerged condition the strength is greater as compared to BURLAP curing.

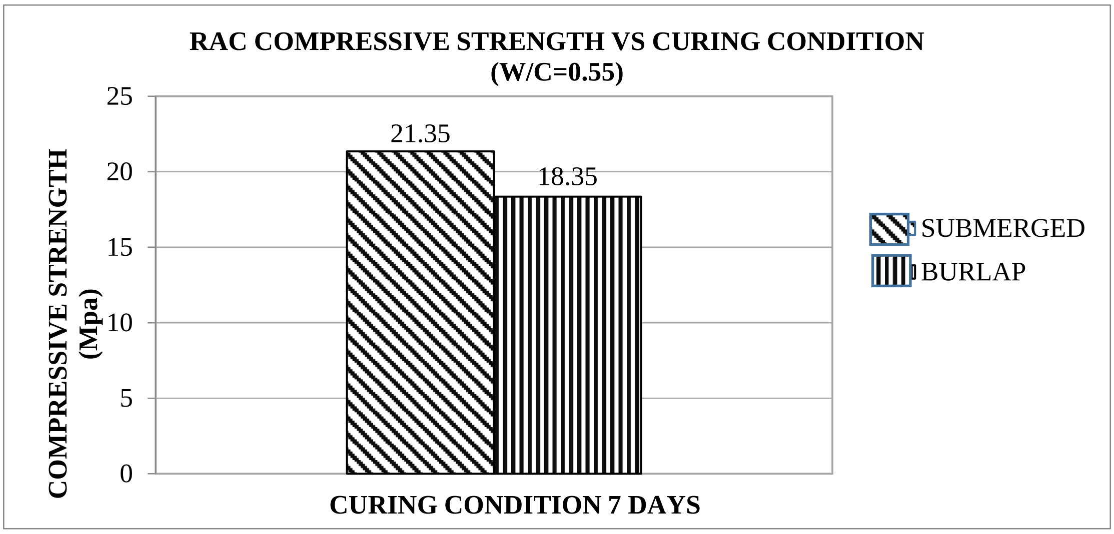

Figure.5 Strength vs Curing Condition for RAC at 7 days

In figure no 6 there is a compression of the strength of 28 days curing. It can be clearly observed the strength of the submerged curing in competitively more than the other curing.

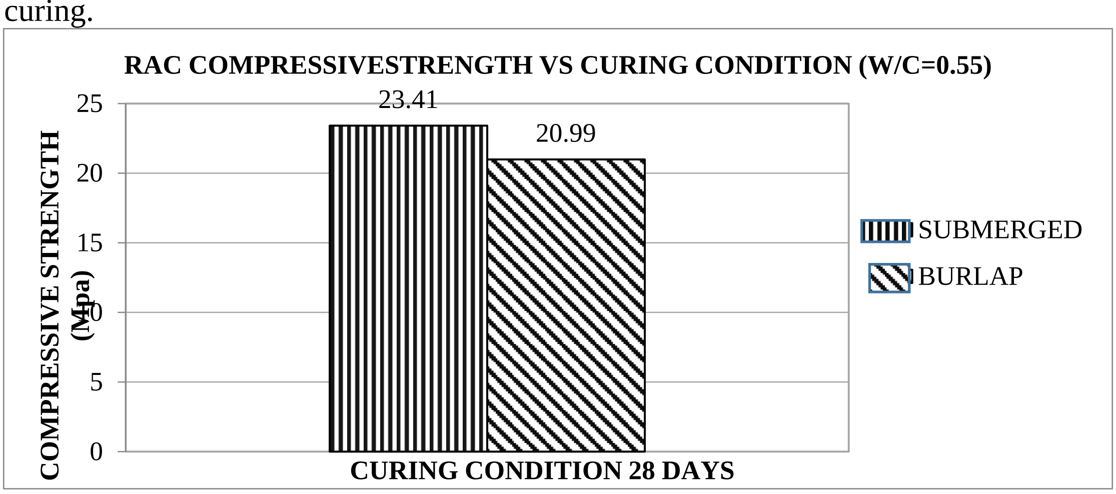

Figure.6 Strength vs Curing Condition for RAC at 28 days

4. FUTURE RECOMMENDATIONS:

- For this project the strength should also be checked for steam curing condition.
- For 0.50 w/c ratio where the concrete was not workable, admixtures should be used to increase the workability.
- At the same water cement ratio, the workability of Recycled concrete is higher than that of control concrete. The reason for which is the smooth impervious surface of recycled aggregate.

5. CONCLUSIONS:

The results of experimental work have been analysed and conclusions drawn from this study are presented below.

- For the same w/c ratio and changing curing condition the compressive strength obtained by submerged curing at 28 days is 11.5 % higher than burlap curing.
- For the same w/c ratio and changing aggregate type the compressive strength obtained by natural aggregate at 28 days is 7.13 % higher than recycled aggregates.
- The highest compressive strength was obtained by immersion of specimens in water tank, submerged curing gives more strength as compared to burlap curing because of enough water for hydration process.

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Use of Banana Fibres in Concrete to Mitigate Shrinkage-Crack Propagation in Concrete Roads

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Abstract

Sustainable development of nation’s infrastructure in developing countries is a major challenge in this era. There is an emerging trend of using locally available natural fibres in structural applications to support sustainable development in these countries. Banana fibre is abundantly available and can be utilized in concrete roads. These roads are prone to hair line micro cracking i.e., shrinkage cracking, instigated by drying shrinkage due to volumetric changes in concrete during the curing. The overall aim of this study is sustainable development of concrete roads by using Banana Fibre Reinforced Concrete (BFRC). And the specific aim is to investigate the tensile behaviour of BFRC in comparison with conventional plain concrete (PC). Mix design of 1:2:4 is adopted and 0.5% banana fibre by mass of concrete is used in BFRC specimen. Standard cylinder specimens (100mmx200mm) of PC and BFRC each are casted and cured for 28 days. Split tensile test is performed on these specimens. BFRC depicted decreased tensile strength and energy absorption as compare with PC. On the other hand, BFRC showed an immense increase in toughness value. This improved behaviour of BFRC toughness can help in reducing the resulting shrinkage cracking propagation. It is recommended that, mechanical properties of BFRC in addition with so other strengthening admixtures and its usage as a commercial product should be explored in depth.

Keywords: Shrinkage cracking, Crack propagation, Banana Fibre Reinforced Concrete, Split tensile strength, Tensile energy absorption, Bridging effect.

1. INTRODUCTION:

Concrete pavements are durable structures and preferred more over flexible pavements. Concrete pavements provide more strength. These pavements need less maintenance. But these are also prone to number of pre mature failures. Shrinkage cracking is one of these premature failures, which later on results in various distresses of pavements and ultimately lessens the design life of the structure (Kumar et al. 2012). These cracks are also responsible for creation of chain and transverse cracking in concrete pavements (Niken et al. 2016). Figure 1(a), (b) and (c) shows shrinkage crack propagation after six months, one year and two year of construction, respectively. The thickness of the lines indicate the width of cracks. These issues are resolved by many researchers by use of fibres to lessen the impact of damage caused due to poor construction practices.

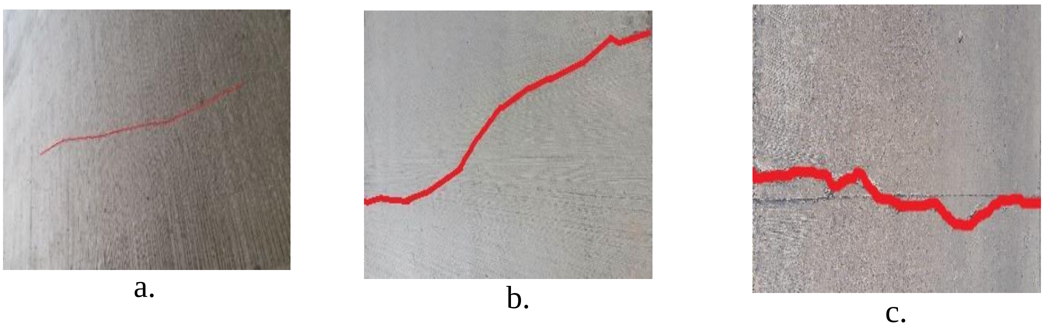


Figure 1: Shrinkage-crack propagation, a. after six months of construction, b. after one year of construction and c. after two years of construction

Rai and Joshi (2014) conducted a research to study the effect of fibre reinforced concrete on the shrinkage cracking in comparison with conventional concrete. They concluded that fibre reinforced concrete showed resistance against shrinkage cracking as compared to plain concrete. Fibres like polypropylene in restrained concrete can lessen the drying shrinkage percentage (Yousefieh et al. 2017). Use of mineral fibres and nano silica showed improved behaviour in mechanical properties of concrete (Larisa et al. 2017). Debate on natural fibres all over the world is increasing. Utilization of natural fibres as a source of raw materials for the different industries due to their eco-friendly nature and for accomplishing the idea of global sustainability has been increased. Experts are looking for the new economical, environment friendly and sustainable techniques in construction. Researchers are focusing on agricultural waste as one potential of source energy. Today throughout the global, increase in awareness about issues of environment is acting as the strong motivation for the utilization of natural fibres in various kinds of industries. However, there is a changing trend in use of natural fibres in structural applications. Banana fibre is abundantly available and can be utilized in concrete roads. Mostafa et al. (2015) expressed that banana fibres can be utilized in compressed earth block creation, and they reported improved compressive and flexure strength of banana fibre reinforced specimens in comparison with unreinforced specimens. Binici et al. (2005) reported that in an immense amount of agricultural by-product created each midyear in Turkey. Agriculturists consume it, causing environmental harm. Rather than being scorched, agricultural fibres can be utilized as a part of mud block creation. In the same way, banana fibre is abundantly available as an agricultural by-product in Pakistan and being used as export product. Sakthivel et al. (2019) did research study on the mechanical properties of banana fibre reinforced concrete. Apart from enhanced mechanical properties, they reported banana fire was found good in fire resistance. Keeping in mind, enhanced properties of banana fibre reinforced concrete, they recommended to explore its future prospects for construction industry as an environment friendly material. To the best of author's knowledge, the research conducted to control shrinkage cracks by using BFRC in concrete roads is limited. Therefore an experimental study is planned to investigate the potential of BFRC in concrete roads. The overall aim of this research is sustainable development of concrete roads by using Banana Fibre Reinforced Concrete (BFRC). And the specific aim is to investigate the tensile behaviour of BFRC in comparison with conventional plain concrete (PC). Percentage of banana fibre used is 0.5% by mass of concrete is used in BFRC specimen.

2. EXPERIMENTAL PROCEDURES:

2.1 Raw materials:

Raw materials which are used to prepare specimens comprises of ordinary portland cement, sand having fineness modulus of 2.6 and specific gravity of 2.63, aggregates of maximum size 19 mm, and banana fibres of maximum length 50 mm.

2.2 Mix design and casting procedure:

In the mix design ratio of PC, the ratio of 1:2:4 is used for cement, sand, and aggregate respectively with a water-cement (W/C) ratio of 0.6. The mix design and (W/C) ratio for BFRC is equivalent as compared to PCC irrespective to that 50 mm long fibres having 0.5% proportion, by mass of concrete. All quantity of materials is mixed by mass. For preparation of concrete rotating drum concrete mixer is used. In making plain cement concrete, raw materials including water are transferred in the drum of the conventional concrete mixer following the sequence of aggregates first, sand second, cement third and water in the last, and it is rotated for three minutes. Slump test is performed before filling moulds and the result of slump is 2-cm.

For preparation of banana fibre reinforced concrete, an alternate approach is embraced to avoid the accumulation of banana fibres in blender bars of concreting drum. The materials are placed in drum in three layers. In first layer, 33% of total aggregates is spread in the drum followed by cement, fibres and sand. In this way, fibres are sandwiched between aggregates and sand. A similar strategy is embraced for the remaining material. The blender is halted at the point where addition of layer is to done and remaining water is included. Concrete mixing machine is rotated again for 1 minute.

2.3 Specimens:

The cylinders were casted for conduction of split-tensile strength test for PC and BFRC; having standard size (100mm x 200mm). Total 4 specimens were casted (2 PC + 2 BFRC). For BFRC, filling and compaction of the mix in the moulds requires special consideration. Therefore, another approach is adopted to remove air voids and to achieve self-compaction, in addition to standard compaction method. The mould lifted up to a height of 200mm approximately and afterward dropped to the floor. After 24 hours, the specimens are de-moulded and placed in the curing tank for 28 days. The density of PCC and BFRC is determined by using ASTM C138/C138M-16 procedure.

2.4 Split-tensile strength test:

The split-tensile strength is performed on the test specimens as indicated by the ASTM standard C496/C496M11. The split-tensile strength, split-tensile behaviour, split-tensile energy absorbed before crack and split tensile toughness are determined.

3. RESULTS:

3.1 Split-tensile behaviour:

Load versus time curves under split tensile loading for PC and BFRC are shown in

Figure 2(a). The crack development in specimens under split-tensile loading at initial, peak and ultimate loads is presented in Figure 2(b). The upper two images i.e., Figure 2(b) demonstrate the underlying crack in PC and BFRC. The underlying crack in the specimens of PC and BFRC is noted at 100% and 95% of their comparing maximum loads. The underlying crack length and width in BFRC is lesser when contrasted with PCC. The length of initial crack around 60 mm to 75 mm is observed in BFRC. At this point, the separated pieces of PC can be around the first crack observed with no time distinction, while pieces of BFRC are held together as a result of the interlocking impact of fibres. At the maximum load, when contrasted with PCC, the observed number of splits, their length and width at the maximum load are less in BFRC as shown in Figure 2(b). At this stage, the greatest length of crack in the specimen of BFRC is amplified up to around 75 mm. The test is proceeded even after the maximum load to see the sample behaviour. At the ultimate load, there are various cracks and the most extreme split lengths for the example of BFRC is developed up to around 90 mm as shown in Figure 2(b). For observation of fibre failure, specimens of BFRC are deliberately broken into two pieces after the ultimate load application.

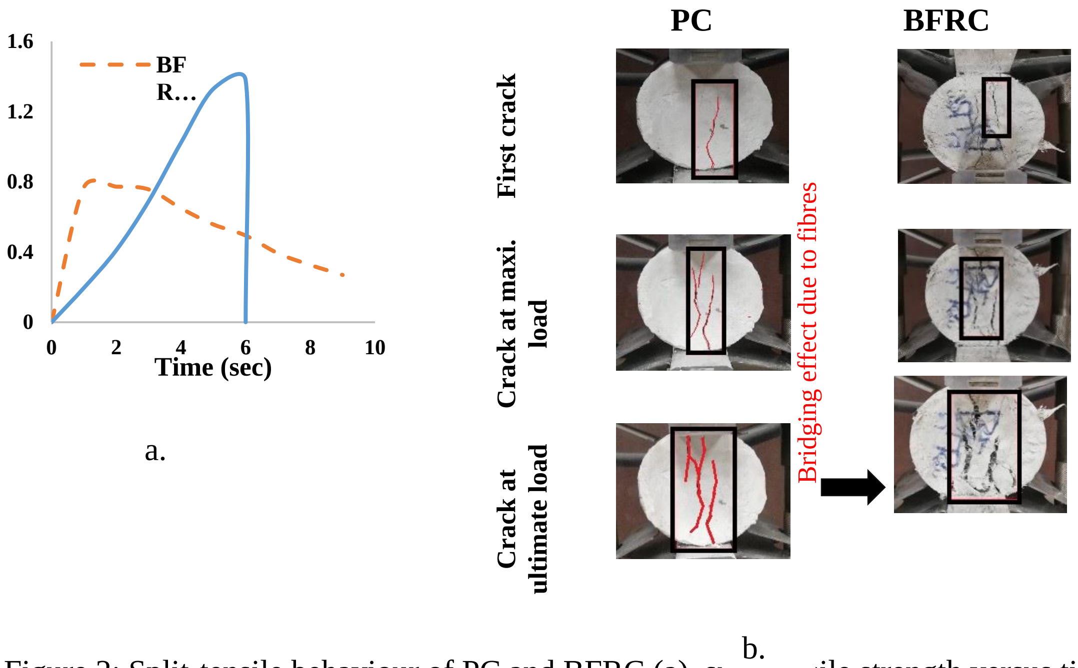


Figure 2: Split-tensile behaviour of PC and BFRC (a). split tensile strength versus time, and (b). crack development

3.2 Split-tensile strength (SS), split-tensile energy absorbed before crack (SE_1) and split-tensile toughness (SE_T):

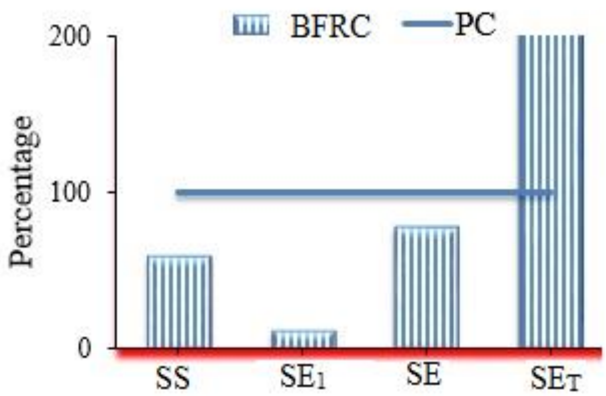
For computing the split-tensile strength (SS), the peak load value from split-tensile load versus time curve is utilized according to ASTM standard. For split-tensile absorbed energy before crack (SE_1) calculation, the portioned part of area under split-tensile load versus time curve up to the first crack load is utilized. Similarly, the split-tensile energy absorbed after crack (SE_2) is computed by using the area under split-tensile load versus time curve from the first crack load to the peak load. Splitting-tensile absorbed energy up to the peak load (SE) is taken as the entire area under split-tensile load-time curve from zero to the peak stress. Splitting-tensile toughness (SE_T) is taken as the ratio of the splitting tensile absorbed energy up to peak load to the splitting-tensile energy absorbed before crack (SE/SE_1).

Table 1: SS, SE_1, SE_2, SE, SE_T of PC and BFRC

Parameter	Concrete type	
	PC	BFRC
SS (MPa)	1.43	0.83
SE_1 (kN.s)	176.67	18.8
SE_2 (kN.s)	0	118.3
SE (kN.s)	176.67	137.1
SE_T	1	7.29

The split-tensile strength SS of PC and BFRC are 1.43 MPa and 0.83 MPa respectively, which means the split tensile strength of the BFRC is decreased by 42% in comparison with PC. The comparison of SS, SE_1, SE and SE_T is shown in Figure 4. BFRC showed decreased results in results of SS, SE_1 and SE_T for BFRC in contrast with PC by 42%, 90% and 33% respectively. The split-tensile toughness of BFRC showed extreme increase in its value as compare with PC.

Figure 3: Comparison of SS, SE_1, SE and SE_T of PC and BFRC



4. KEY ASPECTS FOR USING BFRC IN CONCRETE ROADS:

Shrinkage-crack propagation in concrete pavements can be bonded with different factors which include shrinkage, penetration of water, and tensile strength etc. Due to volumetric changes, shrinkage cracking results, which can be eliminated, if the split-tensile strength of concrete is greater than the tensile stresses induced by volumetric changes due to shrinkage. It depicts that shrinkage cracking can be controlled by tensile strength of concrete. Bending stresses in concrete structures are introduced by differential settlement in these structures. Similarly, cracks due to bending stresses would be controlled if bending strength of the concrete is greater than these stresses. Therefore, bending strength of the concrete is also needed to highlight in controlling cracks. In order to avoid formation of severe distresses in concrete pavements, it is necessary to increase the absorption, toughness and energy absorbed after ultimate loads of concrete. For reducing the rate of cracking in concrete pavements, it is essential to investigate materials by means of less shrinkage in addition with better mechanical properties (tensile strength, flexural strengths and toughness).

Experimental behavior of PC and BFRC to reduce the rate of shrinkage cracking is inspected, in the current study. When contrasted with that of PC, the BFRC demonstrates ineffective results in split-tensile strength SS, split-tensile energy absorbed before crack SE_1, total split-tensile energy absorbed SE. An extreme enhancement in SE_T is seen when contrasted with that of PC. Though, the split-tensile strength of BFRC is not found sufficiently effective but the increased toughness of

BFRC can help in reducing propagation of shrinkage cracking which ultimately will stop different distresses in concrete pavements.

Figure 4: Fibre behavior in broken test specimen

BFRC gives a bridging effect while failing, hence changing the failure mode and pattern. Figure 4 shows fibre pull out and fibre de-bonding. It is observed that, nearly 70% of fibres are pulled out and that of 30% are broken on the broken surface of specimen. The pulled out observed in broken specimen is due to lesser bond strength between banana fibres and concrete matrix. The reason can be that fibres were not properly dispersed while mixing

5. CONCLUSIONS:

This study highlights the usage of banana fibre reinforced concrete to avoid shrinkage-crack propagation in rigid pavements. Mix design of 1:2:4 with water cement ratio 0.6 was adopted to prepare PC and BFRC specimens. BFRC specimens included banana fibres of 5 cm length. Split-tensile behaviour of samples was judged. Following conclusions can be drawn from the conducted study:

- The increased toughness value of BFRC can help in controlling the propagation of early age shrinkage cracking in concrete pavements.
- The bridging effect of BFRC prevented the shrinkage cracks from getting deep under increased loading during testing. BFRC specimen did not break into two pieces like PC specimen.

The results show a potential in BFRC to be used for mitigation of shrinkage cracks in concrete roads. It is recommended that, mechanical properties of BFRC in addition with so other strengthening admixtures and its usage as a commercial product should be explored in depth.

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Study of Mechanical Properties of Concrete developed using Metamorphosed Limestone Powder (MLSP), Burnt Clay Pozzolana (BCP) & Wood Ash (WA) as Partial replacement of Cement

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Abstract

In this growing era of advancements and infrastructural growth, concrete has become one of the most utilized construction material across the globe. The environmental issues may be caused by direct discarding waste materials into the environment. Waste materials are now being employed to develop eco-friendly products and can be directly used as effective additives in many materials to develop efficient characters and durability in them. Stone slurry and solid marble waste are the two most produced waste materials of Metamorphosed limestone industry. Because of growing inflation in prices of raw materials researches are being made to replace the expensive materials with a low cost or waste materials in order to achieve eco-friendly yet cost-effective materials. In this paper, cement is being replaced by the metamorphosed limestone powder, burnt clay pozzolana, and wood ash. The tests are carried out for concrete developed in combination with the replacement of 0, 5, 10, 15 and 20% cement by metamorphosed limestone powder, burnt clay pozzolana, and wood ash. The main focus of the current study is to effectively identify the optimum range of percentage replacement that can be practically useful to achieve high-performance mechanical properties of concrete.

Keywords: Concrete; Waste Materials, Cement; Mechanical Properties; Strength

1. INTRODUCTION:

Concrete is an amalgamated material prepared with cement, sand, aggregates, admixtures or superplasticizers and water. In concrete, cement is used as a binder to hold the aggregates while fine aggregate (sand) acts as a filler material to increase the density of concrete and water is used to hydrate the cement(Coo & Pheeraphan, 2015). The effective properties of cement paste in concrete decides the efficient behaviour of concrete. In order to prepare a durable , low cost yet high-performance concrete many studies are being conducted around the globe to explore this particular aspect (Almusallam, Beshr, Maslehuddin, & Al-Amoudi, 2004). Numerous new skyscrapers strengthened bond solid structures have utilized cement with a compressive quality of more than 100 MPa. With increased usage of high-quality concrete, comprehensive research work has been conducted on base mechanical properties of concrete.
Research pertaining to the use of waste materials which are the by-products produced

by different manufacturing processes to enhance the properties of concrete. It clearly shows the fact that human activities in different aspects of production and usage create a very strong impact on procuring waste materials in significantly higher volumes more than 2500 MT per year. Different detailed studies prove the fact that these waste materials are way more profitable in usage nature if treated properly to utilize in effective ways. The solid wastes are one of the most prominently produced wastes that may include silica fume, fly ash, blast furnace slag, burnt clay pozzolana, metamorphosed limestone powder, wood ash and waste construction materials. These waste materials are used to improvise the strength of concrete, workability behaviours, increase the water tightness, and to reduce the heat of hydration and significant thermal shrinkage for concrete.

The civil engineers are primarily concentrating to develop the advanced concrete to enhance the service life of the structures and provide satisfactory performance under different climatic conditions. In the past, the proper disposal of such industrial waste materials was indeed one of the significant environmental issues across the world (Aliabdo, Elmoaty, & AbdElbaset, 2015). But now the engineers are utilizing them for plausible purposes. The fundamental objective here is to study the suitability of the metamorphosed limestone powder, burnt clay pozzolana and wood ash as pozzolanic materials to replace the cement by weight up to a certain amount in concrete. However, it is anticipated that the employ of metamorphosed limestone powder, burnt clay pozzolana and wood ash in concrete improves the base mechanical properties of ordinary concrete.

These pozzolanas are verily considered as effectively acting source materials to be used for the partial replacement of bonder in concrete. While Mineral Additives are also known as Pozzolanic Materials. They are the natural residues of different processes and they can fulfill our demands and can be replaced with chemical admixtures. Pozzolanic materials are: (i) Natural Pozzolanas (Shales and Clay, Cherts, Diatomaceous Earth and Pumicites and Volcanic Tuffs) (ii) Artificially existing Pozzolanas (Metamorphosed limestone powder, Burnt clay pozzolana, Wood Ash, Slag (GGBFS), Fumes of silica, Rice Husk and commonly available Metakaolin).

These improve numerous characteristics of concrete, for example, these reduce the warmth of hydration and warm shrinkage, effectively increases the water snugness impact , reduces the plane soluble base total response, improvises the protection from assault by sulphate soils and ocean water, elevates the potential extensibility, lowers the defencelessness ability towards the disintegration and filtering, improvises efficient workability and lowers the gross costs (Agarwal, 2006; Aliabdo, Elmoaty, & Auda, 2014; Cheah & Ramli, 2011; Coo & Pheeraphan, 2015; Ergün, 2011; Kiran & Sharma, 2017; Krishnasamy & Palanisamy, 2014; Mansoor et al., 2018; Mohamed, 2011; Omar, Elhameed, Sherif, & Mohamadien, 2012; Puri & Srinivasan, 1959; Ranjan, Srivastava, & Kumar, 2015; Shirule, Rahman, & Gupta, 2012; Siddique, 2007, 2012; Siddique & Klaus, 2009; Torkaman, Ashori, & Momtazi, 2014; Turk, Karatas, & Gonen, 2013; Vardhan, Goyal, Siddique, & Singh, 2015).That is why these materials have been tested by researchers as partial replacement to cement and sometimes sand/filler as well.

In this study, three different waste materials have been tested for strength performance of concrete as a replacement of a binder i.e. cement. This procedure is designed to utilize the sustainable waste management in the field the of construction materials as well.

2. EXPERIMENTAL PROCEDURES:

2.1 Mechanism

Usage of different wastes in concrete and enhancing the base mechanical properties of concrete helps to produce eco-friendly yet efficient structures. Usage of waste marble powder, wood ash, and burnt clay pozzolana was explored the an aspect of strength improvisation by replacement of cement in varying percentages of the volume of cement being used. Cement has been tested to be by using M25 grade concrete in combination with the replacement of 0, 5, 10, 15 and 20% cement by metamorphosed limestone powder, burnt clay pozzolana, and wood ash. It improvised the binder properties and effectively reduced the usage of cement. Addition of pozzolana clay allows better binding properties to prevail leading to achieve higher strengths and low cracking concerns. Wood ash was finely ground to powder in order to achieve required consistency to be utilized in concrete. Fine powder of wood ash serves as replacement of vital ingredients of concrete. In this way, we can develop fine quality, durable and all above sustainable Eco-friendly concrete building blocks. After the development of concrete, the mechanical properties of concrete were tested to understanding the strength behaviour of concrete.

2.2 Materials

Across Pakistan large volumes of metamorphosed limestone powder, burnt clay pozzolana and wood ash are generated that went untreated into river waters or landfills. Similarly, there is a large industry of brick production in Pakistan that produces tons of wastages per day across the country. Utilization of wood ash is also an efficient improvisation in concrete that renders finely reliable characters to concrete. All these materials were analyzed for detailed properties and their respective behaviors as shown in Table.1.

Table 1. Properties of utilized Cement & Additives

Materials	Fineness	Consistency	Initial setting time (minutes)	Final setting time (minutes)	Compressive Strength (MPa)	
					3 days	7 days
CEM	4%	32%	60	265	16.29	23.14
MLSP	6.85%	34%	75	275	9.73	14.89
BCP	5.6%	30%	45	255	12.08	17.41
WA	7.49%	28%	40	225	8.67	11.93

2.3. Testing

The use of materials like metamorphosed limestone powder, burnt clay pozzolana and waste wood ash in the production of concrete had been presented in this paper. Waste materials that were to be added were analysed for their base properties like consistency measures, fineness, and sieve analysis were conducted before the utilization of these materials. For the understanding the impact of cement replacement with these materials on strength of concrete, three major mechanical properties were to be tested that are explained as under:

2.3.1. Compressive Strength

The 7, 28 days of curing led to the compressive strength of concrete (ASTM C 39) using burnt clay pozzolana at 10% by weight in place of cement to be effectively greater than the normal compressive strength achieved by ordinary concrete. While in all other cases, the compressive strength of concrete using metamorphosed limestone powder, burnt clay pozzolana, and wood ash at different proportions (5, 10, 15 and 20%) by the weight as a replacement of cement that was lesser than the effective compressive strength of normal concrete. Compressive strength is evaluated by using the following formula:

$$\text{Compressive Strength (MPa)} = P/A, \tag{1}$$

Where,
P = compressive load at failure (N)
A = cross-sectional area of cylinder, (mm^2)

2.3.2. Tensile Strength

The 7- and 28-days tensile strength of concrete (ASTM C 496) using burnt clay pozzolana at 10% by weight in place of cement was greater than the tensile strength of normal concrete. While in all other cases, the effective tensile strength of modified concrete using metamorphosed limestone powder, burnt clay pozzolana, and wood ash at different proportions (5%, 10%, 15%, and 20%) by weight in place of cement was lesser than the tensile strength of normal concrete. In general, the tensile strength of the cylinder is 10% of its compressive strength. The effective tensile strength of concrete can be evaluated by the following formula:

$$\text{Tensile Strength (MPa)} = 2P/\pi DL, \tag{2}$$

Where,
P = compressive load at failure (N)
L = length of the cylinder (mm)
D = diameter of the cylinder (mm)

2.3.3. Flexural Strength

The flexural strength of concrete(ASTM C 78) using metamorphosed limestone powder, burnt clay pozzolana, and wood ash was higher than flexural strength of normal concrete when cement is replaced by weight up to 15% and the flexural strength was reduced by adding further metamorphosed limestone powder, burnt clay pozzolana and wood ash. The flexural strength is computed by the following formula:

$$\text{Flexural Strength (MPa)} = 3Pl/bd^2, \tag{3}$$

Where,
P = total load at failure point (N)
l = effective beam span between supports (mm)
b = total width of beam (mm), d = measured depth of beam
Flexural strength can also be computed as:

$$\text{flexural strength (MPa)} = 0.7\sqrt{fc'}, \tag{4}$$

3. RESULTS:

3.1. Performance Analysis

In the current effectively conducted evaluation of mechanical properties the maximum compressive strength is achieved at 7 days for 10% replacement level of burnt clay pozzolana with cement so this is considered as optimum replacement level and the optimum gained strength is 13.50 MPa after 10% replacement while the compressive strength of specimen at control mix i.e. 0% replacement level was 12.26 MPa. For effective compressive strength of the cylinder specimen at 28 days, the maximum strength is achieved for 10% replacement level of burnt clay pozzolana with cement so this is considered as optimum replacement level and the optimum gained strength was 18.21 MPa after 10% replacement while the compressive strength of specimen at control mix i.e. 0% replacement level was 15.91 MPa as shown in Fig.1. In case of the tensile strength of cylinder specimens cured at 7 days, we have the strength of 1.165 MPa at the control mix i.e. 0 % replacement level while the maximum tensile strength was 1.25 MPa at 10% replacement level of burnt clay pozzolana. For the tensile strength of cylinder specimens cured at 28 days, we have the strength of 1.58 MPa at the control mix i.e. 0 % replacement level while the maximum tensile strength was 1.67 MPa at 10% replacement level of burnt clay pozzolana as shown in Fig.2. For the flexural strength of beams, the maximum 7 days strength was achieved for 5% replacement level of burnt clay pozzolana with cement which is 3.36 MPa while the flexural strength of beam specimen at control mix i.e. 0% replacement level was 2.62 MPa. For the effective flexural strength parameter of casted beams at 28 days of curing, the maximum strength was achieved for 5% replacement level of metamorphosed limestone powder with cement which was 4.58 MPa while the flexural strength of beam specimen at control mix i.e. 0% replacement level was 3.38 MPa as shown in Fig.3.

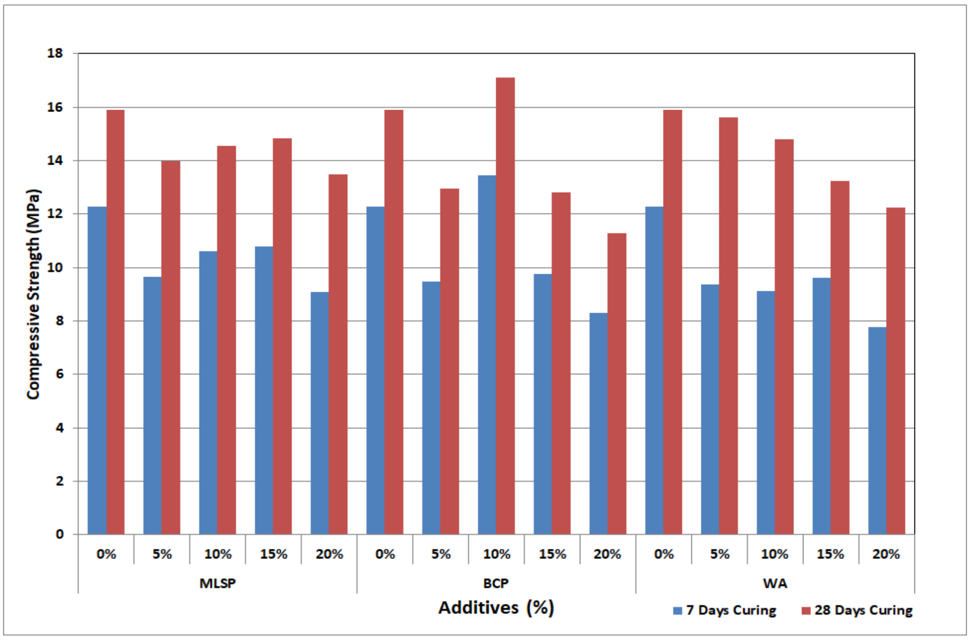


Figure 1. Compressive strength of Concrete Cylinder (MPa) under 7 days and 28 days curing

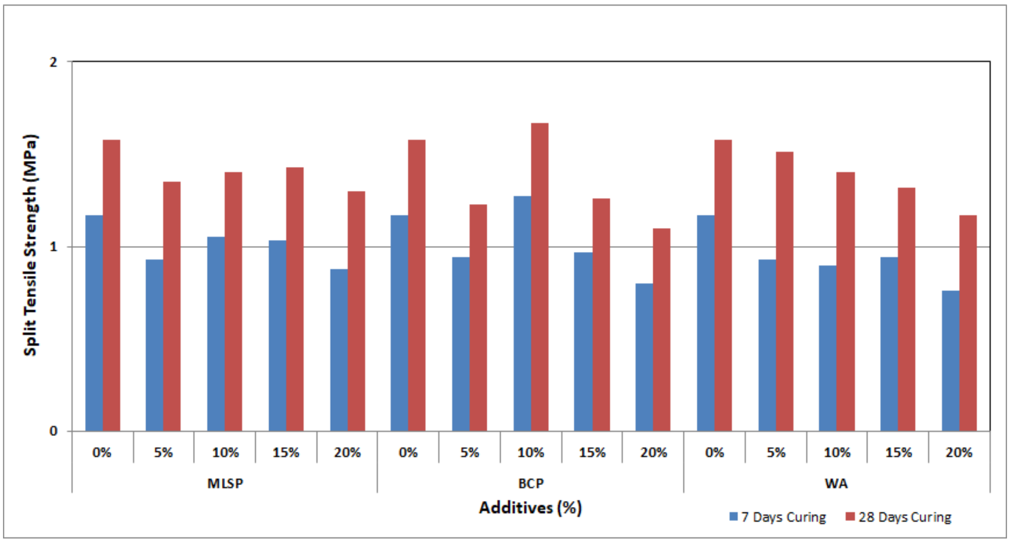


Figure 2. Tensile strength of concrete cylinder (MPa) under 7 days and 28 days curing

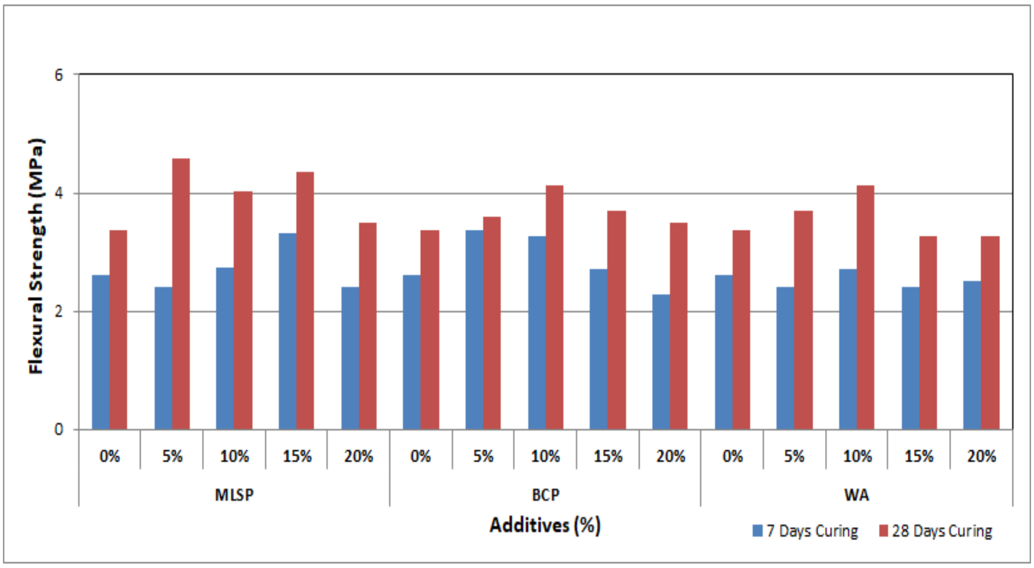
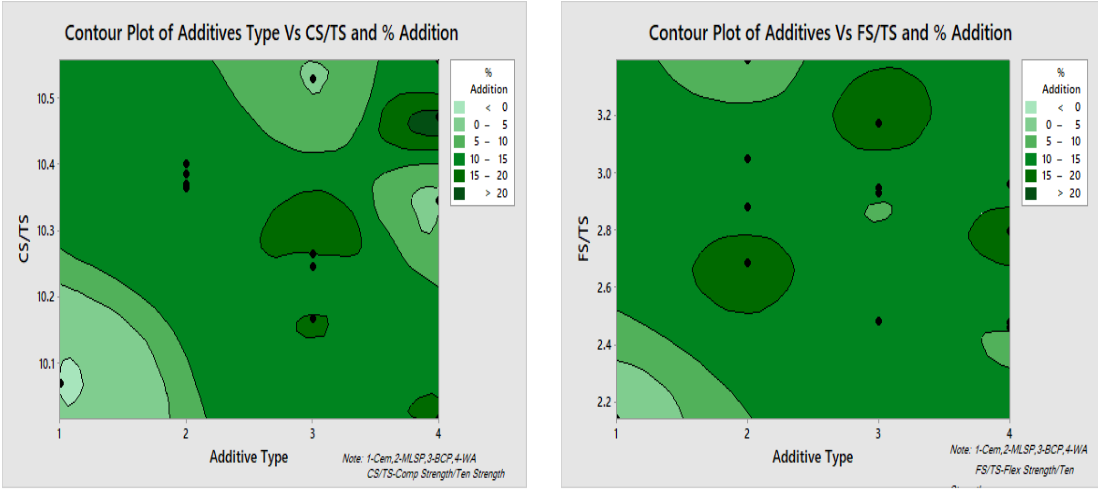


Figure 3. Flexural strength of concrete cylinder (MPa) under 7 days and 28 days curing

3.2 Analysis of Factors

During the analysis major focus was to find the replacement material for cement as it is a costly material and need manufacturing process while industrial wastes are the available raw materials. To investigate the impact of the change of additive and replacing it with cement needs further confirmation before deciding about the commercial use of the additives. To further deep analysis a relationship has been built via contour plots which are basically between ratios. Compressive/Tensile strength ratios and Flexural/Tensile strength ratio have been plotted against the types of additives. From Plot (Fig.4.a) we can observe that the highest strengths are with cement, but only dominant replacement is additive 3 (i.e. BCP) is true alternative. Similarly, in Plot (Fig.4.b) again clear dominance can be observed by additive 3(i.e. BCP).



(a) (b)

Figure 4. Contour Plot: (a) Comp/Tens Vs Type of Additives (b) Flex/Tens Vs Type of Additives

4. CONCLUSIONS:

Concrete is one of the most used materials in construction industry comprises of cement, sand, aggregates, and water as major ingredients. For centuries it is under discussion that what are the major influencing factors which contributing the strength of concrete. Research has started by testing its mechanical to physical properties and later fresh and hardened properties. Sometimes it tends to physics and everything is led by forces and revolving terms are stress and strain. Sometimes it starts with chemistry like chemical reactions. In this research focal point is a replacement is of cement as from all ingredients cement is one of the costly items and need an industrial production. Cement has been tested by incorporating it in the concrete ineffective combinations of replacements ranging from 0, 5, 10, 15 and 20% of used cement by metamorphosed limestone powder (MLP), burnt clay pozzolana (BCP), and wood ash (WA). These three materials are usually considered industrial waste are available at a lower cost than that of cement. For effective compressive strength of the cylinder specimen at 28 days, the maximum strength is achieved for 10% replacement level of burnt clay pozzolana (BCP) with cement so this is considered as optimum replacement level and the optimum gained strength was 18.21 MPa after 10% replacement while the compressive strength of specimen at control mix i.e. 0% replacement level was 15.91 MPa. For the tensile strength of cylinder specimens cured at 28 days, we have the strength of 1.58 MPa at the control mix i.e. 0 % replacement level while the maximum tensile strength was 1.67 MPa at 10% replacement level of BCP. For the effective flexural strength parameter of casted beams at 28 days of curing, the maximum strength was achieved for 5% replacement level of MLP with cement which was 4.58 MPa while the flexural strength of beam specimen at control mix i.e. 0% replacement level was 3.38 MPa. Although the highest strengths are with cement, only dominant replacement is MLP but BCP was found satisfactory in all three-test condition. This study can help in the construction of concrete structure like rigid pavements which are saved and need rough construction pattern. In case of 100 km of long road a large quantity of cement can be saved by applying 15% replacement of cement with MLP or 10% replacement of cement with BCP. It can be successful as loading conditions for rigid pavement roads are different than that of building and structures.

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Dispersion of Multi Wall Carbon Nano Tubes using Hybrid Surfactants.

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Abstract

Carbon Nano Tubes adhere together due to strong Vander wall forces, so it is inevitable to disperse Carbon Nano Tubes before using them in the cement matrix. In this paper, the dispersion of MWCNTs with Arabic gum, Polycarboxylate ether (PCE) based superplasticizer and both acting concurrently are described. The initial sonicated suspensions were too concentrated; they were diluted according to Lambert beer law to molarity of 0.18, 0.14 and 0.1mg/ml. Ultra Voilet-Visible spectrophotometry technique was used to check the dispersion of diluted samples. The peak absorbance values of Arabic gum, superplasticizer, and their synergistic suspension were measured respectively and a graph of these different surfactants was plotted according to Lambert beer law which depicts maximum dispersion in case of synergistic suspension of both surfactants.

Keywords: Multi-walled carbon nanotubes, Ultraviolet-visible spectrophotometry, Dispersion, Surfactants, dilution.

1. INTRODUCTION:

Carbon nanotubes are held by high Vander wall forces making it inferior soluble in water or organic solvent, resulting in difficulties in the preparation of stable carbon nanotube dispersions. There are many methods used to achieve dispersions like mechanical dispersion (Elias et al., 2005), ultrasonic dispersion (Liang & Li, 2015), chemical modification (Ou et al., 2009), dispersants (Yurekli, Mitchell, & Krishnamoorti, 2004), polymer coating method (Star et al., 2001), and metal coating method (Lordi, Yao, & Wei, 2001), of which the chemical treatment and ultrasonic process will cause damage to the structure of carbon nanotubes (Lu et al., 1996). Depending on the available equipment and needs, the researcher may use any one or combinations of any prior mentioned methods for dispersion of carbon nanotube. Furthermore, any kind of surfactant may be used to help in breaking the pi-pi bond between each layer and keep the individual layer stable.

A homogenous suspension of carbon nanotubes is inevitable for the reinforcing action of carbon nanotubes in the cement mortar. However due to strong van der wall forces in the carbon nanotubes due to its high surface area causes the CNTs to agglomerate together and adhere to form bundles in the matrix, originated from there polarizable extended pi-electron systems. Different techniques are used to disperse carbon nanotubes in the aqueous medium, such as the use of solvents, surfactants, functionalization with acids, amines, fluorine, plasma, microwave, moieties, and non-

covalent functionalization. CNTs form agglomerates due to the adhesion between them which is caused by strong Vander wall forces in between the tubes. Surfactants have two characteristic features they adsorb on the interface of the CNTs and self-accumulation into a supramolecular structure.(Vaisman, Wagner, & Marom, 2006).In contrast to covalent sidewall functionalization, the noncovalent can immobilize the organic molecules on the sidewalls of the nanotubes without destroying the geometric and electronic structure of the carbon nanotubes(Zhao, Lu, Han, & Yang, 2003).
The most common method of dispersion employed these days is the use of chemicals in the form of surfactants in conjuncture with ultrasonic energy (Parveen, Rana, & Fangueiro, 2013). However, these chemicals must be very carefully selected as they may sometime interfere with hydration kinetics of cementitious matrices. Various cationic, anionic and non-ionic surfactants have been explored to evaluate their influence on the dispersion of functionalized and unfunctionalized MWCNTs.Arabic gum has also been effectively used as a surfactant to disperse the CNTs. W.Rashmi (Rashmi et al., 2011) reported in his work that significant dispersion of the CNTs can be achieved by the use of Arabic gum as a dispersing agent. They used CNTs 0.01-0.1% by weight of water and Arabic gum in the concentration of 0.25-5% by weight of water. Dispersion can also be achieved by the use of chemical admixtures. Superplasticizer (polycarboxylate based) is commonly used to achieve enhanced workability can also act as an effective dispersant of carbon nanotubes (Collins, Lambert, & Duan, 2012). Ultraviolet-visible spectroscopy (UV-vis spectroscopy) shows a clear difference in dispersion capability of different surfactants. The behavior of dispersion may be governed by the chemical interaction of CNTs and surfactants and electrokinetic properties (Parveen et al., 2013). The phenomenon of proper dispersion of CNTs with the help of a surfactant is also reported by (Sasmal, Bhuvaneshwari, & Iyer, 2013). It is evident from the above discussion that not much work has been done on the dispersion of CNTs in the combination of two or more surfactants. In this research, we try to evaluate the different suspensions containing surfactant individually and their synergistic effect based on their UV-vis spectra.

2. MATERIALS:

The CNT was purchased from SAT nanotechnology material CO. LTD. The properties of carbon nanotubes are listed in table1. Arabic gum was purchased from the local grocer and later on, was passed through sieve # 200. Similarly, Polycarboxylate ether based superplasticizer (master polyheed 996) conforming to ASTM C-494 and acacia gum was used for dispersion mechanism. The detail properties of acacia gum and superplasticizer are listed in table 2 and table 3.

Table 1: Properties of CNTs.

Test items	Dia.	Length	Appearance	Surface Area	Ash	Fe	Al	M
Test results	10-20nm	10-30um	Blackpowder	90-350m2/g	<2%	0.4%	0.5%	0.2%

Table 2: XRF of Arabic Gum

Analyte	CaO	K_2O	Fe_2O_3	SO_3	MnO	F_2O_5	CuO	TiO_2	ZnO	SrO	Rb_2O
Result(%)	49.748	28.114	8.441	5.108	5.001	1.088	1.009	0.679	0.403	0.258	0.150

Table 3: properties of SP.

Form	**color**	**density**	**PH value**	**Chloride content**
Light	Light Yellow	1.08± 0.02	≥6	Chloride free

2.1 Sample preparation.

To measure the comparative dispersion of two different surfactants and the combination of these surfactants. Three different composites were made, MWCNTs to water ratio was 0.2% in every composition. In the case of Arabic gum and superplasticizer individual compositions, the ratio of MWCNTs to surfactant was 1:4, while in the case of the combination of both surfactants the ratio was 1:4:4. In which 1 represent MWCNTs and 4 represent Arabic gum and superplasticizer consecutively. The compositions were placed on a magnetic stirrer for 10 minutes. After dispersing the samples with a magnetic stirrer, further dispersion was carried out with the help of a bath sonicator. The samples were placed in the bath sonicator for 1 hour. After sonication, the samples were diluted, and the diluted samples were placed in a spectrophotometer in quartz cuvette to check the absorbance value. The detail process of mixing can be seen in Figure 1.

Figure 1: (a) Stirrer mechanism; (b) Bath sonicator

3. RESULTS AND DISCUSSIONS:

3.1 Effects of dilution:

The comparative degree of dispersion MWCNTs in an aqueous medium can be judged by the measurements of the UV-Vis spectra of the dispersions since multiwall CNTs show characteristic bands in the UV region(Miyata, Mizuno, & Kataura, 2011; Saito, Fujita, Dresselhaus, & Dresselhaus, 1992). In Ultra Violet-Visible spectrophotometry, the measured absorbance at a specific wavelength can be related to the degree that how much Carbon Nano Tubes are dispersed (Grossiord, Regev, Loos, Meuldijk, & Koning, 2005; Grossiord, van der Schoot, Meuldijk, & Koning, 2007). the three different suspensions after the sonication process were taken for the Ultra Violet-Visible spectrophotometry. The composites were too concentrated that's why they were diluted according to Lambert beer law. Each sample was diluted to get the molarity of 0.18, 0.14 and 0.1mg/ml. quartz cuvette was used because plastic and glass cuvettes don't give significant results in the UV region. The results are shown in Figure 2.

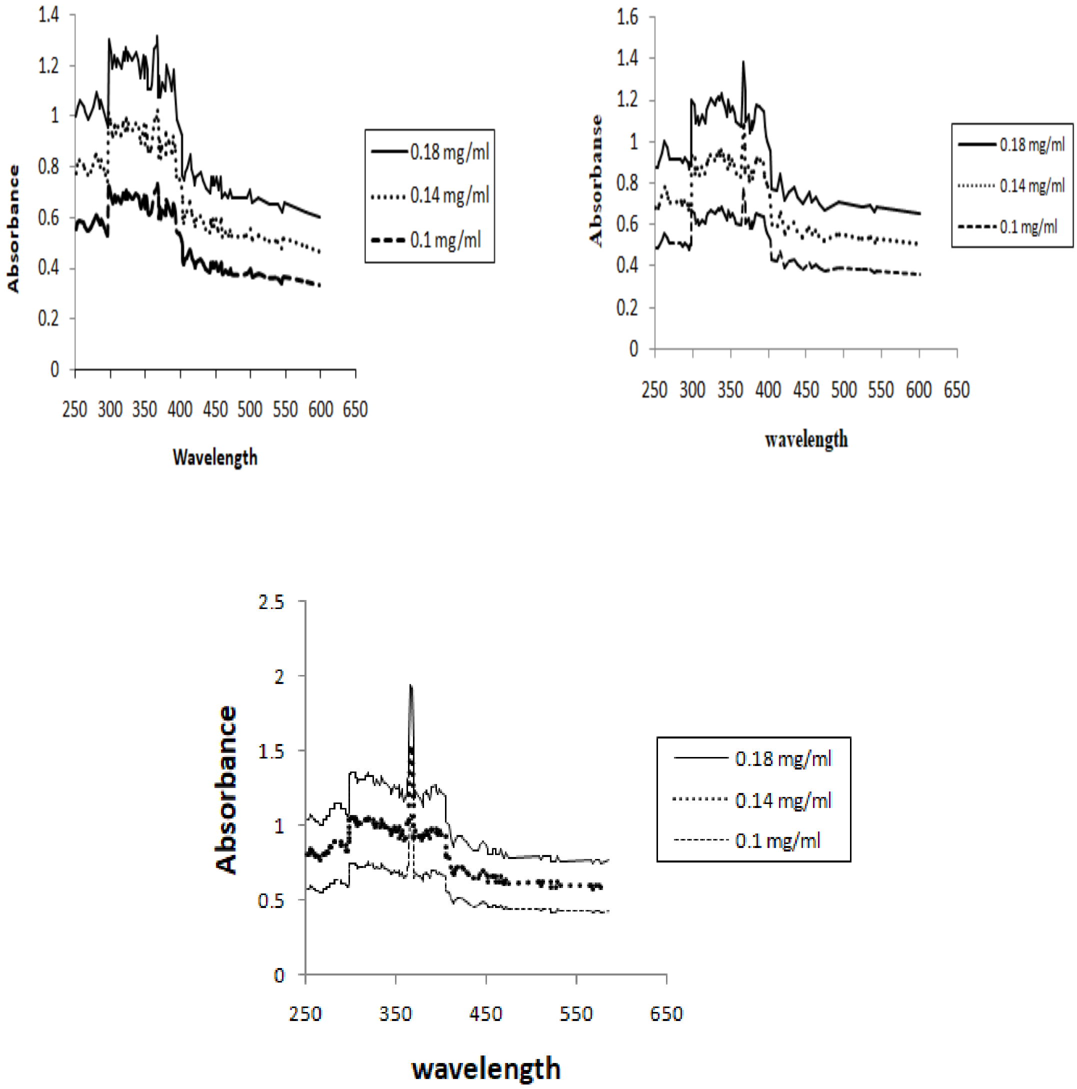

Figure 2: Absorbance spectra of the aqueous solution of CNTs in the presence of, (a) Arabic gum, (b) superplasticizer and (c) Combination of Arabic gum and superplasticizer.

3.2 Effect of surfactants:

From the Lambert beer law equation, the graph for each sample (Arabic gum, superplasticizer, and their combination were plotted). This graph shows the absorbance at 0.18, 0.14 and 0.1mg/ml. The straight line of Arabic gum and superplasticizer is showing maximum slope and depicts maximum absorbance.

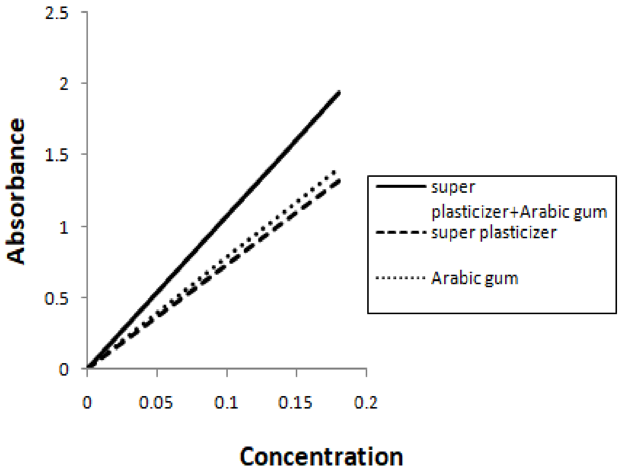

Figure 3: Absorbance spectra at different concentration.

In this study, different surfactants individually and there combined action on the dispersion of MWCNTs in aqueous medium was monitored using the technique of UV-, Visspectrophotometry. From the results, it is quite evident that when both of the surfactants, Arabic gum and superplasticizer were used concurrently the absorbance value was maximum which is a sign of thorough dispersion in comparison to individual action of both surfactants. It can be because of the reason that both surfactants rigorously disperse the MWCNTs, overcoming the Vander wall force. Moreover, the relation between dilution and absorbance in the spectrophotometry is also shown which shows the direct relationship between the concentration and absorbance

4. CONCLUSION:

To the authors' knowledge, both of these surfactants had not been used together to disperse carbon nanotubes. The current findings do not only add up to the growing body of the literature on multiwall carbon nanotubes dispersion but also confer the opportunity to perform further extra functionalization steps due to the carboxylate reacting group facing the surface. Further, this study concludes that, the peak value at 362 nm wavelength and molarity of 0.18 mg/ml depicts that on comparison synergic composition of both surfactants gives maximum dispersion.

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Use of Steel Mill Slag in Concrete as Fine Aggregates

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Abstract

Waste materials and some industrial by-products are hazardous to environment and health of living beings. But many of these waste materials can be recycled or reused. Some of these can be used in concrete as a construction material, that can reduce land filling expenses and protect the environment from injurious effects. In this study, natural fine aggregates were replaced with steel mill scale, which is produced as a waste material in steel industries. A detailed experimental study was carried out to determine the effect of replacement of fine aggregates in high strength concrete with steel waste. Three grades of fine aggregates and steel waste were used for attaining optimum packing density. Compressive strength test was conducted on specimens with varying steel waste content. Flow and dry density tests were also performed on these mixes. It was observed that an increase in the percentage of steel scale waste increased the flow as well as the dry density of the concrete mixtures. The compressive strength of concrete mixtures also increased due to steel scale waste content up to a certain percentage.

Keywords: Steel scale waste, recycling, packing density, compressive strength test.

1. INTRODUCTION:

The increase in population, development of industries and escalation of consumerism have produced a rapid growth in the production of waste materials. Some of the waste materials can be reused or recycled or can be used as a raw material in different productions, whenever it is possible. Iron and steel industries are the main producer of steel scale waste (SSW) [Furlani and Maschio]. Steel mill scale is produced in all steel industries but only small fraction is used in Pakistan. Previously, in Pakistan, mill scale was also exported to China, but now huge amount of this waste material is being dumped as a landfill. Land-filling is not a suitable solution because some of the portion of this metallic waste leaches into the groundwater, thus deteriorating the environment. On the other hand, natural resources are depleting day by day due to the development of construction industry. Therefore, we need to look for an alternative of the natural materials in concrete. Limited researches have been conducted on the use of steel mill scale in concrete. Pradip et al. (1990) determined the effects of steel mill scale in the production of cement mortar. Al-Otaibi (2008) used the SSW as aggregate replacement, where the effect of SSW on concrete was observed by replacing it in percentages of

0%, 20%, 40%, 50%, 70% and 100%. Compressive strength tests were conducted at 3,7 and 28 days. It was seen that compressive strength increased with up to 40% replacement. Moreover, reduction in drying shrinkage was observed. Another study determined by Pereira et al. (2011) concluded that greater water content is required in order to maintain the workability of the mix. It was seen that concrete mix having water/cement ratio of 0.55 and 0.65 resulted in high water consumption and greater compressive strength.
The significance of this study is to use steel mill scale as replacement of sand, providing an environment friendly solution for waste management. It will help prevent natural resources from further depletion. The objective of this study is to evaluate the effects of partial as well as full replacement of natural fine aggregates with steel mill scale waste on the mechanical properties of high strength concrete (HSC). Combination of different grades of SSW and fine aggregates which gave the highest packing density are used for preparing concrete mixtures. Flow, hardened density and compressive strength of the concrete samples are tested.

2. METHODOLOGY:

Steel mill scale filler was collected and separated into different grades. The optimum packing density of steel mill scale was determined and a suitable mix design was developed for the optimum mix. Compressive strength test, hardened density test and flow table tests were conducted according to ASTM specifications.

3. MATERIAL PROPERTIES:

Commercially available Fauji brand ordinary Portland cement (OPC Type-I) was used. Silica fume and quartz powder were added as 25% and 40% of OPC to prepare HSC. Locally available sand was used as a fine aggregate. SSW of different grades was prepared by sieving (similar to fine aggregates). Fine aggregates/SSW were divided into three different grades i.e., Grade I (2-1.18mm), Grade II (1.18-0.6mm) and Grade III (0.6-0.075mm). Particle density (PD) of various combinations of these grades were checked and the combination with the highest packing density (PD) was used in the mix composition. Super-plasticizer was used to increase workability of the concrete.

Figure 1: Determination of packing density using vibrating table

3.1 Mix compositions:

It was observed that G1G3 combination of fine aggregates resulted in the highest PD for both sand and SSW. Based on the PD results, G1 (50%) and G3 (50%) of sand/SSW were used for final HSC mix. First mix was prepared with 100% sand (0.5 G1 & 0.5 G3). Then sand was replaced with SSW (0.5 G1 and 0.5 G3) by 20%, 40%, 60%, 80% and 100%. w/c ratio of 0.22 was used for all mixtures. Flow of all concrete mixes was tested as per ASTM C-1437. Concrete specimens (2”x2”) of all mixes were cast and cured as per standard specification. After curing, dry density of concrete specimens was checked and later on samples were subjected to compressive strength test.

Figure 2: Samples of HSC before compressive strength testing

4. RESULTS AND DISCUSSION:

4.1 Flow table test:

Figure 3 shows the comparison of flow of HSC made with various combinations. It was observed that flow increased with the increase in SSW content in the HSC due to the reason that SSW absorbs less water as compared to sand. Mixture with 100% sand exhibited flow of 195mm and it increased up to 235mm for the mix in which sand was totally replaced by SSW (S0 SSW100). Mixtures with the varying replacement percentages have the flow in between 195mm and 235mm

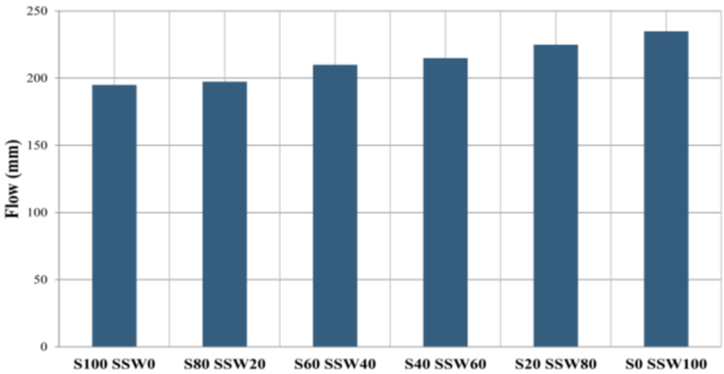

Figure 3: Flow of various HSC mixtures waste

4.2 Dry density test:

The dry density test was performed as per ASTM C138 after 28 days of curing. Figure 4 shows that the increase of SSW content increased the dry density of the HSC, due to higher specific gravity of SSW as compared to the natural fine aggregates. The control specimen exhibited a dry density of 2427 kg/m^3 and the mixture with 100% SSW showed around 17.5% increase in dry density as compared to the control specimen. Therefore, SSW increased the weight of the concrete.

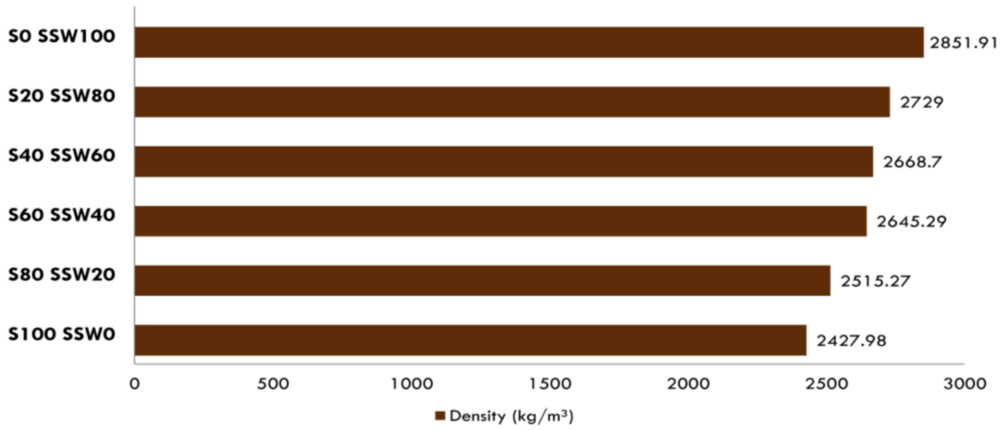

Figure 4: Dry densities of various HSC mixtures at 28 days

4.3 Compressive strength test:

Figure 5 compares the 28 days compressive strength of control mixture (S100SSW0) and mixtures containing SSW as a fine aggregate in different proportion. It was observed that the compressive strength increased with the addition of SSW up to 40% replacement, which was the optimum level. S60SSW40 experienced 27% increase in the compressive as compared to the control mix. After 40% replacement level, although the compressive strength started decreasing, still the mixture prepared with 100% sand replaced with SSW (S0SSW100) exhibited 1.8% higher strength than the control mix. The higher compressive strength in the case of steel slag mixtures can be attributed to higher density of these mixtures.

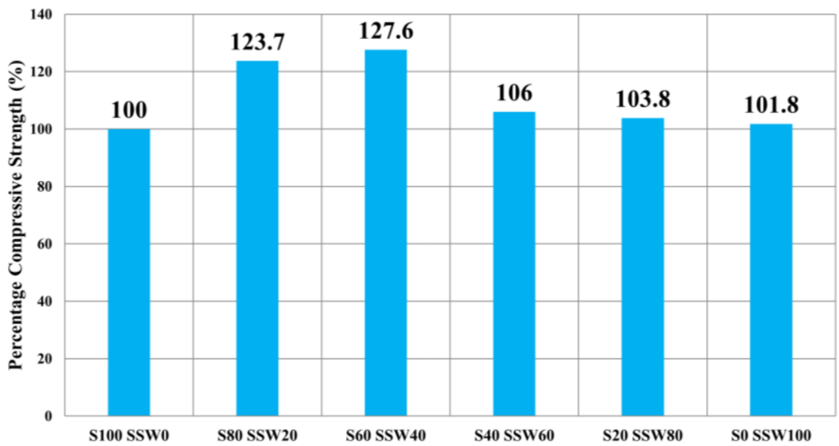

Figure 5: Percentage compressive strength of concrete mixtures

5. CONCLUSIONS:

Following conclusions can be withdrawn from this study:

- Due to different sizes of fine aggregates, varying packing densities were observed.
- Flow of concrete mixture increased as percentage of steel scale waste increased.

- With the addition of SSW content in mixture, increase in dry density was observed.
- Optimum compressive strength was observed at 40% replacement of sand with SSW.
- 100% replacement of sand with SSW exhibited comparable compressive strength with the control mix.

6. FUTURE RECOMENDATIONS:

Any future work in the same direction should also consider deriving stress-strain plot for the compression test on concrete cylinders.

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Effect of Various Combination of Aggregates from Different Sources on Properties of Concrete

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Abstract

Concrete is a fundamental constitute of construction industry across the globe. The bulk of concrete is made up of aggregates, hence structural behaviour of concrete significantly relies on the quality and properties of aggregates. This paper presents best combination of fine and coarse aggregates that yields the maximum compressive and tensile strength. A research is carried out on coarse and fine aggregates from different sources in Pakistan. Primary laboratory analysis was conducted to establish the suitability of the aggregates from various sources in construction work. Tests conducted include sieve analysis, bulk density, and specific gravity. Nominal mix (1:2:4) was adopted for this work and mix compositions were calculated by absolute volume method were cast to compute the compressive strength to be monitored at 28 days. Test result show that concrete made from has the highest workability followed by and crushed granite aggregates. Experiments were conducted on eighteen different combinations concrete and a constant W/C ratio of 0.59 for each batch sample. Highest compressive strength at all ages was noted with concrete made from Margallah crush with blend of 50% Lawrencepur sand and 50% Kashmore sand. Also, the combination of Haripur crush with Kashmore sand yields second best value. A common practice is to mix Chenab sand with Lawrencepur sand to attain good strength and workability.

Keywords: Concrete, Aggregate, Potential Sources in Pakistan, Strength.

1. INTRODUCTION:

Concrete is composed of the combination of cement, coarse aggregate, fine aggregates and water. It can be said that concrete is a combination of a material that consists of a binding medium which are embedded particles of aggregates (Li, 2011). Moreover, it is most economical construction material, good in compression, durable and good fire resistance, environment friendly and the maintenance of concrete is also easy to conduct. Pakistan is a developing country and there are lots of construction projects currently running in different areas so there is a huge demand of coarse and fine aggregates for those projects. Research has been carried out on chemical and mechanical properties of aggregates available in the country. At the start, bio-materials like leaves, branches and so on were utilized, which later on changed to more tough materials like mud, stone and timber (Khitab et al. 2015). The most extensively used coarse aggregate source in most of the Pakistan is Margallah Hill Limestone, most of

the quarrying activity is observed in the Margallah hill source located at Hassan-Abdal and Taxila regions of Punjab. The Sakesar limestone of Salt Range is also feeding most of the southern Punjab and vicinity. There are a lot of construction material sources situated in Azad Jammu and Kashmir. Khurshidabad Dist. Haveli and Bakot area near Kohala has millions of tons of limestone (Afzal, 2017). Takial and Khairabad in KPK has huge resources of coarse aggregates similarly Khairpur Mir's, Sind has vast potential of coarse aggregates which are used for local construction needs. The sand from Lawrencepur is one of the best sand available to be used for construction works. Other sand types are also used in construction works in their relevant areas (Ghaffar et al., 2016). Margallah aggregates resulted in better concrete mechanical properties, while Sargodha aggregates showed improved physical characteristics (Abbas, et al., 2017). Bara River, Basai, LoyeKhawar and Zangali/JaniKhawar can be safely used in structure concrete works. Out of these four sources, Basai is found as the best source for coarse aggregates (Ayub, et al., 2012). The Chattan Shah Quarry aggregates showed the higher specific gravity compared to all other aggregates, therefore higher compressive strength (38.2 MPa) was achieved as expected (Aslam et. al, 2015).
Despite of the availability of number of researches on the utilization of coarse aggregates of different sites/areas, the most suitable and economical combination of coarse aggregates of different quarrying sites is still not envaulted. This research work will be helpful in finding the optimum combination of coarse aggregates of different sites.

2. MATERIALS AND METHODOLOGY

2.1 Materials

Material testing is the process to determine the engineering properties and the characteristics of material. Aggregate play important role in the strength of concrete so their properties really matter a lot. Therefore, it is necessary to test them in the lab for determining their shape, size, gradation, fineness modulus, specific gravity and the water absorption. This article describes the procedure as well as the mix proportion of the materials used in the research work. The section also elaborates the material properties of concrete mix. Ordinary Portland Cement (OPC), provided from Fauji Cement Company according to the requirements of ASTM C-150 was used. The following Table lists both chemical and physical properties of OPC.

Table 1:Chemical Composition and Physical Characteristics of OPC

Constituent (wt. %)	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	SO_3^{--}	Cl^-	Na_2O	K_2O	L.O.I	Total
Mass (%)	21.26	4.49	3.49	63.81	2.02	3.11	0.03	0.14	0.09	1.57	99.98

Physical Characteristics	Specific gravity	Consistency	Blaine's specific surface (cm^2/kg)	Initial setting time (min)	Final setting time (min)
Values	3.15	24%	2415	27	134

2.2 Sieve Analysis Test

Sieve analysis id widely used test in civil engineering for finding the gradation of aggregates to check whether they can be capable to be used in concrete or not. The sieve analysis of fine aggregates was performed as per ASTM C 136 and the gradation results are shown in the figure 1.

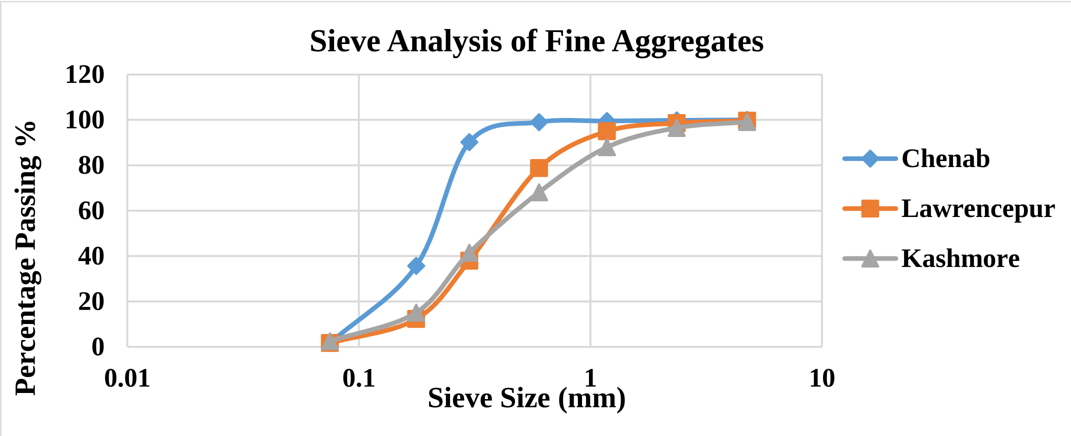

Figure 1: Gradation of Fine Aggregates from various sources

The graph shows that the Chenab sand is gap graded and has a large number of fine particles lesser than size of 0.17mm. Also the fineness modulus of this sand is obtained as 1.74 which shows that the average size of particles is in between sieve 200 and 100. This sand is not best for use in concrete due to very fine particles but can be used with mixing some coarse grained quantity of fine aggregate. A good cement sand mortar can be made using this sand. The Lawrencepur sand is obtained as coarser sand with almost 40% particles retained on sieve 80. Also the fineness modulus is 2.76 which tells its good capability to be used in concrete for better results. The Kashmore sand is well graded sand with having particles of all sizes due to which it can make very good bonding with coarse aggregate when used in concrete. The fineness modulus of this sand is obtained as 2.89 so this is very suitable for high strength concrete. The sieve analysis of coarse aggregates was performed and the gradation is shown in the figure 2.

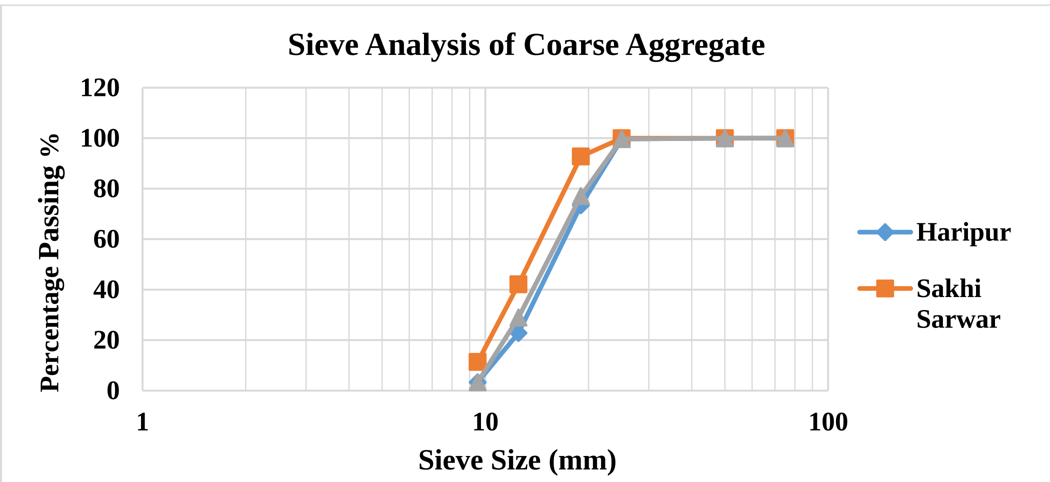

Figure 2: Gradation of Coarse Aggregates from different sources

The sieve analysis of Haripur and Margallah crush has almost the same gradation with well graded nature. Sakhi-Sarwar aggregate also has well gradation.

2.3 Water Absorption, And Specific Gravity Test

Specific gravity (G_s) is the ratio of density of a material to the density of water. This test is used to find out the specific gravity and the water absorption of aggregates. Water absorption values are helpful in determining the water absorbed by the pores available in aggregate particles. The Tests were performed as per ASTM C127 – 15. The values of specific gravity and water absorption of both coarse and fine aggregates are presented in table 2.

Table 2: G_s and water absorption of fine and coarse aggregates

Source Name	Specific Gravity	Water absorption
Margallah Crush	2.61	0.97
Haripur Crush	2.88	0.81
Sakhi-Sarwar Crush	2.63	0.70
Kashmore Sand	2.77	1.28
Chenab Sand	2.60	1.21
Lawrencepur sand	2.69	1.01

2.4 Preparation Of Mixes:

Six combinations were prepared for each type of crush. On these combination tensile strength and compressive strength test were performed.

For Margallah crush the combination used are
1. Margallah-Lawrencepur(ML),
2. Margallah Chenab(MC),
3. Margallah-Kashmore(MK),
4. Margallah-Lawrencepur-Kashmore(MLK),
5. Margallah-Lawrencepur-Chenab(MLC),
6. Margallah -Chenab-Kashmore(MCK).

Likewise, six combinations were prepared for Haripur crush. These combinations are
1. Haripur-Lawrencepur(HL),
2. Haripur-Chenab(HC),
3. Haripur-Kashmore(HK),
4. Haripur-Lawrencepur-Chenab(HLC),
5. Haripur-Lawrencepur-Kashmore(HLK),
6. Haripur-Chenab-Kashmore(HCK).

Six combinations were prepared with Sakhi-Sarwar Crush. These combinations are
1. Sakhi-Sarwar – Lawrencepur(SSL),
2. Sakhi-Sarwar – Chenab(SC),
3. Sakhi-Sarwar – Kashmore(SK),
4. Sakhi-Sarwar - Lawrencepur-Chenab(SLC),
5. Sakhi-Sarwar -Lawrencepur-Kashmore(SLK),
6. Sakhi-Sarwar-Chenab-Kashmore(SCK).

2.5 Slump Test

Slump tests were carried out for each combination prior to the pouring for having best workability of concrete according to ASTM C 143. Figure 3 depict the variation in slump values by using different combination of fine and coarse aggregates. All the slump values were in range of 45mm to 75mm.

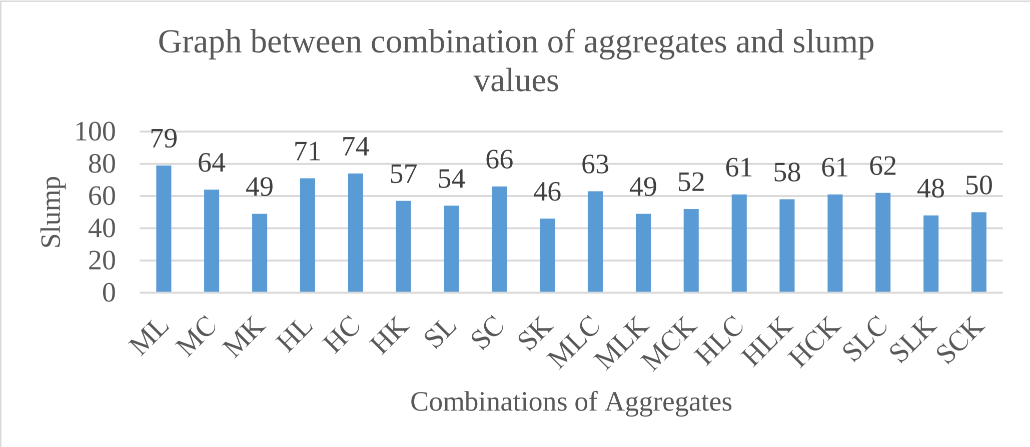

Figure 3: Variation in Slump with different aggregates combinations

2.6 Casting and curing of Concrete Specimens

All the samples were made with standard concrete cylinders of 12” height and 6” diameter. The mix was placed in the mould in three layers of equal depth and compacted using stand size rod with 25 blows on each layer. A total of 108 samples were casted in 6 days with 18 nos. of samples per day. Fig 4 shows the casted samples.

Figure 4: Casting of Test Specimens

The test samples were remoulded after 24 hours and then placed in curing tank for 28 days filled with tap water at room temperature.

2.7 Testing Method

The samples after their curing were tested for their compressive and tensile strengths using compression testing machine and universal testing machine. The compressive strength of 28 days cured hardened cylinders was measured according to the ASTM specification using standard compressive testing machine (CTM) having a loading rate of 100 KN/min For each combination three samples were tested for compressive strength and three were tested for split cylinder test as shown in figure 5..

Figure 5: Testing of Concrete Specimens

3. RESULTS AND ANALYSIS

3.1 Compressive Strength Comparison of Aggregates

Figure 6 present the 28 days compressive strength of all 18 combinations. It can be seen that the combination of Margallah crush with a blend of fine aggregates of Lawrencepur and Kashmore gave highest strength of 4179psi. It is may be due to the presence of all size of particles in Kashmore sand which reduced the voids in concrete and gave very good aggregate interlock.

All other combinations in which Kashmore sand was used gave higher results than the rest of the combinations. The lowest values are from the combinations of Haripur crush which are due to the rounded surface of aggregates which does not give strong interlock of aggregates.

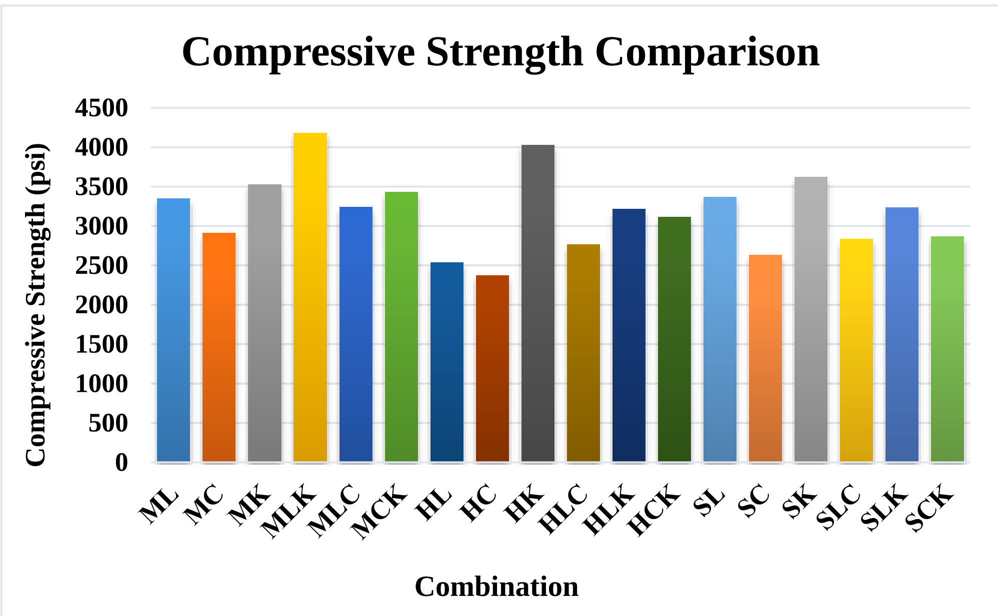

Figure 6: Variation in Compressive Strength with different aggregates combination

3.2 Tensile Strength Comparison of Aggregates

Split tensile strength was determined as per ASTM C 496. Specimens were casted and tested at 28 days of their curing. The results of split tensile strength are shown in figure 7. As for the compressive strength, all other combinations in which Kashmore sand was used gave higher results than the rest of the combinations. The lowest values are from the combinations of Haripur crush which are due to the rounded surface of aggregates which does not give strong interlock of aggregates.

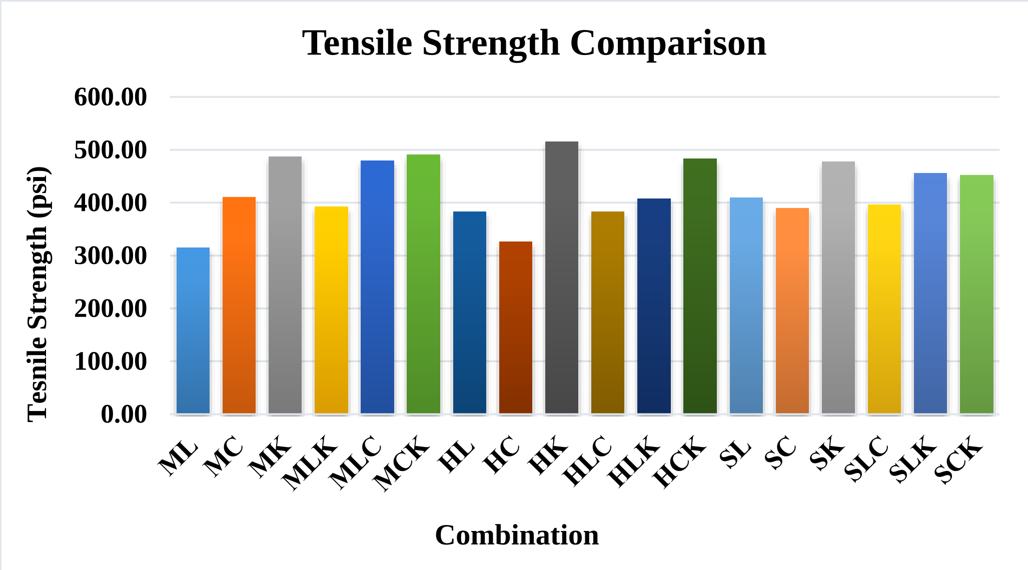

Figure 7: Variation in Split Tensile Strength with different aggregates combination

4. CONCLUSION:

- The compressive strength, combinations made using Kashmore sand either fully or partially, gave the best compressive strength values which was due to its well gradation which enables the great aggregate interlock in concrete due to very less air voids.
- The Haripur river gravel gave second best strength when combined with kashmore sand but gave lowest strength with other types of sands which was may be due to its rounded surface type and less angular structure which cannot make strong aggregate interlock with those sands. Also, the paste failure phenomenon was observed during the compressive and tensile strength tests which clears tells that there were poor bonding in between coarse and fine aggregates.
- Kashmore sand has fineness modulus higher than all three sands which showed that it will be suitable to be used for achieving good high strength concrete.
- In region around Kashmore, specifically in the DG Khan area where Sakhi Sarwar sand was widely used, the Kashmore sand can be used for better results. Other aggregates which were Sakhi-Sarwar crush, Lawrencepur sand and Chenab sand gave good results also but sand of Chenab river needs to be used in combined with other coarse-grained sands for better results as this sand was fine grained which causes air voids in concrete.
- The Haripur aggregates which was actually river gravels showed that it meets the engineering properties, parameters and requirements so it can be used as a

construction material but not for high strength concrete unless otherwise used with combination of Kashmore sand.

5. FUTURE RECOMMENDATION

This research work will be helpful to the construction industry in assessing the best combination of coarse and fine aggregate for the different areas of the country. They can now decide in a better way keeping the view the material and transportation cost without compromising the concrete structural properties.

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Paper ID: 133

Development of Structural Concrete via Waste Hair Fibers

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Abstract

Concrete is weak in tension due to micro cracks, the inherent property of concrete, which cannot be fully prevented in the hardened concrete. However, the occurrence of micro cracks can be mitigated by introducing human hair as an organic fiber. Human hair being a waste material, cause environmental pollution and badly disturbed the aquatic life. It is a cheap material and works as a function of steel reinforcement due to its contribution in the tensile strength of concrete. Hair also increases the compressive strength of concrete to some extent and reduces the permeability and volumetric expansion of concrete. For the purpose of this research, hair fibers have been collected from the barber shops near UET Taxila. The size of the hair fibers used ranges from 1.5 to 2 inches and added in the concrete mix by weight of the cement. Concrete mix of 1:2:4 were prepared for the target mean strength of 3000 psi. Cement was replaced with human hair weight of 65 grams, 130 grams, 195 grams and 260 grams in different batches. Compressive Strength Test, Splitting Tensile Strength Test, Permeability Test and Slump test were performed on 6 x12 inches cylinders for each batch at the age of 14 and 28 days. As a result, the tensile and compressive strength of concrete decreased at replacement of cement with 260 grams of hair by same amount. Water Permeability test performed on concrete for 3 days at 30 psi pressure showed that the permeability of concrete decreases by increasing the percentage of hair fiber.

Keywords: Hairs, Permeability, Compressive strength, Tensile Strength, Concrete,

1. INTRODUCTION:

Human hair is a material considered useless in most societies and therefore is found in the municipal waste streams in almost all cities and towns of the world [kumar, 2009]. Fiber are used in construction from old days. However, then only the small pieces of ropes were used in mud for construction of mud structure. Those small pieces increased the overall strength of the mud. Hair is used as a reinforcement in construction due to high frictional coefficient and tensile strength. In different areas of many European countries, India and Syria [Heyman, 2002] human hair is used in plastering of walls. Research shows that hair reinforcement increases the insulation capacity of structures [Pillai, 2012]. Human hair reinforcement reduces the cracks in cement mortar due to plastic shrinkage up to 92% [Al-Darbi, 2006]. Due to hair reinforcement compressive strength of concrete increases by over three times [Akhtar, 2009]. Fibers are used in plaster to increase its properties. In 1950s the concept of fiber reinforcement was a point of interest, but later on due to its health risks the use of asbestos fibers in concrete was discouraged [Naveen, 2015]. A problem discussed in local newspaper, the barber shops and parlors in and around UET Taxila caused serious environmental issues. There was no system present in Taxila Punjab to safely treat and decompose the human hair waste generated by the locals. Currently, this waste is thrown in the Gumrah Kas (near UET Taxila), which pollutes and disturbs the aquatic life adversely. After conducting a survey of the barbershops and hair salons near the UET Taxila, most of the shops agreed that the presence of human adversely affects the lives of locals. It was in this context that we decided to use hair as a reinforcement, in order to have safely channeled this waste for further use. Hairs are elastic in nature. In dry condition they increase their length 20 to 30% under stress, but in wet state the increase can be observed up to 50%. The resistance to breakage of hair is function of hair and diameter of thread. When the length of hair increases its elasticity decreases. Hair reinforcement also reduce the erosion of concrete by increasing its durability. Another additive feature of hair reinforcement is that if small micro-cracks appear in concrete, there is no chance of corrosion. It is better to use hair reinforcement in Hydraulic structures such as piers, sluice gates, where structural member are subject to high velocity water. Finally, human hair reinforces the concrete and prevents the sapling effect [Jain, 2012].

2. RESEARCH SIGNIFICANCE:

The main purpose of this research is to investigate the properties of hardened cement concrete after introducing specific percentages of human hair through performing the Compressive Strength test, Splitting Tensile Strength test, Permeability test and Slump test. The second major purpose is to introduce the idea of using human hair in concrete as a reinforcement material to decrease the environmental pollution. No work has been carried out in Pakistan on the utilization of human hair in concrete as organic fiber, so this work might improve the properties of the concrete by utilizing the waste material. Moreover, it will also reduce the environmental pollution caused due to non-utilization of human hair.

3. MATERIALS:

The material used in the test specimens were human hair (as fiber reinforcement), cement, sand, coarse aggregate and water. Hair were collected from barber shops and from hair salons around UET Taxila Punjab Pakistan. Sand source was of Lawrencepur and the coarse aggregate were brought from Margalla hills having size of less than 20 mm. Hydraulic Fauji cement was used as binding material. Initial and final setting time of cement were determined using ASTM C 191-04a [ASTM C191]. The initial setting time was found to be 34 minutes and final setting time came out to be 175 minutes. The consistency of the cement paste was 30. Hair were washed with acetone, separated, and left for drying purposes before utilizing for concrete preparation [Neville, 1990].

4. METHODOLOGY:

Compressive Strength, Splitting Tensile, Permeability and workability test were performed on each concrete batch containing human hair as fiber reinforcement. Concrete Cylinders of 6 inch diameter and 12 inch height were casted for compressive strength and splitting tensile test. For permeability test, cylinders of 6 inch diameter and 6 inch height were used. The amount of cement in each batch was replaced with hair fiber by 65g, 130g, 195g and 260g. Control mix of concrete was also prepared to compare results of different tests. For each batch, three samples were casted with mix ratio of 1:2:4 and fixed water cement ratio of 0.58.

5. TEST RESULTS:

Results of various tests performed on concrete sample containing human hair are discussed as under.

5.1 Workability Test:

For ensuring the workability of hair reinforced concrete, slump test and compacting factor test were performed as per ASTM C143-90a and British standard respectively. Results of compacting factor and slump test are tabulated in table 1.

Table 1: Comparison of the slump and compacting factor test results

Hair (grams)	Slump(mm)	Compacting Factor
00	28	0.81
65	31	0.85
130	33	0.91
195	30	0.88
260	26	0.76

Table 1 showed the workability of fresh concrete and it indicates that the workability of concrete increased initially till 130 grams' addition of hair fibers and then it started to reduce by increasing hair fiber as a reinforcement for the development of structural concrete. When slump decreases workability of concrete also decreases [Neville, 1990]. The value of compacting factor test also reduced to 0.76 which shows the decrease in workability.

5.2 Compressive strength Test:

Concrete cylinders were prepared for compressive strength of concrete by using different weightage of hairs with respect to mix ratio for different formulations. The compressive strength test was performed according to ASTM C 39-04a. [ASTM C39] at the age of 14 and 28 days. Results are tabulated in table 2 and are compared in fig 1.

Table 2: Compressive strength of concrete at 14 days & 28 days

Hairs by weight of cement (grams)	**Mean Compressive Strength (psi)**	
	at age of 14 days	**at age of 28 days**
0	2236.48	3237.24
65	2268.39	3285.1
130	2238.01	3280.83
195	2387.20	3427.24
260	2324.95	3321.36

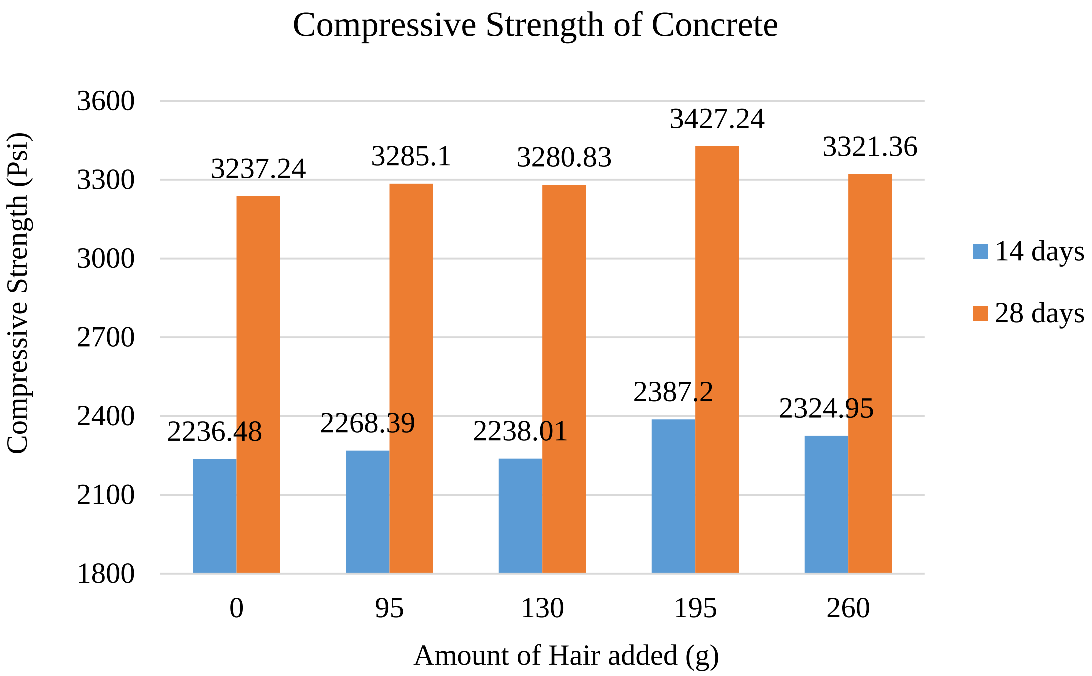

Figure 1: Comparison of Compressive strength of Concrete at the age of 14 and 28 days

From table 2 and figure 1, it was inferred that the compressive strength of concrete initially increases but when the amount of hair fibers exceeds 195 grams, it starts decreasing at the age of 14 days and 28 days. This can be due to excessive volume of hair fibers in the cement paste matrix which ultimately decrease the density and hence compressive strength of concrete. Moreover, figure 2 showed the failure/cracking pattern of concrete cylinders and it was obvious that there was resistance against cracks due to hair reinforcement otherwise specimen would split into pieces.

Figure 2: Failure pattern of concrete cylinders

5.3 Split Tensile Test:

Splitting tensile test was performed on cylinders according to ASTM standard [ASTM C-496], indirect tensile strength of concrete was determined at age of 14 days and 28 days as shown in table 3. The splitting tensile strength of concrete increased with increasing weight of hairs upto 195g. After that, the tensile strength of concrete decreased due to weak bonding between hair fibers and cement. During the testing of hair reinforced concrete, it was noticed that concrete showed only cracks without splitting into pieces contrary to as it split in control mix (concrete without reinforcement). This could be the reason hair reinforcement reduced swelling in concrete as shown in figure 3.

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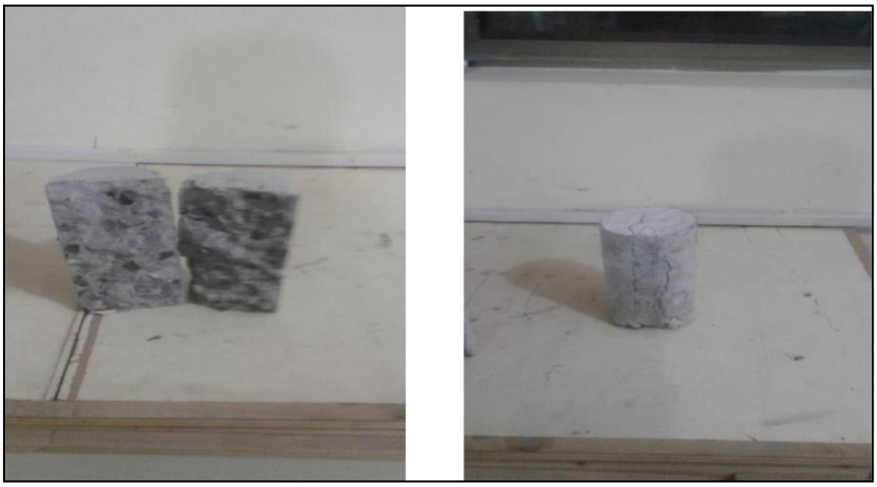

Figure 3. Internal structure of Concrete Cylinders

Table 3: Tensile strength of concrete at the age of 14 and 28 days

Hairs by weight of cement (grams)	**Tensile Strength (psi)**	
	at age of 14 days	**at age of 28 days**
0	494.6	603.4
65	509.0	617.9
130	530.8	639.6
195	546.8	655.5
260	514.9	623.7

5.4 Water Permeability Test:

The "Four Cell Automatic Concrete Water Permeability Apparatus" shown in figure 4 was used for investigating water permeability of concrete specimens. Figure 4 and figure 5 showed the pictorial and schematic layout of apparatus along with testing cell details. Cylindrical specimens of 6x6 inch were used to evaluate water permeability of concrete. Permeability test was performed as per ASTM standards [ASTM D2324-68]. Prior to test, samples were oven dried at 105˚C. Blue Emulsion paint was used to coat sides of specimens so that water can permeate through top and bottom Surfaces only. The specimens were subjected to hydrostatic pressure and water permeated was directly collected in graduated cylinder and measured. Table 4 showed the results of permeability test. For research purpose split the cylinder and measure the depth of penetration from those samples to measure the penetration of water.

Figure 4: Water Permeability Test apparatus

Figure 5: Application of Blue Emulsion pain to prevent penetration of water from sides

Table 4: Water permeability Test Results

Hairs by weight of cement (grams)	Penetration depth (mm)
0	18
65	13
130	9
195	6
260	3

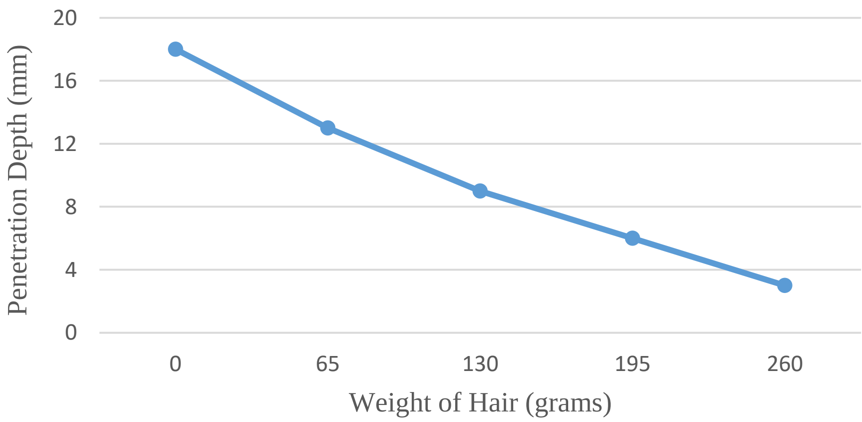

Figure 6: Variation of penetration depth with amount of hair reinforcement added

The graph between weight of hair and penetration depth, shown in figure 6, depicted that permeability of concrete decreased by increasing weight of hair in hair reinforced concrete. Initially in control mix, penetration depth of water is more but with the increment of hairs, penetration depth of water decreases. In other words, permeability of water decreases in hair reinforced concrete by increasing amount of water.

6. CONCLUSION:

The following conclusion can be drawn from the study:

- Workability of hair reinforced concrete initially increases then decreases by increasing the amount of hair fibers.

- Compressive strength of concrete increases at 65 grams, 130 grams and 195 grams of hair fibers addition while it decreases at addition of 260 grams possibly due to decrease in the density of concrete. Maximum 7.26 % increment in compressive strength of concrete was observed with addition of hair fibers as compared to control mix.
- Due to elastic behavior of hair, tensile strength of concrete increased at 65 grams, 130 grams, and 195 grams of hairs but decreased at 260 grams of hairs addition in the cement matrix. Tensile strength of hair reinforced concrete increased by 8.64% as compared to control mix.

- Permeability of hair reinforced concrete decreased by increasing weight of hair because hair fibers act as bridging elements and reduce the porosity of concrete.

7. FUTURE RECOMMENDATION:

Concrete is week in tension. To improve tensile properties of concrete we use steel reinforcement. Due to micro-cracking, there is a possible chance to corrode steel embedded in concrete. If we use hair as a reinforcement in concrete it reduced the propagation of cracks and prevent steel from corrosion.

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Structural Analysis and Design

Paper ID: 204

Lateral Resistance of Interlocking Stabilized Soil Block Walls with Different Geometrical Wall Configurations

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Abstract

Interlocking masonry walls exposed to lateral loads have reduced lateral resistance due to lack of its tensile strength, as reported in the literature. Different techniques have been used to enhance the lateral resistance of such structure which include use of reinforcement, grout within interlock, plastering and rendering etc. In this study 1500 mm high interlocked masonry walls mostly used in poor developing countries are experimentally tested. The research focus was to evaluate the structural parameters like stiffness, load at first crack and toughness by changing the geometrical plan of walls. Two walls of 3000 mm length were tested which include straight wall and non-straight wall. It was noted that first crack lateral stiffness of non-straight wall as compared to straight wall was improved by 800%. Non-straight wall also undertook twice the load than straight wall failure load without occurring failure.

Keywords: Interlocking stabilised soil wall, mortar-less, non-straight wall, lateral load, lateral stiffness.

1. INTRODUCTION:

One of the oldest construction materials is masonry. It is still being widely used with some improvements. Different types of the unit developed over the time from simple block to interlocking blocks have led to mortar-less construction. The types of the blocks used in the mortar-less construction around the globe include Haenar system, Mecano system (Vargas 1988), Putra Block (Thanoon et al. 2004), Bamba system, Tanzanian interlock block (TIB) system (Kintingu 2009) etc. Among these block systems, mostly have similarity with conventional units with the differences in projections and keys which provide interlocking mechanism for mortar-less construction (Safiee et.al, 2011). The performance of masonry interlocking walls is being widely explored (Velazquez and Ehsani 2000, Baqi et al. 1999, Rodriguez et al. 1998). Wind and earthquake loadings are typical examples of lateral loading, which is required to resist by masonry walls. Various research works were conducted to study the behavior of masonry walls exposed to lateral loads (Sokairge et al. 2017, Griffith et al. 2004, Uzoegbo 2001, Velazquez and Ehsani 2000 and Drysdale and Essawy 1988). In the research work by Uzoegbo 2001, experimental work is performed for mortar-less wall due to lateral loading and also plastering of walls are considered. The result indicated that the addition of plaster affected the lateral capacity of interlocked walls and it was noted that load capacity increased by 20%. In another study by Safiee et al. 2011, interlocking system was used and the behaviour of masonry wall under lateral load was experimentally investigated. It was concluded that the lateral behaviour was mainly controlled by large displacement and opening of block joint. In another research work by Sokairge et al. 2017, mortar-less blocks were used and tested for lateral loading. It was discussed that this system has some disadvantages like low bending capacity and also interlocking units had to settle down to balance uneven surfaces which could result in low strength of the walls. Therefore, some other mechanism or methods are required to overcome these deficiencies. In this study, Interlocking soil stabilized blocks (ISSBs) system was used to build the wall and was subjected to lateral load. In order to enhance the lateral resistance of these walls, technique of changing the geometric plan of the walls was explored. Comparison of structural parameters of both walls system was carried out which include peak load, lateral stiffness, energy absorption and toughness. This study will enable to explore the technique of enhancing the strength of the masonry wall by changing the geometry with similar material cost.

2. EXPERIMENTAL PROCEDURES:

2.1 Wall preparation and specifications:

The dimensions of interlocked stabilised soil block (ISSB) are shown in the Figure 1. These blocks were prepared in a machine with manual compression. Two walls (3000 mm long x 150 mm wide x 1500 mm high) were built with ISSB without any mortar. 600 mm block returns at the ends were given to avoid wall instability. Both walls tested against monotonic lateral load. Sample labelling is shown in Table 1. Sample A represents the straight wall whereas sample B has got return in the middle and classified as non-straight wall.

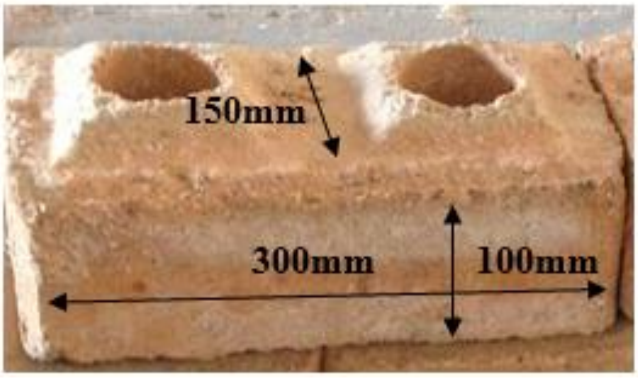


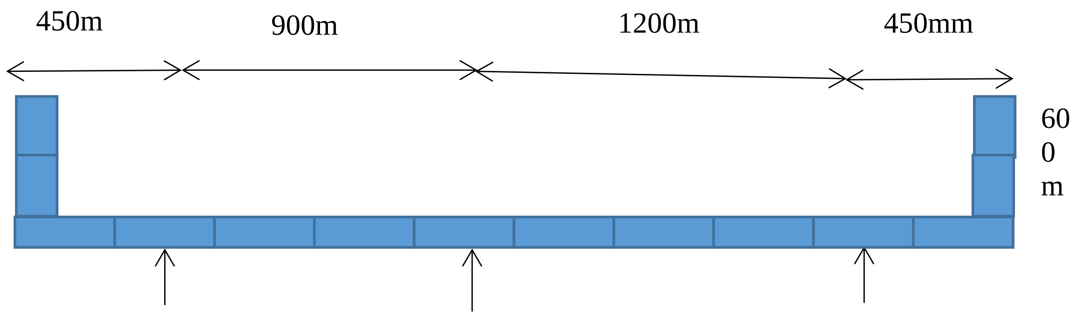
Figure 1: ISSB with dimensions

Table 1: Sample Labelling

Specimens	Straight wall	Non-straight wall
Label	A	B

2.2 Testing Procedure:

The straight and non-straight wall testing are shown in Figures 2 and 3, respectively. The application of lateral loading by using pulley at 1000 mm height is also demonstrated. Displacements at the top of wall are measured with non-digital theodolite. Smaller loads were chosen initially to get the smaller displacement. Load was then applied in increments of 40 N so as to observe first crack and peak load resistance.



a.

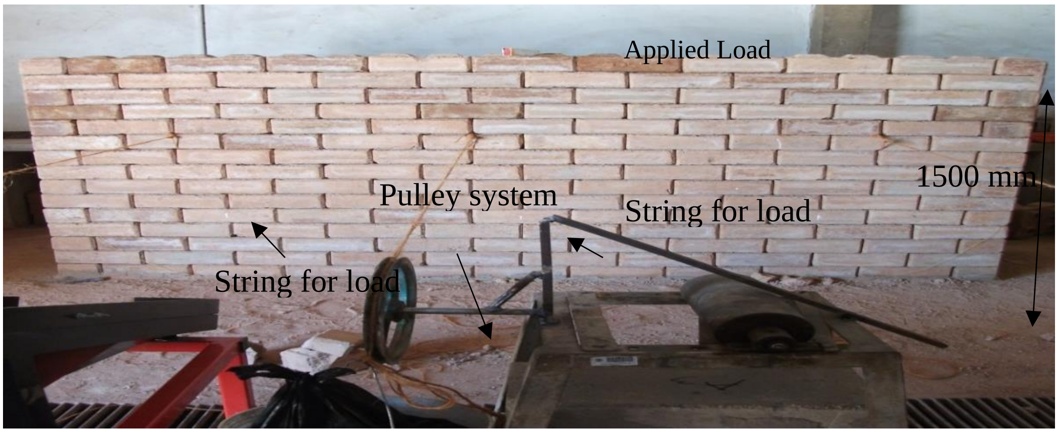


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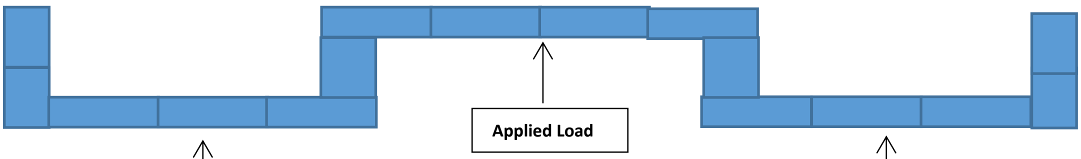
Figure. 2: Straight wall testing: (a) Schematic figure and (b) Test setup displaying system for lateral load



a.

Figure. 3: Non-straight wall testing: (a) Schematic figure and (b) Test setup displaying system for lateral load

3. RESULTS:

3.1 Behaviour against lateral load

The load displacement plot of each wall is detailed in Figure 4. It may be noted that around 5 mm displacement is observed against a load of 100 N for straight wall whereas loads for non-straight walls are much higher for the similar displacement. Non-straight wall showed very high stiffness. The highest load was found much higher for non-straight wall as compared to straight wall and increased to 617 N. This showed that by changing the geometric plan of the walls increased the stiffness and peak load.

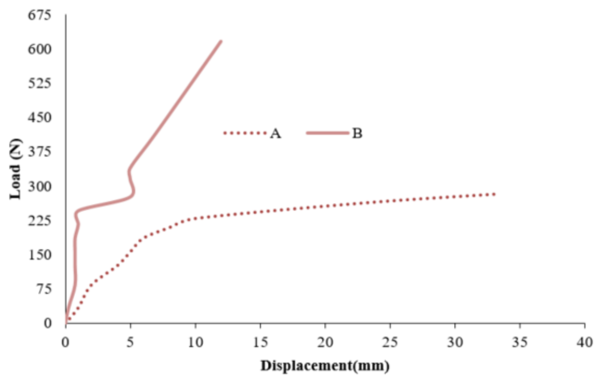


Figure 4: Load displacement curves for straight and non-straight walls

Figure 5 explains the highest loads for the straight and non-straight wall. There is an increase in failure load from 314 N for straight wall to 617 N for non-straight wall. Around two times increase in lateral resistance is observed.

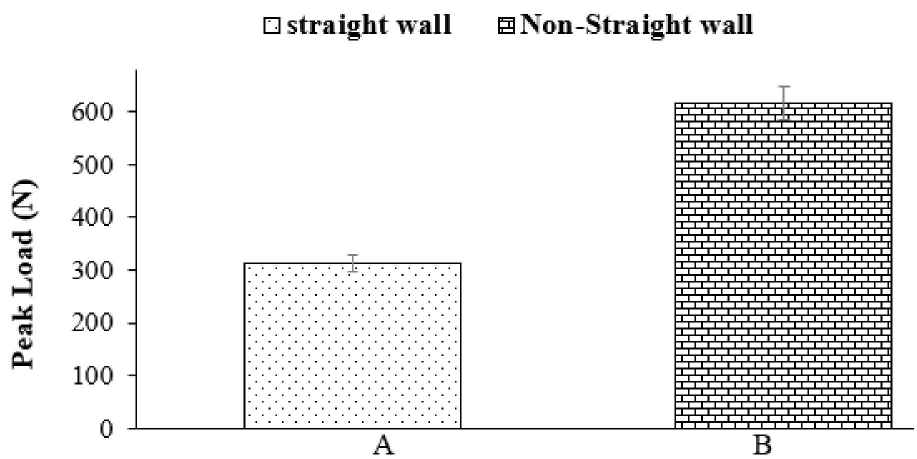


Figure 5: Comparison of peak load

3.2 Load, Stiffness and Energy Absorption Properties

The investigated parameters of both walls are presented in Table 2. These are load at first crack, peak load, first crack stiffness and energy absorption related to before and after crack. The load-displacement slope up to load at first crack is taken as the first crack stiffness. First crack and after crack energy absorbed are calculated as the areas of load displacement plots up to initial crack load and from initial crack to highest load, respectively. Overall energy absorbed was obtained by adding before and after crack energy. The fraction of overall energy to first crack energy is considered as toughness. It is noted that the first crack stiffness is enhanced from 8.48 N/mm for straight wall to 68 N/mm for non-straight wall. This is around eight times increase. It can be noted that the significant increase in first crack stiffness is achieved by changing the geometry of the wall. The before crack energy absorbed for non-straight wall is showing less value as compared to straight wall. This is due to limitation of loading as non-straight wall could not reached to failure load. However non-straight wall showed after crack energy absorbed and toughness index of 2.05. This showed ductile behaviour of non-straight wall as compared to straight wall after first crack. The straight wall showed value of toughness index 1.0, indicating brittle failure. The non-straight wall showed higher failure load but a lower strain, this is due to the fact that stiffness of the non-straight wall enhanced by changing the geometry of the wall.

Table 2: Investigated parameters of straight and non-straight walls

Specimen	**First crack load (N)**	**Peak load (N)**	**Frist crack stiffness (N/mm)**	**Before crack energy absorbed (N-mm)**	**After crack energy absorbed (N-mm)**	**Overall energy absorbed (N-mm)**	**Toughness (-)**
A	284	314	8.48	7526	0	7526	1.00
B	340	617	68	1888	1987	3875	2.05

4. CONCLUSIONS:

Experimental work was conducted to find the improvement in structural parameters by the changing the geometry of ISSB walling system. Two walls with different geometry were considered for lateral loading. The conclusions are found as below:

- First crack stiffness of non-straight wall as compared to straight wall was improved by 800%.
- Non-straight wall undertook twice the load than straight wall failure load without occurring failure.
- Non-straight wall showed ductile behavior after first crack as compared to straight wall with toughness index of 2.05.

From the results, it can be concluded that non-straight wall enhanced the lateral stiffness and lateral load as compared to straight wall. Further investigation required to observe the effect on strength of non-straight wall by inclusion of plastering and fibrous plaster. Thus, cement content can be reduced with reduction in plaster thickness by utilizing the strength gain due to different geometry for economical solution.

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Effect of Lift Core Wall Location in High Rise Buildings

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Abstract

High Rise buildings are designed as frame structures with shear walls to provide sufficient strength, stability and stiffness against these lateral earthquake forces acting on the building. For the convenience of users, a Lift Core wall (LCW) is provided instead of shear wall which serves the same function as that of shear wall. In this study an attempt is made to study the different locations of the LCW in a 10 storey building, analysed using ETABS 2016 Static Force Method as per UBC-97 in Seismic Zone-3 of Pakistan. A LCW is provided at 4 different locations and the results are compared on the basis of displacement, Storey drift and Storey stiffness to select the best location of a LCW. It was found that LCW offers maximum seismic resistance at the centre of the building.

Key Words: Lift Core Wall (LCW), Lateral Stiffness, Storey Drift, Torsional irregularity, Static force method.

1. INTRODUCTION

The damages caused by an earthquake must be kept in mind before designing a building because it has caused enormous calamities in Asia and other continent (Azad & Gani, 2016). Lateral earthquake forces are greater in case of Reinforced Concrete multi-storey buildings, so they are designed to resist these loads. A Shear wall is the best solution to give structural stability to high rise buildings (Schodek, Subagdja, & Suryoatmono, 1999). Shear wall is a structural element which provides stiffness, strength and stability against the lateral forces that are acting upon it. So, high rise buildings are designed as framed structures with shear walls to resist the cracks or bending in order to ensure the stability of the tall buildings (Chandiwala, 2012). For architectural purposes and for the convenience of users, a shear wall is replaced by LCW, which provides the desired strength and stability to the buildings and with open sections it also accommodates an elevator shaft or a staircase (Constantin & Beyer, 2012; Goud & Pahwa, 2016). A LCW must be provided at such a location where it provides maximum seismic resistivity (Varna & Bhavana, 2017).In this paper, we will study the effect of location of a LCW on the stiffness, lateral displacements and storey drift of the building.

2. RESEARCH METHODOLOGY

In this study, ten storey commercial building was analysed by Static Force Method. The building was assumed to be situated in seismic Zone 3 of Pakistan using ETABS 2016. Rectangular LCW was provided at 4 different locations and earthquake forces in both the x-direction and y-direction were considered.

2.1. Building Layout

The building considered had 5 bays in x-direction and 5 bays in y-direction. Length of each bay was 20 ft, so the total area of the building was 10,000 ft^2. Centre to centre height for each storey was 12 ft, so the total height of the building was 120 ft.
15 in x 18 in concrete beams, 18 in x 18 in columns, 7 in thick slab and 9 in thick rectangular LCW of 10 ft x 7 ft with 4 ft x 7 ft openings in y direction was selected for analysis. Column supports were assumed to be fixed.

2.2. Lift Core Wall Locations

LCW was provided at four different locations (Fig-1) and the building was analysed according to UBC-97. These locations were named as:
L_0 – Frame with no LCW
L_1 – First location of LCW (Corner)
L_2 – Second Location of LCW
L_3 – Third Location of LCW
L_4 – Fourth Location of LCW (Centre)

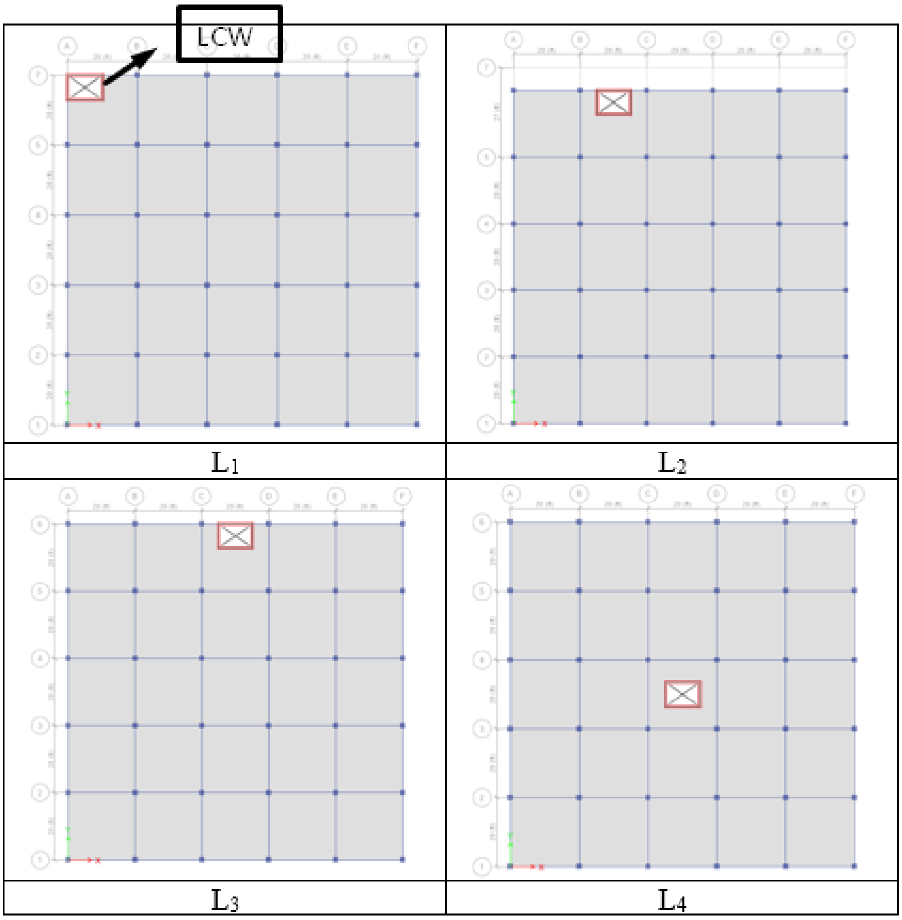


Figure 1- LCW Locations

2.3. Material Properties:

Table 1-Material Properties

Material Properties		
Material	Properties	Consider Value
Concrete	Concrete compressive strength, fc'	3000 psi
	Concrete Modulus of Elasticity, Ec	57000 √fc' psi
	Weight per unit volume	150 pcf
	Poisson's Ratio	0.2
Steel	Steel Type	Grade 60, ASTM A165
	Reinforcement Yield Strength,fy	60 ksi
	Tensile Strength, Fu	90 ksi
	Specific Weight	490 pcf
	Steel Modulus of Elasticity, Est	29,000 ksi
Site	Seismic Zone	3
	Soil Type	SD

2.4. Loads on the building:

Load for the structure was considered as per UBC-97. Self-weight of the structure was considered as Dead load. 60 psf Live load was considered for typical floors, while for the roof it was reduced to 40 psf. Similarly, 3 in thick marble and 1 in thick tiles were considered as Floor finish loads whose value was found to be 43.75 psf. This value was considered for typical floors but for roof it was increased to 60 psf. For typical floors 9 in thick brick wall was considered with a height of 10.5 ft and its load was calculated to be 1 k/ft while for roof 4.5 in thick brick wall was considered whose load was calculated to be 0.135 k/ft (Table-2).

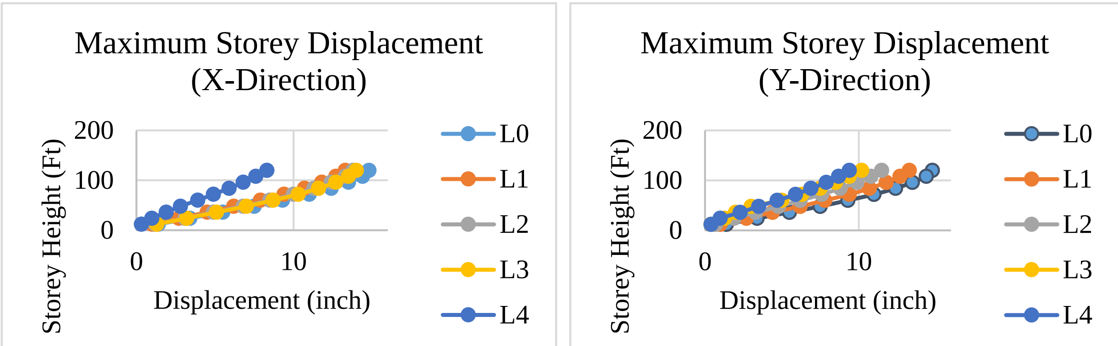
Table 2- Different Loads

Load type	Load Concentration	
	Typical Floor	Roof
Dead Load	Self-Weight of the structure	Self-Weight of the structure
Live Load	60 psf	40 psf
Partition Walls Load	21 psf	21 psf
Floor Finish (Mortar+Tile)	43.75 psf	60 psf
Wall Load on beams	1 kip/ft	0.135 kip/ft

3. RESULTS AND DISCUSSIONS:

3.1. Maximum Storey Displacement:

The maximum storey displacements were decreased by incorporating LCW in the building (Fig-2). In both x and y direction, the maximum storey displacements were effectively reduced by the incorporation of LCW at L4. In x-direction, the second best location was L1 (Fig-2a) while in y-direction; the second best location was L3 (Fig-2b).



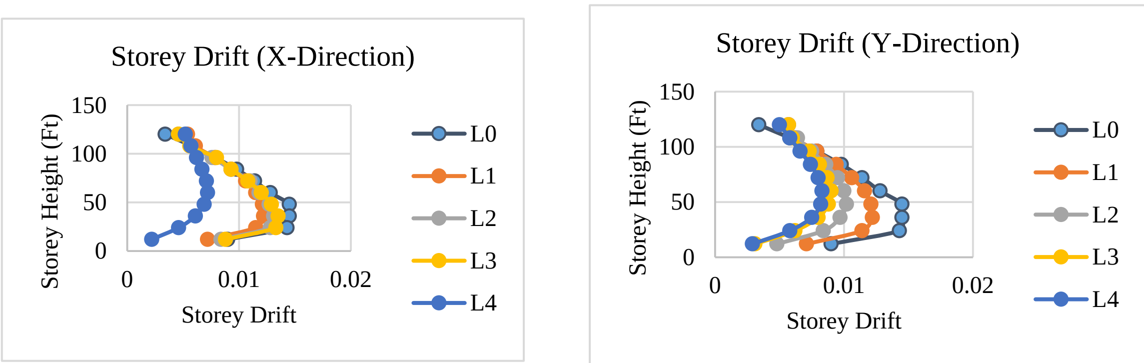
a. b.

Figure 2-Maximum Storey Displacement

The maximum displacements in y direction at L_3 and L_4 were almost the same because the LCW at both the locations is located at the centre of the building corresponding to earthquake.

3.2. Storey Drift

The storey drift was reduced by incorporating LCW in the building (Fig-3). At L4, the storey drift was minimum in both x and y directions. However, at locations other than L4, the results were different in x and y direction because of the different distribution of forces in both directions. For example, L1 had second minimum value of drift in x-direction (Fig-3a) but it also had the largest drift value in y-direction (Fig-3b).



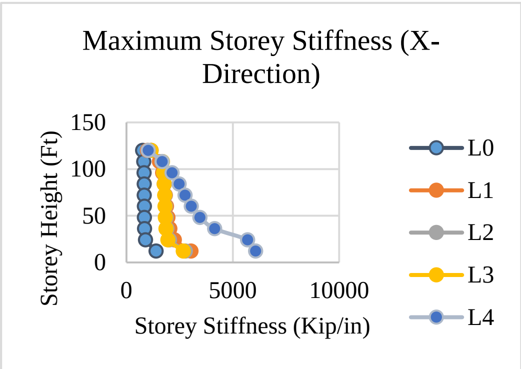
a) b)

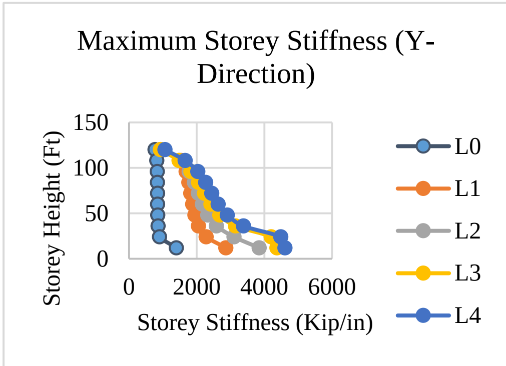
Figure 3-Storey Drift

Similarly, L3 had second minimum drift value in y-direction while maximum drift value in x-direction.

3.3. Storey Stiffness

The storey stiffness in both the x and y direction was found at L_4 (Fig-4). The stiffness of the building in x-direction was more than that of y-direction because of the in-plane behaviour of LCW. The LCW is provided for in-plane forces. As the length of wall is more in x-direction compared to the length of wall in y-direction, so it will take more loads in x-direction and the stiffness of the building will be increased. At locations other than L4, the maximum stiffness in x-direction was found at the corner of the building (L_1) and this value decreased as the LCW location was changed from L_1 towards L_3 (Fig-4a). While in y-direction, the stiffness value was minimum at L_1 and it was increased as the LCW location was changed from L_1 towards L_3 (Fig-4b).



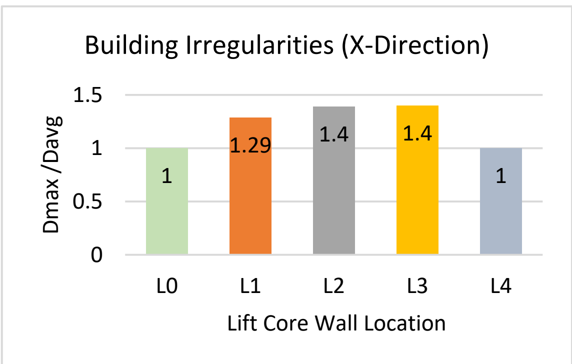


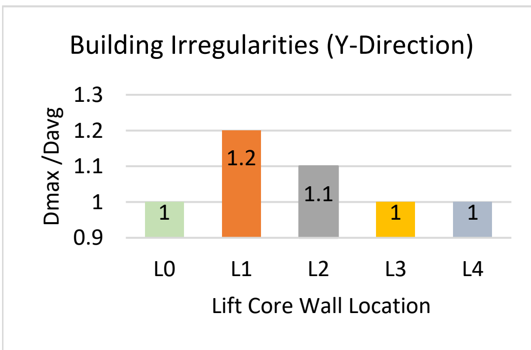
a. b.

Figure 4-Storey Stiffness

3.4. Torsional Irregularities:

When a building does not have the same centre of mass and centre of rigidity, torsional irregularities are produced in the building. These torsional irregularities might cause the failure of the structure. Dmax/Davgis a factor that is used to find the torsional irregularities in buildings.





a. b.

Figure 5-Irregularities in building

If this value is more than 1.4, the building will have extreme torsional irregularities, if it is greater than 1.2, the building have moderate torsional irregularities, while if it is less than 1.2, the building does not have any torsional irregularities.
From Fig-5, it can be seen that: At L_0 and L_4 there were no torsional irregularities in building because of the same centre of mass and centre of rigidity. At L_1, the building had moderate torsional irregularities in both x-direction and y-direction. At L_2 and L_3, the building had extreme torsional irregularities in x-direction, while there were no torsional irregularities in y-direction, because, there was more distance between the centre of mass and centre of rigidity in x-direction while in y-direction there was less distance between them.

3.5. Base Shear

In static force method Natural time period is a function of height. As height of the building is same so the natural time period T was also same (1.088 sec) for all the locations of LCW. Other factors Cv, R, Importance factor and weight of the building was same for all the locations of LCW, so the Base Shear for all the locations was also almost the same (fig. 6).

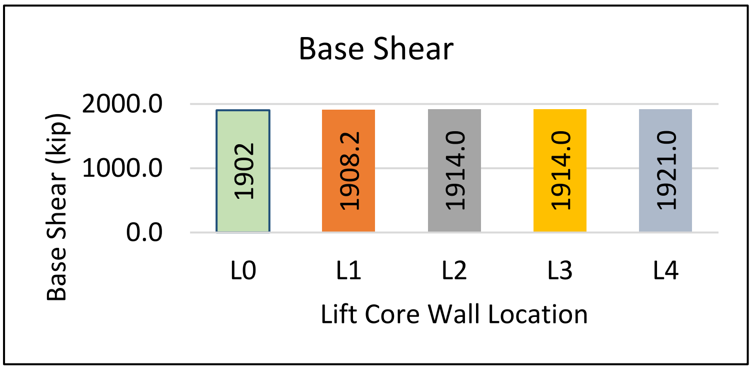


Figure 6-Base Shear

4. CONCLUSIONS

1) LCW reduces the displacement and storey drift and increases the lateral stiffness of the building, so it is better to use LCW in buildings to resist earthquake forces.
2) LCW must be provided at the center of the building because it gives maximum stiffness, minimum displacement and minimum storey drift and produces no torsional irregularities.
3) Building must be checked for torsional irregularities if LCW is to be provided at different location other than the center of the building because LCW produces torsional irregularities.

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Effect of Fundamental Period on Seismic Design of Reinforced Concrete Structures

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Abstract

Seismic design of reinforced concrete structures is becoming more important in high seismic areas of developing courtiers, because of increased seismic activity. There are number of well defined design codes like Uniform Building Code (UBC) 1997, Federal Emergency Management Agency (FEMA), Washington, 1997, and International Business Code (IBC) 2000, etc. which are used in developed countries for seismic design. Seismic design depends on the base shear (V) of the building which acts on the building when any seismic activity happens. UBC 1997 gives empirical equations for calculation of 'V'. The coefficients involve in calculation of (V) depend upon the construction practices and design technique prevailing in the developed countries. Similarly this code gives two methods for the calculation of fundamental period 'T' of the building. This paper describes the influence of structure's fundamental period on the seismic design characteristics. Two different methods define by UBC 1997 have been used in the paper to estimate the fundamental period of the structure. Based on the analytical findings, the research concludes the ineffectiveness of method B for structures with large fundamental period in high seismic zones. T_A and T_B are discussed in detail along with the factors on which T_A & T_B depend. Graphs between base shear coefficient (Vc) and period (T) are developed and discussed for all seismic zones. Moreover, a regular high rise reinforced concrete building is analyzed, designed and compared for both fundamental periods. Comparison shows an ample variation in the forces, design and civil cost of same building for the two cases.

Keywords: Uniform building code, seismic design, fundamental period, building height, reinforced concrete structures

1. INTRODUCTION

The frequency of occurrence of earthquake is increasing day by day. The buildings designed according to prevailing codes are also damaged by these off & on jolts. Many lacunae in construction as well as design have come to light while analyzing the failures due to earthquake. In order to design a structure to withstand an earthquake, the forces on the structure must be specified. The exact forces that will occur during the life of the structure cannot be known. When any earthquake hits the structure, seismic forces arise from the vibration of the mass of the structure. The frequency of these vibrations and corresponding period play an important role in response of the structure. The period can be determined from the equations defined in specified building codes. It is therefore important that careful consideration should be given to the fundamental period of a building in its planning and design stage.

According to UBC-1997, the world is divided into different seismic zones with respect to the intensity of seismic hazards. For a particular zone when a maximum intensity of earthquake jolts any building, the code gives formulae to estimate maximum limit of base shear. Base shear generates seismic forces which will act on a building. Base shear is the only factor which makes the seismic design of the structure different from its gravity design. Base shear is the combination of base shear coefficient (will be termed as Vc) and building dead weight. Vc is multiplied by building's dead weight to find out the magnitude of Base Shear. The coefficient of Base Shear is dependent upon i) seismic zone coefficient Z, ii) soil profile coefficient Ca & Cv, iii) Building response coefficient Rwx & Rwz, iv) building importance factor I, v) fundamental period T. All of above mentioned parameters excluding fundamental period are constant, for any Intermediate Moment Resisting Frame (IMRF) or Special Moment Resisting Frame (SMRF), in a particular zone & soil profile except fundamental period. Code defines two methods for determination of period either T_A or T_B, and variation among both periods may be up to 30 % for zoned 4 and 40% for rest of the zones. This variation generates a marginal difference in design forces which are addressed in this study.

In this regards B N Pandya (4) has published a paper presenting a study carried out to compare the fundamental natural period (FNP) obtained by free vibration analysis of reinforced concrete buildings considering various configuration irregularities with the values of FNP obtained from empirical formulae given by Indian Standard Code IS 1893 (Part 1): 2002, International Building Code IBC 2000 and Federal Emergency Management Agency FEMA 368. It was found that structural configuration irregularities tend to increase the FNP and that the IS 1893 (Part 1): 2002 empirical formula gives FNP which is almost half the computed values. The IBC 2000 and FEMA 368 empirical formula gives FNP, which varies marginally from the computed values.

2. PERIOD AND STATIC LATERAL FORCE PROCEDURE

The period 'T' of the structures is defined as "elastic fundamental period of vibration, in seconds, of the structure in the direction under consideration"

UBC-1997 presents a stepwise procedure for determination of lateral forces.

The flow diagram to represent the calculation procedure for different seismic parameters is shown here. The following equations represent the relationship of base shear with period.

CALCULATION FLOW
I → R → Zone Z → Soil Profile Type → Cv → T → W → V → Ca → V(max) → V(min) → Ft → Fx → Vx → Mx → Drift → Reliability

$V = C_v I W/RT$ eq. (1) $V_{max} = 2.5C_aW/R$ eq. (2) $V_{min} = .11C_aW$ eq. (3)

Where: V = Base shear, W = Total dead load, R = Response modification factor, T = Time period. C_a = Acceleration based ground response coefficient, C_v = Velocity based ground response coefficient, I = Importance factor,

In this paper combination of C_v, I, R & T is considered as base shear coefficient (V_c).

2.1 Governing Parameters Of Period By Method 'A'

In UBC 1997 following formula is used for the calculation of Fundamental period (T) for all the buildings from Method 'A'.

$$T = C_t (H_n)^{3/4} \qquad \text{eq. (4)}$$

C_t = .035 (.0853) for steel moment resisting frames.
C_t = .030 (.0731) for reinforced concrete moment resisting frames.
C_t = .02 (.0488) for all other buildings.
H_n = Height in feet (meters)

The value of Ct for structures with concrete or masonry shear walls may be taken as $.1/\sqrt{Ac}$ (For SI: $.0743/\sqrt{Ac}$ for A_c in m^2).

The value of A_c shall be determined from the following formula

$$A_c = \sum A_e [0.2 + (D_e/H_n)^2] \qquad \text{eq. (5)}$$

The value of D_e/h_n use in formula shall not exceed 0.9.

Fundamental period calculated form Method 'A' is termed as T_A depends upon height, so it remains constant for a particular building. C_t is dependent upon type of building, type of material and also upon construction methodology.

2.2 Governing Parameters Of Period By Method 'B'

In UBC 1997 the calculation of Fundamental period from Method 'B' is termed as T_B calculated following the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The value T_B shall not exceed 30 percent greater than the value of T_A obtained from Method A in Seismic Zone 4 and 40 percent in Seismic Zone 1, 2 and 3. The fundamental time period may be T_B computed by using the following formula.

$$T = 2\pi \sqrt{(\Sigma w_i \delta_i^2 \div g\, \Sigma f_i \delta_i)} \qquad \text{eq. (6)}$$

The value of f_i represents any lateral force distributed. The elastic deflection δ_i shall be calculated using the applied lateral forces, f_i.

As T_B is dependent upon weight & deflection of the structure, so it is highly variable. Deflection can be reduced by increasing stiffness of the structure and weight is also variable due to architectural considerations or any other building's usage requirement.

Base shear calculated from T_A is normally more than that calculated from T_B. Apparently, T_B seems to be more realistic than T_A as it considers many factors as shown in its equation. But codes give freedom to use any one method, so importance of its use is of much interest. Selection of T (either by method A, or by method B makes a marginal change in design) is made in this research.

The coefficients C_t, R and I depends on the construction methodologies and construction techniques. The construction techniques in developed countries are very well defined and followed. In developing countries these design codes are applied in design but importance is not given to construction techniques, which make the design vulnerable and doesn't give the same level of safety against earthquakes. Therefore it is vital requirements to either strictly follow the same level of construction methodologies or revise those coefficients against prevailing construction practices in developing countries.

2.3 RELATIONSHIP BETWEEN BASE SHEAR COEFFICIENT AND PERIOD

The relationship between base shear coefficient & period is shown in eq. (1) to eq. (3). In these equations all the parameters are fixed for a particular zone and soil profile except period will depend upon the structure. Thus graph can be plotted for determination of base shear coefficient against different values of period. Graphs are plotted against different values of period.

These graphs show the range of period for any building vary from T_A to $T_B \leq 1.4T_A$ in Zone 1, 2A, 2B & 3, for zone 4 period ranges is T_A to $T_B \leq 1.3T_A$. These graphs are developed for all zones and soil profiles but only graphs for soil profile E in all zones are presented here due to insufficiency of space available for this paper. If the building need to design against T_A
Another interesting thing presenting in these graphs is same base shear coefficient against two different periods. As in zone 3 V_C is .00618 against period 1.6 if T_B is considered and 1.15 if T_A is considered. This shows that 201ft high RC frame building will have same V_C with T_B, as it is for 129ft high building with T_A.

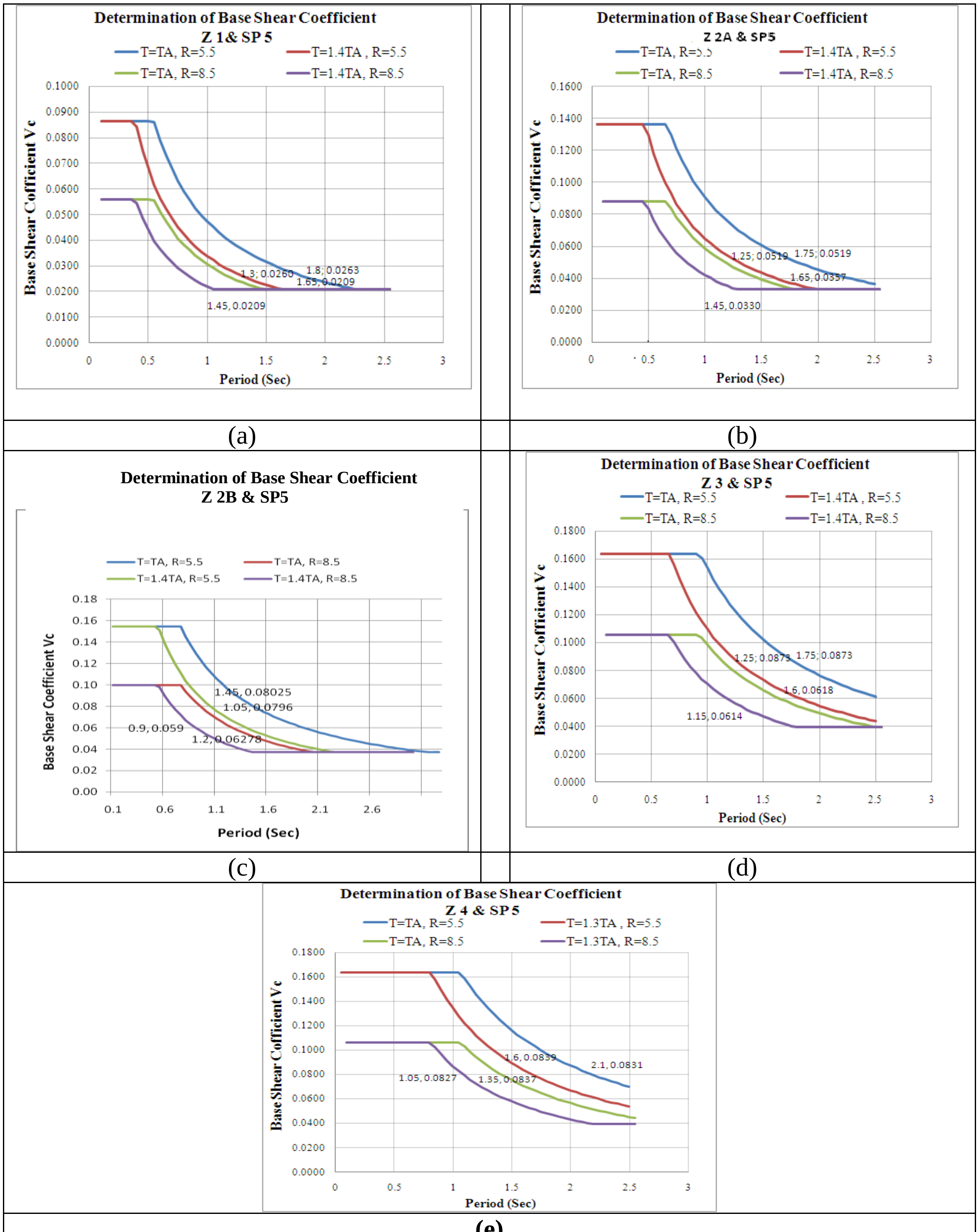


Figure 1: Graphs Between Base Shear Coefficient and Period for Soil Profile 'E' Shown as SP5 and Seismic Zones Zone 3 and & Zone 4

3. CASE STUDY

Regular Structure selected for this research is (3B +G+ 16) 20 storey office building to see the effect of fundamental period on the structural design. Different structural elements of the building have following properties.

i) All basement beams are 12”x27”, ii) All peripheral floor beams are 13½”x36”, iii) All internal floor beams are 12”x36”, iv) Columns from lower basement to 2^{nd} floor are 48”x48”, v) Columns from 3^{rd} floor to 6^{th} floor are 42”x42”, vi) Columns from 7^{th} floor to 10^{th} floor are 36”x36”, vii) Columns form 11^{th} floor to 14^{th} floor are 30”x30”viii) Columns from 15^{th} floor to 20^{th} floor are 24”x24”, ix) Thickness of Pile Cap/ Raft is 66”, x) Thickness of Basement Wall is 12”.ix) Building is 243ft high. Penthouse height is 18.5ft.

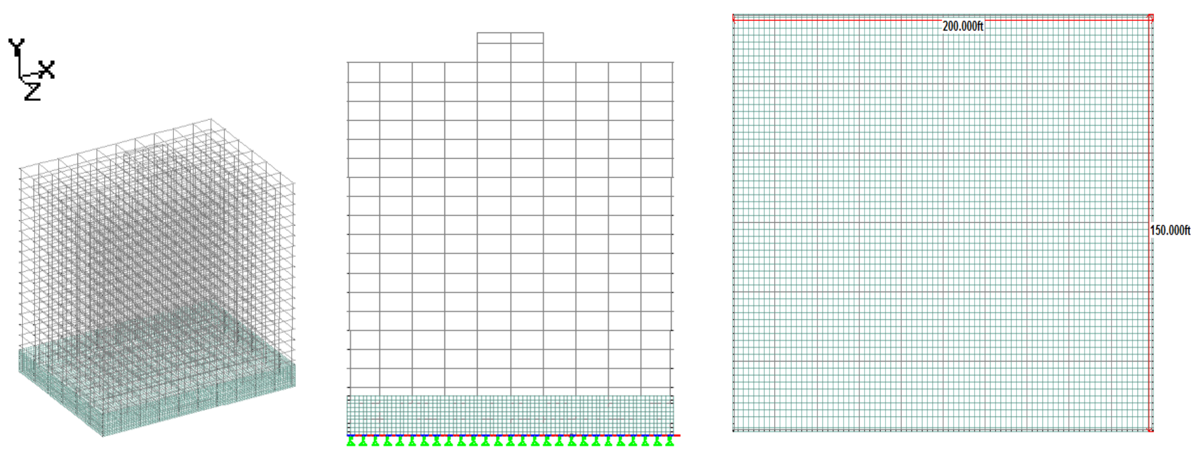
3.1 GENERAL PARAMETERS

Building is designed for basic five loads i) Seismic Load in X-dir, ii) Seismic Load in z-dir, iii) Dead Load, iv) Live Load, v) Roof Live Load. Seismic and factored Load combinations are determined from basic load combination 1612.2.1 of UBC-1997. Seismic parameters considered are i) R_{WX} & R_{WZ}= 8.5, ii) I=1, iii) N_A & N_V= 1, iv) S=5, v) Z=.3 & .4. The schematic views of the building are shown in Fig-02.

Pile foundation is provided as recommended by Geo-Tech investigation report. Fig 02 (b) shows the spring which are designed against piles stiffness calculated with following formula using values given in report.

K (FY) = Pile Capacity/Settlement.

In material properties all main reinforcing steel is deformed bars with 60 ksi yield strength, where as for secondary steel is mild with 40 ksi yield strength, fc’ compressive strength of concrete for columns is 4 ksi & 3 ksi for other structural members.



(a) 3D View (b) Front View (c) Top View

Figure 2: StaadPro Model of Structure

3.2 STRUCTURAL ANALYSIS

In structural analysis following parameters are determined, which are used in calculation of design forces for all structural elements.

3.2.1 FUNDAMENTAL PERIOD

Time period provided for structural design is calculated against total height of building excluding penthouse, shown as follows

- T_A 1.795 in X & Z DIR. for Zone 3 & 4 (considering building height only)
- T_B 2.34 (1.3 T_A) in X-DIR & Z-DIR for Zone 4,
- T_B 2.34 (1.3 T_A) in X-DIR 2.513 (1.4T_A) in Z-DIR. For Zone 3

Comparison of both designs with time period T_A and T_B is shown in terms of percentage reduction in forces.

3.2.2 BASE SHEAR & DEFLECTION

Base shear determined in both directions is tabulated in Table (01). For a particular building displacement of nodes make a considerable variation in design forces due to PΔ effect. Therefore values of maximum displacement are also noted for each model. Displacements are only noted against factored loads with seismic force combination to make a comparison.

Table 1

SOIL PROFILE 'E'	BASE SHEAR (KIPS)				DEFLECTION (INCHES)			
	T_A		T_B		T_A		T_B	
	X DIR	Z DIR	X DIR	Z DIR	X DIR	Z DIR	X DIR	Z DIR
Zone 3	10248.44	10248.44	7861.51	7371.52	10.978	14.32	9.178	11.607
Zone 4	11712.5	11712.5	9003.83	9003.83	12.545	16.365	10.503	13.78

3.3 STRUCTURAL DESIGN

3.3.1 PILES / PILE CAP-RAFT

Piles design is not in the scope of this project, only forces acting on piles are shown. In building in Zone 3 & 4, almost 70% piles have governing forces against gravity loads so design remains same either T_A or TB is used. For 30% piles governing forces are against seismic load combinations. Thus when pile designed with T_A is compared with T_B the reduction in axial forces is 6% and in plane forces are also reduced up to 25%.Piles with 30" diameter and 76 ft length is assumed to be sufficient for both designs.

Pile cap/raft with 66" thickness is provided. The reduction in design of raft for a building in Zone 3 with T_B is 8% & 16% for top moment in longitudinal & transverse direction, where as bottom moment is reduced 13.5% & 58.7% for exterior & inner column strip in longitudinal. For transverse direction it is 21% & 27% respectively. Area of steel is reduced in only on those locations where flexural moment governs and reduction is almost in same percentage as moment reduced. The reduction in design of raft for a building in Zone 4 with T_B is 9% for top moment in longitudinal & transverse direction, where as bottom moment is reduced 20% for exterior column strip in longitudinal & transverse direction. Area of steel is reduced in only on those locations where flexural moment governs and reduction is almost in same percentage as moment reduced.

12” thick retaining wall is provided all around the basement. For design of retaining wall moment does not govern and only minimum reinforcement against temperature and shrinkage is provided. Results for columns and beams are tabulated below.

3.3.2 COLUMNS

Design of 20 storey columns is divided into 5 parts. Each 4 storey have same X-section and almost they have similar results, so same design is used for 4 storey column. In general almost all columns up to 16 storeys have minimum 1% area of steel. So there is no comparison. Whereas last four storey column are designed against governing forces and area of steels varies marginally. Some results are presented in Table (02) shown above.

Table 2

Typical Column Represents	Percentage reduction with TB in Z=3				Percentage reduction with TB in Z=4			
	P	**MX**	**MY**	**As**	**P**	**MX**	**MY**	**As**
Exterior Column	-1%	-	-3%	-3%	-1%		-3%	-3%
Inner Column	-1%	-	-2%	-3%	-1%		-2%	-3%

3.3.3 BEAMS

Beams results also tabulated to represent the comparison given in Table (03). Where (M_{+ive}) & (M_{-ive}) represents bottom and top moments of beams. (T) represents torsion and (V) represents shear in beams.

Table 3

Average Beam Results with Storey Level	Percentage reduction with T_B in Z=3						Percentage reduction with T_B in Z=4					
	M_{+ive}	**M_{-ive}**	**T**	**V**	**Bottom Steel req.**	**Top Steel req.**	**M_{+ive}**	**M_{-ive}**	**T**	**V**	**Bottom Steel req.**	**Top Steel req.**
1ST TO 5TH	19%	25%	20%	12%	17%	26%	18%	24%	19%	12%	19%	26%
5TH TO 10TH	21%	28%	13%	17%	20%	29%	19%	25%	12%	16%	20%	26%
11 TH TO 15TH	18%	27%	3%	14%	18%	29%	17%	24%	3%	13%	18%	28%

16TH TO 20TH	5%	5%	2%	3%	4%	5%	5%	5%	2%	3 %	7%	8%

4. COST ANALYSIS

In both cases concrete outline of all structural members is kept same. Therefore comparison is only possible among area of steel ratio of all structural elements of the structure. The accumulative concrete quantity for both cases is 423,000 CFT. Whereas G-60 steel required for case study in Zone 3 is 5435 tons with T_A and 16% reduction is found for T_B. Similarly for case study in Zone 4, G-60 steel required is 5877 ton for T_A with 12% reduction for T_B. Total civil cost for both cases is 926million PKR with T_A & 846.8 million PKR with T_B for case study in Zone 3 and for case study in Zone 4 total civil cost is 968 million PKR with T_A and 904.5 million PKR with T_B. Thus 8.6% cost is reduced if building is designed with T_B as compare to T_A in zone 3 & soil profile E, where as 6.6% cost is reduced for same building in zone 4 & soil profile E. Cost is only reduced due to steel, if concrete outline does not kept same there will be further reduction in cost. This cost analysis was done in 2009 and schedule of rates prevailing at that time in Pakistan were used.

5. CONCLUSIONS & RECOMMENDATIONS

The following conclusions have been made from this study.

1. The base shear coefficient V_C of the building with Intermediate moment resisting frame (IMRF) designed with T_A is 54.5% more than Special Moment Resisting Frame (SMRF) system. Similarly if same building is designed with T_B ($1.4T_A$) than V_C will be 40% reduced, giving the design near to SMRF design without any SMRF detailing.
2. The graphs in Fig (01) depicts that building with Time period 0.5 sec. to 2 sec. which corresponds to 5 to 20 storey height buildings, there is marginal increase in base shear if T_A is used as compared to T_B.
3. Reinforced concrete building with height 60 ft or less has T_A & T_B equal to 0.6 sec, as shown in Fig 01, (d & e). So the building V_C will remain same for zone 3 & 4 in soil profile 'E'. Thus five storey height building will have same design either in zone 3 or 4. For the two most critical zones in loose soil profile, our design will not change. This needs reconsideration of factors for calculation of V_C.
4. The present study shows that if building is designed with larger period T_B the result shows less deflection see Table 1, and also more economical design as compared to design of same building with higher deflection and lower period T_A. The other formula for calculation of time period T_B as shown in eq. (6) depends on displacement. So when the lateral displacements will be increased then Time period will also be increased. This contradicts to each other.
5. The height of the building used in Method 'A' for time period is also contradictory. Design code says if penthouse area is less than 10% of the total area of the building than penthouse height should not included in total height of building for calculation of time period with Method A. The height of the building plays important role in calculation of base shear so the percentage of penthouse are in relation to total area of building should be reconsidered to make the design more economical.

6. Similarly if the building have a basement without seismic/expansion joints, we must consider that height in total height of the building.
7. The variable involves in a calculation of base share with eq. (1) to eq. (3) are C_a & C_v should also be reconsidered especially for zone 3 & 4 in soil profile 'E' to make the design different in these zones.
8. Construction material and techniques are different for all over the world. Therefore value of C_T may not be constant for all regions. It should be estimated for a particular area according to prevailing practices.
9. For any value of period corresponding height of the building can be determined using following Eq.

$$H = {}^{4/3}(T/Ct) \qquad \text{eq.(7)}$$

Where H is in feet and T is in seconds. Thus for assisting quick design graph between V_C & H can also be developed and can be presented in further studies.

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To evaluate the effect of Synthetic polymers to control Dampness in Structural Members

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Abstract

The quality of construction projects can be enhanced by reducing the defects in the projects during and after construction. Some defects in the construction projects are harmful for the durability and structural stability and some of them are harmful for the indoor air quality and building aesthetic. Dampness is main defect that reduces the life of building as well as the comfort level for the resident of the building. Bitumen coating has been used for number of years to control dampness. But this technique is not efficient and not good for the health as it produces hazardious gases which are harmful for the human health. In this study synthetic polymers were used to control the dampness by improving the water proofing quality of the material. The effect of the polymers on the compressive strength of concrete was also checked by adding these polymers into concrete during concrete mixing. Structural members like wall footing, concrete slab, concrete cubes and septic tank were casted and tested to check the efficiency of synthetic polymer against seepage or leakage of water. The precast concrete slabs were also tested by using these polymers. The crack was produced along the length of the sewerage pipe and also tested for water leakage after applying multiple coats of polymers. It was found that these polymers are equally efficient for water proofing, crack filling and also improving the compressive strength of the concrete. The water absorption of brick and concrete was also tested by applying multiple coats on each side of the brick and concrete cube. These polymers reduced the water absorption of both bricks and concrete to almost zero. So, these polymers are highly recommended to the construction industry to improve the project life, durability, strength and aesthetic.

Key Words: Defects in concrete, Dampness, Water Absorption, Strength Improvement, Crack Filling and Aesthetic.

1. INTRODUCTION

The development of any country highly depends upon the development of construction industry. Construction industry comprises six to nine percent of the gross domestic products of developed countries. Buildings are the most important part of the constructions industry. There are many defects in buildings like structural and temperature cracks, dampness, settlement and failure. These defects produce due to poor quality workmanship. A typical example is water infiltration through some portion of the building structure that is called dampness, which may create an environment for the growth of mold. Most common defect in buildings is the dampness. Dampness is the existence of unsolicited moisture in different members of a buildings, either because of interruption from outside or condensation of water vapors from within the structure. Many techniques are there to control dampness. In this study new waterproofing techniques are introduced using synthetic polymers. In order to control the dampness bitumen coating is preferred over polymers as some of the polymers are costly and require skilled labor. Conventional techniques of waterproofing are still used in different developing countries. So, there is a need to introduce the new technique to control the dampness that must be equally economical as well as environment friendly. In this study, modern techniques of waterproofing are introduced by using synthetic polymers like crack filler (PP 007), sealer, hyper coating, Concrete mix (PP 500) and PP 50. The polymers are, Acrylic Polymers, Super Absorbent Polymer (SAPs), Epoxy, Dr. Fixit, Kembond K Fix 75, Polypropylene (PP), Polyvinyl Chloride (PVC) and Polystyrene (PS).

Structural defects are those which arises in the structural member of the building to cause these defects is due to poor design, use of poor materials and human errors these defects arise separately or may be some time in combination. Structural defects caused in members of building including beams, columns, retaining wall and slab. Defects in brick work, dampness in old structure and defect in the plaster work are some type of non-structural defects (Kofi Agyekum, 2013). Another study in Malaysia on seven hostel buildings identified some common defects in the buildings. That are the leakages of water, erosion in steel, penetration of rain water and other types of water in the buildings, horizontal cracks in the interior and exterior walls. These all defects need proper maintenance. Buildings need proper waterproofing to stop water from penetration to stop the damages and to protect the health of users of the building from harm (Wahab, 2011). Crakes are caused in many construction materials because it is nature of construction materials to crake after some time. Contraction and expansion take place due to variation in temperature during winter and summer seasons respectively. This contraction and expansion produce cracks in concrete members which allow the water to enter the structural members through these cracks. As walls cover the major area of buildings that is why thermal cracks are more critical in walls. The cracks in wall are produced due to over loading of wall, use of poor material and poor workmanship. The other reason of wall cracks might be settlement of underneath soil (Nurul Nadia Omar Bakri, 2014).

Different studies exposed that the defects also effect directly users of the building that is some of defects create health and economical problem for users. So, these defects should have identified first and then remedial measure should use. These defects should deal separately. The focus will be on the dampness which is one of the most damaging defects in buildings (Ogunoh P. E., 2016). Roof leakage accident arises many times due to poor and improper waterproofing system. As they are using bituminous membrane which causes a lot of problem for them because the bonding between roof concrete and bitumen was poor due to these cracks appears. The water leakage through those cracks produce dampness which damages the building and effects the health of the users of the building (Suffian, 2013). Dampness sources are classified into four major types are, Mounting dampness, Penetrating dampness, Condensation, Pipe leakages. Minor sources or causes are, not proper drainage system at the building site. Poor orientation of building, Flawed slope of the roof (fat slab), Poor construction. Indications of dampness are dull spots on the building, paint detaching, sometime plaster flaking, growth of Fungi etc. (Kofi Agyekum, 2013). Buildings should be such that it prevents any kind of water to penetrate through its element. A research was done on six room residential building in which they use three different stages to investigate the dampness. The first one is visual inspection on which they found from the result that interior and exterior walls are damp the symptom they found is dirty spots, sweltering of paint and efflorescence. The second stage is non-destructive test for which they use moisture meter which give the result is that the dampness is very much noticeable. The third and final stage is destructive test in which they found that the dampness in the kitchen and bathroom is due to the pipe leakages and precipitation penetration. From this it is concluded that when there is not proper waterproofing system to each element or where needed this problem will be damage the building and the health of user also. In spatially in residential building if there is not a proper ventilation then the condensation dampness will occur which disturb the health of users. (Agyekum, 2014). Results from the walls test were carried out like on the double side cladding the free twist limited by 75 % and on the single sided cladding free twist restrained only by 13 %. In order to achieve more rigorous demands from the end users there should be a better interaction between timber producers and contractors of structures and building (Kliger, 2006). Paint was blistered due to the dampness; plaster was damaged and surface efflorescence could be seen on most of the external and internal wall surfaces. Dampness could be seen on the walls having a height of approximately 1200mm in the walls inside the apartment the dampness could easily be seen on the walls of the bedrooms. Partition walls between the bedrooms and the washrooms were showing serious symptoms of Dampness (Kofi Agyekum, 2013). The effect of dampness and mold is increasing because the bacteria are becoming stronger than previous many years and there is need to improve lifestyle and stop dampness to cause such types of diseases at the end the researcher suggested that indoor dampness and mold problems constitute an important health hazard. And there is a need to prevent dampness and molds production in workplaces and especially in homes to prevent dangerous diseases of asthma and allergy (Maritta S. Jaakkola, 2009).

Indoor dampness and mold problem are universal and major problems for health diseases and structural members which affects the lives of people, their money and resources. The major problem of dampness and mold is health diseases like asthma. These types of issues are very common in countries where temperature remains very low. insufficient maintenance, improper construction work and construction of tight building to conserve energy by ventilation might be the critical factors for the severity of these issues specially in the cold climatic areas and countries. The dampness in residential buildings is significantly dangerous because it increases the chances of asthma specially in the children (Martín Sevilla, 2014). Polymer Modified Cement Impermeable Coating Material: It is a type of sealing material used in construction and engineering work. When the polymer emulsion and the cement mixed with powder are mixed and took on site and applied to the base, the cement is hydrated to form a seal coat. Polymer modified cement impervious coatings are commonly used as reliable and safe impervious materials because they have the quality of combining polymer flexibility, hardness and tackiness of cement and they do not require the use of an organic solvent or naked flame (Jack J. Fontana, 2005).

A special type of Geo-Polymers has been produced which can be used to increase the strength of structural members and also provide better waterproofing. With the increase in contact angle on Geo-Polymer surface from 21^0 to 22^0, the specimen will float in water causing low water adsorption. This is called surface hydrophobic modification of material. These polymers will improve the different properties of materials like short setting time, significantly high flow rate, improvement in compressive strength, high bond strength through interfacial bonding of particles which can be shown through dense microstructure. These polymers can also repair concrete of rigid pavement or it can also provide protective coatings for concrete for marine structures (Ping Duan, 2016). With the application of these polymers on samples, the reduction of water level adsorption can be achieved around 0.5% within 28 days. These polymers repair the materials and improve the properties like short setting time which is only 24 minutes, significant high flow rate which is around 212 m, high early compressive strength of concrete and also high bond strength of concrete (A. Kamel, 2016).

2. RESEARCH METHODOLOGY

The research is based on the experimental study on the effect of synthetic polymers on strength and dampness of concrete structures. Most effective polymers are, Acrylic modified polymers, and epoxy modified polymers, PP 50, PP30 crack fillers, and crack sealer. These polymers are equally good for improving the strength as well as for improving the resistance against dampness. For water proofing, sealer is a Liquid synthetic polymer used as primary coating. The function of sealer is to seal the pores on the surface of concrete and plastered surface. Crack filler is a solid synthetic polymer made up with cement. It has property to make grout or slurry flexible throughout its life. It is used to fill cracks, gaps between different structural members and expansion joints.
It is manufactured in Pakistan by Pakistan Phthalates Ltd. Company. They are manufacturing and selling it in market also as crack filler PP 007. Concrete mix is a liquid synthetic polymer manufactured by PPL. It is used in concrete as self-leveling agent and for reducing water quantity in concrete. It is also increasing the strength of concrete. Primer is also a liquid synthetic polymer by PPL available in white color used to close pores in plastered surface while painting it. It can also be manufactured of different colors. Hyper is a liquid synthetic polymer by PPL have transparent properties use after Crack filler to make the surface hard like glass and waterproof it. This polymer is many times used in plumbing work in core filling in kitchen, washrooms and in inlet/outlets of water.
Acrylic modified synthetic polymers are also available in liquid form. The use of these synthetic polymers is to increase strength of mortar and concrete by 3 time of its compressive strength. Epoxy modified synthetic polymers are manufactured in liquid form along with epoxy. These are also used in concrete and cement mortar to increase the compressive strength of concrete by 6 times the compressive strength of concrete.
Seven samples are selected including Spread Footing of Wall, R.C.C Slab with R.C.C Parapet Wall, R.C.C pre-cast Slab with Brick Parapet Wall, Septic Tank (Bricks), P.C.C Blocks (for strength in Compression), P.C.C Blocks (Waterproofing) and Sewerage Pipe. First of all, construction of 3 steps wall footing of 4’x18”, 3.75’x13. Polymers are also used for increasing compressive strength of concrete for that purpose 3 cubes were casted at same W/C ratio and mix design for each test and compare the strength of it.
In order to check the effect of three different polymers total nine cubes were casted having three cubes for Acrylic Modified Polymers, three cubes for Epoxy modified polymers and three cubes without edition of any polymers to compare the strength of polymers edition concrete with this one. To check the effect of polymer in water proofing, a cube with normal water to cement ratio was casted and then polymer was applied at its all sides and put it in the pond for one month for water absorption test. These polymers are good in filing the cracks in structures. For this purpose, a major crack was produced in the asbestos cement pipe and then that crack was filled with this polymer to check its effect in crack filling. Water was pored in pipe and kept it for several days to check the leakage of water from crack.

3. RESULTS AND DISCUSSION

After the preliminary tests on the materials, concrete mixed design was carried out to prepare the concrete samples. Samples were casted and cured in the pond for further tests. The samples were cured for prescribed 28 days before the application of polymers. Then the samples were tested by using ASTM method of concrete cubes testing procedure. The results are shown in the below table.

The table undoubtedly directs that there is a substantial increase in the compressive strength of concrete after the addition of polymer. It means that these polymers are equally good for water proofing and strength improvement. The polymer almost doubled the compressive strength of concrete.

Table 4: Compressive Load Capacity of Cubes

S/N	Normal concrete KN	Acrylic Modify Polymers KN	Epoxy Modify Polymers KN
1	362	659	860.1
2	349.59	665.78	871
3	342.98	658.35	867.8
Avg. Load	**351.52**	**661.04**	**866.3**

The sample of the cube which was simple of concrete had the compressive strength of maximum load 362 KN, other went to 349 KN and the last one went down on 342 KN. Now when the acrylic modified cubes were tested a big difference was seen in load bearing as the first sample took load up to 659 KN second one took 665 KN load and the last one took 658 KN. Now the last three cubes of epoxy modified cubes were tested and major difference in load bearing could be seen.

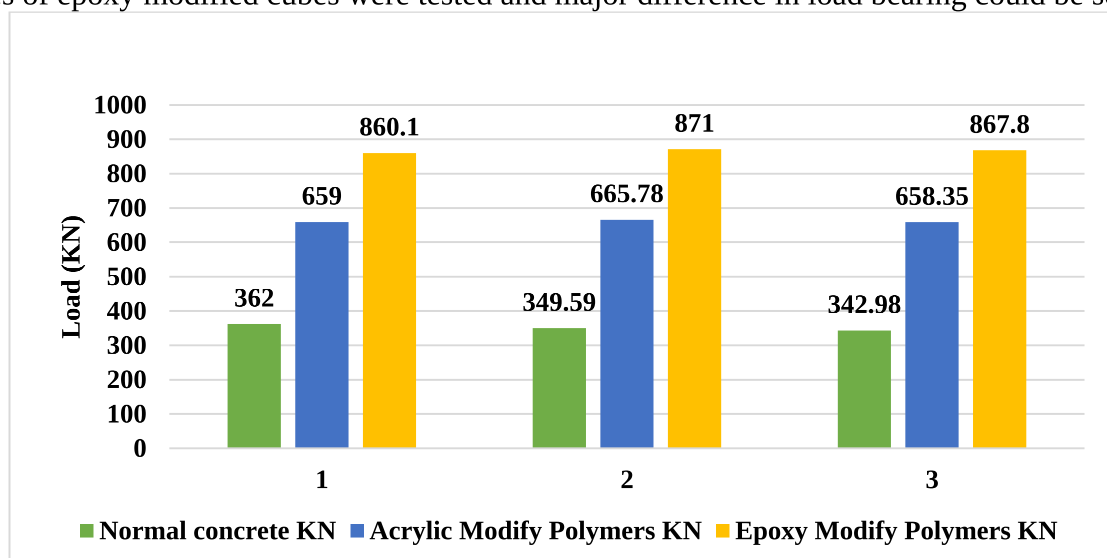


Figure1: Load Carrying Capacity of Concrete Cubes

Compressive strength of concrete expressed in Psi (pound per square inches), KN/sq.m (kilo newton per mete square) and different other units. As polymers used for waterproofing have dual performance waterproofing as well as strength enhancement. Results shows that acrylic modified polymers and Epoxy modified polymers increase the strength of concrete. Table 2 shows that with the help of synthetic polymers compressive strength will increase.

Table 5: Compressive strength of Cubes with polymers

S/N	Normal concrete	Acrylic Modify Polymer	Epoxy Modify Polymer	Area	KN-lb. factor	Normal concrete	Acrylic Modify Polymer	Epoxy Modify Polymer
	KN	KN	KN	Sq.in	-	Psi	Psi	Psi
1	362	659	860.1	36	224.82	2260.78	4115.62	5371.54
2	349.59	665.78	871	36	224.82	2183.28	4157.96	5439.61
3	342.98	658.35	867.8	36	224.82	2141.99	4111.56	5419.63
Avg.	**351.52**	**661.04**	**866.3**	**Avg. Strength**		**2195.35**	**4128.38**	**5410.26**

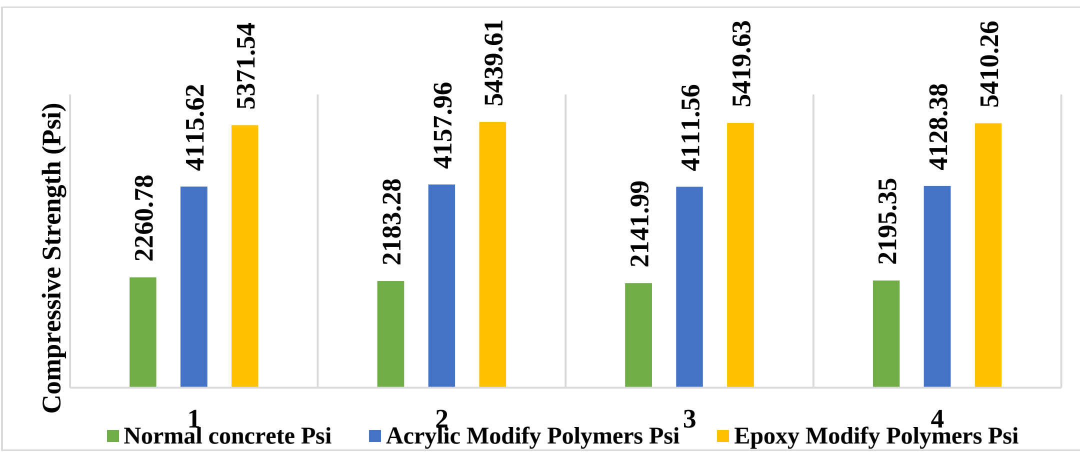
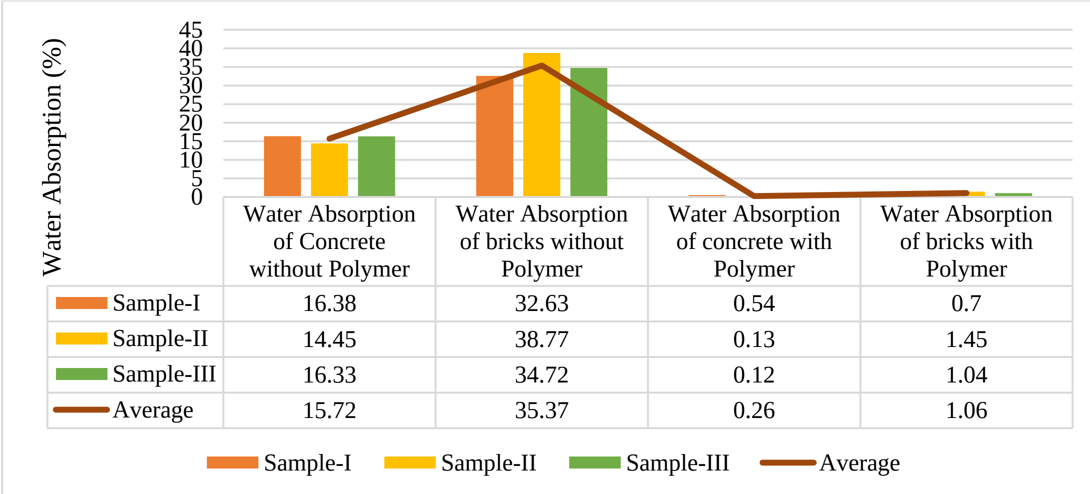


Figure 2: Compressive strength of Cubes with polymers

The compressive strength as well as other properties of the structural members highly depends upon the water absorption power of those members. Amongst all construction material brick have maximum tendency to absorb water. An attempt was made to check the same property of brick but after applying the polymer the water absorption power of brick was significantly reduced to almost zero. After polymers coating bricks do not absorb more water. Results are clear that before polymers coating bricks absorb water more than 35% water and after polymers coating same bricks absorb only 1% water on avg.

Below figure indicated that these polymers are really good in reducing the water absorption of concrete as well as in bricks. These polymers reduced the pores in concrete and bricks that is why water cannot seep through those pores and hence water absorption of material is reduced.



	Water Absorption of Concrete without Polymer	Water Absorption of bricks without Polymer	Water Absorption of concrete with Polymer	Water Absorption of bricks with Polymer
Sample-I	16.38	32.63	0.54	0.7
Sample-II	14.45	38.77	0.13	1.45
Sample-III	16.33	34.72	0.12	1.04
Average	15.72	35.37	0.26	1.06

Figure 3: Water absorption of bricks and concrete with and without Polymer

Similarly, visual inspection was carried out by apply polymers on cast on site concrete slab, asbestos cement pipe and on wall footing. After a month of visual inspection, it was observed that water was not penetrated through the layer of polymer and not a single dampness spot as observed on the surface of these structural members.

4. CONCLUSIONS

From the results discussion, it is concluded that the synthetic polymers are the good waterproofing agent that control the seepage and leakage of the water. These synthetic polymers are equally good for the improvement of the compressive strength of concrete as these polymers almost doubled the compressive strength. The water absorption of the material defines its strength. So, the water absorption of structural members can be reduced by using these synthetic polymers. The efficiency of these polymers is equally good for the sewerage pipe where the leakage effects the durability of the pipes. The damp proof course can be improved by applying some coating of synthetic polymers at DPC level.

5. RECOMMENDATIONS

From the results and conclusions, the recommendations are made that Polymers are recommended for the thin walled structures like warehouses and other types of shells because they are prone to cracks, so the use of polymer will improve the strength of these members and also control the cracks. Footing pads is required for the heavy machineries used in any industry. Dampness in the footing pads will cause the rusting of the machine parts. So, application of the polymers will reduce the chance of rusting by improving the waterproofing. For the future studies, the existing building can be used to improve the waterproofing by applying these polymers on the defective areas. Future studied can be done by using other types of polymers like polymer for the insulation.
The other properties of concrete can be studied like durability, weathering and soundness. Polymers should be applied outside where there is direct sun rays or heat so that it should take less time to dry and to complete the desired project on time and to see the full results on the time or schedule. Proper coating should be done there should be no such space remaining on the surface where polymers should be applied if space is remaining it would cause the seepage or dampness

which will fail the desired project. Strength will be lesser if dampness is caused as polymers increase the strength also which can be seen in the testing of the polymers on the concrete and the bricks. If the project or samples are made inside the building or any room having stoke or heater type instrument could be used to dry them as would not increase much cost of the project. Cost doesn't very much in using polymers instead of the bitumen or any other materials to stop the seepage. Polymers work 100 % if they are used in the initial stage of the project or construction of any building.

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Paper ID: 211

Retrofitting of Damaged Gravity Designed Reinforced Concrete Exterior Connection using Energy Dissipating Haunch

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Abstract

This research aims to compare the seismic response parameters of gravity designed model tested by Rizwan et. al (2018) by introducing novel haunch retrofit technique in already damaged exterior reinforced concrete connection. The model tested by Rizwan et. al was 1/3rd scale bay 2 story RC frame with deficient connection design. Scaled testing was performed on a quasi-static assembly installed at Earthquake Engineering Centre (EEC), UET Peshawar. Dissipating haunches were installed by first removing damaged concrete from the joints. Afterwards, damaged portion was replaced with rich concrete and haunches were anchored in them to reduce demand on beam-column joints. The Quasi static cyclic load was applied to damaged beam-column assembly by deforming the structure from elastic to inelastic state under displacement control condition. The ACI ITG-5.1-07 protocols were used as loading history, in which different target roof displacements equivalent to target drifts were applied. The structure force deformation capacity curve was derived, for the computation of Response Modification Factor (R) and global structure ductility (μ). The comparison of the retrofitted and as-built model shows that the retrofitted model not just regained its capacity but increased its stiffness, ductility, strength and response modification factor by 70%, 20%, 40% and 70% respectively.

Keywords: beam-column joint, haunch retrofit technique, response modification factor, quasi-static.

1. INTRODUCTION:

Severe deficiencies have been found in moment resisting frames (MRF) built before 1970s owing to inadequate shear resistance of their beam-column connections. In many developing countries, the RC structures are still constructed without considering the design guidelines for beam-column joints provided by their respective building codes. This is primarily due to the unfamiliarity, lack of skill workmanship and cost insinuation as shown in **Figure 1** and **2**. The gravity load designed structures are not designed for lateral actions due to earthquakes and also there is no such load transfer mechanism available in these structures to resist the lateral actions.

Figure 1: Manual batching and mixing

Figure 2: Bent Reinforcement and Non-Seismic Hooks

2. EXPERIMENTAL PROCEDURES:

2.1 Test Specimen

Rizwan et. al (2018) tested five scaled RC portal frames amongst which only one was code compliant RC portal frame, designed according to the BCP-SP 2007. The other four frames were incorporated with some deficiencies in reinforcement detailing and material strength. For the purpose of this research, the model designed for gravity loads was considered for retrofitting.

2.2 Retrofitting of Model:

The retrofitting process of damaged RC portal frame consists of four steps i.e. removal of concrete, model repositioning, concrete replacement and haunch installation. Initially the model was supported by applying jacks to both the floors and lateral support was provided to avoid the collapse of model during retrofitting process as shown in Figure 3(a). The damaged concrete from the joint region, columns and adjacent beams was removed up-to 2 times the depth of the joining members as shown in Figure 3(b) and 3(c). Repositioning was performed with the help of belts to remove the out of plane tilt from the model. Joint concrete was replaced with rich concrete (34.5 MPa) in order to increase the joint's shear capacity and to provide better anchorage to the haunch element by joining members Figure 3(d) and 3(e).

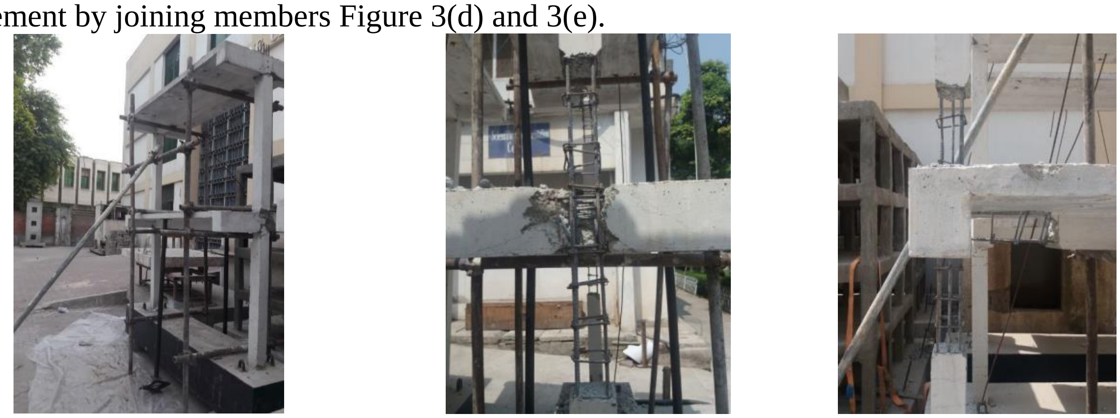

Figure 3: (a) Damaged Frame, (b)Damaged concrete removed from joints, (c) Joint after concrete removal


(d) Formwork for concreting, (e) Joint after replacing damaged concrete

2.3 Haunch Design:

Design of energy dissipating haunch used in this research work was based on the design procedure specified in the research work of Genesio (2012). The idea of haunch retrofit technique revolves around relocation of the plastic hinge from the joint region. Application of haunch to the joint region will reduce the flexural moment at the beam-column interface with the goal of reducing the shear stress in the joint region. Considering the structural and architectural requirement, the length of the haunch element is kept in range of 0.1 to 0.2 times the length of the beam (Sharma et al, 2012). In design of haunch element, certain geometric parameters of trial haunch member are considered i.e. projected length (L'), angle of haunch (α) and haunch stiffness (K_d). The strength hierarchy of the mechanisms involved in beam-column joints are given from least severe to most severe on the structure: Vc, beam-hinge $\leq \Phi_1$Vc, column-hinge $\leq \Phi_2$Vc, joint shear $\leq \Phi_3$Vc, column-shear $\leq \Phi_4$Vc, beam-shear. Haunch element consists of a dog-bone specimen which is enclosed in a steel cylinder which is then filed with rich mortar to resist buckling during compression of the haunch element subjected to lateral loading. Figure 4 shows the designed haunch element diagrams. Haunches were placed both at top and bottom of the beam in order to get a better performance in resisting the lateral loads Figure 5 shows the haunch placement scheme. The type of anchorage used in this research is through anchorage. It consists of 6 bolts passing through steel plates at both sides of the column and beam Figure 6.

Table 1: Haunch element details

Haunch Parameters	**Anchorage details**	**Anchorage details**
Haunch Diagonal (Lh)= 353mm	Anchorage type= Through Anchors	Fc'= 34 MPa
Projected Length of Haunch (L') = 250mm	Number of anchors (n)= 6	Fy= 414 MPa
Haunch angle (α)=45°	Anchors diameter(d_{nom})= 6 mm	
Es= 200GPa		
Cross sectional area of Haunch (Ad)= 30 mm^2		
Haunch Stiffness (Es.Ad/Lh)= 16.9 KN/mm		

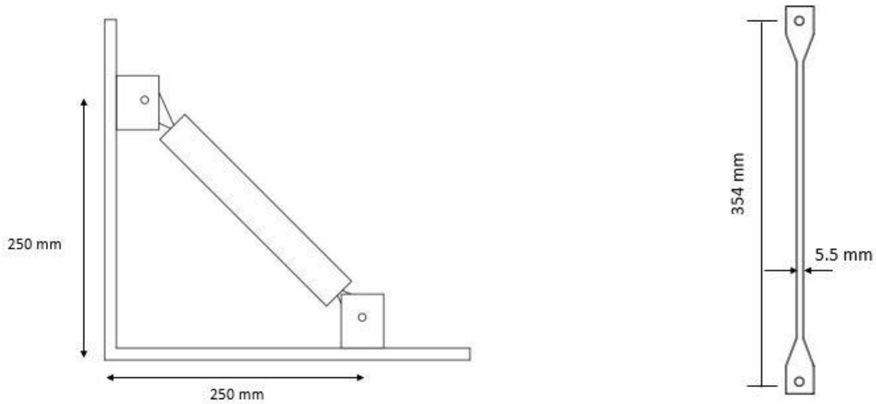

Figure 4: Haunch schematic diagrams

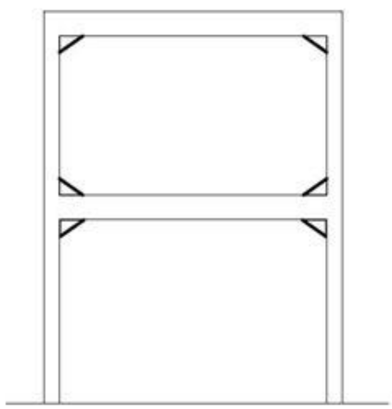

Figure 5: Placement scheme

Figure 6: Through Anchorage in beam and columns

2.4 Test Setup and Loading Protocols

Quasi static lateral cyclic load was applied to the test model with the help of displacement-controlled hydraulic actuator. The actuator consists of a ram and load cell having capacity of 50 tons and having ± 6in (150 mm) displacement capacity of the shaft. The hydraulic actuator is connected to the vertical distribution beam at 1/3rd span from the top end, to apply lateral loads simulating triangular lateral force distribution. The test setup of the retrofitted specimen is shown in Figure 7.

The loading history consists of series of three cycles at increasing level of target roof drift (0.4%, 0.5%, 0.7%, 1%, 1.5%, 2%, 2.5%) prepared as per ACI ITG-5.1-07 protocols (Figure 8). To record the in-plane lateral displacement, two displacement transducers were attached to each floor, while one displacement transducer was attached to the base pad of the structure in order to record any sliding in the structure. The applied load was recorded by the load cell attached to the hydraulic actuator.

Figure 7: Test Setup

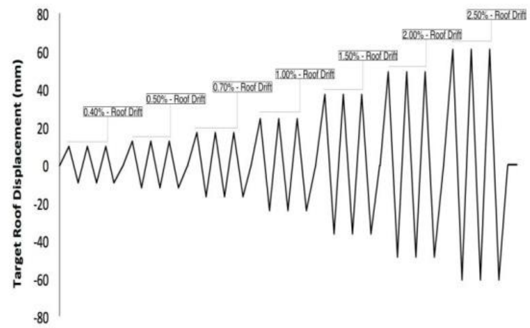

Figure 8: Loading History

3. RESULTS:

3.1 Damage Behaviour:

Damaged behaviour was observed in relation to story drift. At 0.4 % drift, pre-existing cracks reopened in columns. Further increase in drift to 1% led to development of shear cracks in joint region. At 1.5% drift, haunch fractured at ground storey while further increase to 2% led to further widening of shear cracks in column base and joint. Widening of flexural and shear cracks continued at 2.5% drift accompanied by severe damage to column base.

3.2 Comparison of As-Built and Retrofitted Structure:

The as-built structure of Rizwan et al (2018) was not designed for any kind of lateral loading so it withstood 50% of intensity of Northridge accelerogram and showed very lower stiffness and severe joint panel damage due to lateral loading conditions. Due to the use of lower strength concrete and the lack of stirrups in the joint region, it has very less tensile strength resulting in a brittle failure mechanism in the form of joint shear cracking which ultimately led to lower overall lateral load capacity of the structure.

Table 2: Structural Properties

Structural Properties	As Built Model	DH Model	% Increase
Stiffness (N/mm)	817.51	1399.87	71.24
Ductility (μ)	1.65	1.99	20.46
Ductility Factor (Rμ)	1.65	1.99	20.46
Overstrength Factor (Rs)	1.73	2.46	42.15
Response Modification Factor (R)	2.86	4.89	71.24
Strength (KN)	90.85	129.18	42.15

In the retrofitted structure with rich concrete and haunch retrofit technique, slight shear cracks were observed in joint region. The observed ultimate mechanism was hinging and the core concrete crushing at the base of the columns under large lateral displacement cycles. The haunch retrofit technique significantly improved the seismic performance of a damaged reinforced concrete gravity designed structure.

3.3 Force-Deformation Capacity Curve

The force-deformation capacity curve was developed from the recorded data of the quasi-static testing of the retrofitted frame. The capacity curve shows that severe pinching effect (Figure 10) reduced the energy dissipation

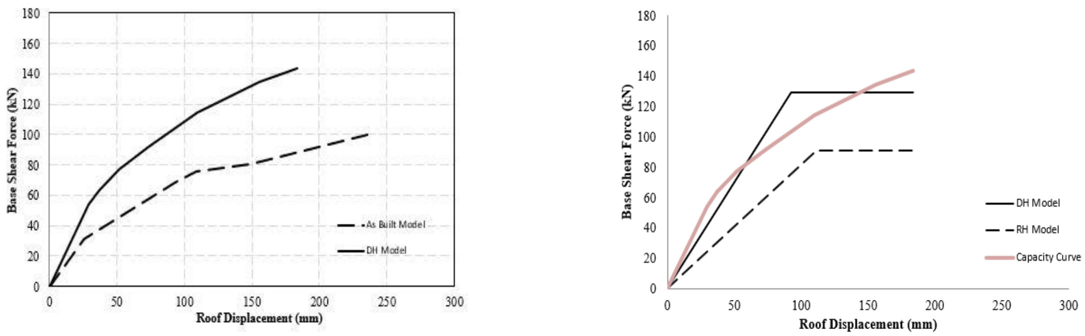

Figure 9(a): Roof Displacement, (b) Comparison with Rizwan et al

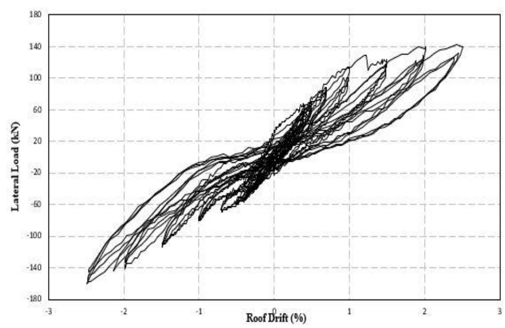

Figure 10: Hysteretic Curves

n in the structure. Both the positive and negative lateral roof displacement and lateral loads were plotted and then averaged to obtain the mean capacity curve for the prototype structure shown in Figure 9(a). The retrofitted structure shows substantial increase in the lateral load carrying capacity as compared to the as-built structure. In order to compute the structural response parameters, bilinear idealization of the capacity curves was performed using equal area principle. Figure 9(b) compares the bilinear idealized curves for as-built and retrofitted structure.

4. CONCLUSIONS:

The following conclusions are derived from this research work:

- The retrofitting technique substantially improved the strength and stiffness of the retrofitted structure by approximately 42% and 71% respectively as compared to the as-built structure.
- The retrofitting technique increased the overall response modification factor (R) of the structure by 70%.
- The proper anchorage design made the retrofit technique more effective. Through anchorage was provided in this research. The drilling through the members is risky, if not done properly can weaken the members internally by introducing pre-test damages and is

ultimately the failure of the retrofit technique.

ACKNOWLEDGEMENTS:

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Repair and Retrofit of Beam-column Joints of a Damaged Two Story RC-frame

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Abstract

A 1/3rd reduced scale two story reinforced concrete frame tested on shake table was repaired and retrofitted using steel haunches, to check the efficiency of haunch retrofitting in restoring seismic capacity. Repairing was done by removal of damaged concrete from the joints and replacement with rich concrete. Haunches were installed at each joint to change the hierarchy of strength from brittle shear damage in joints to ductile flexure damage in the beam. Testing was performed using quasi-static cyclic loading setup. From the comparison of the tests it was observed that the repaired and retrofitted frame showed a ductile failure mechanism of beam flexure cracking as campaired to joint shear cracking in as-built frame. Also a substantial increase in the seismic response parameters of the retrofitted frame was noted showing the effectiveness of the repair and haunch retrofitting technique.

Keywords: Haunch retrofit technique, quasi-static cyclic loading, seismic response parameters.

1. INTRODUCTION:

Due to earthquakes every year a large stock of buildings is damaged in addition to the loss of many human lives. The risk of damage to the buildings and lives is increased further if non-seismically designed or some type of construction deficiency is found in it, which is the case of a huge stock of buildings in developing countries. Mostly attributable to non-seismic design and construction deficiencies, the Kashmir 2005 earthquake killed more than 73,000 people in addition to severely injuring 70,000 people, while making 2.8 million people homeless. According to an estimate some US$5.2 billion loss to the economy was attributed to the Kashmir earthquake (Asian Development Bank and the World Bank, (2005)).

The study in this work will consider such deficient structures with beam-column(BC) joints in focus, as in past beam-column joints were assumed as to behave in an elastic nature during an earthquake event, that assumption proved to be wrong as when the joints were observed to be one of the most vulnerable part of a structure and become the reason for brittle failure of many structures, either designed as per old colds with improper seismic provision, or with construction deficiencies (Pampanin et al. 2006). Either by increasing the seismic capacity or decreasing the demand on BC joints its brittle failure mechanism in the form of shear cracking can be prevented. To prevent the shear failure of joints many techniques were developed, the applicability of which depends on materials, expertise, cost considerations, architectural and aesthetics requirements. Haunch technique due to its less expensiveness, less invasiveness and easily applicability was proposed by Pampanin et al. (2006) for reinforced concrete(RC) BC joints, while for the haunch technique to be faster, easier and lesser invasive post-installed anchors were used for fastening of the diagonal haunch with the beam and column by Genesio, G. (2012).

2. EXPERIMENTAL PROCEDURES:

2.1 Introduction:

For the purpose of investigating the effectiveness of haunch retrofit technique in restoring seismic capacity of a deficient structure and already damaged during shaking table test performed by Rizwan et al (2018) on the as-built model to be repaired and retrofitted, was tested under quasi-static cyclic loading conditions. In addition to a two story control frame designed according to Seismic Provision of BCP-SP (2007) by Rizwan et al (2018) four other frames were constructed considering the deficiencies found in field practices. The model frame considered in this study was the one with reinforcement details (Figure 1) as of the control frame with concrete strength of 2000psi as campaired to the design strength of 3000psi. The frame constructed and tested, were 1/3rd reduced scale simple model idealized.

2.2 Model Repairing:

The damaged frame was repaired by firstly repositioning the frame from the induced tilting due to testing, then the removal (Figure 2) of damaged concrete from joint and the adjacent beams and column region, which was replaced (Figure 3) with rich concrete of 5000psi of compressive strength. Concrete of higher strength was used in the joints to increase its shear strength, and in the adjacent parts of the beams and columns for better anchorage of the haunches to the beam and column.

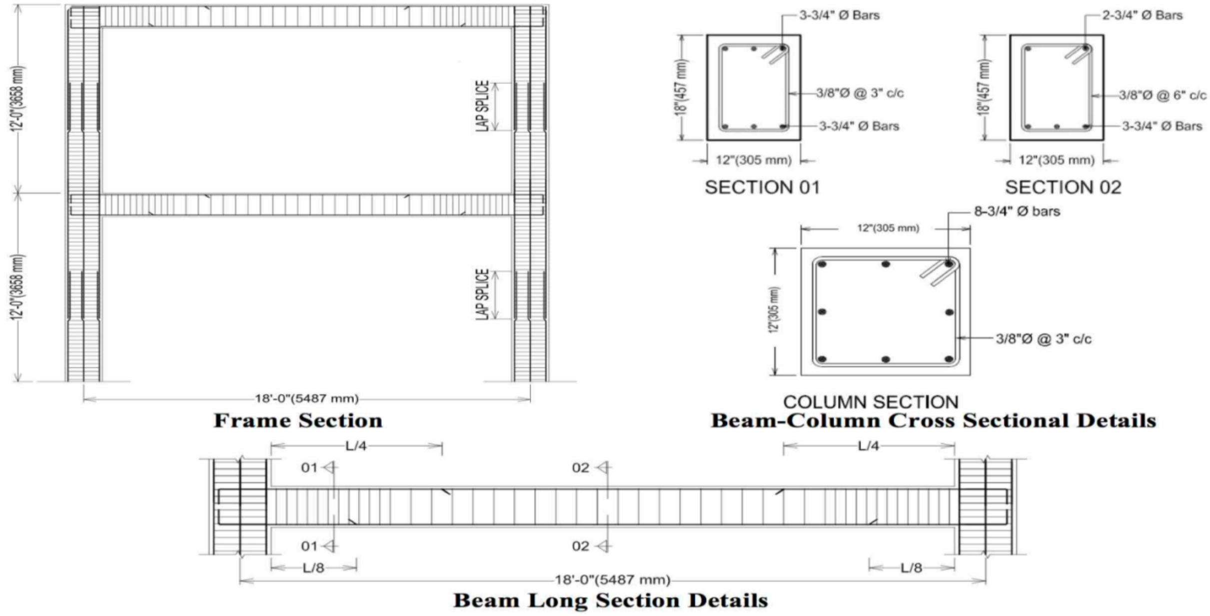

Figure 1: Steel reinforcement details of control frame used by Rizwan et al (2018) as well as candidate model in this study.

Figure 2: Removal of damaged concrete. Figure 3: Replacement with rich concrete.

2.3 Haunch Design and Application:

The haunch design performed in this study was based on the procedure of fully fastened haunch retrofit solution (FFHRS) introduced by Genesio, G. (2012), that intern is an extension to the procedure used by Pampanin et al (2006). The effectiveness of the haunches in diverting the shear demand from the joint towards the beam depends on length, the angle it makes with the beam and stiffness of the haunch element, where the stiffness of the haunch element depends on its dimensions, material and also anchors stiffness. The dimensions, geometry, and material properties of members that connects with the joint in addition to the trial haunch element and stiffness of the anchors, were the parameters used for designing of the FFHRS. The haunches were connected to the beam and column by drilling of holes, where the anchors passing through the haunches base plates are bonded to the concrete using epoxy.

Table 1: Details of the haunches used for retrofitting.

Haunch Parameters	**Anchorage details**	**Anchorage details**
Haunch Diagonal (Lh)=353mm	Anchorage type=Bonded anchors	Fc'=34MPa
Projected Length of Haunch (L')=250mm	Number of anchors(n)=6	Fy=414MPa
Haunch angle=45°	Anchors effective depth(hef)=60mm	
Es=200GPa	Anchors diameter(dnom)=8mm	
Cross sectional area of Haunch(Ad)=1080mm^2		
Haunch Stiffness (Es.Ad/Lh)=612KN/mm		

2.4 Test methodology, setup and Loading Protocol:

Tests on the repaired and retrofitted frames were conducted by applying quasi-static cyclic loading through a hydraulic actuator of 50-tons of load and 12 inches (300mm) of displacement capacity (Figure 4). Two pin connections were provided with the actuator assembly for avoiding any accidental eccentricity that can be induced due to loading.

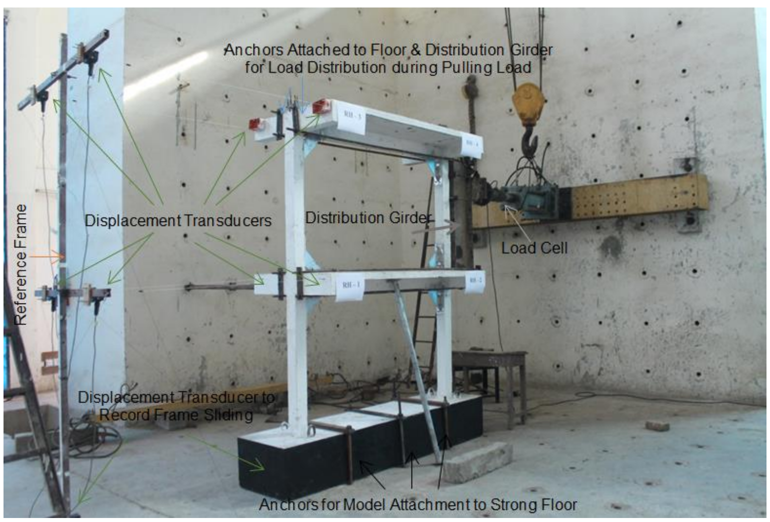

Figure 4: Test setup and instrumentation plan.

The actuator was attached at 1/3rd height from the top to a distribution girder attached to the top of first and second stories, so as to create loading conditions equalling to that of code linear triangular load distribution between the floors. Loading history as per ACI ITG-5.1-07 protocols was applied, where 0.4, 0.5, 0.7, 1.0, 1.5. 2.0, 2.5% of roof drift cycles were applied with each drift cycle repeated three times. Based on the code allowed maximum inter story drift for low rise structures, a maximum to be applied roof drift was selected. Load cell to the actuator was attached for recording load, while displacement transducers were attached to the floors for recording displacements. Also to record any horizontal sliding of the retrofitted frame another displacement transducer was attached to its base pad.

3. RESULTS:

3.1 Damage behavior:

By starting with the application of 0.4% of roof drift and increasing to 2.5% as per ACI ITG-5.1-07 protocol, the damage started with hair line cracking in the beam at the end of haunch plates that widened significantly in the subsequent drifts up-to 2.5%. Also some hair line shear cracking in the joints and shear cracking in the columns of ground story at the end of haunch plates was observed during the application of the last drift cycle, which can be attributed to the detachment of the haunches through concrete pry-out that further weakens the substrate member. Also some pinching behavior in the load-deformation response (figure 5) of the frame was observed that can be attributed to the opening and closing of already existing cracks, which can be controlled/minimized by injecting epoxy.

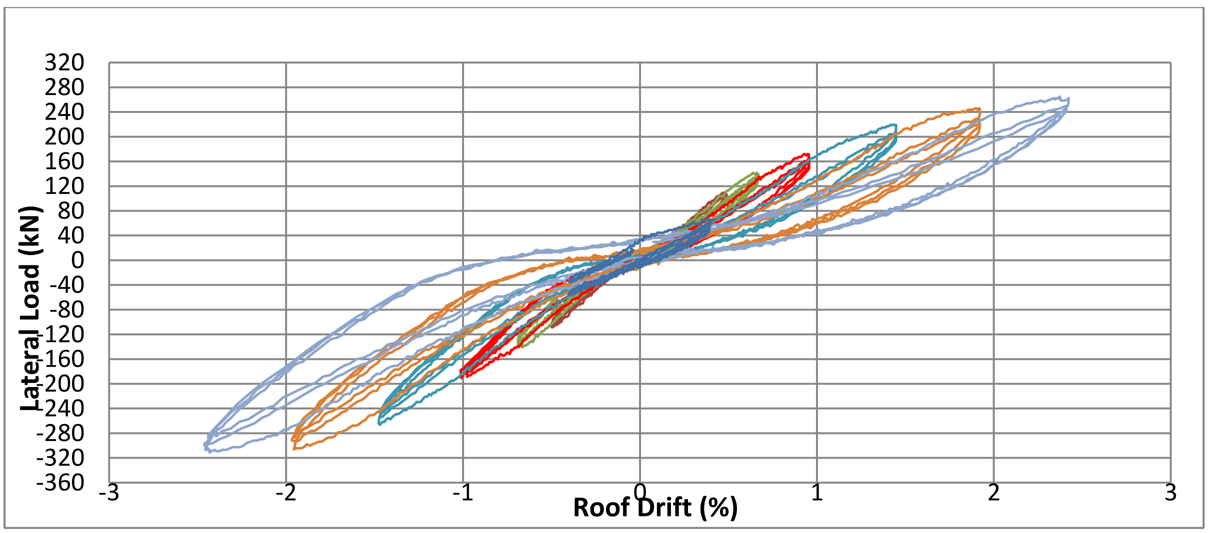

Figure 5: Force-displacement hysteretic response of prototype of retrofitted test frame.

3.2 Comparison between Retrofitted and As-Built RC Frames:

In case of the as-built frame tested by Rizwan et al (2018) through a shaking table, that was built using low strength concrete, had lower load resistance and higher deformation under same loading conditions. This frame has also lower lateral load capacity due to early joint shear failure (Figure 6) due to the use of low strength concrete. The frame when then repaired using higher strength concrete and with haunches installed showed much higher stiffness and lateral load capacity. This can be attributed to the higher joint shear strength due to the use of rich concrete and also due to the shear transfer mechanism produced by the haunches installed at beam-column joints, from joint to the beam. That resulted into flexure plastic hinge (Figure 7) formation in beam as opposed to joint shear damage mechanism in as-built frame.

Figure 6: Damage in joint of as-built frame

Figure 7: Flexure cracking in beam of retrofitted frame.

3.3 Seismic Response Parameters:

The capacity curves were generated from the hysteretic behaviour of the frames and then this capacity carves were bi-linearly idealized using equal energy principal. From the Idealized Elastic-Plastic Capacity Curves the different Seismic response parameters were calculated as per procedure followed by Rizwan et al (2018), so as to have quantitative comparison between the seismic response of the as-built and repaired plus retrofitted frame. From this comparison it was found that the repair and retrofitting not just restored but increased stiffness, strength, ductility and response modification factors of the deficient and damaged frame by 75%, 52%, 15% and 75% respectively.

4. CONCLUSIONS:

The objective of transferring damage from brittle shear cracking in the joint was achieved in the form of ductile flexure cracking in the beam. Due to which a reasonable increase in structure stiffness, strength, ductility and response modification factor was observed. It was observed that the attachment of fully fastened haunches to beam and column, which is through drilling of holes, can further weaken it and in case of earlier detachment of anchor through concrete pry-out can further weaken the beam or column (especially) and can lead to its shear failure. Also as pinching behaviour in the hysteretic response of the structure was observed due to opening and closing of already existing cracks, epoxy injecting is recommend. Additionally to avoid anchors failure and weakening of beam and column due to drilling done for anchors attachment, attachment of haunches through external anchors is recommended.

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The authors would like to thank every person/department who helped thorough out the research work, particularly PG Adviser Earthquake Engineering department UET Peshawar Engr. Dr. Naveed Ahmad. The careful review and constructive suggestions by the anonymous reviewers are gratefully acknowledged.

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Seismic Performance Assessment of Existing Mid-Rise RC Buildings in Pakistan

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Abstract

Recent disastrous earthquakes reveal the vulnerability of the existing reinforced concrete (RC) buildings in Pakistan. The increased level of awareness about possible seismic hazard raises serious concerns about the structural performance of RC buildings. Most of the RC buildings in Pakistan are not in compliance with the prevalent stringent seismic requirements. In the current study, a 13-story RC shear wall building, located in the capital city of Pakistan, is considered as case study to evaluate the structural performance of existing buildings. The case study building is categorized as mid-rise RC building. Nonlinear response history analysis (NLRHA), as per ASCE-41-06, is used to evaluate the seismic performance of the case study building. The result shows that the case study building will be severely damaged In the case of an event of an earthquake. This study concluded that more studies are needed to access the seismic performance of the existing RC buildings in Pakistan so that suitable retrofitting measures can be devised.

Keywords: Seismic Evaluation, NLRHA procedure, RC Shear walls

1. INTRODUCTION

Natural disasters have always been threatening to human civilization. Earthquake is one of the most devastating natural hazards. Pakistan geographically situated in a region of vigorous seismic activities. Pakistan has a long history of seismic events essentially because of the interaction of the plates in the Karakoram Range. In recent times Pakistan has faced many major earthquakes, 80,000 people died in Kashmir earthquake 2005 and nearly 2000 causalities were reported in Peshawar earthquake in 2014. These events of ground shaking is related with complex plate boundary conditions, which encompasses Pakistan. Indian plate and Eurasian plate are moving towards each other at 3cm and 1.3cm per year respectively. This opposite movement of plates has cracked Indian plate into many slices.

In a developing country like Pakistan, we are lacking far behind in technical skills and expertise to understand the seismic activities and accordingly designed safer structures, the situation is much worse than we thought. Recent earthquake raised serious concerns about the structural performance of reinforced concrete (RC) building in Pakistan.

After the Kashmir earthquake in October 2005, Government of Pakistan directed national engineering services of Pakistan (NESPAK) to develop new seismic codes for the country to save the buildings during earthquakes. Due to the delay in the process of developing the new codes, Earthquake Reconstruction and Rehabilitation Authority (ERRA) started using Uniform building

codes (UBC-97, 1997). Later on, building codes of Pakistan (BCP, 2007) was published in 2007. Although this building code named as Building code of Pakistan, due to lack of ground motion filed data and experimental lab data, it is almost similar to the UBC 1997.
Recent earthquake raised serious concerns about the structural performance of reinforced concrete (RC) building in Pakistan. Severe damage was reported in the mid-rise buildings in Rawalpindi/Islamabad region. The incident of margalla tower collapse in prominent. So, there is a serious need for assessment of existing RC buildings in Pakistan.

2. BUILDING INFORMATION

AWT plaza is a thirteen story building, and it is located on the Mall road in Rawalpindi. The building was built in the 1980s. The building consists of two parts separated by 1-inch wide seismic joint. One part of the building is thirteen-story, while the other part is a single story. The thirteen story part of the building has a footprint of about 140’ x 120’; the building’s area decreases at each floor after the fifth level, making it an irregular structure in planar, as well as, vertical sense. The total height of the building is about 156ft (47.5m); its story height is 12 ft. (3.66m), and columns of the building are spaced at 20 ft. (6m) for the selected floor plan. Reinforced concrete walls have been employed in the building to cater to the strength and stiffness requirement of the structure against lateral loads. The building has reinforced concrete frames which enables it to transfer gravity loads from floors to the foundation. The floor system consists of reinforced concrete beams between columns and, predominantly, six inches thick reinforced concrete slab; the foundation of the building mainly consists of 53 inches thick mat for 13 story part.

3. COLLECTION OF BUILDING DATA

The data regarding the structural system and sizes of structural members are taken from structural drawings. A visual inspection of the building was also made, and the size of shear walls, columns, and spacing between columns was verified with structural drawings. The strength of rebar is also taken from the structural drawings. The strength of concrete for different structural members is not mentioned in the structural drawings, only the class of concrete is mentioned, without reference to any structural member. The strength of concrete for foundation, beams, and slabs was assumed, while the strength of columns and shear walls was taken from the results of non-destructive tests.

3.1 Cross-section of columns

The size of columns has been taken from structural drawings and was confirmed with measurements while doing a visual inspection of the building. The detail of reinforcement of the column at a particular location could not be determined from structural drawings because of non-availability of the column layout plan. The reinforcement of a column at a particular location was assumed with the help of results of the ETABS model of the building and the data of column cross-sections on structural drawings. Table 1 shows the cross-section of columns.

3.2 Cross-section of Shear Walls

All shear walls in the building are 8 inches thick and have the same amount of flexural and shear reinforcement. Table 2 shows the detail of transverse and longitudinal reinforcement of the walls.

Table 1: Reinforcement detail of columns

Column ID	**Size (inches)**	**Longitudinal reinforcement (in^2)**	**Transverse reinforcement**
C1	24 x 24	6 at all levels	#3@8'' c/c ; 0.33 in² in each direction
C2	24 x 24	6 (1-3 levels); 7 (remaining levels)	#3@8'' c/c ; 0.33 in² in each direction
C3	24 x 24	12.64 (1-3 levels); 7 (remaining levels)	#3@8'' c/c ; 0.33 in² in each direction
C4	40 x 8	13 (1^{st} level) 5.5 (remaining levels)	#3@8'' c/c ; 0.22 in² parallel to short direction 0.44 in2 parallel to long direction
C5	Triangular 39 x 31 x 47	22 at all levels	#3@8'' c/c ; 0.44 in² in both directions
C6	24 x 24	26.6 (1-2 levels) 12.6 (3^{rd} level) 6 (remaining levels)	#3@8'' c/c ; 0.11 x3 in² in each direction

Table 2: Reinforcement detail in shear walls

Member	**Concrete, *f'*c (ksi)**	**Main bars, *fy* (Ksi)**
Slab	3	60
Beams	3	60
Columns	4	60
Shear Walls	4.5	60
Foundation	3	60

Table 3: Material strengths

Level	**Longitudinal reinforcement**	**Transverse reinforcement**	**thickness**
1-2	#5 @ 8 in c/c	#12 @ 8 in c/c	8 in
3-4	#4 @ 8 in c/c	#3 @ 8 in c/c	8 in
5-till end	#3 @ 8 in c/c	#3 @ 8 in c/c	8 in

4. NONLINEAR SEISMIC EVALUATION

4.1 Performance Objective

The main objective is to evaluate the structural performance of the existing building under gravity and seismic loadings. For seismic loading, the building shall be checked to satisfy Basic Safety Objectives, with a goal to provide a low risk to life safety for any seismic event likely to affect the building. "Life Safety" performance level shall be checked against 475-year return period earthquake the earthquake that has a 10% probability of exceedance in 50 years, also Known as Design basis earthquake (DBE), and "Collapse Prevention" performance level shall be checked against the 2475-year return period earthquake the earthquake that has a 2% probability of exceedance, also known as Maximum considered earthquake (MCE).

4.2 Seismic Loads

Uniform Hazard spectra used in the Pakistan Building Code-2007 (BCP, 2007) resulting from a probabilistic seismic hazard is used in this evaluation. Effective viscous damping of 5% of critical damping in considered in both 475-year (DBE) and 2475-year (MCE) return period earthquakes.

4.3 Expected Seismic Hazard and ground motion Selection

As required by the Building Code (TBI-2010, 2010), seven accelerogram sets were used for Nonlinear Response History Analysis (NLRHA). Keeping in mind this, deaggregation analysis is performed to identify the sources of the expected seismic hazard. This will help to select the suitable ground motion for AWT building. Results of deaggregation analysis, are shown in Figure 1 and Figure 2. The result of geographic deaggregated seismic hazard map for AWT Plaza shows that the 0-50Km seismic source dominates the seismic hazard (M6.60 at a distance of 14 km) and the Main Boundary Thrust is the single fault which shows a little contribution (M7.78 at a distance of 13 km). Based upon these results following ground motions were selected from the PEER NGA data base, Table 4.

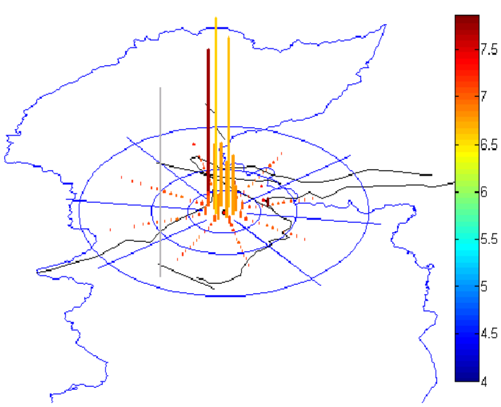

Figure 1: Deaggregation analysis for the earthquake with a 10% probability of exceedance in 50 year

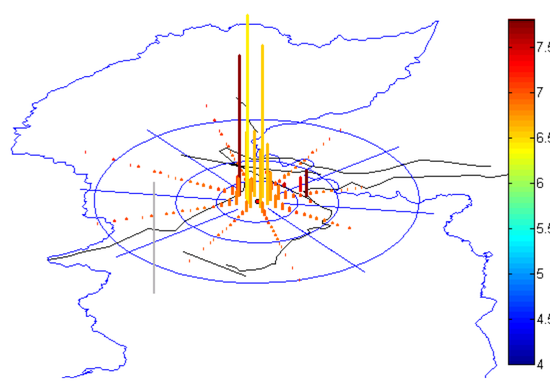

Figure 2: Deaggregation analysis for the earthquake with 2% probability of exceedance in 50 years

Table 4: Selected ground motions

No	Earthquake Event	Year	M_w	PGA (g)	Duration (sec)
1	Loma Prieta	1989	6.93	0.14	60
2	Loma Prieta	1989	6.93	0.21	59.88
3	Chi-Chi_ Taiwan	1999	7.62	0.27	80
4	Chi-Chi_ Taiwan	1999	7.62	0.15	70
5	Chi-Chi_ Taiwan	1992	7.62	0.13	70
6	Cape Mendocino	1999	7.01	0.33	28
7	Iwate_ Japan	2008	6.9	0.35	47

4.3 Acceptance Criteria

4.3.1 Adequacy of Components against seismic + gravity loads

The response of the components is checked based on the type of action, namely force control, and deformation control. For force control actions, the expected strength of the component should be less than the demand due to gravity and seismic forces. The expected strength of the component is calculated according to the procedures of ACI-318. The response of components is checked against "Life Safety" performance level under 475-year return period earthquakes while "Collapse Prevention" performance level will be checked under 2475-year return period earthquakes. The axial strain in shear walls is compared against the maximum allowed to check their adequacy against flexural-demand.

4.3.2 Story drift

Story drifts shall not exceed the following limits of table 11-2 in (ATC-40, 1996)

Load Case	Story Drift Limit
DBE Seismic (10%/50 yr)	2%
MCE Seismic (2%/50 yr)	$0.33\frac{V_i}{P_i} = \frac{0.33 \times 0.08}{100} = 2.7\%$

5. NONLINEAR MODELING OF THE AWT BUILDING

A nonlinear model of the AWT building is created to Perform 3D version 5.0 (Perform3D, CSI). Each structural wall is modeled by nonlinear fiber elements over the entire height since flexural cracking and yielding may occur at any location. The wall is divided into many horizontal layers, where each layer is modeled by a newly developed fiber model called Multi Vertical Line Element Model (MVLEM) (Orakcal & Wallace, 2012). This model is made of a large number of vertical concrete and steel fiber elements to simulate the combined axial and flexural behavior of the wall. It also has a horizontal shear spring to simulate the elastic shear response. A bilinear hysteretic model of non-degrading type is used for the steel fibers. The post-yield stiffness is set to 1.2 percent of the elastic stiffness. In making concrete fiber elements, the Mander's stress-strain (Mander, Priestley, & Park, 2008) model for either confined or unconfined concrete is approximated by a tri-linear envelope. Each RC column is modeled by a combination of a linear elastic beam-column element with nonlinear plastic zones at its two ends. The un-cracked flexural rigidity is assigned to the linear element. The plastic zones are assumed to have a length of 0.5D, where D is the lesser cross-sectional dimension of the column. They are modeled by concrete and steel fibers in a similar manner to RC walls. By this way, the un-cracked (linear elastic), cracked, yielded, and post-yielded states of the column can be fully simulated(Najam & Warnitchai, 2018).

RC beam is modeled by a combination of a linear elastic beam-column element with nonlinear plastic zones at its two ends. The un-cracked flexural rigidity is assigned to the linear element. For nonlinear plastic hinge zone on both ends of beams, moment rotations relationships were developed using available construction detail(Najam, Warnitchai, Qureshi, & Mehmood, 2018).

The concrete slabs are assumed to remain elastic and are modeled by using rigid diaphragm floor constrains command. The mat foundation is treated as a rigid boundary, which is displaced horizontally by the input ground motion.

5. RESULTS OF NONLINEAR SEISMIC EVALUATION

5.1 Story Shears

Figure 3 presents the comparison of story shears obtained from the NLRHA procedure and from the code based Equivalent Lateral Static Force procedure. This comparison clearly indicates that there is significant shear amplification due to the negligence of higher mode contribution in the design procedure. Approximately a shear amplification factor of 2.8 and 3.6 was observed at the base of the structure. Usually, a shear amplification factor of 1.3 to 1.5 is anticipated to strength reduction factors and strain hardening. This shows that no shear amplification due to irregularities of the buildings are included in the design procedure.

6.2 Story drift

Story drift ratios are checked for MCE level ground shaking and found to be within the specified limit. Results are presented in Figure 4. Story drift ratios are lesser in the x-axis, which is obviously due to higher lateral stiffness contribution from RC walls in the x-axis direction.

6.3 Component responses

6.3.1 RC shear walls

Shear strength of RC walls is checked against the seismic shear demand obtained from both MCE and DBE level ground shaking. Results are presented in Figure 3. Five walls are estimated to fail in the shear mode of failure at several levels at MCE level seismic hazard, while 3 walls are estimated to fail in shear at DBE level seismic hazard.

The axial strain is an important seismic performance response parameter to assess the deformation related damage to a structure in an event of an earthquake. To avoid the crushing of RC shear walls, compression strain should be within the specified limit by ACI-318, which is 0.003.

6.3.2 RC Columns

Shear strength of columns is calculated based on ACI-318 (ACI-318-14, n.d.) equation and compared with the seismic shear demand obtained from both DBE and MCE level ground shaking.

It was observed one of the RC columns, shown in the red in Figure 6, is expected to fail in the shear mode of failure under both MCE and DBE level at several floor levels. All other columns possess sufficient reserve capacity against the expected level of seismic hazard. Average demand to capacity ratio (D/C) is approximately 0.5. Flexural yielding occurs only in few columns at upper floor levels due to an increase in inter story drift ratios.

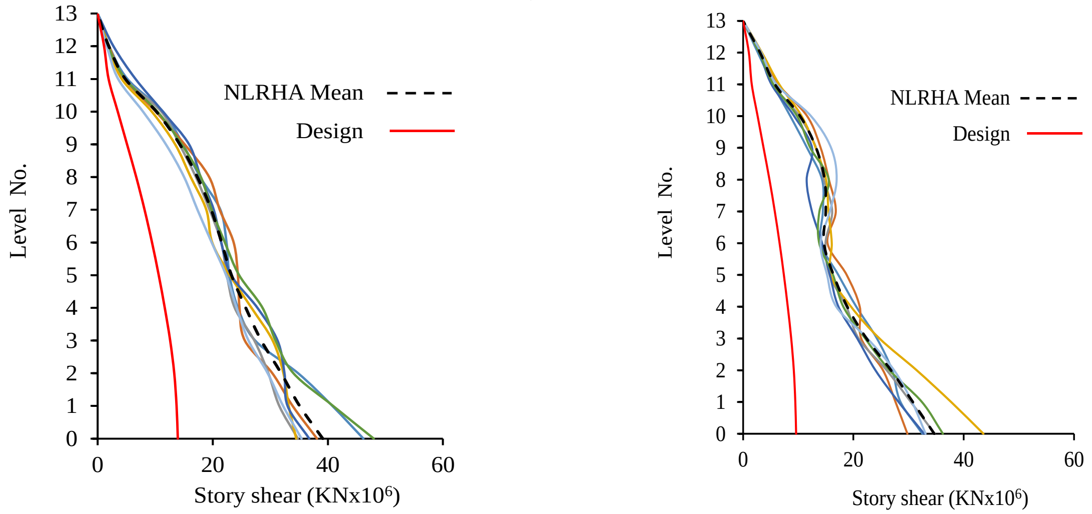

Figure 3: Story shear (a) X-axis (b) Y-axis

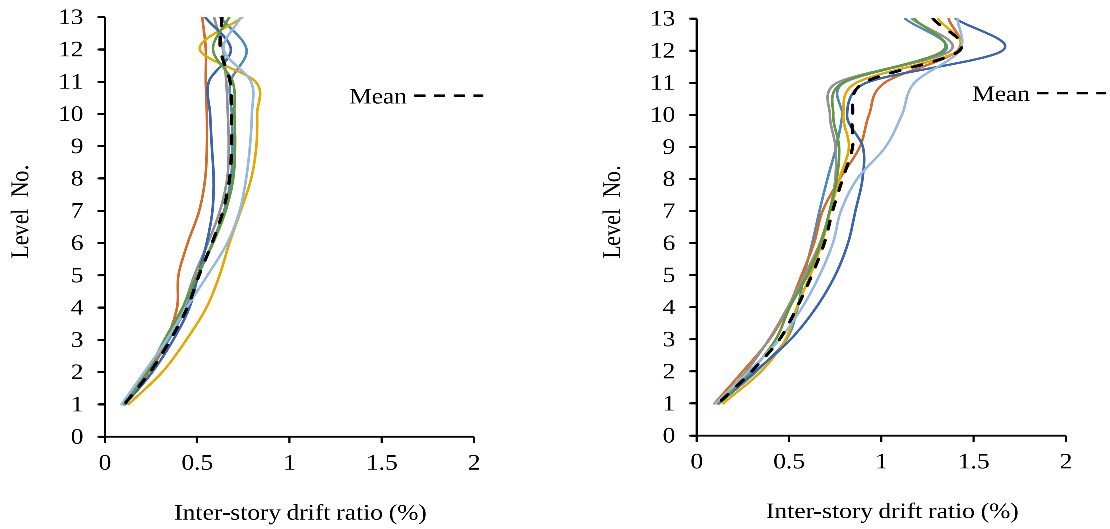

Figure 4: Inter-story drift ratio at MCE (a) X-axis (b) Y-axis

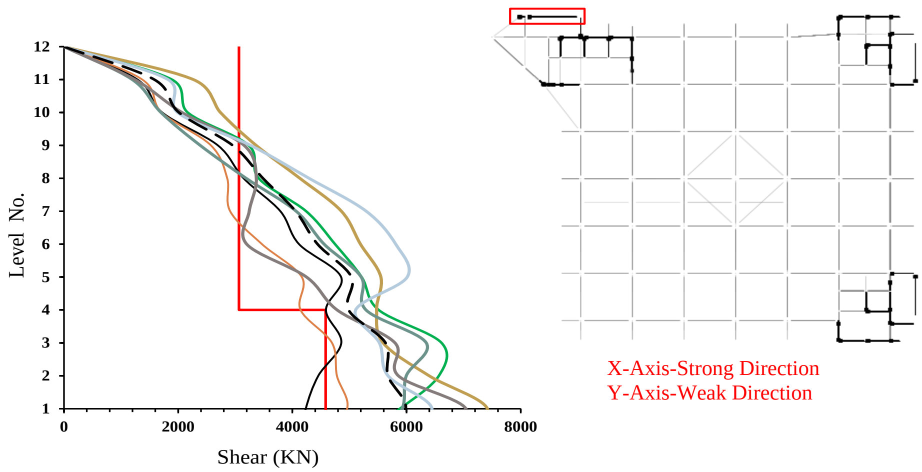

Figure 5: Shear demand versus shear capacity of RC walls at MCE level

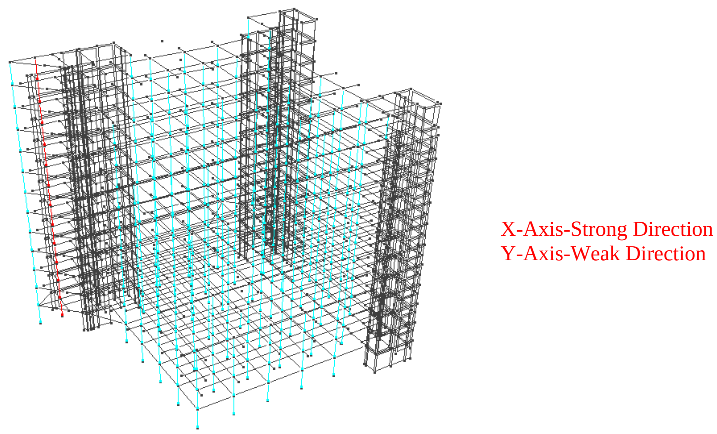

Figure 6: Shear failure in RC columns at MCE and DBE level

7. CONCLUSIONS

Following conclusions can be made based on the result of the nonlinear seismic evaluation.

- Structural walls are expected to fail in shear against MCE, as well as DBE, level seismic hazard.
- Structural walls are expected to behave favourably in flexure against both levels of seismic hazard.
- Beams should respond, to both levels of seismic hazards, favourably against force controlled, as deformation controlled, actions.
- One column is expected to fail in shear, as well as flexure, against both levels of seismic hazards. All other columns should respond favourably.
- The diaphragm is found to be adequate in transferring inertial forces to vertical members of the structure.
- The building should not be in serviceable condition after experiencing both levels of seismic hazards.
- The building poses a life safety hazard in the event of DBE level earthquake due to shear and flexural failure of one column; the failure of that column can lead to the partial collapse in that part of the building. The column is highlighted by an oval in figure 6.

- The building should not collapse under MCE level seismic forces as the drift ratio is under control, compression strains are within allowable limits and the bare frame can take at least 50% of the seismic force in both directions.

8. RECOMMENDATIONS

- It is recommended that the column, discussed in the previous section, be retrofitted to avoid a possible partial collapse in that part of the building.
- Pakistan building codes need to be developed and implemented efficiently and more studies are needed to assess the seismic performance of existing RC buildings in Pakistan So that suitable retrofitting measurements can be devised.

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Estimation of Material Properties of SDOF Structures using Visual Vibrometry

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Abstract

In order to evaluate the present status of aging structures, destructive and non-destructive testing (NDT) techniques are employed. Visual testing, as one of the oldest methods for NDT, plays a great role in the inspection of civil infrastructure. As NDT has evolved, more quantitative techniques have emerged, such as vibration analysis. New computer vision techniques for analyzing the small motions in the videos have been recently developed, allowing quantitative measurement of the vibration behavior of structures from videos. Video cameras offer the benefit of long-range measurement and can collect a large amount of data at once because each pixel is effectively a sensor. This work presents a video camera-based vibration measurement methodology for civil infrastructure. By projecting the vibrations of objects, we offer cameras as low-cost vibration sensors. The work includes the estimation of material properties for a variety of rods with known geometry by passively observing their motion in a regular frame rate video. Centering on the case where geometry is known or prepared, we indicate how information about an object's mode of oscillation can be excerpted from video and used to calculate the object's material properties.

Keywords: Visual Vibrometry, Material properties, Vibration analysis; Image processing

1. INTRODUCTION:

In the last century, there has been an unprecedented growth in infrastructure. With the passage of time, physical conditions and properties of structures change due to aging and environmental effects. Therefore, there is a need to assess the residual strength and stiffness of the existing infrastructure. Performance evaluation of the existing infrastructure is crucial to assess the strength/stiffness degradation of the structures to anticipate their performance under future extreme events. This identifies the structures under risk of major harm and even collapse in the aftermath of future extreme events which may result in loss of lives and damage to property. Structural health monitoring (SHM) field has therefore been developed in the past to come up with effective techniques to predict the performance of existing structures against extreme events. Different techniques developed for the SHM consist of both destructive and non-destructive testing (NDT) methods. NDT is the process of inspecting, testing, or evaluating materials, components or assemblies for discontinuities, or deviations in characteristics without destroying the serviceability of the component or system. NDT includes rebound hammer test, ultrasonic pulse velocity test, half-cell Penetrometer test, carbonation test, etc. These tests usually predict the residual compressive strength, stiffness and durability properties of structures. However, the results usually provide an assessment of local properties of different members while an explicit assessment of the global behavior of structures is limited. Degradation of the stiffness of existing structures will result in an increase in the natural time periods and change of mode shapes which may result in an increase or decrease in the force and displacement demands against seismic and wind actions depending upon the seismic characteristics and soil conditions of the area, manifested by the design response spectra. Certain methods have been developed in the past which relies on the calculation of natural time periods and mode shapes of existing structures to relate it to the health of the structure (Carden et al. 2016 and Doebling et al. 1996). It is well recognized that structures vibrate at preferred frequencies and mode shapes. The frequencies and forms of these modes mainly depend on the type of structure, material properties, mass, and boundary conditions. Field testing with the help of sensors placed at predetermined locations to record the vibrations is required to estimate the properties of these dominant vibration modes through vibration analysis. Different sensors with different arrangements have been utilized in the past for this purpose (Lynch JP. 2006). However, these sensors are usually limited in number, giving limited data and their cost and availability, especially in developing countries, may limit their use. In the recent past, ambitious works have been performed in the area of image and video processing where the video of a vibrating object has been used to get the natural frequencies (Davis et al. 2015). Use of such techniques is mostly limited in structural engineering, but it has huge potential as an alternative to traditional structural health monitoring of existing structures using sensors. Image processing has been used in the past works to measure deflection and crack width in civil engineering structures. However, use of video processing to calculate modal properties of structures is quite limited. The current work aims to explore the possibilities of using visual vibrometry techniques in predicting the natural frequencies of vibrating structures and ultimately assessing their material properties, resulting in a cost-effective and more detailed health monitoring of existing infrastructure.

2. METHODOLOGY:

2.1 Scheme of the experimental setup:

The experimental setup is presented in . In this experiment, three rods of different materials were used with boundary condition at the base as fixed and free end at the top mounted with an accelerometer at the tip of the rod, which assist the two roles: firstly it works like a lumped mass in order to get the rod to vibrate in the first mode and second to record acceleration data. A small video camera of 60 frames per second (fps) was clamped with the aid of a stand such that the camera was perpendicular with respect to the rod. A black dot target was attached at the tip of the rod. The background was made clear in order to get our target more visible. Improper setup could compromise the accuracy of the results, therefore, the apparatus chosen and setup established was considered to avoid any error. The oscillations were induced in the rod by using a sound amplifier and by hand.

Figure 1: Experimental Setup

2.2 Material of the rod and dimension:

Three rods of different material, i.e. Steel, aluminium, and brass were used in the experiments. The length and diameter of all the rods were 533.4 mm and 6.35 mm, respectively.

2.3 Experimental Procedure:

In the first experiment, vibrations were induced in the rod by giving it an initial deformation and then letting it go. At the same time, the accelerometer was switched on to record acceleration data. By using a stopwatch, the time span between on/off of the accelerometer was noticed. In the 2nd experiment, vibrations were induced by using the sound of frequency starting from 32 to 2000 Hz by using a sound amplifier. The accelerometer recorded the acceleration in rods due to these vibrations. Same experiments were done on all three types of rods.

2.4 Calculation of Natural frequency of rod:

Natural frequency or time period is an inherent property of a structure as it depends upon the stiffness of the structure which is a function of material and geometric properties of the structure. If the mass, geometry and boundary condition of a structure are known, its natural frequencies can be used to calculate its material properties/stiffness. For this purpose, three procedures were employed. In the first procedure, the video of the vibrating rod was analyzed manually to calculate the time required for a complete cycle of vibration. In the second approach, shaking in the rod was induced by using a sound amplifier and accelerometer data was used to find out the dominant frequencies by using Fast Fourier Transform. In the 3rd approach, the video of the rod was analyzed by using a code in MATLAB developed by Kashif (2014). The Graphical Use Interface (GUI) of the MATLAB code is shown in Figure. The MATLAB code first divides a video into several images depending on the fps rate of the video. These images are then analyzed to find out the displacement of the target.

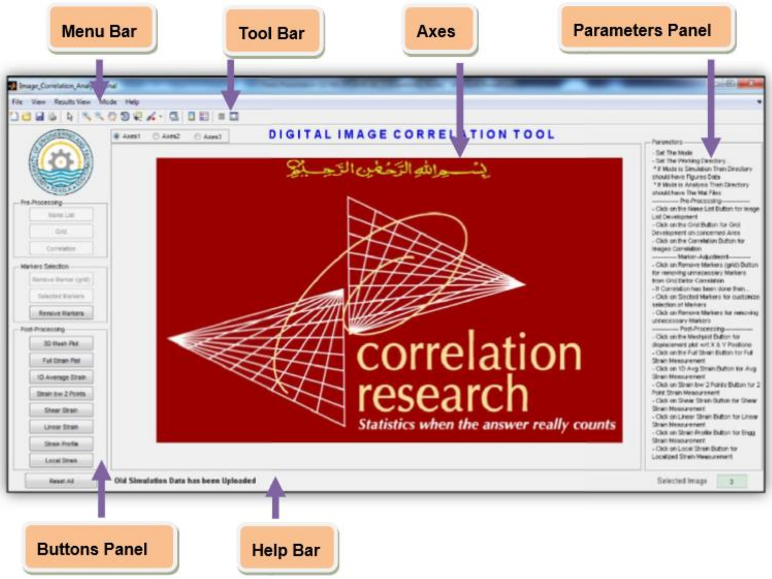

Figure2: Digital Image Correlation Software Interface

2.5 Estimation of modulus of elasticity

After calculating the fundamental natural frequency and using fixed geometric parameters of the rods i.e. Length and diameter, the following formulas were used to calculate the modulus of elasticity of the rod.
Bending frequencies of beams, rods, and pipes (Irvine et al. 2012) is given by:

$$f_1 = \frac{1}{2\pi}\sqrt{\frac{3\mathrm{EI}}{(0.2235\rho L + m)L^3}} \qquad (1)$$

Where f_1 is the fundamental frequency, E is the modulus of elasticity, I is the area moment of inertia, L is the unsupported length, ρ is the mass density (mass/length) and m is the mass of the attached objects.

Another formula for calculating natural frequencies introduced by (Shabana and Shabana. 1991) is as follows:

$$\omega_1 = 3.51563\sqrt{\frac{EI_Z}{ml^3}} \qquad (2)$$

Where, ω_1 is the angular frequency equal to 2πf, E is the modulus of elasticity, I_Z is the area moment of inertia, l is the length and m is the mass of the rod.

3. RESULTS AND DISCUSSION:

The results from both the Eq (1) and Eq (2) are quite close, but Eq (1) gives more accurate results due to the involvement of additional important parameters like it consider the mass of the object (rod) and the end mass which is attached at the end of the object (rod) while these parameters are not considered in Eq (2). Thus, these two main equations are applied to obtain the material properties of the targets.

Different materials have different strength properties. Rods made of steel, aluminium, and brass used in the present study have material properties shown in table 1. After going through certain experiments and by adopting different methodologies as explained in the methodology section, results were obtained from the accelerometer and form processing the video of the vibrating object. Modulus of elasticity calculated by using accelerometer results is shown in Table 1, which shows a reasonable accuracy level with aluminium results showing minimum error while brass showing more than 20% error.

Table 1: Comparison between Results from Accelerometer and Actual values

Material	**Experimental E (GPa)**	**Actual E (GPa)**	**% Error**
Steel	164.5	193	14.77%
Brass	127.93	105	21.84%
Aluminium	72.13	69	4.54%

The MATLAB code used is not able to capture the video of vibration of the objects against sound vibrations generated by a sound amplifier. So, the vibrations of the bars were processed by using the video in which an initial displacement was given to the bar top by hand. The MATLAB code provided the displacement time history of the top end of the bars. Figure 3 shows the displacement time history of aluminium and steel bars. Once the displacement time history is known, it is easy to find out the fundamental time period/frequency of the bar, which is then used to calculate the modulus of elasticity using EQ 1 as shown in Table 2.

The results exhibit a reasonable accuracy with the percentage of error for modulus of elasticity ranging from 3% to 20% for different material types. The results can be further refined by minimizing the impact of different sources of errors. Like the use of a high-speed camera capable of recording largest possible frequencies along with a more sophisticated testing apparatus. Furthermore, future studies can be focused on simple multi degree of freedom structures to find out natural frequencies as well as the mode shapes.

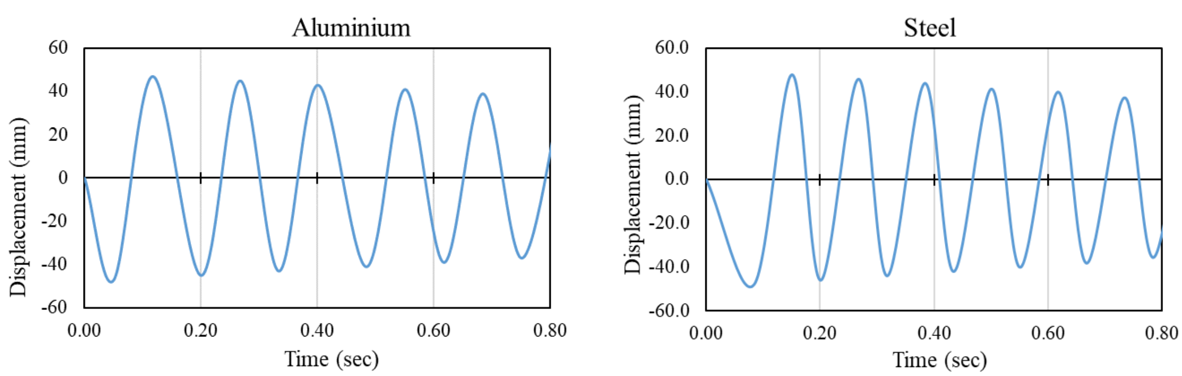

Figure 3: Displacement time history of a. Aluminium and b. Steel

Table 2: Comparison between Results from MATLAB and Actual values

Material	Experimental E (GPa)	Actual E (GPa)	% Error
Steel	163.247	193	15.42%
Brass	125.9	105	19.90%
Aluminium	71.23	69	3.23%

A comparison of % error in both the cases is shown in the form of a histogram as shown in Figure 4.

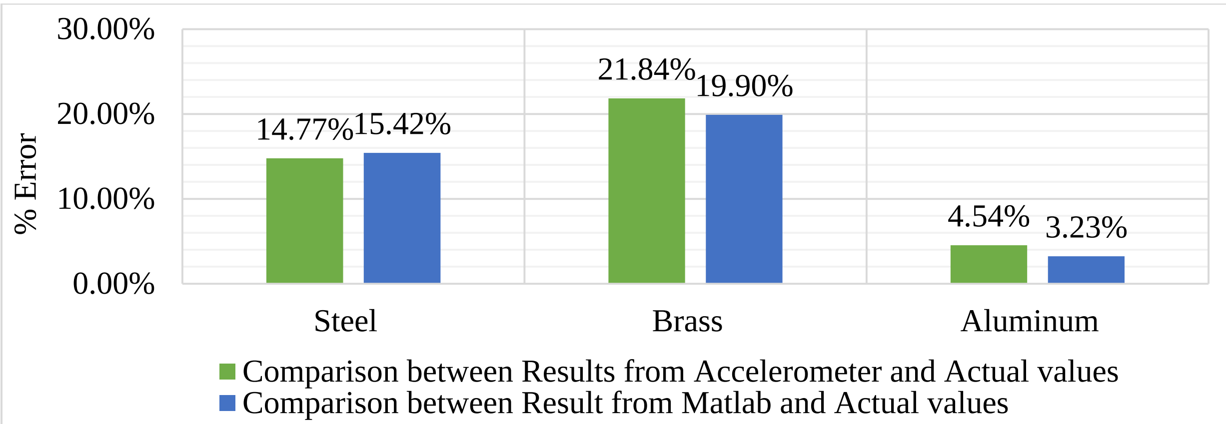

Figure 4: Comparison of error in two methods

Visual vibrometry has several applications for civil engineering works including the health monitoring of buildings, bridges, flexible structures, billboards etc. A lot of work has been done in the past to find out the deformation of structures using target or target-less approach with the algorithms being developed for target tracking. The current work aims to monitor and record this deflection in time domain, which may then be used to find out natural time period of structures. Also, such techniques can be extended to even calculate the strains in concrete using high speed video cameras. Currently, this work is underway in our research group for calculation of axial strain in concrete cylinders and the initial results are showing promising results.

4. CONCLUSIONS:

In the current study, the use of visual vibrometry to calculate the material properties of single degree of freedom structures is discussed. The results show that video-based evaluation of natural frequency gives comparable results to that of the accelerometer. The results for three different materials show a percentage error of 5% to 20%. However, the current code is able to analyze the vibrations where displacement is relatively high and visible through the naked eye. Future works should explore the efficiency of this technique for relatively smaller displacement values usually expected in building structures.

5. ACKNOWLEDGEMENTS:

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Paper ID: 234

Assessment of Existing Concrete Half-Joint Structures using Strut and Tie Analysis and The Development of The Yield Assumption Method

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Abstract

During the 1960s, half joints were commonly used in the design of concrete bridges. Due to the age and condition of such structures, it was necessary to carry out assessments of structural resistance of the half joint structures. The most commonly used method of assessment for half joint structures is the Strut &Tie Method (STM). However in many cases a simple application of the STM without further iteration can result in an underestimate of the structural resistance. For this reason, other analysis methods were developed over the period. Alternative methods include the upper bound collapse mechanism approach (CMA) and the development of the yield assumption method (YAM) as described in this paper. Experimental work was carried out by Desnerck et. al 2016 on a series of half joint beams. The aim of this paper is to compare the experimental results with analytical methods and to ascertain the efficiency of recently developed assessment method YAM. The reinforcement layout and details are taken from the experimental work and analysed using STM and YAM. It was found that the resistance obtained from YAM matches the experimental work within an error of 7%.

Keywords: Half joint, strut and tie method, yield assumption method.

1. INTRODUCTION:

The half joint form of structure was commonly used in bridges during the 1960s (Northing 2015). This concept enabled efficient installation of a centrally supported deck, and allowed a reduced construction depth by recessing the supporting corbels into the depth of the beams supported. (Mattock 1979). The other benefits of half-joint structures include the ability to standardise the design of the supported span, enabling a modular design approach to be used for a series of bridges. However, there are some disadvantages associated with this form of construction. The most prominent is related to the lack of water tightness of the joint itself resulting in deterioration (Desnerck et. al 2017). Seepage of chloride-contaminated water can accumulate in the lower half joint, resulting in corrosion of steel reinforcement. Due to access issues, half-joints can be difficult to inspect as easily as other structures and therefore there can be uncertainty about the condition of reinforcement. Limited guidance is available for the assessment of concrete half-joints. In the UK, the DMRB standards and advice notes BD44, BA39, BA51, and IAN 53 give some guidance about the consideration of section loss of reinforcement and method of assessment for half joint. These documents are very useful but are based on limited data and do not give specific guidance on the combined effect of reinforcement deterioration, concrete spalling and improper detailing at the same time.

In the practical assessment of half joint structures various analytical methods have been proposed. These include: strut and tie method (STM); collapse mechanism approach (CMA); and yield analysis method (YAM). STM is a well-known method, most commonly used for design of new structures as set out in EN1991-1-1, and is based on the lower bound theorem of limit analysis, so it generally provides conservative estimates of resistance. CMA is an implementation of the upper bound theorem of limit analysis, therefore it can give unsafe estimates of resistance unless fully optimised. Nevertheless it has been proposed in the assessment of some half-joints as a departure from the standards. The third method YAM is presented in this paper, and is a development of STM that enables the conservative estimates from STM to be improved, whilst still giving a safe estimate of resistance. The brief procedures of these three methods are detailed in section 2.

Experimental work carried out by Desnerck et. al (2016 & 2018) considered four large scale half joint details with different reinforcement arrangements. The size of specimen and reinforcement details have been taken from the experimental work by Desnerck et. al (2016 & 2018) for this study. The outcome of the experimental work by Desnerck et. al (including failure load and failure mode) have been theoretically analysed using the STM and YAM methods. The reinforcement details and layout considered for these two methods are shown in Figure 1. Comparison was carried out with the experimental output and the efficacy of each method is discussed. The aim of this paper is to compare the experimental results with analytical methods and to ascertain the efficiency of the recently developed assessment method YAM.

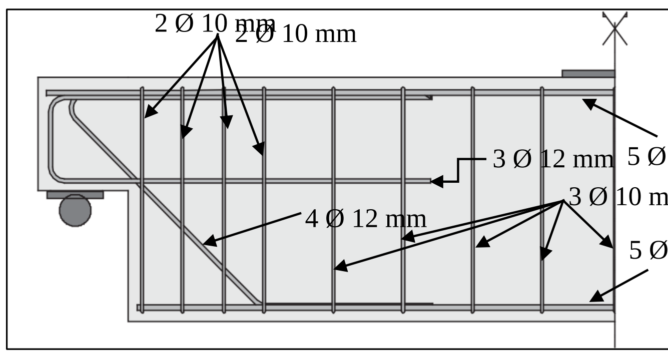

Figure 1: Reinforcement layout (Desnerck et al 2016)

2. METHODOLGY

2.1 Assessment of half joint structure in accordance with STM (using the default models in BD 44/15)

According to BD 44/15, assessment of half joint structures should be done using the Strut-and-Tie Method (STM). This method comprises idealising the forces in the half-joint using concrete struts, reinforcement ties and connecting nodes. STM is a lower bound method which requires that the equilibrium and yield criteria to be satisfied.

BD 44/15 permits two simple strut and tie models (orthogonal and inclined) to be analysed. The load carrying capacities of the half-joint structure is obtained by taking the results of the two models. Based on the reinforcement layout of Figure 1, the models, as shown in the Figure 2, are used for assessing the capacity of half joint in accordance with BD 44/15.

It is recognised that the STM can also be carried out using alternative arrangements of struts and ties, which might give a better result. However, in this study the results for the STM method are based on an assumption that the simple models of BD44/15 are applied directly without further iteration. This is a common interpretation of the BD44/15 content by practising engineers. It should be noted that a length of horizontal reinforcement was assumed as it was not indicated in the study by Desnerck et. al 2016. The assumed length will provide a full anchorage condition for the horizontal tie at the node in the orthogonal model. Hence, this may overestimate the load carrying capacity of the structure. The same assumption has been adopted for the Yield Assumption Method (YAM).

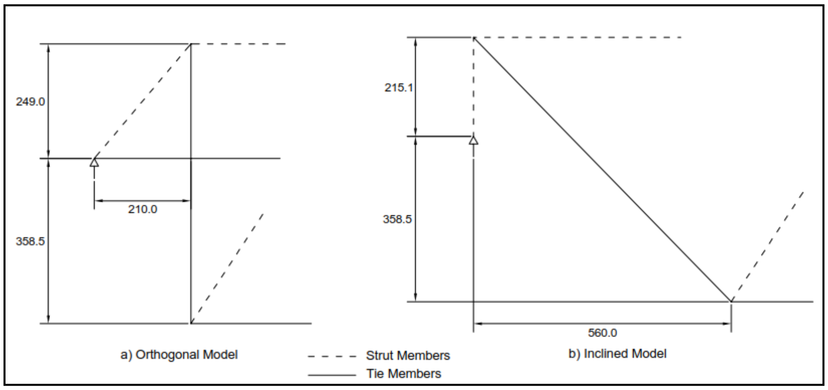

Figure 2: Strut-and-tie models in accordance with BD 44/15

2.2 Assessment of half joint structure in accordance with YAM (DAS ID:80895)

The existing STM (BD 44/15) has been adopted due to the simplicity of this approach and the capacity of the structure can be calculated by hand. Nowadays, a complex strut-and-tie model can be proposed and analysed with the help of computer aids (e.g. MIDAS Civil 2018 v2.1). The Yield Assumption Method (YAM) proposes a combined strut-and-tie system to estimate the capacity of the half-joint structure. In the system, the tie members that are fully anchored can yield one by one starting with the most critical member. Once a tie member reaches its yield condition, this tie, will be replaced with a force equal to the yield capacity of the reinforcement and applied to the relevant nodes to represent the tie in the system. Figure 3 shows the combined strut-and-tie system which is used to calculate the capacity of the half-joint structure.

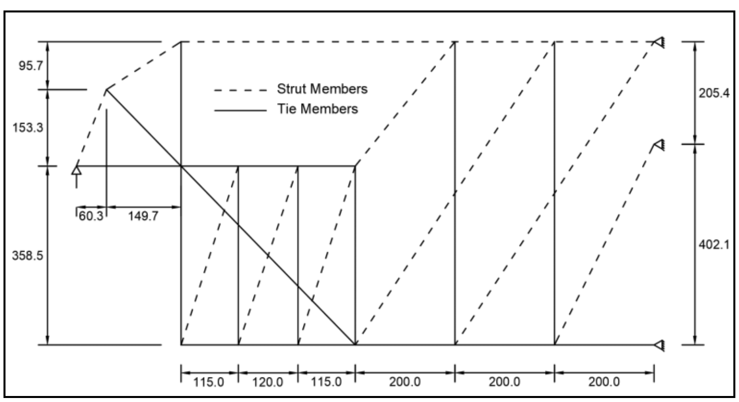

Figure 3: Proposed YAM model

3. RESULTS AND DISCUSSION

Table 1 details the results from the three methods of half joint assessment as explained in section 2. It should be noted that all the partial safety factors have been set to 1.0 for the evaluation purpose. It can be observed that difference of STM and YAM are 36%, and 7%, respectively, as compared to the experimental work. The results demonstrated that the capacity obtained from the YAM are matching with the experimental results within an error of 7%. It should be noted that STM is underestimating the capacity of the half -joint structure whereas the YAM is slightly over estimated the capacity of the half-joint structure. This may be due to the assumption made for length of the horizontal reinforcement. However, once including the partial safety factors and design strength of the materials, the STM will well underestimate the capacity of the half-joint structure.

Failure mode for each method and experimental work are also presented in the Table 1 and Figure 4. It can be observed that the failure mode determined by the YAM is comparable to the experimental work. It should be noted that failure cannot be determined by STM. In both cases, horizontal and inclined reinforcement reached to its limit and resulted in the failure of the half joint. This has developed further confidence in utilising the YAM for the assessment of half joint structures.

Table 1: Half Joint Assessment Results

Method	Maximum Half Joint Capacity (kN)	Ratio (Analytical / Experimental) %	Failure Mode
STM	256.7	64	Yielding of the inclined bars and the 1st vertical link.
YAM	431.9	107	The top 2 elongated reinforcement are the horizontal reinforcement and the inclined bars based on the strain analysis.
EXP	402.3	-	Due to the rupture of inclined bars and the horizontal reinforcement.

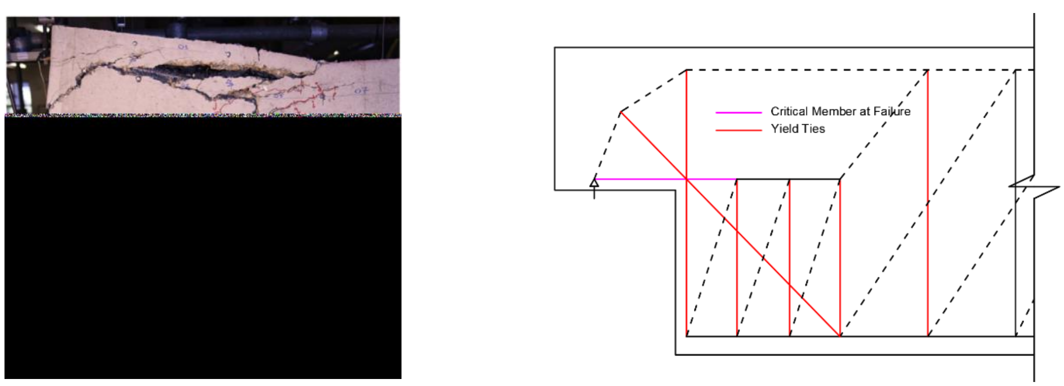

Figure 4: Comparison of Failure Mode (Experimental vs YAM)

5. CONCLUSION

The outcome of the experimental work by Desnerck et. al which include failure load and failure mode were taken in this study and theoretically analysed using STM and YAM methods. The conclusions are as follows:

1. The difference of failure load for the half joint from STM and YAM are 36% and 7%,

respectively, as compared to the experimental work.

2. The capacity obtained from YAM is matching with the experimental work within an error of 7% difference.
3. Once including the partial safety factors and design strength of the materials, the STM will well underestimate the capacity of the half-joint structure.
4. Failure mode was observed by the YAM is comparable to the observation in the experimental work.

This study has suggested that YAM is more suitable method for the assessment of half joint structures than STM. However, only one experimental result has been assessed in this study, therefore, it is recommended to carry out further checks with other experimental results to confirm the adequacy of YAM.

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The authors would like to thank Jon Shave (WSP UK Ltd) to give suggestions and advice for improving this paper.

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Seismic Isolation of RC Bridges using Low-Cost High Damping Rubber Bearings

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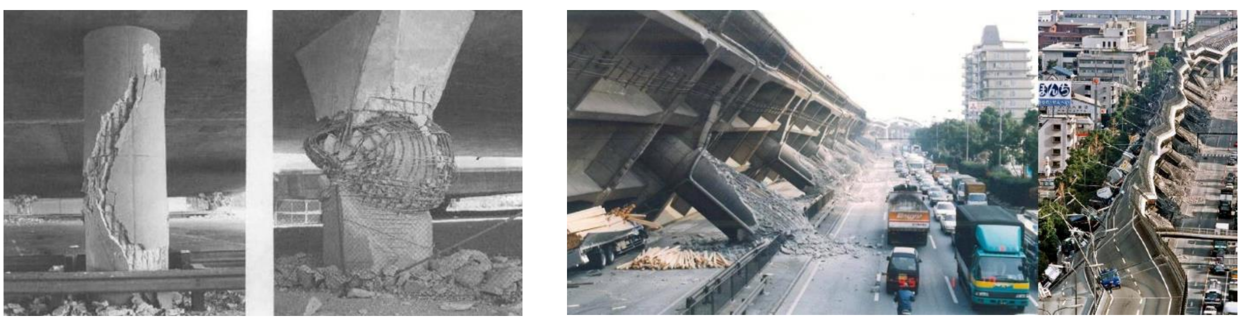
Abstract

This paper presents a simplified seismic design procedure for the seismic analysis and design of HDRB isolation for reinforced concrete (RC) bridges. Reduced-scale HDRBs were locally fabricated in Pakistan, which were investigated through shake table tests at the Earthquake Engineering Center of UET Peshawar. A natural acceleration time history of 1994 Northridge earthquake was used for multi-levels excitations from 0.10g to 1.0g. The essential mechanical properties of HDRB were obtained; including shear moduli, shear stress-strain relationship and hysteretic response curves. A simplified Bi-Linear hysteretic model was calibrated, which was incorporated within the fiber-based nonlinear finite element numerical model of representative bridge, for nonlinear time history analysis. An example bridge studied for seismic isolation design is presented, which was verified through nonlinear time history analysis procedure using design spectrum compatible natural acceleration time histories. This preliminary research have shown promising behavior of the locally fabricated HDRBs in limiting chord rotation demand on bridge piers, essential for controlling damage, under representative design basis earthquakes.

Keywords: Seismic isolation, reinforced concrete bridges, HDRBs, risk mitigation

1. INTRODUCTION:

Bridges are very critical structures and should be designed with great care, which otherwise could result in catastrophic failure, causing human and economic losses. It is not only common to developing countries, but countries with cutting edge technology and research in the field of structural and earthquke engineering are continuously suffering from the damaging earthquakes. For example, the 1989 Loma Prieta, 1971 San Fernando and 1994 Northridge earthquakes in USA and 1995 Kobe earthquake in Japan, among others, where number of important bridges collapsed, or severely damaged, and resulted in huge economic losses (Figure 1). The structural collapse mechanisms of such bridges included; fall of deck, pounding, flexure or shear failure of bridge piers, foundation or soil failure and failure of abutments. Seismic codes worldwide don't allow the collapse of bridge in any earthquake, thus, design and construction procedures were stipulated over the course of time to safeguard bridges against typical modes of collapse and limit structural damages under seismic actions, through establishment of desired strength hierarchy in the bridge to ensure damage occur where designer intends (Priestley et al., 1996). Further, the bridge should sustain functionality for emergency traffic and repair must be easy, in case a bridge incurs damages duirng an earthquake. In response to this, bridge piers were considered appropriate with the emphasize to design these for adequate inelastic deformation and seismic energy dissipation. Such ductile behavior of bridge piers can be promising in moderate to high seismicity region, however, alternative techniques will be needed to avoid or at least control damages in piers. Recent studies have focused on the development of design procedures and isolation devices, and their verification through experimental testing (Priestley et al., 1996; Christopoulos and Filiatrault, 2006; Kawashima, 2004; Skinner et al., 1994; Constantinou et al., 2007), which have shown excellent seismic performance in limiting actions on bridge components. Unfortunately, majority of reinforced concrete bridges and flyovers in Pakistan have been provided with non-seismic elastomeric bearings without proper design. Further, the installtion of such bearings is also carriedout without proper care i.e. bridge girders are directly placed on bridge pads without proper connection between pad–pier or bent beam and pad–girder. Shear keys are provided with marginal clearance between girder and keys, that prevent transverse movement of girder, thus, transfer lateral seismic force to bridge piers. The present research focuses on the investigation of low-cost seismic isolation using high damping rubber bearings (HDRBs), locally produced in Pakistan. These HDRBs have been recently installed also in Gulpur Hydropower Project in District Kotli, Azad Jammu Kashmir, which is an area of high seismicity. The present pilot research focus on shake table testing of these HDRBs for seismic qualification and to obtain the essential mechanical properties of low-cost indigenous HDRBs, which can facilaite design of seismic isoaltion for structures.



(a) - 1971 San Fernando, USA (b) - 1995 Kobe, Japan

Figure 1: Critical damages observed in some important bridges in past earthquakes.

2. PILOT RESEARCH BACKGROUND AND SCOPE:

The deck inertial forces generated during transverse seismic excitation are fully transferred to the bridge piers and abutments through structural connections, achieved through monolithic connections or bearings supplemented with shear keys. Depending on the type of bridge (e.g. steel or concrete) and expected loads, the engineer can select among various types of bearings like elastomeric, pot, line rocker and spherical bearings. In case of reinforced concrete bridges and flyovers, to provide comfort to the passing vehicle, elastomeric bearings made of natural or synthetic rubbers are widely used in Pakistan. These bearings are placed between pier and girders to allow translation and rotation movements in the longitudinal direction, however, shear keys are provided between bearings and girders, with a marginal separation, to restrain horizontal translation in the transverse direction but permitting rotation. Depending on the vertical load, the elastomeric bearings may be provided also with thin steel shim plates to avoid bulging of elastomers. Such bearings are ideally used for short span bridges located in low seismicity regions. As the conventional constructions of bridges involving bearings transmit the total inertial force to the supporting components: abutments and piers, which can incur severe damages under extreme seismic actions. This can be catastrophic in case of deficient bridges where low quality construction materials (low strength concrete and low quality re-bars) are coupled with poor quality of reinforcement detailing, as commonly observed in the existing bridge stock of Pakistan (Ali, 2009). Alternatively, this seismic action on supporting components can be reduced by permitting horizontal translation of bridge deck, yet, within the allowable deformation limit. An economical solution is to modify the design and materials of existing laminated rubber bearings; by altering the geometry, selected based on appropriate design for seismic loads, and using high damping rubber bearing (HDRB) materials instead of ordinary elastomers. Elastomers exhibiting damping in excess of 6 precent can be regarded as HDRBs (EN 15129). In this regard, the Rainbow Rubber Industry in Karachi was contacted to produce indigenous high damping rubber bearings, which were tested at the Earthquake Engineering Center of UET Peshawar, in order to retrieve their essential mechanical properties for facilitating seismic analysis and design of bearing isolations for bridges. The following sections describe the initial findings from this pilot research conducted on the indigenously produced HDRBs

3. SHAKE TABLE TESTS ON INDIGENOUS HDRBs:

3.1 Test model, instrumentation and loading protocols:

The test model comprised of a wide-deep beam with a superimposed load of 1.20 tonne, resulting in a total weight of 3.66 tonne, which was acting as a simply supported beam (Figure 2). Bearings of cross-sectional area 135 mm x 135 mm and height 90 mm were provided at both the ends. These bearings were resisting a vertical stress of about 2.00 MPa, considering the limit state displacement, which can be experienced in short-span bridges subjected to lower gravity loads. The bearings were provided with 30 thin steel shim plates (1 mm), dividing the total bearing height into 31 layers of rubber of about 2.00 mm thickness. This gave shape factor “S” of bearing of about 16. The bearings were provided with steel plates both at the top and bottom to facilitate connectivity. For testing, the bearings were mounted on the shake-table and fully secured through anchor bolts. The beam was also attached to the bearing through anchor bolts. The test model was instrumented with displacement transducers and accelerometers both at the top and bottom, in order to record the seismic input and model response under each test run. The test model was subjected to acceleration time history of 1994 Northridge earthquake, scaled to multi-levels of excitation from 0.10g to 1.0g in order to retrieve the full response of bearings. The data obtained under each test run was processed for the required baseline correction and filtering, in order to derive lateral force-displacement hysteretic response of bearings under seismic excitations.

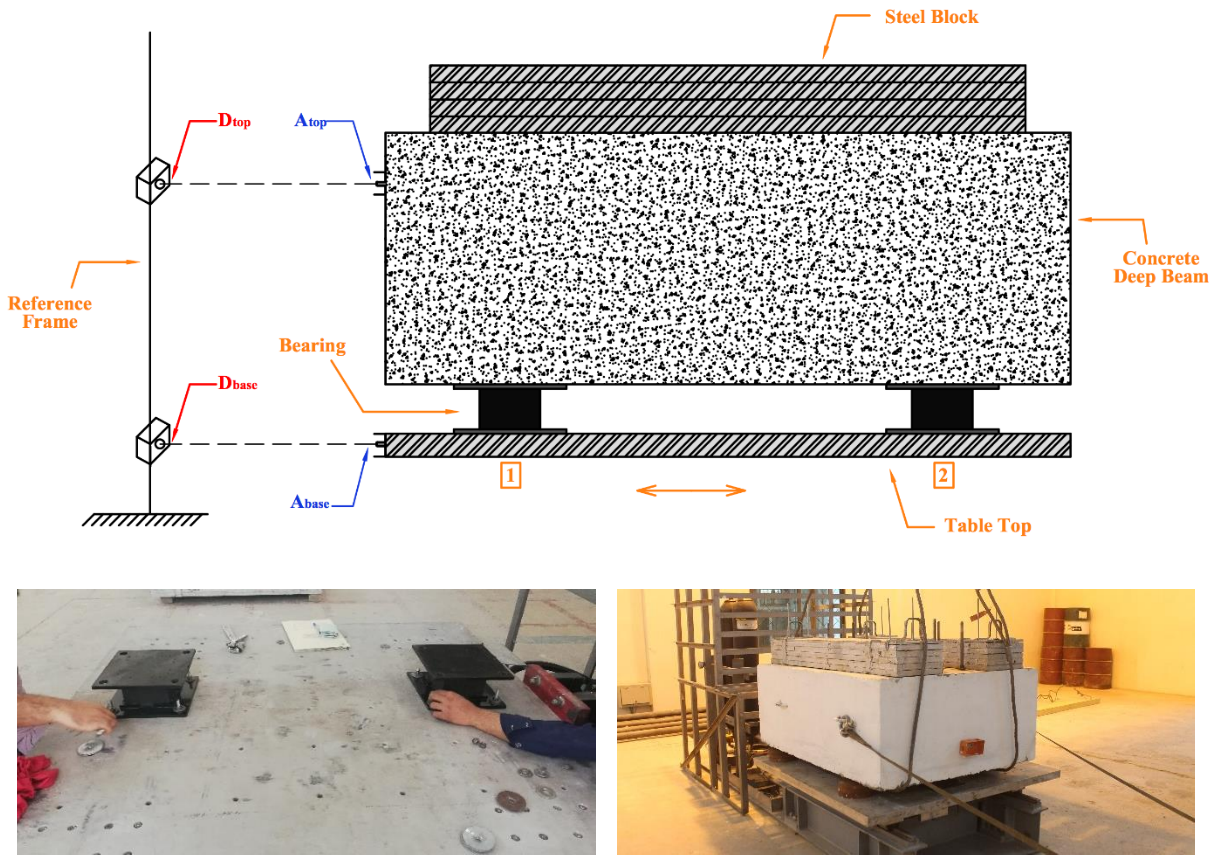
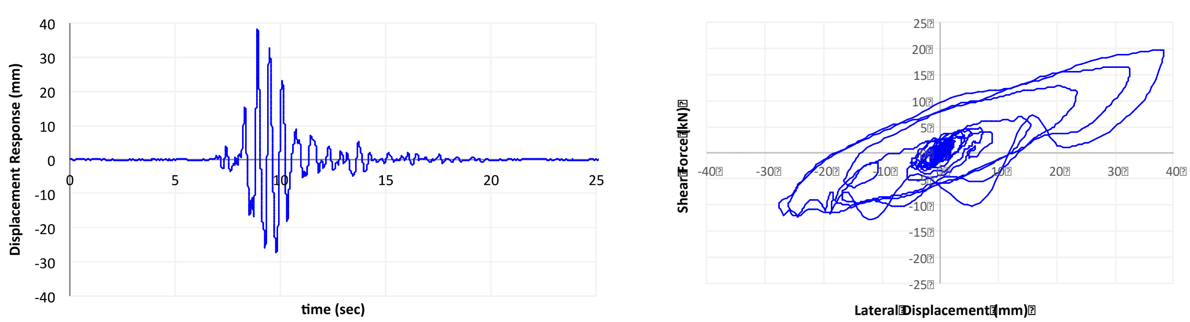


Figure 2: Experimental test model.

3.2 Observed behavior and seismic response:

The bearings under low intensity shaking were not observed with any apprecial deformation. However, the bearing exhibited signficant lateral deformation, and even rocking, under extreme shaking. Rocking of bearing was experienced due to the low-vertical stress, nevertheless, the bearings avoided overturning and were still able to recover their lateral deformation. Figure 3 shows the displacement response and force-displacement hysteretic response of the tested model under extreme level shaking (1.0g). Despite the extreme level shaking, the bearing was able to control lateral displacement demand, which was due to the high damping exhbited by the bearings. This can be evidenced also from the force-displacement response of the test model (Figure 3b).



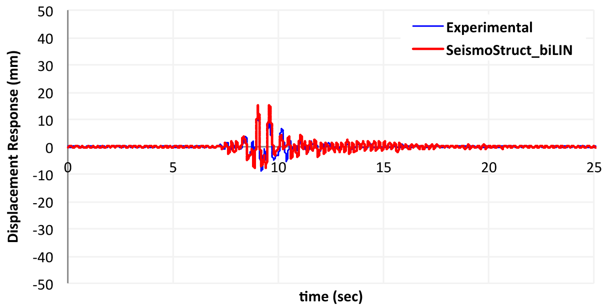
(a) – Displacement Response (b) – Hysteretic Behavior

Figure 3: Response of bearings under shake-table test with extreme shaking 1.0g.

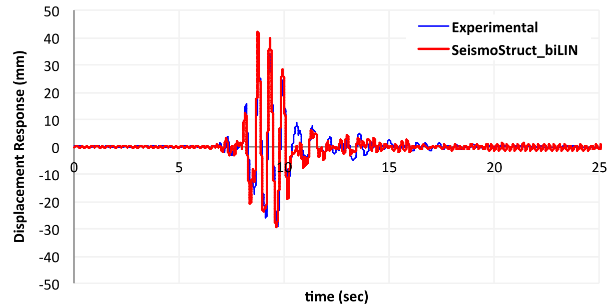
4. VERIFICATION OF HDRBs ISOLATION FOR RC BRIDGE:

4.1 Nonlinear modelling of HDRBs:

The Bi-Linear hysteretic model available in SeismoStruct was adopted, which was calibrated with the observed experimental data. The calibration involved calculating the yield force, yield stiffness and post-yield stiffness of the numerical hysteretic model. These parameters were obtained through regression analysis performed on data obtained experimentally. The calibrated numerical hysteretic model was tested against each run, which has shown excellent performance in predicting the displacement time history response of test model (Figure 4).



(a) – Displacement Response, 0.50g



(b) – Displacement Response, 1.0g

Figure 4: Comparison of numerically predicted-to-experimental displacement response

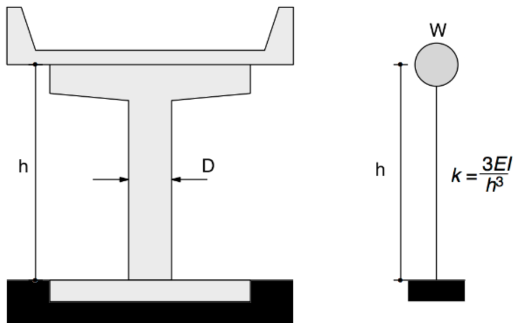
4.2 Selection, Scaling and Matching of Accelerograms:

A suite of 07 acceleration time histories, compatible with the regional tectonics, were obtained from the PEER strong ground motions data base. These accelerograms were scaled and matched to the seismic design spectrum specified in the Building Code of Pakistan (BCP-SP 2007) for highest seismic zone i.e. Zone 4, for type D "Stiff Soil". SeismoMatch program of SeismoSoft (2016) was used for scaling and matching of accelerograms. The matched accelerograms, simulating the design basis earthquakes, were used for the nonlinear time history analysis of an example bridge.

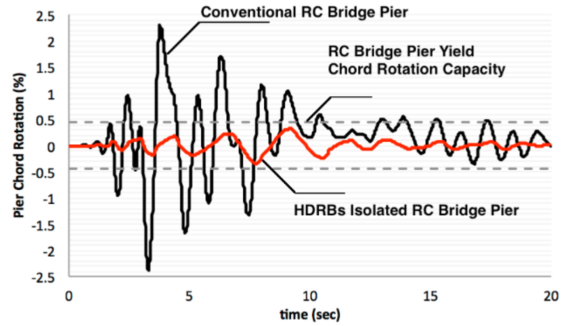
4.3 Pier Chord Rotation under Design Basis Earthquakes:

An example bridge with single cantilever pier of 4 m height and diameter 1.50 m, supporting superstructure with total weight of 190 kN/m, was considered for seismic analysis in transverse direction (Figure 5a). The pier is considered with longitudinal reinforcement of 1%, which gives lateral yield strength of pier F_y = 1715 kN. The initial yield stiffness of pier is 967 kN/cm, resulting in the initial time period of 0.56 sec for bridge pier in transverse direction. The bridge was analysed through nonlinear time history analysis procedure, using accelerogram of 1971 San Fernando earthquake, scaled and matched to seismic design spectrum. Figure 5b shows the pier chord rotation demand, with a maximum observed chord rotation of 2.40% and ductility (ratio of maximum displacement demand to yield displacement capacity) of 5.40. This chord rotation demand is clearly a larger demand for bridge pier having construction deficiencies.

Initial design of HDRBs was carried out for target period of 3.0 sec, to obtain the geometric dimensions (length, width and height) of required bearings. A total of four bearings were considered on the top of pier; two bearings supporting girder on either side. The initial design was verified through nonlinear time history analysis procedure; checking the displacement demand to remain within the allowable displacement capacity of HDRBs. The final design suggested four HDRBs with dimensions 700 mm x 700 mm and height of 470 mm. The bearing need to be reinforced with a total of 36 steel shim plates having 4 mm thickness. The isolated bridge was observed with maximum chord rotation of 0.35%, hence, the bridge pier was responding elastically (Figure 5b).



(a) – Example Bridge, Cantilever Pier



(b) – Bridge Pier Chord Rotation (%)

Figure 5: Nonlinear time history analysis of example cantiliver bridge pier, under design basis earthquake. Max. chord rotation of 2.40% and ductility of 5.40 were observed for conventional pier while chord rotation of 0.35% was observed for isolated bridge.

4. CONCLUSIONS:

The following conclusions were drawn on the basis of research conducted on low-cost indigenous HDRBs, designed and fabricated locally in Pakistan:

- The HDRBs exhibited significant viscous damping of about 16%, which helped the test model to control lateral displacement demand.
- The Bi-Linear numerical hysteretic model calibrated with the experimental data has shown excellent performance in predicting the displacement response of test model.
- Nonlinear time history analysis performed on Example Bridge has shown promising behavior of HDRBs in controlling pier damage under design basis earthquakes. A reduction of more than 80 percent was observed in the pier chord rotation demand.

ACKNOWLEDGEMENTS:

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Influence of Earthquake Direction on Nonlinear Seismic Response of Plan-Asymmetric Structure

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Abstract

This research presents the influence of varying orientations of ground motions on the global seismic demands of a mono-symmetric structural model using a validated numerical model. The considered structure is a ¼-scaled frame shear-wall model. The numerical model was established based on the response validation with the experimental findings. Based on the validated numerical model, seismic response variation at the flexible and stiff edges were compared to present the influence of stiffness eccentricity. It has been concluded in this research that such kind of structures are more sensitive towards rotational response variability compared with the translational response variability. Finally, a conclusion pertaining to the non-conservatism of the principal axis excitation is established from statistical viewpoint.

Keywords: Varying seismic orientation; Asymmetric structure; Seismic response; statistical evaluation

1. INTRODUCTION

The seismic motion is recorded in the form of two horizontal and one vertical direction. In general, these seismic components are correlated, but according to Penzien and Watabe (1974) there exist uncorrelated seismic components which could be used to determine the critical orientation of an earthquake. The determination of critical seismic response can be obtained using response spectrum method. Numerous researchers have demonstrated the influence of varying orientations of ground motion on elastic and inelastic seismic responses. For elastic seismic response, various analytical formulae have already been established (Athanatopoulou 2005) to investigate the critical seismic response of three correlated seismic components. The research concluded that critical direction of a ground motion changes with the response quantity of interest and characteristics of seismic excitation. These conclusions have also been presented in literature (Kalkan and Kwong 2013 and Alam et al. 2016) where the influence of orientation of seismic excitation on various response quantities have been illustrated based on a linear 3-D structure. Kostinakis et al. (2008), examined the critical orientation of seismic excitation and the corresponding peak response quantity on the basis of the formulae (Athanatopoulou, 2005) for special classes of buildings subjected to isotropic bidirectional ground motion (Kalkan and Kwong, 2013).

This research however, emphasizes the assessment of seismic response uncertainty from statistical viewpoint to simplify the directionality problem. In this regards, this research demonstrates seismic response variability at both flexible and stiff edges of a plan-asymmetric structure. The

research findings conclude that in torsionally-stiff plan-asymmetric structures, rotational response variability is more sensitive compared with the translational response variability at both flexible and stiff edges when the issue of seismic directionality is considered.
This research highlights a basic design problem as the conventional design practice of considering seismic excitations along the reference axes of the structure may potentially lead towards a non-conservative design estimate. Based on the statistical evaluation presented in this work, the significance of variation in seismic orientation for peak seismic response estimate is highly evident. The presented work is useful mainly for the design engineer in decision making during the design process of critical asymmetric structures.

2. TORSIONAL VIBRATION UNDER VARYING SEISMIC ORIENTATIONS

Figure 1 below shows a schematic representation of the multi-storey plan-asymmetric structure tested on a shake table along transverse direction for El Centro 1940 seismic motion. The details on the experimental setup is already available in a companion paper (Zhang et al. 2018). The structure's equation of motion (Chopra, 2001) can be expressed as follows:

$$\mathrm{M\ddot{u}\,(t)\; + C\dot{u}\,(t)\; + Ku\,(t) = F_{eff}} \tag{1}$$

where,

$$\mathrm{F_{eff} = \; -MI\ddot{U}_g} \tag{2}$$

M, C and K represent the global mass, damping and stiffness matrices of dimensions 3ψ where ψ is the degree of freedom. Each floor has been considered to have three degrees of freedom. After decomposing the external force component of equation 1, the final undamped general equation of vibration of the considered plan-asymmetric structure is expressed in equation 3.

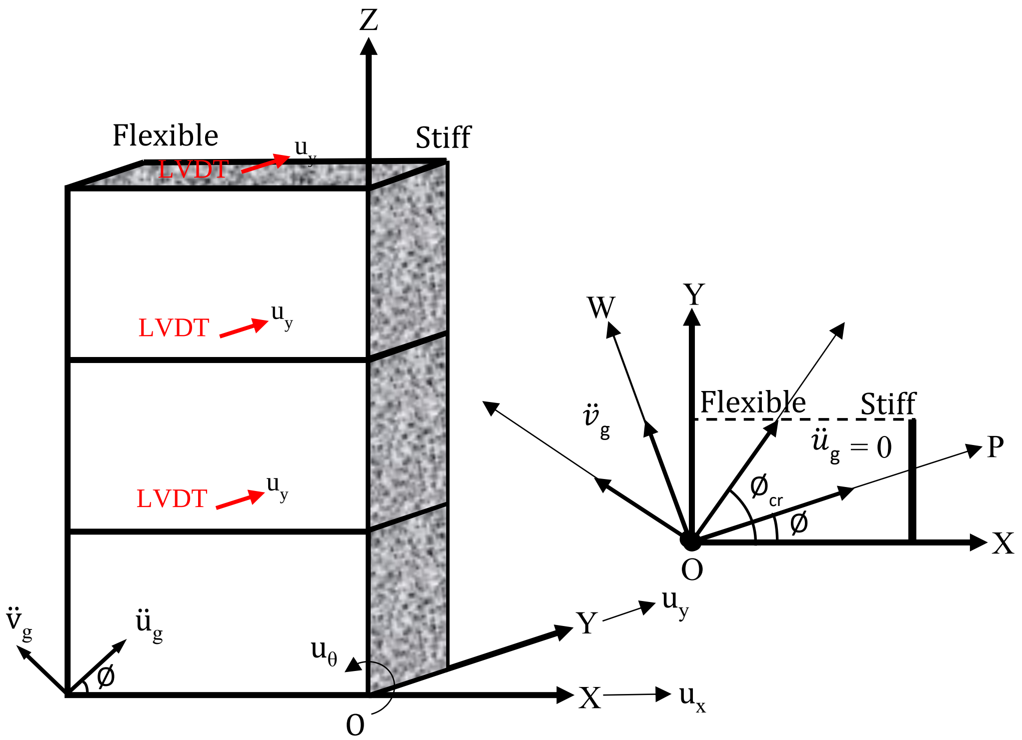


Figure 1: Schematic representation of the experimental model in perspective of varying seismic orientations

$$
\begin{bmatrix} [m] & [0] & -[m][\alpha_{ym}] \\ [0] & [m] & [m][\alpha_{xm}] \\ -[m][\alpha_{ym}] & [m][\alpha_{xm}] & [\Omega_{cm}] \end{bmatrix} \begin{bmatrix} [\ddot{u}_x] \\ [\ddot{u}_y] \\ [\ddot{u}_\theta] \end{bmatrix} +
\begin{bmatrix} [K]_x & [K]_{xy} & [K]_{x\theta} \\ [K]_{yx} & [K]_y & [K]_{y\theta} \\ [K]_{\theta x} & [K]_{\theta y} & [K]_\theta \end{bmatrix} \begin{bmatrix} [u_x] \\ [u_y] \\ [u_\theta] \end{bmatrix} =
$$

$$
\left[-\left(\begin{bmatrix} [m] & [0] & -[m][\alpha_{ym}] \\ [0] & [m] & [m][\alpha_{xm}] \\ -[m][\alpha_{ym}] & [m][\alpha_{xm}] & [\Omega_{cm}] \end{bmatrix} \left(\begin{bmatrix} I_X\ddot{u}_g \\ 0 \\ 0 \end{bmatrix} + \begin{bmatrix} 0 \\ I_Y\ddot{v}_g \\ 0 \end{bmatrix} \right) \right) \cos\theta + \right. \tag{3}
$$

$$
\left(-\begin{bmatrix} [m] & [0] & -[m][\alpha_{ym}] \\ [0] & [m] & [m][\alpha_{xm}] \\ -[m][\alpha_{ym}] & [m][\alpha_{xm}] & [\Omega_{cm}] \end{bmatrix} \left(\begin{bmatrix} -I_X\ddot{v}_g \\ 0 \\ 0 \end{bmatrix} + \begin{bmatrix} 0 \\ I_Y\ddot{u}_g \\ 0 \end{bmatrix} \right) \right) \sin\theta -
$$

$$
\left. \left(\begin{bmatrix} [m] & [0] & -[m][\alpha_{ym}] \\ [0] & [m] & [m][\alpha_{xm}] \\ -[m][\alpha_{ym}] & [m][\alpha_{xm}] & [\Omega_{cm}] \end{bmatrix} \begin{bmatrix} 0 \\ 0 \\ I_Z\ddot{\varphi}_{gz} \end{bmatrix} \right) \right]
$$

Where

$$F^{gx}{}_{eff} = -\begin{bmatrix} [m] & [0] & -[m][\alpha_{ym}] \\ [0] & [m] & [m][\alpha_{xm}] \\ -[m][\alpha_{ym}] & [m][\alpha_{xm}] & [\Omega_{cm}] \end{bmatrix}\left(\begin{bmatrix} I_X\ddot{u}_g \\ 0 \\ 0 \end{bmatrix} + \begin{bmatrix} 0 \\ I_Y\ddot{v}_g \\ 0 \end{bmatrix}\right) \tag{4}$$

$$F^{gy}{}_{eff} = -\begin{bmatrix} [m] & [0] & -[m][\alpha_{ym}] \\ [0] & [m] & [m][\alpha_{xm}] \\ -[m][\alpha_{ym}] & [m][\alpha_{xm}] & [\Omega_{cm}] \end{bmatrix}\left(\begin{bmatrix} -I_X\ddot{v}_g \\ 0 \\ 0 \end{bmatrix} + \begin{bmatrix} 0 \\ I_Y\ddot{u}_g \\ 0 \end{bmatrix}\right) \tag{5}$$

$$F^{gz}{}_{eff} = -\begin{bmatrix} [m] & [0] & -[m][\alpha_{ym}] \\ [0] & [m] & [m][\alpha_{xm}] \\ -[m][\alpha_{ym}] & [m][\alpha_{xm}] & [\Omega_{cm}] \end{bmatrix}\begin{bmatrix} 0 \\ 0 \\ I_Z\ddot{\varphi}_{gz} \end{bmatrix} \tag{6}$$

After simplification, the general equation of motion of the structure would become:

$$M\ddot{u}\,(t) + C\dot{u}\,(t) + Ku\,(t) = (F^{gx}{}_{eff})\cos\theta + (F^{gy}{}_{eff})\sin\theta + F^{gz}{}_{eff} \tag{7}$$

For a response quantity Θ, utilizing the principle of superposition, the response-history for any arbitrary seismic orientation $\emptyset$ may be reflected as a linear combination of three-response histories. Since in this research the uni-directional seismic excitation is in the translation direction is considered, seismic component along the longitudinal direction is ignored ($\ddot{u}_g$ (t) = 0), therefore, the seismic responses $\Theta_{,1X}$ (t) and $\Theta_{,1Y}$ (t) can be reduced using the formulation presented by Song et al., 2007. Thus, the typical response quantities for the case considered in this research can be expressed as follows:

$$\Theta_{,X}\,(\emptyset, t) = \Theta_{,1X}\,(\emptyset, t) \tag{8}$$

$$\Theta_{,Y}\,(\emptyset, t) = \Theta_{,1Y}\,(\emptyset, t) \tag{9}$$

3. VALIDATION OF NUMERICAL MODEL WITH EXPERIMENTAL RESULTS

The numerical structure was exposed to the same El-Centro 1940 earthquake record. The achieved displacement response in the time domain are illustrated as sum square amplitude in Figure 2 for comparison between the numerical and experimental response. Numerical findings demonstrate fairly good agreement with the experimental results.

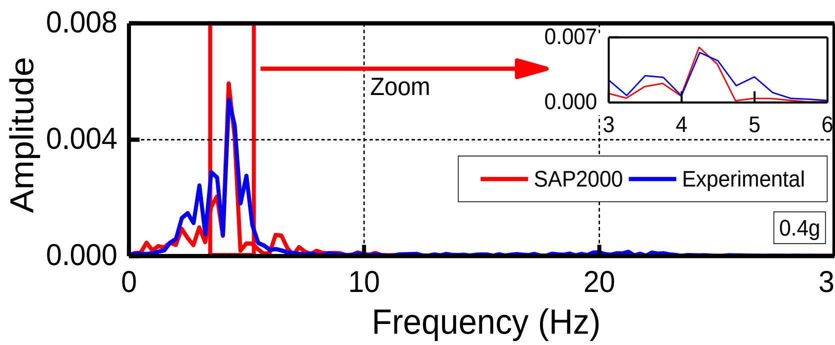
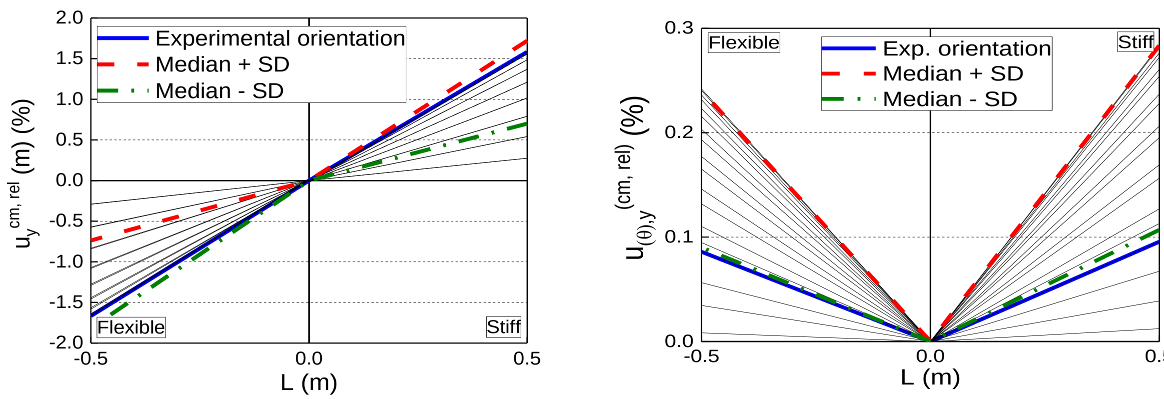


Figure 2: Numerical and experimental response validation as sum square amplitude

4. RESPONSE UNDER VARYING ORIENTATIONS

Response variability depends upon both the seismic excitation and the response quantity. This fact has been illustrated in Figure 3 in which flexible and stiff side-wise distribution of response quantities over all possible seismic directions has been presented for the validated numerical model with PGA = 0.4g as shown in Figure 3. The seismic direction that was considered along the transverse direction of the structure to induce excitation during the experimental testing is termed as exp. orientation so that the seismic response from other orientations could easily be distinguished. The distribution of seismic response quantities corresponding to exp. orientation has been highlighted with a dashed blue line whereas the black lines demonstrate the seismic response from varied orientations. It is evident from the presented illustrations that for almost all the seismic response quantities, exp. orientation has not led to the maximum response except. Moreover, rotational response has demonstrated higher seismic response variability compared with the translation response.



(a) (b)

Figure 3: Seismic response quantities under varying seismic orientations (a). Maximum translational displacement (b). Maximum rotational displacement; the blue line corresponds to the response obtained when the seismic excitation was considered along exp. orientation; the red line corresponds to median + standard deviation response; the green line corresponds to the median – standard deviation response; the black lines corresponds to other possible orientations

5. SEISMIC RESPONSE UNCERTAINTY FROM STATISTICAL VIEWPOINT

To quantify this observation, the idea of evaluating the seismic response in terms of probability of exceedance is explored. In Figure 4, the solid red line indicates the response obtained when the seismic excitation was considered along the principal axis of the structure. In the mentioned Figure, there lies approximately 20% probability of observing top roof's rotational displacement response when the seismic excitation was considered along the principal axis of the structure. Eventually, this describes the fact that there is an underestimation of rotational displacement response with approximately 80% probability. Hence, it can be said that there is always a possibility of underestimation of the peak seismic response during conventional design practices as the conventional design practices involve the use of seismic excitations only along the principle directions of a structure.

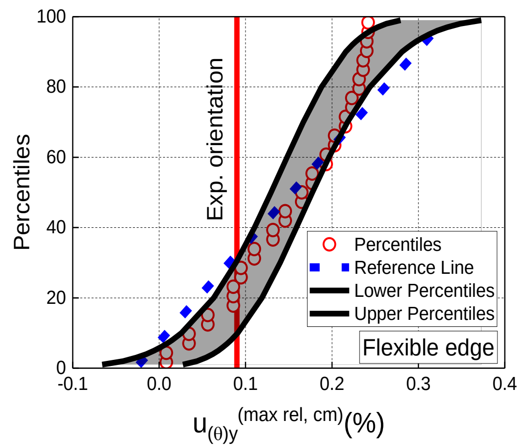


Figure 4: The probability of randomly observing rotational drift at top roof level of the plan-asymmetric structure; the red line corresponds to the response quantity obtained from the seismic excitation along principal axis of the structure

6. CONCLUSION

In this research the influence of varying orientations of ground motion using a validated numerical model was carried out on a mono-symmetric torsionally stiff structure. It has been concluded that the investigated mono-symmetric frame shear-wall structure appears to have more sensitivity towards rotational response variability compared with the translational response variation under varying seismic orientations. Both translational and rotational seismic demands from excitation along the principal axis of the plan-asymmetric structure do not demonstrate peak structural response, which implicates the non-conservatism of the traditional design approach of considering seismic excitation along the principal axis of the structure for design purposes. Treating the structure's principle axis excitation as a randomly selected orientation, there exists approximately 80% probability that for most of the rotational response quantities, the seismic response will exceed the principal axis response.

Since the peak seismic values are underestimated while considering seismic orientation only along reference axes of the structure, conventional design practices may therefore, lead to an unsafe seismic design. Based on the findings of this research, the variation in the seismic orientation is recommended for the determination of peak seismic response during the design process of critical asymmetric structures.

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Seismic Performance of Multi-Storey Torsionally-Unbalanced Torsionally-Stiff (TU-TS) Structures

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Abstract

Aesthetics and functionality requirements have turned most of the building structures to be asymmetric in recent times. These asymmetric structures have demonstrated poor seismic performance while experiencing major earthquakes in the past. Such buildings exhibit complex vibration characteristics under seismic shaking as there is coupling between the lateral and torsional components of vibration. These coupled vibrations tend to cause weak locations under torsional distress, which eventually lead towards local damage in the asymmetric structures. The identification of such weak locations is critical in nature when an asymmetric structure experiences seismic shaking. In this regard, this research demonstrates damage characteristics and global seismic behaviour of torsionally unbalance torsionally-stiff (TU-TS) systems with planar and vertical irregularities and evaluated the potentially vulnerable behaviour of TU-TS systems.

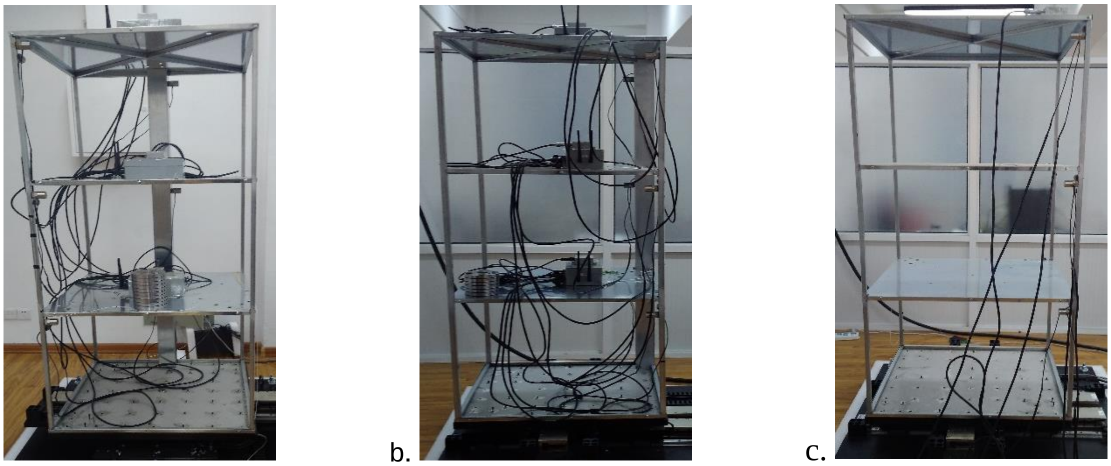
Keywords: Shake table test; Asymmetric structures; Local damage response; Global seismic response

1. INTRODUCTION:

The potential for structural failure in asymmetric structures is higher compared with symmetric structures (Oyguc et al., 2018) because torsional coupling with translation response. Numerous studies have been carried out in the past to evaluate the seismic response of asymmetric structures (Zhang et al., 2016, Anagnostopoulos et al., 2015, Duan and Chandler, 1997, Georgoussis, 2014, Tezcan and Alhan, 2001, Alam et al., 2016). However, majority of the previous studies are limited to the simplified single storey structure with global seismic effects. Research on the local damage response and its correlation with the global effects in multi-storey TU-TS systems is nearly none. This research demonstrates local damage characteristics and rotational behaviour of TU-TS structures under bi-directional seismic excitations.

2. EXPERIMENTAL MODEL:

To investigate the damage characteristics and global behaviour of TU-TS structures, three 1/6-scaled, three-storey steel structures were designed and fabricated. The fabricated TU-TS models along with their regular counterpart are illustrated in Figure 1.



a. b. c.

Figure 1: Experimental models (a). Mono-symmetric steel model (b). Bi-eccentric steel model (c). Counter-symmetric steel model

The TU-TS model were designed for both mono-eccentric and bi-eccentric stiffness eccentricities at all floor levels. Mass eccentricities (e_m) were introduced manually by shifting the centre of mass (C_M) during the shake table testing. The C_M of these structures was designed to be located at the geometric centre (C_G) of the structure while the centre of stiffness (C_S) was displaced from the C_G to form a normalized stiffness eccentricity of 0.45 (e_s / L = 0.45) at all floor levels. The state of the TU-TS system where all floors have uniform normalized stiffness eccentricity is termed as reference state of TU-TS system (Table 1). Since in the reference state, the TU-TS system possess regular floor-eccentricity along the height of the structure, the asymmetric system in this case is characterised as regularly irregular (RI) system (De Stefano et al., 2006). It is worth mentioning that the designed system by default is an RI system and after the introduction of mass eccentricities, the TU-TS systems were transformed into irregularly irregular (IRI) systems (Bosco et al., 2013). Damage characteristics and global behavior of TU-TS systems for various structural asymmetries are representative of both IR and IRI states and their detailed description is reported in Table 1. Each floor of the TU-TS system is designed such that its uncoupled torsional frequency ratio (Ω) is greater than unity thereby leading to a torsionally-unbalanced torsionally stiff (TU-TS) system. The uncoupled torsional frequency ratios (Ω) for the fabricated models were determined using equation 1 and 2 (Hejal and Chopra, 1989):

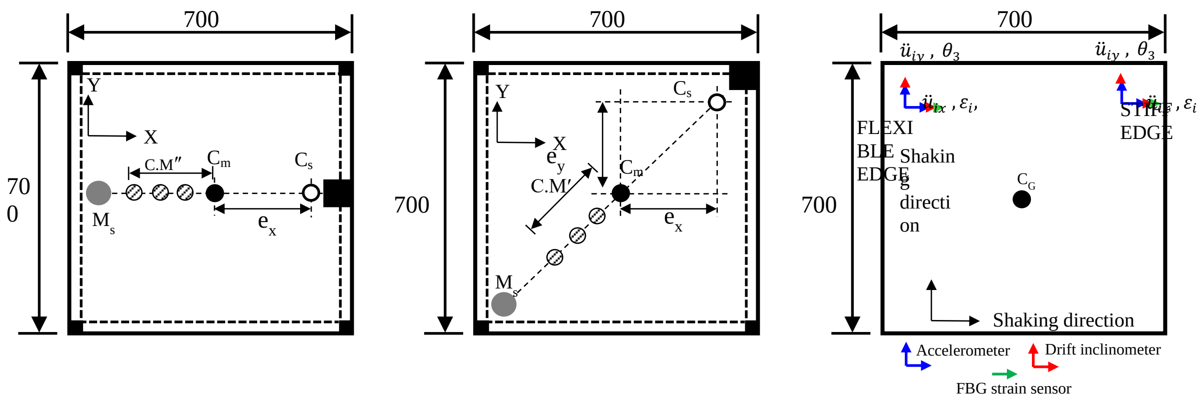
$$\Omega = \sqrt{(\omega_{\emptyset,G} \,/\, \omega_{x,G})} \tag{1}$$

$$\Omega = \sqrt{(\omega_{\emptyset,G} \,/\, \omega_{y,G})} \tag{2}$$

Where $\omega_{x,G}, \omega_{y,G}$ refer to the global translational frequencies and $\omega_{\emptyset,G}$ demonstrates the global torsional frequency of the assumed single DOF, system.

3. STRUCTURAL ASYMMETRIES AND INSTRUMENTATION:

The eccentricities in each of the experimental models were varied by shifting the C_M of the asymmetric structures to investigate the damage characteristics and global behaviour of the TU-TS systems under four different seismic excitations.



(a) (b) (c)

Figure 2: Experimental models and instrumentations (All dimensions in mm): (a) Schematic representation of eccentricities in mono-symmetric model (b) Schematic representation of eccentricities in bi-eccentric model (c) Schematic representation of deployed instruments

Based on the schematic representation of eccentricities in Figure 2, nine-asymmetric cases were developed as reported in Table 1, which can be observed in combination with Figure 2 to evaluate the structural asymmetries.

Table 1: Details of planar and vertical irregularities in TU-TS systems

Case No.	Characteristics of eccentricity	Asymmetric state
1	Reference state	Regularly-Irregular (RI)
2	a. Mass and stiffness eccentricity (e_m and e_s) variation at the 1st-floor level b. Stiffness eccentricity (e_s) at 2nd and 3rd floor level: e_s/L = 0.50	Irregularly-Irregular (IRI)
3	a. Mass and stiffness eccentricity (e_m and e_s) variation at the 2nd-floor level b. Stiffness eccentricity (e_s) at 1st and 3rd floor level: e_s/L = 0.50	Irregularly-Irregular (IRI)
4	a. Mass and stiffness eccentricity (e_m and e_s) variation at the 3rd-floor level b. Stiffness eccentricity (e_s) at 1st and 2nd floor level: e_s/L = 0.50	Irregularly-Irregular (IRI)
5	Mass and stiffness eccentricity (e_m and e_s) variation at 1st, 2nd and 3rd-floor level	Irregularly-Irregular (IRI)
6	Centre of mass (C_M) and centre of stiffness (C_S) converged at one point but dislocated from the geometric centre (C_G) of the structure	Regularly-Irregular (RI)
7	a. Third floor's mass three times higher than the adjacent lower floors b. Constant stiffness eccentricity (e_s) at 1st and 2nd floor level: e_s/L = 0.50	Irregularly-Irregular (IRI)
8	a. Second floor's mass three times higher than the adjacent upper and lower floor	Irregularly-Irregular (IRI)

	b. Constant stiffness eccentricity (e_s) at 1st and 3rd floor level: $e_s/L = 0.50$	
9	a. First floor's mass three times higher than the adjacent upper floors b. Constant stiffness eccentricity (e_s) at 2nd and 3rd floor level: $e_s/L = 0.50$	Irregularly-Irregular (IRI)

The deployed instruments to monitor the structural response include inclinometers at the top roof level, accelerometers, and bare Fibre Bragg grating (FBG) strain sensors at all floor levels. For a better comparison of the seismic responses, the instrumentations were arranged at both the flexible (FS) and stiff sides (SS) of TU-TS systems.

4. SEISMIC LOADING PROGRAM:

The described TU-TS systems were exposed to bi-directional seismic excitations for four different ground motion inputs as illustrated in Figure 3.

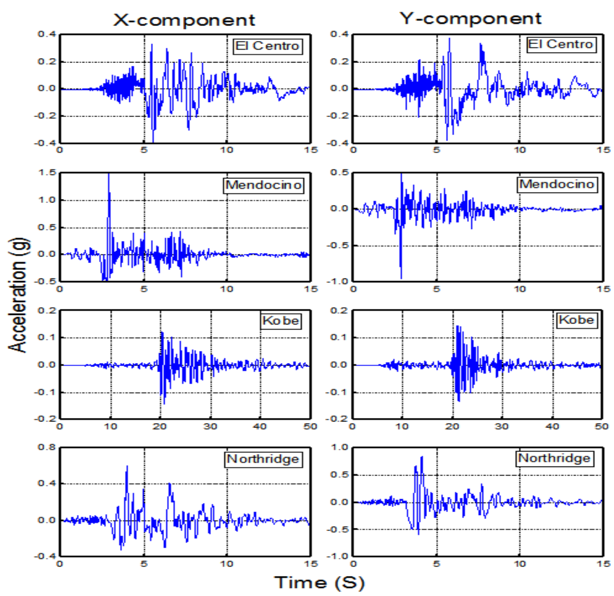
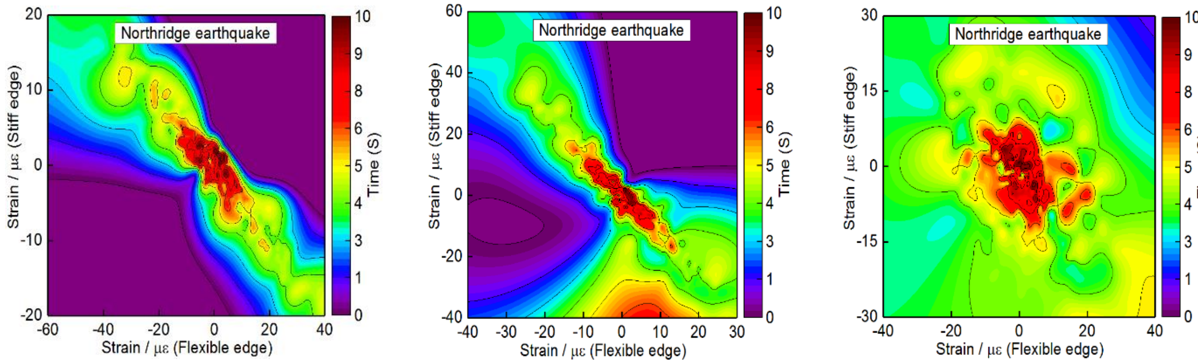


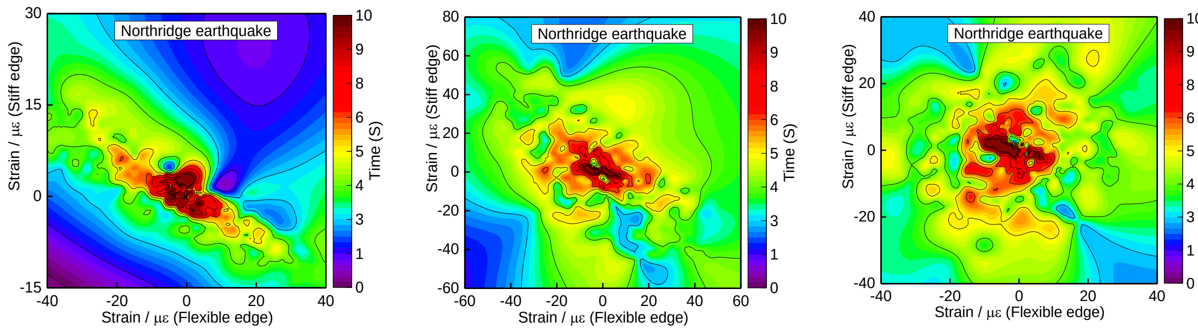
Figure 3: Input seismic motions in the time domain

5. LOCAL DAMAGE CHARACTERISTICS OF TU-TS SYSTEMS:

This section demonstrates the damage characteristics of the TU-TS systems for both mono-symmetric and bi-eccentric models at the flexible and stiff edges (Figure 4).



Bi-eccentric structure



Mono-symmetric structure

Figure 4: Damage response at the flexible and stiff edge of the TU-TS systems for the reference state under Northridge earthquake: First column corresponds to the damage response at first-floor level; Second column corresponds to the damage response at the second-floor level; Third column corresponds to the response at first-floor level

It can be seen that for TU-TS structures, simultaneous occurrence of tensile deformation at first floor level for both FS and SS is negligible. However, at the intermediate and top roof levels, simultaneous occurrence of compressive and tensile deformations is highly evident at both FS and SS of the TU-TS system in the reference state. This observation is important in regards to the damage response correlation with the global rotational response of the asymmetric systems. Besides, it can be observed that first floor demonstrates equally negligible compressive and tensile deformations whereas; the top roof demonstrates higher tensile deformations at the FS of the TU-TS systems. In general, for this particular case, top roof level is expected to experience higher amount of local damage because of higher tensile deformations. Moreover, the abrupt change in the local deformation demands at the intermediate floor highlights the fact that the seismic response is dominated by the second mode of vibration. This observation can be correlated with the global behaviour of the asymmetric structures presented in the next section where second mode dominance is highly evident especially for mono-symmetric structures.

6. GLOBAL RESPONSE CORRELATION WITH DAMAGE CHARACTERISTICS:

In the case of bi-eccentric and mono-symmetric TU-TS systems, it can be seen that the acceleration demands at the flexible and stiff edges have demonstrated similar response trends under far-field seismic excitation (Kobe earthquake).

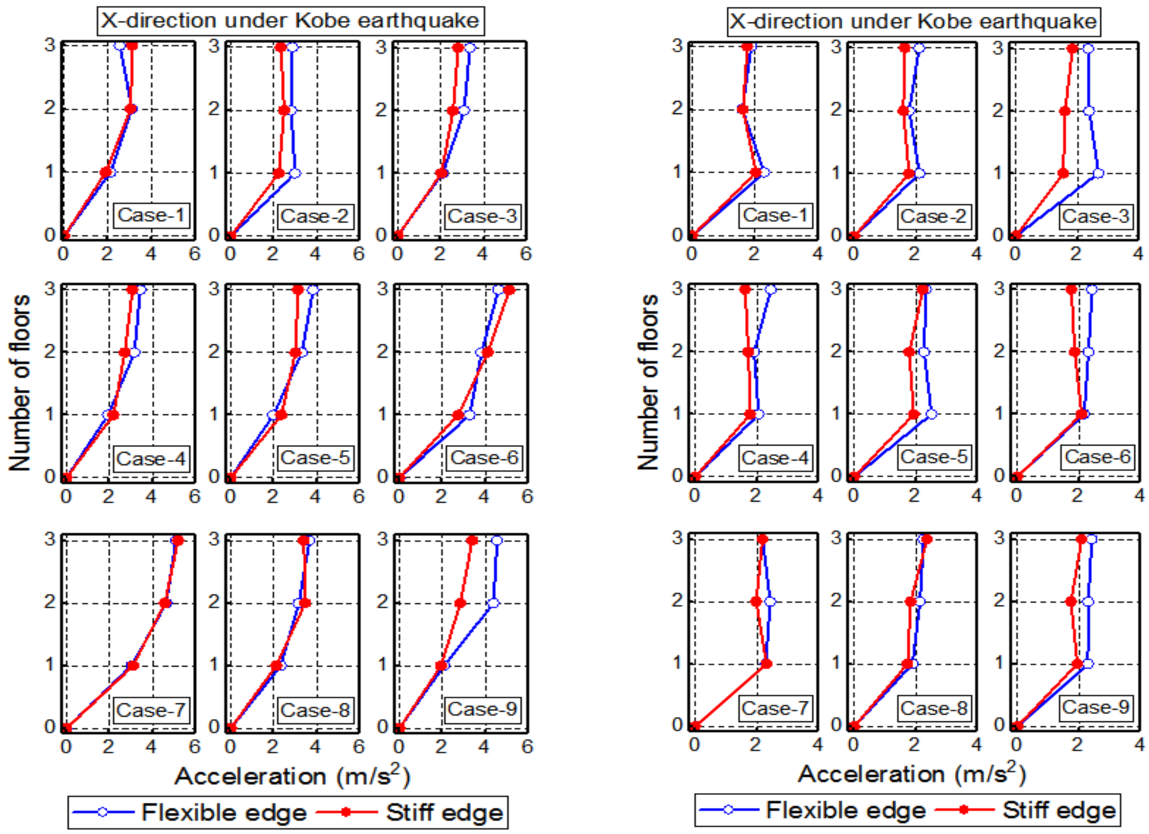


Figure 5: Amplified acceleration response of mono-symmetric and bi-eccentric structures (a). X-direction response of mono-symmetric structure under Kobe-earthquake (b). X-direction response of bi-eccentric structure under Kobe-earthquake

It is worth mentioning that lower floor eccentricities have the highest influence on the top floor's acceleration demands. Similarly, top floor's eccentricities have the highest influence on the lower's floor acceleration response. It should be noted that the described observations for mono-symmetric structure is only true for the asymmetric direction. The symmetric direction of the same system remained least affected under torsional vibrations. Furthermore, rotational response is highly evident for the IRI state of structural asymmetry. For the case of uniform eccentricities along the height, floor rotation response is observed to be minimum.

7. CONCLUSIONS:

Based on the detailed experimental investigations, following conclusions are established:

- In terms of local damage response, both mono-symmetric and bi-eccentric TU-TS systems are likely to form weak locations at the flexible edge of the intermediate and top roof levels. Lower order floors are the least affected in such TU-TS systems.
- In bi-eccentric TU-TS system with mass and stiffness eccentricities, top floor eccentricity has the highest influence on the maximum global seismic demands at the lower floor levels. Conversely, lower floor eccentricity has the highest influence on the top floor's global seismic demands. In the case of mono-symmetric structures, similar trends were monitored

with an exception that the observed influence was dominant only in the direction of eccentricity. The symmetric direction was the least affected under seismic shaking.

- In both bi-eccentric and mono-symmetric TU-TS systems with mass and stiffness eccentricities, top roof experiences the highest influence of asymmetry when the planar eccentricities are non-uniform along the height of the structure. This can be observed from appreciably variant seismic responses at the FS and SS of the of TU-TS systems.
- Top roof eccentricities tend to cause highest rotational response at top roof level in TU-TS systems from global response perspective.
- Eccentricities on a floor tend to transmit their influence to the adjacent lower/upper floors.

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Seismic Performance of Multi-Storey Torsionally-Unbalanced Torsionally-Flexible (TU-TF) Structures

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Abstract

Asymmetric structures have demonstrated poor seismic performance under coupled torsional vibrations. These vibrations tend to induce stress concentration at weak locations and eventually cause damage to the structural components. From seismic performance perspective, such weak locations are challenging to be determined in advance. However, with effective monitoring of the local deformation behaviour correlated with the global response of the structure, such estimations can be a possible realization. In this regard, this research experimentally evaluates the potential weak locations and damage characteristics under stress concentration in 1/6-scaled torsionally-unbalanced torsionally-flexible (TU-TF) systems. It has been concluded that TU-TF systems are vulnerable to damage appreciably at both flexible and stiff edges under sudden changes in the seismic demands under higher-mode effects.

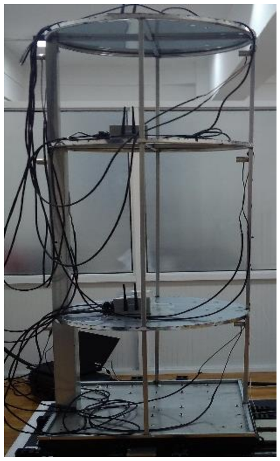
Keywords: Shake table test; torsionally-flexible structures; local seismic damage; Global seismic response.

1. INTRODUCTION:

The potential for structural failure in asymmetric structures is higher compared with symmetric structures (Oyguc et al., 2018) because of torsional coupling with the translation response. Numerous studies have been carried out in the past to evaluate the seismic response of asymmetric structures (Zhang et al., 2016, Anagnostopoulos et al., 2015, Duan and Chandler, 1997, Georgoussis, 2014, Tezcan and Alhan, 2001, Alam et al., 2016). However, majority of the previous studies are limited to the simplified single storey structure with global seismic effects. Research on the damage concentration and its correlation with the global effects in multi-storey asymmetric structures is nearly none. This research demonstrates potentially vulnerable locations in TU-TF structures evaluated through local damage response correlated with the global seismic behaviour.

2. EXPERIMENTAL MODEL DESIGN:

To investigate the damage characteristics and global behaviour of TU-TF structures, two 1/6-scaled, three-storey steel structures were designed and fabricated. The fabricated TU-TF model along with its regular counterpart is illustrated in Figure 1.

a. 

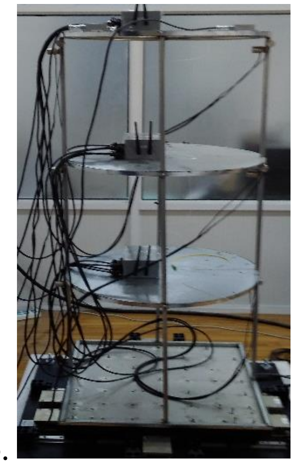
b. 

Figure 1: Experimental models (a). TU-TF bi-eccentric steel model (b). Counter-symmetric steel model

The TU-TF model were designed to contain bi-directional stiffness and strength eccentricities at all floor levels. Mass eccentricities (e_m) were introduced manually by shifting the centre of mass (C_M) during the shake table testing. The C_M of these structures was designed to be located at the geometric centre (C_G) of the structure while the centre of stiffness (C_S) was displaced from the C_G to form a normalized stiffness eccentricity of 0.45 (e_s / L = 0.45) at first floor, 0.35 (e_s / L = 0.45) at second floor and 0.30 (e_s / L = 0.45) at third floor. The described state of the TU-TF system where all floors have non-uniform normalized stiffness and strength eccentricity is termed as reference state of TU-TF structure (Table 1). Since in the reference state, the asymmetric system possess irregular floor-eccentricity along the height of the structure, the asymmetric system in this case is characterised as irregularly irregular (IRI) system. It is worth mentioning that the designed system by default is an IRI system due to varying stiffness and strength eccentricities along the height of the structure. Therefore, all the investigated cases in this research pertain to an IRI system. Each floor of the TU-TF system is designed such that its uncoupled torsional frequency ratio (Ω) is less than unity thereby forming a torsionally-unbalanced torsionally flexible (TU-TF) system. The approximate global translational frequencies ($\omega_{x,G}, \omega_{y,G}$) and torsional frequency ($\omega_{\emptyset,G}$) of the 3-DOF system can be expressed as follows:

$$\omega_{x,G} = (K_x / M)^{0.5} \tag{1}$$

$$\omega_{y,G} = \left(K_y / M\right)^{0.5} \tag{2}$$

$$\omega_{\emptyset,G} = \left(K_{\emptyset} / (M\,.\,e^2 + J'_{\emptyset,G}\right)^{0.5} \tag{3}$$

Where K_x, K_y and $K_{\emptyset}$ are the translational stiffness in the X-direction, Y-direction and about the vertical direction respectively. M refers to the floor mass and e describes the eccentricity between C_M and C_S. In addition, $J'_{\emptyset,G}$ refers to global-polar moment of inertia and can be expressed as follows:

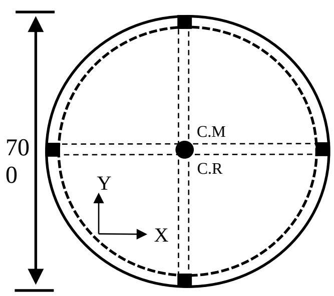
$$J_{\emptyset,G} = \left(\sum_{i=1}^{n} J'_{\emptyset,i} + m_i\left((\alpha_{xmi} - \alpha_{xG})^2 + (\alpha_{ymi} - \alpha_{yG})^2\right)\right) \tag{4}$$

In the above equation, $J'_{\emptyset,i}$ refers to the polar moment of inertia of the respective floor at C_M where α_{xG} and α_{yG} are the global coordinates at C_G of the n^{th}-DOF system. The uncoupled torsional frequency ratios (Ω) for the fabricated models were determined using equation 5:

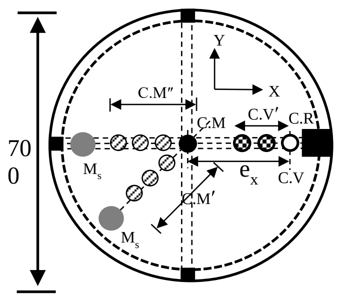
$$\Omega = \left(\omega_{\emptyset,G} \,/\, \omega_{x,G}\right)^{0.5} \text{ and } \Omega = \left(\omega_{\emptyset,G} \,/\, \omega_{y,G}\right)^{0.5} \tag{5}$$

3. STRUCTURAL ASYMMETRIES AND INSTRUMENTATION:

The eccentricities in each of the experimental models were varied by shifting the C_M of the asymmetric structures. Therefore, the assessment of damage characteristics and global behaviour of these models for various asymmetric conditions were evaluated after exciting the TU-TF systems under four different seismic inputs with their dominant vibration periods illustrated in Figure 2.



a



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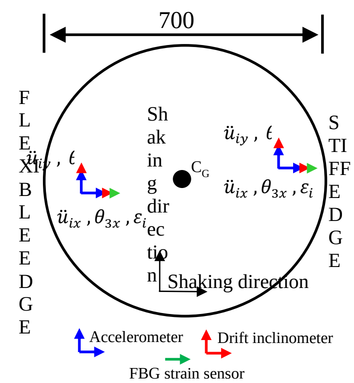

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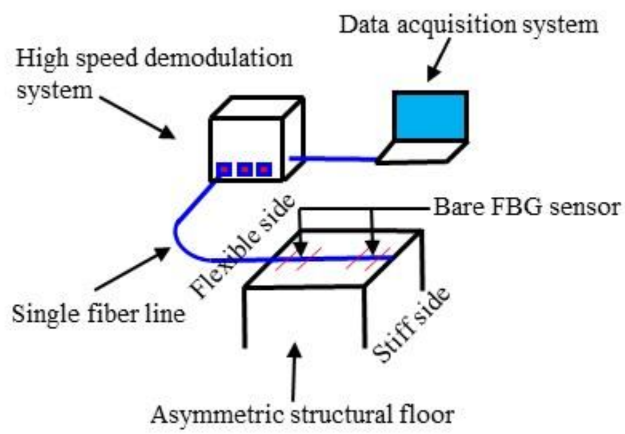


d

Figure 2: Experimental models and instrumentations (All dimensions in mm): (a) Schematic representation of symmetric model (b) Schematic representation of eccentricities in TU-TF model (c) Schematic representation of asymmetric structure equipped with instruments; accelerometers, FBG strain sensors, Drift inclinometers (d) Data acquisition mechanism for FBG sensors

Based on the schematic representation in Figure 2b, twenty-four asymmetric conditions were established for experimental evaluation. Seismic response from similar asymmetric conditions were averaged at the end of experiment, which eventually transformed the experimental findings into nine-asymmetric cases. These nine asymmetric cases are presented in Table 1, which can be observed in combination with Figure 2 to evaluate the structural asymmetries. It should be noted that C_V in Figure 2b refers to the strength eccentricity whereas C_V' refers to the changing state of strength eccentricity in the adjacent upper floors.

Table 1: Details of planar and vertical irregularities in TU-TF models

Case No.	Characteristics of eccentricity	Asymmetric state
1	Reference state	Irregularly-Irregular (IRI)
2	a. Mass and stiffness eccentricity (e_m and e_s) variation at the 1st-floor level b. Stiffness eccentricity (e_s) at 2nd and 3rd floor level: e_s/L = 0.50	Irregularly-Irregular (IRI)
3	a. Mass and stiffness eccentricity (e_m and e_s) variation at the 2nd-floor level b. Stiffness eccentricity (e_s) at 1st and 3rd floor level: e_s/L = 0.50	Irregularly-Irregular (IRI)
4	a. Mass and stiffness eccentricity (e_m and e_s) variation at the 3rd-floor level b. Stiffness eccentricity (e_s) at 1st and 2nd floor level: e_s/L = 0.50	Irregularly-Irregular (IRI)
5	Mass and stiffness eccentricity (e_m and e_s) variation at 1st, 2nd and 3rd-floor level	Irregularly-Irregular (IRI)
6	Centre of mass (C_M) and centre of stiffness (C_S) converged at one point but dislocated from the geometric centre (C_G) of the structure	Irregularly-Irregular (IRI)
7	a. Third floor's mass three times higher than the adjacent lower floors b. Constant stiffness eccentricity (e_s) at 1st and 2nd floor level: e_s/L = 0.50	Irregularly-Irregular (IRI)
8	a. Second floor's mass three times higher than the adjacent upper and lower floor b. Constant stiffness eccentricity (e_s) at 1st and 3rd floor level: e_s/L = 0.50	Irregularly-Irregular (IRI)
9	a. First floor's mass three times higher than the adjacent upper floors b. Constant stiffness eccentricity (e_s) at 2nd and 3rd floor level: e_s/L = 0.50	Irregularly-Irregular (IRI)

The deployed instruments to monitor the structural response include drift inclinometers at the top roof level to measure the angular drift (rotation) response at the flexible and stiff edges, accelerometers, and bare Fibre Bragg Grating (FBG) strain sensors at all floor levels. For a better comparison of the seismic responses, the instrumentations were arranged at both the flexible side (FS) and stiff side (SS) of the TU-TF systems.

4. SEISMIC LOADING PROGRAM:

TU-TF systems were exposed to bi-directional seismic excitations for four different ground motion inputs as illustrated in Figure 3.

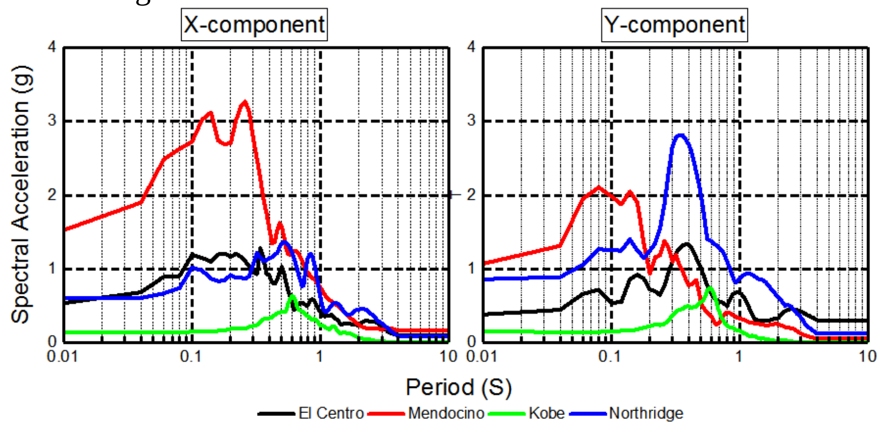
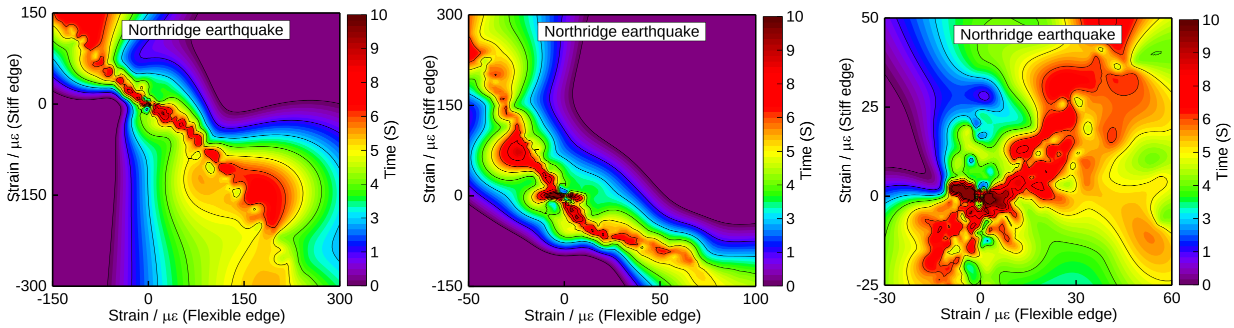


Figure 3: Seismic excitations

5. LOCAL DAMAGE CHARACTERISTICS OF TU-TF SYSTEMS:

The presented results in Figure 4 are representative of the reference state (case-1) of the TU-TF system only. It can be seen that for TU-TF systems with stiffness and strength eccentricities, lower floors are expected to experience higher damage compared with the upper floors. However, in terms of simultaneous compressive and tensile deformations at the FS and SS of the structure, first floor and intermediate floor demonstrate negligible influence as compared to the top roof level. The first floor is sensitive towards higher tensile deformations at the FS whereas in terms of compressive deformations, SS appears to be more sensitive compared with the FS of the structure. The intermediate floor experienced similar response with a difference in the behaviour of the two edges. At intermediate level, stiff edge of the structure appears to experience higher damage response mainly because of the dominance of the second mode, which has appreciably transmitted the damage response from first floor level to the intermediate floor level. The sudden change in the damage response at top roof level is attributed to the contribution of higher modes effect in such kind of highly torsionally flexible systems. Based on the experimental findings, it can be concluded that torsionally flexible structures demonstrate quite abnormal local stress concentration response along the height of the structure when observed in the reference state (Stiffness and strength eccentricities only). Moreover, the presented findings are helpful in determining the potentially weak regions involving local stress-concentration in torsionally flexible asymmetric structures.



a b c

Figure 4: Damage response at the flexible and stiff edge of the TU-TF structures for reference state under Northridge earthquake: (a) damage response at first-floor level; (b) Second column corresponds to the damage response at the second-floor level; (c) Third column corresponds to the response at first-floor level

6. GLOBAL RESPONSE CORRELATION WITH DAMAGE CHARACTERISTICS:

In the case of bi-eccentric TU-TF systems, global acceleration demands at the two edges have approximately similar response trends. For all cases of irregularities, maximum response occurred only at the FS compared with SS of the asymmetric structure with an exception to few cases of asymmetry. It is worth mentioning that the torsional influence in terms of response transition between the FS and SS is considerably high in the direction of major component of seismic excitation (Y-direction for Kobe earthquake). Similarly, in terms of maximum response it can be seen that the seismic response is influenced at first-floor level under top floor's planar and vertical, mass and stiffness eccentricities. Moreover, it can be seen that for top floor eccentricities, both the edges have induced relatively higher seismic demands at the first floor compared with the rest of the floors. This leads to the conclusion that top floor's planar and vertical mass eccentricity in TU-TF systems transmit its influence to the adjacent lower floors. Similarly, lower floor eccentricities have demonstrated higher influence on the seismic response at top roof level. Both local and global behavior of the TU-TF systems suggest that stiff edge is equally vulnerable to damage as the flexible edge under intense seismic shaking.

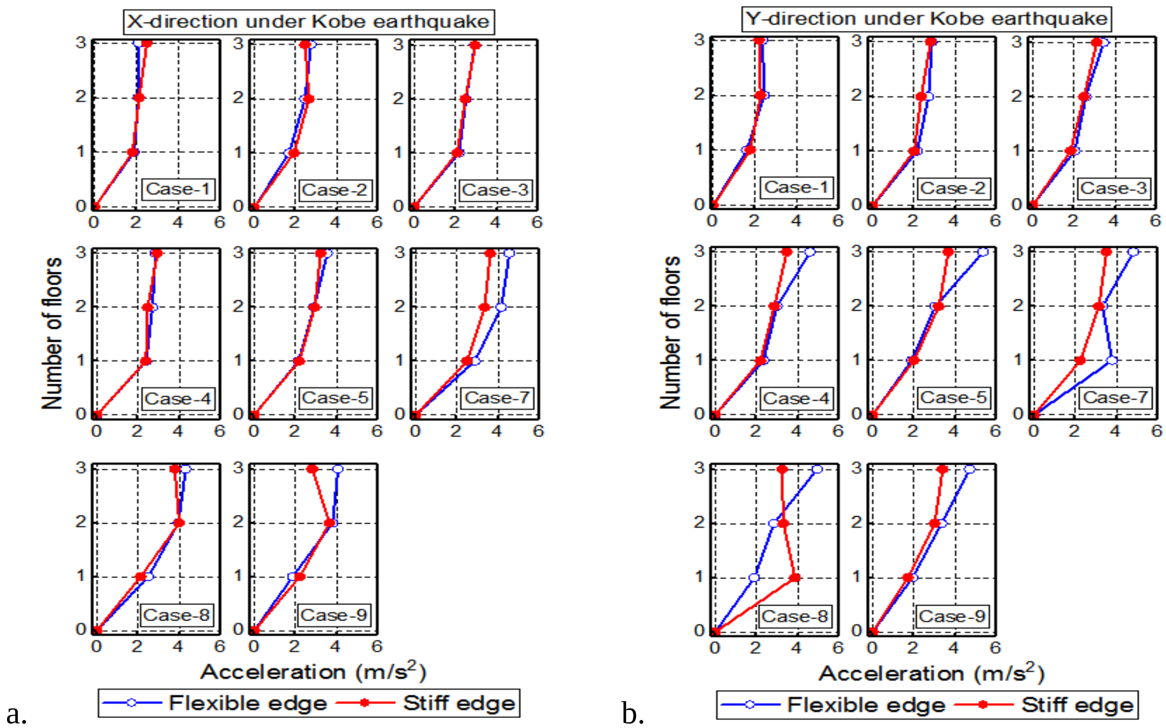


Figure 5: Amplified global acceleration response (a). X-direction response of the TU-TF system under Kobe-earthquake (b). Y-direction response of the TU-TF system under Kobe-earthquake

Moreover, the sudden reduction in the seismic demands for the reference state of the structure can be correlated with local damage response described in the previous section. This endorses the contribution of higher mode effects. Modal characteristics of various TU-TF systems have not been presented here because of space limitation. Moreover, the presented global behaviour implicates higher rotational response at the top roof level under primary component of seismic excitation. It should be noted that because of space limitation, only few representative results are presented here.

7. CONCLUSIONS:

Based on the detailed experimental investigations, following conclusions are established:

- TU-TF systems demonstrate quite abnormal local damage response pattern because of the contribution of high torsional flexibility and higher mode effects.
- Global behavior of the TU-TF system suggests higher floor rotational response at the top roof level, which eventually leads to simultaneous occurrence of compressive and tensile deformations at both flexible and stiff edges of the structure.
- In terms of local damage response, lower order floors of TU-TF systems are more likely to experience seismic damage under intense seismic excitations compared with the top roof level.
- Eccentricity at a floor may likely cause response reduction at the adjacent floor level. However, this is attributed to the presence second mode dominance.
- Floor eccentricities transmit their influence to the adjacent lower/upper floors.

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The careful review and constructive suggestions by the anonymous reviewers are gratefully acknowledged.

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Construction Management

Paper ID: 304

Use of Digital Engineering in Ancillary Civil Design

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Abstract

Word is changing in the digital Era characterized by the technology which increases the speed and breadth of knowledge turnover within the economy and society. There was always a need to develop a digital plan to assist engineering design process. This paper particularly discusses the digital plan implemented on East West Rail re-signalling scheme. East West Rail Phase 2 (EWR2) is progressing from GRIP4 (optioneering) into GRIP5 (detail design), during this cross-over period there is an opportunity to make the Ancillary Civils' team processes more efficient and streamlined. Lessons learnt at GRIP4, the need for an Ancillary Civils Digital Engineering delivery plan become apparent. The purpose of the digital plan is to define the requirements and processes that all Ancillary Civils design and station design teams adhered. Jawaid Malik is Alliance Responsible Engineer, he is delivering the ancillary civil design packages digitally.

Keywords: Digital Plan, Digital Engineering, Ancillary civil design, Platform extensions.

1. INTRODUCTION:

The East West Rail Alliance believed that the integration of digital engineering in to design is essential for the successful delivery of the East West Rail Phase 2 programme.By selecting and applying the latest digital technologies and systems, there was an opportunity to develop, improve and advance the processes required to deliver design work throughout the project lifecycle. This will result in a significant improvement in the quality of information produced by the alliance, leading to better decision making and delivering best value throughout the programme. In turn this will allow the alliance to interrogate information more effectively and to increase reliability and predictability within the project. The successful implementation of the Digital Engineering on the programme is based on a culture change within the Alliance team (as well as construction industry) to apply the process and integration of technology into all functional activities. This requires the enthusiasm, passion and commitment of all Alliance members and suppliers to adopt and apply new ways of working. The proactive adoption of digital engineering will drive innovation, enhance the capabilities and reputation of our team and support the future competitiveness of the Rail sector. This will be achieved through the creation of a high performance collaborative culture within the team and through the utilisation of Digital Engineering systems and processes. This in turn will safeguard the future competitiveness of the rail sector by investing in new and emerging digital technologies with payback periods that exceed the life of the programme. Digital Engineering can be described as a digital representation of a project's physical and functional characteristics. Digital Engineering creates a shared knowledge and information resource for the project, which can aid the decision making, from earliest conception, through design, delivery, handover, commissioning, operation and maintenance and ultimately demolition. The implication of digital element on East West Rail scheme has proved a positive outcome. Civil design packages at GRIP 5 (Detailed Design Stage) are produced based on 3D modelling and use of several digital tools.

2. PROCEDURES:

2.1 Digital Engineering Enablers and Benefits:

Digital Engineering utilises technology, software and processes to improve design, construction, handover, operation and maintenance activities. It identifies, manages and influences the implementation, use and exchange of digital information which will be produced on the EWR2 Programme.
This information can be:

- Graphical (2D, 3D, Spatial Data)
- Non-Graphical (asset information, databases)
- Documentation (output drawings, schedules and reports)

Successful delivery of Digital Engineering requires the engagement and support from all functions to be successful. This document will be a live document which will be updated over the course of the project lifecycle as required. In the event of any apparent conflict between this document and other Standards, the matter shall be referred to the East West Rail Phase 2 Digital Engineering team for clarification.

2.2 Tools:

2.2.1 ProjectWise:

ProjectWise is used to manage the digital information within the scheme. ProjectWise is a suite of engineering project collaboration software from Bentley Systems designed for the architecture, engineering, construction (AEC) industries. It helps project teams to manage, share and distribute engineering project content and review in a single platform. See Figure 1 from ProjectWise workflow.

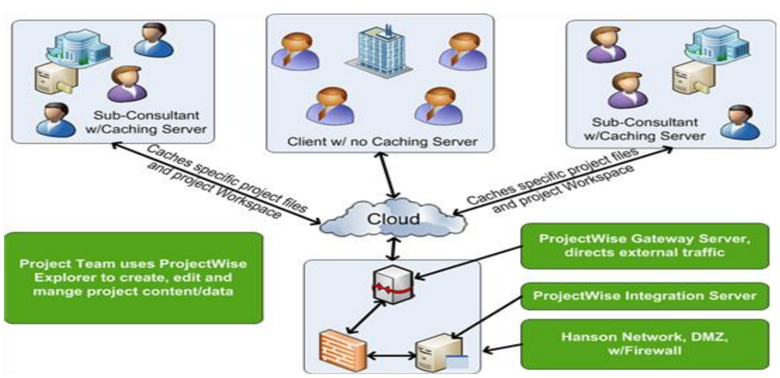

Figure 1: ProjectWise Work flow (Curtsey of Bentley)

2.2.2, Master Information Delivery Plan (MIDP):

The Ancillary Civils Digital Team created a MIDP spreadsheet, capturing all deliverables to be produced in preparation for GRIP 5. The MIDP sets out the deliverables for each asset, e.g. 3-D models, Form F003 documents (Detail design submission document), schedules, specification documents etc.

Each Work Package Manager is tasked to review the MIDP, ensuring that enough deliverables are included, prior to submission to the EIM Team for a final review. The MIDP is a guideline of the Ancillary Civils deliverables and is subject to change. The responsibility of Work Package Managers was set to ensure the MIDP is reviewed periodically and checked against the deliverables saved within ProjectWise. See Figure 2, an extract from level of model defination/Master Input Delivery plan (MIDP).

DELIVERABLE DETAILS (to be completed by Engineering Function)	
Deliverable Title	Title which will be placed within the deliverable
Description	Grouping of deliverables into standard categories e.g. Long Section, Cross Section, Plan, Form 1, Form A
Type	Type of deliverables 3D Models, 2D Models, Reports, Schedule
Native format	Deliverable file format .dgn, .dwg
Exchange format	Exchange format to be delivered .i.dgn, .ifc, .pdf

FILE NAME (to be completed by Engineering Function)	
File	EWR2 BS1192 file naming broken down to individual fields

PROJECT INFORMATION (to be completed by Design Package Managers)	
Risk Classification	Refers to risk profile of the work package the deliverable is associated to, please refer to '7.4 Work Package Classification' of the Digital Engineering Execution Plan P02

Figure 2 - Extract from Level of Model Definition (LOMD) / Master Input Delivery Plan (MIDP)

2.2.3 Model / Drawing Creation:

Quantity, repeatability and scale differ throughout many of the Ancillary Civils assets and the designs can lead to confusion and lost time. This is often as a result of modelling unnecessary detail or modelling with lack of detail that then must be included later. The modelling and drawing process varies for each asset, therefore a clear understanding of the Level of Detail and Level of Information requirements helped the designers and modellers deliver efficiently. Models can come in a variety of forms, different model references will give an indication of what dimension (2D/3D) the model has been drawn in, if the model contains references and if the model is a drawing file (Jones, Daniel, (2019)). An understanding of how different models fit and reference each other, will reduce the time taken in getting a 2D model with no information, through to a fully annotated and rendered model or drawing. Figure 3 below demonstrates the correct process to be followed for completing models.

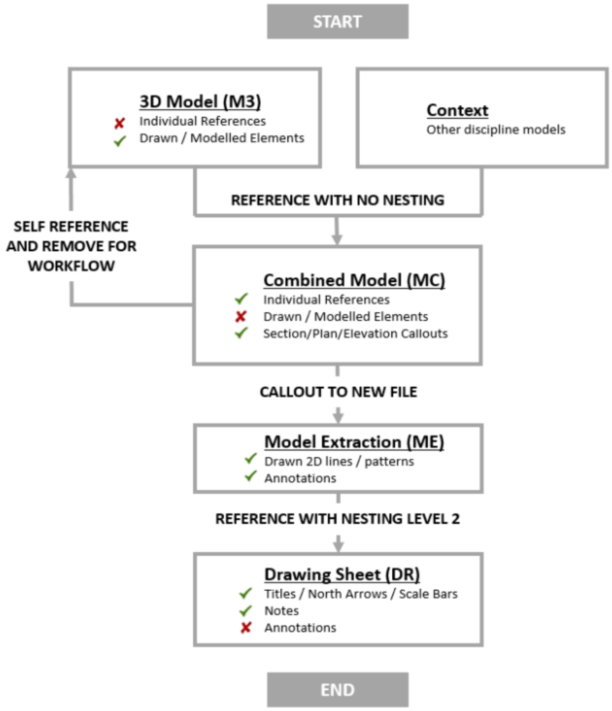

Figure 3 - Model File Composition

2.2.4, LOD/LOI Requirements:

Prior to the commencement of the detailed design, as part of the design start up meeting with the Alliance Engineering Integration Management and Digital Engineering teams, the Level of Detail and Level of Information requirements confirmed and documented by the Works Package Manager. See Figure 4 as an example of LOD for various structures (Alliance DE Team, (2019)).

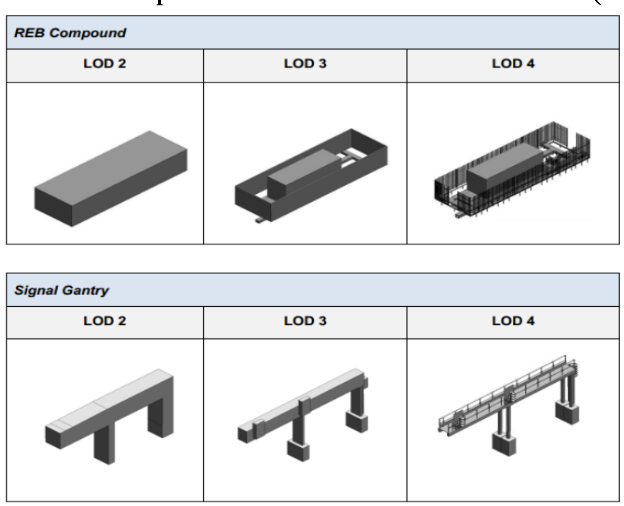

Figure 4 - LOD 04 Requirements

3. RESULTS:

3.1 Master Models

Engineering input required during the detailed design stage to ensure the models are correct. However, to begin the process, GRIP 4 models and information taken from the Technical Notes for Target Cost to initially set up the models. The Master models required updating, as the design progresses. Mark-ups of the Master models are required to provide an audit trail of design / check / review. It was the intention of the Ancillary Civils Digital Team that the Master models will be added to the EWR Alliance project wide ‘catalogue’ of models and could be used on future projects. Simplified models for use in the route sections will be produced following approval of the Master models. An example from the fencing deliverables is shown below in Figure 5.

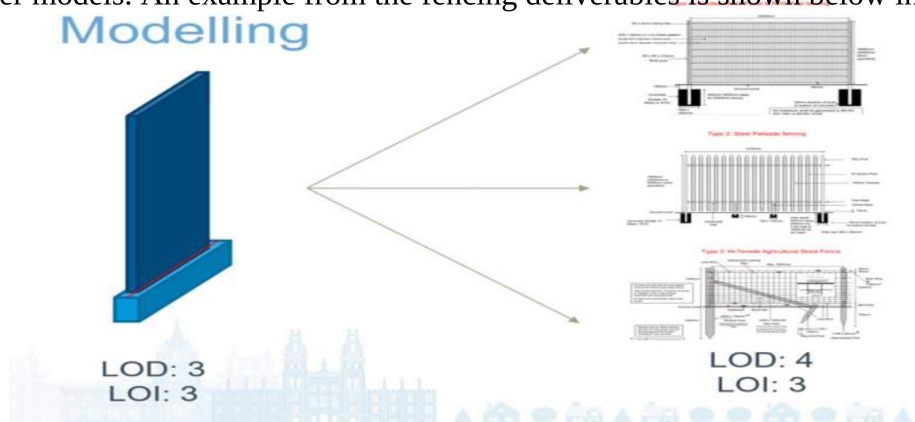

Figure 5 - Examples of the LOD for Fence Panel

3.2 Station 3D Model:

The station 3D modelling produced by integrating 3D models from other disciplines such as earth works, OLE models etc. The clashes of design elements were identified and where required the design was re-produced (Martin, Robert, (2019).). See Figure 6 for examples of Station 3D model.

Figure 6 - Examples of Station 3D model

4. CONCLUSIONS:

Following conclusions can be drawn from the conducted study:

- A concise Digital Model is required to be produced preferably at Feasibility design stage to deliver the scheme digitally.

- 3D models to be produced by each discipline prior to Inter disciplinary Check this includes compatible input data i.e. laser sweep survey etc.
- The survey results acquired from laser sweep method were quick to interpret in 3D model.
- The use Bentley fly over helped in producing 3D detailed flyover video which helped in understanding the present and future constraints.
- A clash analysis to be carried out prior to sharing the models with other parties.
- Design development/Model Update to be shared with other parties periodically.
- Digital models can be used on tablets to for the construction purpose i.e. saving paper print, hence a sustainable process.
- The models help future constructions.
- As build information is readily available with updated construction changes.
- All information saved on ProjectWise is available to be transferred to client in a safe and comprehensive way.

The above approach is more sustainable and safer approach to the design process. The H&S file at the end of the scheme will be helpful for any future refurbishment works etc (Alliance DE Team, (2019)). The digital practice exercised in East West Rail are planned to be used in upcoming schemes. This approach is used widely in aviation and metrology fields. The use of digital practice in Highway and Transportation is the future which can help in achieving a successful, sustainable and safe schemes in future.

ACKNOWLEDGEMENTS:

The authors would like to thank all EWR alliance team members who helped thorough out the research work, particularly East West Rail design teams. The careful review and constructive suggestions by the anonymous reviewers are gratefully acknowledged.

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4D BIM modeling of Insulated Concrete Sandwich Panel Building

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Abstract

Unexpected disadvantages occur in construction industry due to deviation from planned schedule of a project. However, adjustment of these delays and deviation can be made by early assessment of the delays and deviation in the construction phase which is achieved by investigation of any possible delays and deviation that may occur and providing a planned schedule for the client. Adopting Building Information Modeling (BIM) in the construction industry can help in identifying and planning the different challenges faced by construction industry in phase of construction. This paper aims to develop BIM for the early planning, management and scheduling of construction of Concrete insulated sandwich panel building to avoid the wastage of time and resources during the construction phase. 3D model of Concrete insulated sandwich panel building is created in Autodesk Revit and the model is linked with MS Project scheduling using Naviswork Manage. 4D BIM modeling of the model is carried out which helps in identifying delays and reduce sequencing and delays problems.

Keywords: BIM, Resource management, Revit, Naviswork, MS Project, Sequencing, Planning.

1. BACKGROUND AND HISTORY

The demand for housing and industrial building all around the world in construction industry is increasing with time. These increasing demands is affecting the building material and the price to complete these constructions. These demands have given rise to the need of developing new construction methods and products. Researcher have worked continually to develop new products and material which has no effect on the environment and are considered to be sustainable. Sandwich panel is one of the products of the research which is being used in building construction since 1960s (Choi W et al. 2019).

Sandwich panel is a structural product consist of three layers, inner thick insulated Thermal core and two thin structural layers bonded to each side of insulted core. It can be differentiated from other structural product by various characteristic e.g. thermal insulation, light weight, high stiffness, water proof, high strength and ability to be modeled in different shapes and geometry. Due to these characteristics composite sandwich panel are increasingly considered in civil engineering structures. Also, Sandwich panel offer high flexibility, both in term of different

constituent of material and geometry arrangement. To study the behavior of the sandwich panel researcher have performed various experimental and numerical studies.
Yury Solyaev et al. (2019) measured the thermal and mechanical performance of the foam-filled sandwich panel by studying sandwich panel models resting on elastic foundation. They find a new procedure of solution for stability of panel element under compression considering that the foam-filled is reinforced. They introduced Winkler moduli to describe the effect of foundation stiffness on the foam parameters. M Garrido et al. (2019) used Direct MultiSearch method to optimize the sandwich panel system for rehabilitation of building floors. The architectural model of sandwich panel was defined by them using 3 geometric variables, 8 objective functions and 14 variables related to material and concluded that for better performance high density and stiffer material should be used in the panels. H.R Tabatabaiefar et al. (2017) studies experimentally the mechanical properties of the sandwich panel whose inner core is constituent of mixed cement and polystyrene and outer layer is made of thin cement and find out the elastic modulus value and ultimate strength of sandwich panel in saturated and dry conditions and conclude that saturating the sandwich panel will reduce its total strength by 28% of its dry condition strength.
This paper presents the implementation of Building Information Modeling in the Construction of Concrete insulated sandwich panel building. BIM (Building Information Modeling) is a process of digital representation of physical and functional characteristic of a system. Traditional building design were mostly based on 2D drawings and 3D modeling but Building Information Modeling extends this beyond 3rd dimension. 4D BIM modeling of the building is achieved by using three different softwares, Revit, Navisworks and MS Project.

2. BIM TOOLS

The use of BIM had significantly increased in the field of design and construction. To solve different engineering challenges many construction companies especially AEC (architecture, engineering, construction) industry are using Building information modeling (BIM) software. BIM software not only fulfill design requirement but it also helps us in construction management. It is not only limited to 3D modeling but also include 4D (Time),5D(cost) and 6D (Sustainability), 7D (Facility Management/ Operation and Maintenance). Using BIM more work can be carried out with smaller team, greater speed and high quality. BIM provide extensive information about complex structure, detect and solve errors before it occurs on the implementation stage. BIM tools help us to detect possible issues without wasting time and money. Most of the BIM products is built by Autodesk company like Navisworks, Infraworks, Revit etc.
Architectural, Engineering and Construction (AEC) industry has implemented 4D BIM in bunch of mega and small project and concluded positive effect of BIM on the construction. In 2013, BIM was implemented for the sustainable construction of the NeoBuild Innovation center (NIC) in Luxembourg. The building was designed to support research activities related to construction. 4D BIM modeling was implemented for sequencing issues and clash detection in early design stage of the building. HARBOR center is multi use building located in New York. In the construction of this center 4D synchro model was developed in its early design stage. The synchro model was aligned to the weekly schedule of the construction.

3. REVIT FAMILY

Revit is BIM software which has different features for architectural design, construction, structural Engineering and MEP (Mechanical, Electrical and Plumbing). Revit offers libraries of already made object that can be used in Modeling. All the elements that are add to Revit project are created with the help of families. All of these elements have different geometric, physical and thermal properties. There are two types of families use for modeling in Revit. System family and loadable family. system family are the build in family that cannot be created in, deleted from, loaded into, or saved out of the current project these are the predefined families. e.g. walls, roof, floors, stairs etc. loadable families can be created in Revit family builder or loaded in to the project from external source e.g. Doors, Windows, Columns.

4. METHODOLOGY

4.1 Concrete Insulated Sandwich Panel Family Creation

In order to create the section or basic element having properties of concrete insulated sandwich panel, the concept of composite section is used in Revit by creating family for wall and floor which consist of three layers, the middle layer is insulated thick thermal layer of thickness 4 inches and physical and thermal properties of insulation material Polystyrene are assigned to it from the material library. The outer layers are thin structure layers of thickness 2.5 inches each and structure material of Concrete of strength 3.5 ksi is assigned to them as shown in Figure (1).

Layers

EXTERIOR SIDE

	Function	Material	Thickness
1	**Core Boundary**	**Layers Above Wrap**	**0' 0"**
2	Structure [1]	Concrete	0' 2 1/2"
3	Thermal/Air Layer [3]	Polystyrene	0' 4"
4	Structure [1]	Concrete	0' 2 1/2"
5	**Core Boundary**	**Layers Below Wrap**	**0' 0"**

INTERIOR SIDE

Figure1: Concrete Insulated Sandwich Panel Layers

4.2 Modeling in Autodesk Revit

BIM modeling of double story residential building for this paper is carried out in the Autodesk Revit 2017 using Concrete insulated sandwich panel in the walls, slabs and floors of the building. Revit automatically generate 3D view of a building when its 2D plan is drawn. Figure (2) shows 3D view of the building generated by Revit from 2D plane.

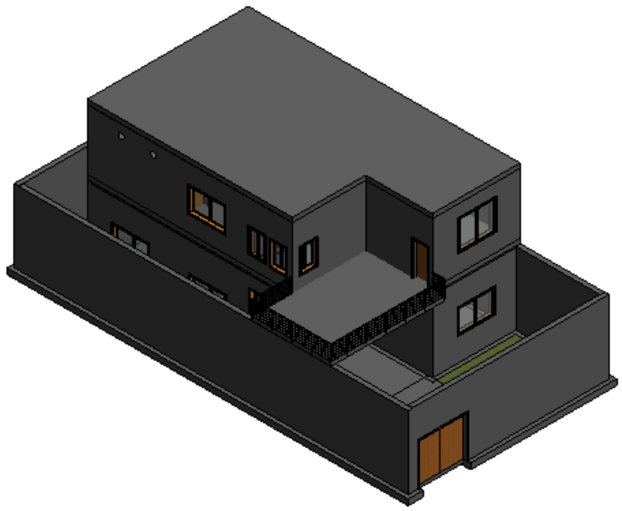

Figure 2: 3D view of Concrete Insulated Sandwich Panel Building

Elevation view of the building is also automatically generated in Revit and using the elevation view the height of different levels can be changed by dragging the level line or by changing its elevation value which will change the height of the objects attached to that specific level. Levels detail i.e. the height of single story and overall building are shown in Figure (3).

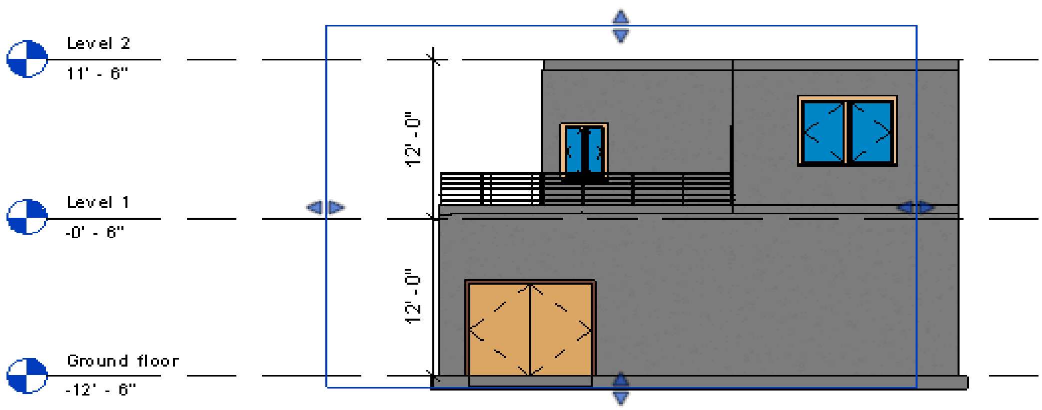

Figure 3: Elevation View of Residential Building

4.3 Linking with MS project

After 3D modeling of the sandwich panel building. Planning and resource management of that building is carried out in MS Project. Estimated duration and various requires resources are assigned to the different tasks and its scheduling is carried out to obtain its Gantt Chart as shown in Figure (4) and (5).

Task Mode	Task Name	Duration	Start	Finish	Predecessors
	Foundation	5 days?	Tue 23/04/1	Mon 29/04/1	
	GF-Sandwich Panel Wall	8 days?	Tue 30/04/19	Thu 09/05/19	2
	Stairs	4 days?	Fri 10/05/19	Wed 15/05/1	3
	GF-Sandwich Panel Slab	2 days?	Thu 16/05/19	Fri 17/05/19	4,3
	1F-Sandwich Panel Wall	9 days?	Mon 20/05/19	Thu 30/05/19	5
	1F-Sandwich Panel Slab	3 days?	Fri 31/05/19	Tue 04/06/19	6
	Floors	4 days?	Mon 20/05/1	Thu 23/05/1	5,3
	Windows	5 days?	Fri 31/05/19	Thu 06/06/1	6,3
	Doors	5 days?	Fri 31/05/19	Thu 06/06/1	6,3
	Boundry Wall	2 days?	Tue 23/04/1	Wed 24/04/1	

Figure 4: Building Tasks in MS Project

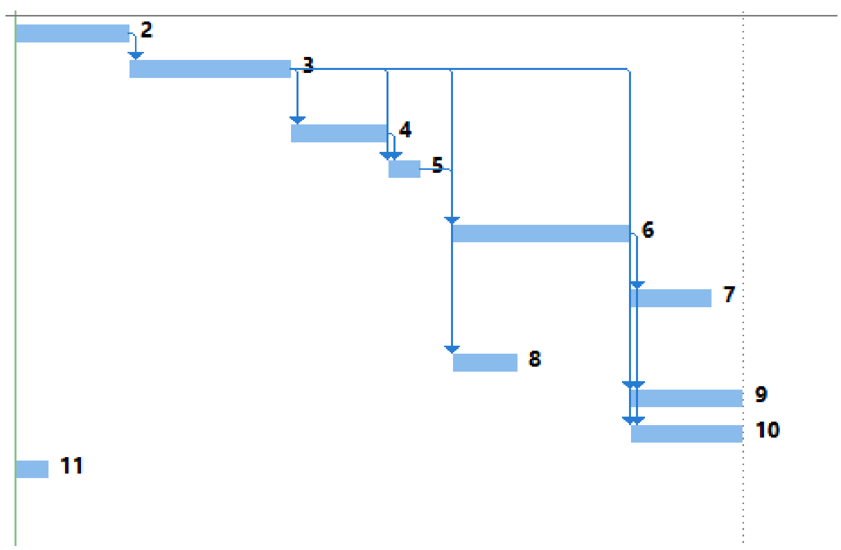

Figure 5: Gantt Chart for Sandwich Panel Building

Navisworks Manage is used as a medium for linking the 3D model and the planning in MS Project. First the 3D model from Revit is inserted to the Naviswork then MS project scheduling is inserted into the Naviswork Manage using Timeliner Feature. Components of the model are attached to its corresponding task which able the model for 4D simulation which give rise to the sequence of building of the model

5. RESULT AND DISCUSSION:

After linking of the model and its planning in MS project. The 4D simulation is run to develop animation which represent the construction work which is in progress or finished by change in visual behavior of the building model. The light green color represents the initial stage of construction of specific building component while the color changes to its material color when the construction of that component is complete i.e. at final stage. These colors can be assigned to every element of the building differently by creating new category of visual behavior for the component.

Figure (6) and (7) shows the initial and final stage of the building components construction by light green color and material own color respectively.

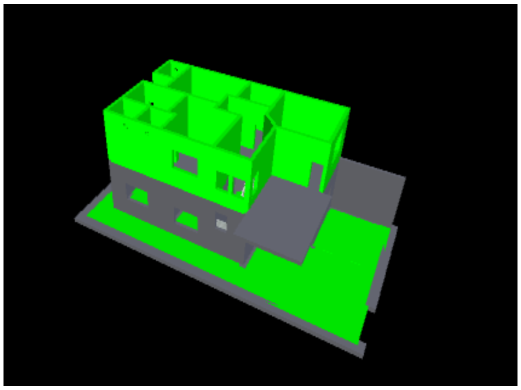

Figure 6: 4D simulation of Ground Floor

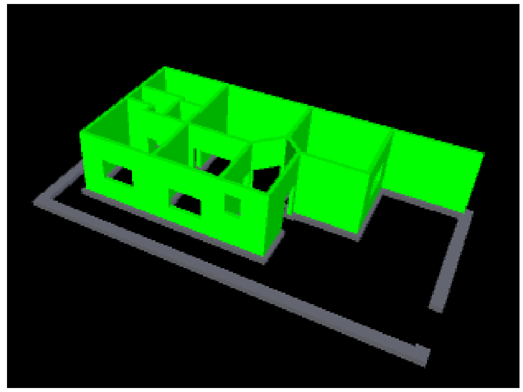

Figure 7: 4D simulation of 1st floor

6. CONCLUSION AND RECOMMENDATION:

Once the model components and the scheduling are linked the advantages of 4D BIM are realized that it helps in Understanding the construction work flow and its sequence. Construction work flow consists of major activities such as Foundation construction, wall construction, slab construction etc. knowing the sequence of the construction reduce the construction time and cost. It is concluded that 4D BIM visualize safety by identifying the shortcoming and by detection of possible delays that may happen during construction phase..

The significant weakness of the Naviswork was identified that its visualization time scale count is one calendar day which is too long time for simple residential building construction.

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Proposed method for Risk Management of small size residential housing construction projects – A Case Study

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Abstract

Risk Management is an organized process used to identify, analyze, and proactively respond to the risks that directly or indirectly affect the project objectives. This process focuses to increase the chances and benefits of positive events and to minimize the likelihood and severity of undesirable events. Risk management has been widely discussed by different researchers. But unfortunately it is not commonly practiced in real life projects. There are various excuses that Construction Managers show to avoid proper risk management. One of the most common excuses is, "RM is just scaremongering". This paper focuses to propose a solution and make it easy to work out Risk Management during project initiation phase. Construction of a residential bangalow has been taken as a case study. A model is developed and explained through a practical example that can be applied to any small scale building construction.

Keywords: Risk, Risk Management, Risk Score, Probability, Impact

1. INTRODUCTION

The execution of construction work is a very complex endeavor. It is linked to numerous potential risks (here, we recognize risk as an event results in negative impact). Risk Management is the integration of all processes needed to identify, analyze, and respond to potential project risks. This paper proposes a model supporting the management of project risk. Unfortunately in most of the developing countries like Pakistan, the construction practices are poorly followed. The factors contributing to the poor management are not well known. To the best of the author knowledge limited research has been done to evaluate the management flaws in construction industry. This paper focuses on the key factors contributing the project failure in the residential construction in developing countries. In developing countries the risk management is an ad-hoc activity. However there is no systematic way of handling risks properly. This paper focuses on the analysis of risks related to the housing construction. The study will provide a reference guideline for all the concerned of the residential sector. It will also be helpful in establishing a basis for decision makers to invest in residential construction sector.

1.1 Risk

Loosemore et.al, (2012) defined risk as a potential event that if occur; will have either a positive or negative impact on the project objectives. The traditional view about risk deals with negativity;

often synonymous to harmful, adverse, hazardous and unwelcome. But some uncertainties may be desirable. Risk is quantified as a magnitude of incapability to accomplish the project objectives within distinct project needs and constraints. Risk consists of three components: (i) the chances of incidence, (ii) the impact of that threaton the project, and (iii) the exposure time- duration in which the risk will impact, if it is not mitigated.

1.2 Probability of Occurrence

The subsequent table defines the probability of occurrence.

Table 1 – Risk Scores for the Probability of Occurrence

Likelihood	Description	Probability	Score
91% - 99%	Almost Certain	> 0.90	5
61% - 90%	"Probably" will occur	0.61-0.90	4
41% - 60%	"Likely" to occur	0.41-0.60	3
11% - 40%	"Unlikely" to occur	0.11-0.40	2
1% - 10%	"Very unlikely" to occur	< 0.05	1

1.3 Risk Impact

Similar to the probability table the risk impact is also divided into five levels. But it is not simple as the probability. Impact depends on various factors such as impact on cost, Schedule and world view. Impact can be of absolute values or some percentages of cost and Schedule. Table 2 defines the risk impact categories and terms. Percent values are used for ease in understanding. These values may vary depending upon the client and the project team risk attitude.

Table 2 – Risk Impact score table against respective parameters.

Impact Description	Parameters	Descriptor	Score
An event that if it occurred, would result in project failure	Schedule delay>2 months Impact on Project Cost> 40%	Extraordinary	5
An event that if it occurred, would cause major cost/ Schedule increases	Schedule delay>1 month Impact on Project Cost> 20%	Major	4
An event that if it occurred, would cause moderate cost/ Schedule increases	Schedule delay>2 weeks Impact on Project Cost> 10%	Moderate	3
An event that if it occurred, would cause minor cost/ Schedule increases	Schedule delay>1 week Impact on Project Cost> 5%	Minor	2
An event that if it occurred, would cause negligible effect on the project objective.	Schedule delay>2 days Impact on Project Cost> 5%	Insignificant	1

**Note: Similar table can be used for the positive risks, but instead of avoiding we wish to exploit them.*

1.4 Risk Score

The magnitude of risk, also known as risk score is the value that can be found out by multiplication of both the probability of and consequences of that particular event. This value is been used to prioritize the risks accordingly. A matrix consisting risk scores is been developed as shown in table-3 and can be used to compare with the risk score. Risks are classified into three categories: Low, Moderate and High risks. Risks having magnitude of less than 10 exclusive are low, between 10 to 16 inclusive are moderate and above 16 are risks.

Table 3 – Risk Score

*		Negligible (1)	Minor (2)	Moderate (3)	Major (4)	Critical (5)
Probability of Occurrence	Very likely to occur (5)	5	10	15	20	25
	Probably will occur (4)	*4*	*8*	*12*	*16*	*20*
	50% chance of occurring (3)	*3*	*6*	*9*	*12*	*15*
	Unlikely (2)	*2*	*4*	*8*	*10*	*10*
	Rare (1)	*1*	*2*	*3*	*4*	*5*

Low, moderate and high risks are illustrated as follows,

- **Low Risks**: It has generally low or negligible threat for cost, no significant schedule or cost effect. Typical management attention would be needed to show ad-hoc response.
- **Moderate Risks:** It might result in raise in expenditure, schedule disturbance, or might affect the performance. There is need of some preliminary studies and plans to overcome these risks.
- **High Risk:** More likely to severely affect the cost, Schedule, or influence the performance. Additional action and high priority management attention will be required to control on high-risk. Proactive action plan is highly recommended.

2. RISK IDENTIFICATION AND ANALYSIS

This section is about the identification of risks, analysis and managing risk. The data process and analysis techniques are described.

2.1. Identification

Risk estimate does not limit its scope to recognize risk and to make a strategy for its response. It represent a ''best estimate'' or a ''best assignment'', depending on the basis of its analysis. For estimating risk the two fundamental parameters cannot be compromised: (a) a probability of that specified event, defined as the event occurrence frequency over a long period. This element is uncertain and is estimated in different ways. In construction management the term subjective probability is commonly used, which is computed by asking some specified questions from a group of experts (Aven, T. 2008). (b) The consequences in terms of benefit and threat of a

potential event. It is defined as the amount of effect on the project objectives especially on cost, time and quality of the project. This effect may be positive called opportunity or negative, known as threat. Brainstorming is an efficient method that uses social interaction for the risk identification process. Using this technique, stakeholders are divided into group of 5-8 people. Each group is briefed about the project. Brainstorming technique of risk identification is being used. These groups are asked to share their experiences they face related to the project. Fishbone diagram is being developed and risks are categorized accordingly. Besides that a risk breakdown structure (RBS) is also being developed cross related with work breakdown structure.
Ishikawa diagrams became famous among the managers in the 1960s.It pioneered excellence in management process, and in the process became one of the most important parts in modern management. It is known as fishbone diagram because of its shape, similar to the side view of a fish skeleton. Figure-1 shows the Ishikawa diagram of identified risks.

2.2. Semi-Quantitative Risk Analysis

The semi quantitative risk analysis is an easier approach of analyzing the risk in which the risk can be very accurately estimated. (Del Bianco, et. al. 2010). In this approach semi-quantitative analysis, the values attributed to different categories of likelihood and consequences reflect the relative magnitude of consequences and likelihood. Although both the percentage and absolute values can be attributed to it. Here in this paper we use percent values to make it easy to understand. Refer to the Section 2 of this document the Risk score is calculated.
Based on the risk score, the risk having the higher value is the “Change in Scope”. In general the risk related to the scope of the project is critical because of the effect on time and schedule is high. Scope sometimes even leads to rework. The second most critical risks are financial risks. Financial risks are directly related to the cost of the project and also effect on schedule in case of risks like project no funded properly.

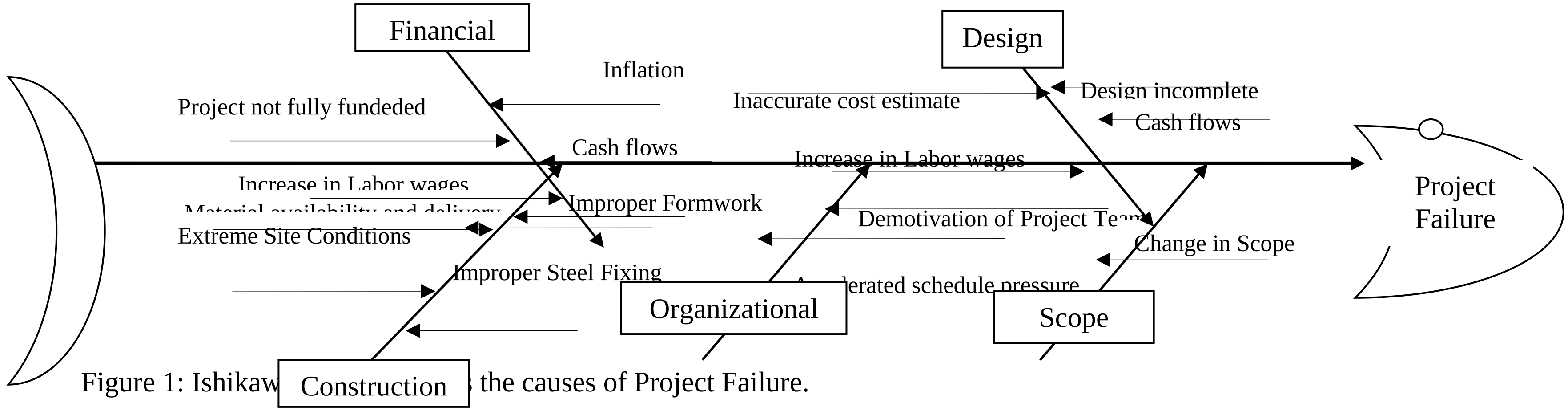

Figure 1: Ishikaw s the causes of Project Failure.

Table 4: Risk Summary after Identification, Analysis and Proposed Strategies for the particular project.

#	Risk Event	Reference	Risk Category	Cause	Effect	Risk Type	Objective	Probability	Impact	Rating	Strategy / Response
R1	Project not fully funded	Nabil and Kartam (2001)	Financial	Stakeholder Interest	Project halted	Threat	Cost	4	3	12	Mitigate
R2	Increase in labour wages	Lo et al., (2006)		Government Polices	Profit Decrease	Threat	Cost	3	2	6	Accept
R3	Inflation	Fang et al., (2004)		Government Polices	Cost Overrun	Threat	Cost	2	3	6	Accept
R4	Cash Flow	Frimpong et al., (2003)		Contract Dispute	Project halted	Threat	Time	2	2	4	Mitigate
R5	Inaccurate cost estimate	Lo et al., (2006)	Design	Un Experienced Staff	Cost over run	Threat	Cost	1	3	3	Accept
R6	Design incomplete	Ayodeji (2006)		Inexperienced Designer	Project Delayed	Threat	Time	2	5	10	Mitigate
R7	Surveys incomplete	El-Sayegh (2008)		Lazy Surveyor	Project Delayed	Threat	Time	2	2	4	Mitigate
R8	Improper Formwork	Fang et al., (2004)	Construction	Unskilled Labour	Project Delayed	Threat	Time	2	2	4	Mitigate
R9	Improper Steel Fixing	Lo et al., (2006)		Complex Design	Cost over run	Threat	Cost	2	4	8	Mitigate
R10	Labour Efficiency	Lo et al., (2006)		New Labour	Quality	Threat	Quality	2	3	6	Mitigate
R11	Extreme Site Conditions	Frimpong et al., (2003)		Heavy Rain Fall	Project Delayed	Threat	Time	2	2	4	Accept
R12	Subcontractor capability	El-Sayegh (2008)		Competition	Quality	Threat	Quality	2	2	4	Mitigate
R13	Material availability	Lo et al., (2006)		Land Slide	Project Delayed	Threat	Time	2	3	6	Mitigate
R14	Demotivation of staff	Nabil and Kartam (2001)	Organizational	wages not paid	Project objective	Threat	Time	2	2	4	Mitigate
R15	Accelerated Schedule	Fang et al., (2004)		Management pressure	Quality affected	Threat	Quality	3	2	6	Mitigate
R16	Change Scope	Lo et al., (2006)	Scope	Inexperienced designer	Project delayed	Threat	Time	3	5	15	Transfer

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3. Conclusion

After studying the past literature and consulting the experts of the field following conclusions are being made,

- Risks may be a threat for the project objective or it can be a good opportunity to exploit. In developing countries risks are commonly considered as threat and that is why people don't invest in it.
- Scope and Financial risks are the top most serious risks. Risks affecting on the scope of the project are the most critical risks affecting budget and schedule of the project.
- Proper risk management leads to the successful completion of the project.

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Challenges in the adoption of Unmanned Aerial Systems (UAS) for health and safety in construction industry of Pakistan

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Abstract

The rate of accidents, fatality & injuries in construction industry are more frequent than any other profession, which causes time delays and cost overruns in the project due to the compensations paid to the injured workers. As there is no authority or organization which monitors health and safety e.g. OSHA in the construction industry of Pakistan, the conditions of health and safety for the well-being of labours are not satisfactory. Adoption of new technology can reduce the number of accidents that occur on construction sites but the rate of adoption of new technology in the construction industry is very slow paced and even slower in area of health and safety. This study provides an insight to the challenges faced by the construction industry of developing country like Pakistan, by promoting innovating technology like unmanned aerial system (UAs) for the purpose of health & safety. Technology such as Unmanned Aerial System (UAS) is being used to identify, report hazards to make zero-accident jobsite & the access the inaccessible areas but this technology is not being adopted in developing countries like Pakistan. A conceptual framework was developed for the adoption of UAS in the construction industry of Pakistan to improve health and safety.

Keywords: Unmanned Aerial Systems, Technology adoption, Health and Safety

1. INTRODUCTION:

Construction industry is an important part of the economy for both developed and developing countries. It contributes about 14% of gross national product and 8% of total employment in the United States (Jochen Teizer 2015) . Developing countries like Pakistan has yet to adopt changes and improvements made in this field. The construction industry of developing countries like Pakistan and India is 2.5-10 times more labour intensive than developed countries (Enno “Ed” Koehn, Rupesh K Kothari et al. 1995).A factory worker in Pakistan is 8 times more likely to killed on job than a factory worker in France. Not only nation but size of economic sector and size of industry also plays a major role in safety against hazards (Benjamin O Alli 2008) .

To abandon old traditions and adopt new innovative approach in the construction industry is a strenuous job but the first step is to comprehend the innovation itself, to gain insight in understanding the hindering factors in adoption (Aletha M Blayse and Karen Manley 2004). As there is much space for innovation in underdeveloped countries like Pakistan. The challenges in adoption of innovation is mainly due to lack of investment in research and development (R&D) and poor communication between industry and academia (Mohammed Fadhil Dulaimi, Florence Y Y. Ling et al. 2002).
This study shows that the UAS can be used as a tool to improve the conditions of health and safety in the construction industry. Using UAS would lead to proactive decision making by the management to reduce time delays and cost overruns.

2. LITERATURE REVIEW:

Health and safety are important aspects for all branches of all parts of life. In case of construction industry, it is more so. All aspects of the works in a construction projects are affected by health and safety (Phil Hughes and Ed Ferrett 2012). (Pakistan Bureau of Statistics (PBS) 2017). The Pakistan's construction industry relies heavily on manual labour and orthodox construction practices which has resulted in poor standard of health and safety (Aftab Hameed Memon, Mohsin Ali Soomro et al. 2017). Hence ensuring proper health and safety measures of the people that contribute to this sector is a major challenge.

2.1 Importance of health and safety:

According to the US bureau of labour statistics (Bureau of Labor Statistics (BLS) 2016) construction and civil works remains the occupation with the highest number of on-job casualties. There is no organization or agency which enforces health and safety in construction industry of Pakistan e.g. OSHA in US. Safety managers are always trying to move towards a safer and zero-accident jobsite (Javier Irizarry, Masoud Gheisari et al. 2012) but still onsite accident and injury is an occupation hazard of the construction industry.

2.2 Labour laws to ensure safety:

In Pakistan, there are no specific legislations or authority that enforces the safety at construction site. This means that each contractor employs safety measure subjectively and generally the cheapest and low-cost measure are taken to provide safety whether they be in terms of personal protective equipment or insurance.

2.3 Cost of accidents:

The cost of accident may just not be a direct cost such as employee compensation damages to building, equipment etc. but also could be indirect or hidden cost such as business loss, time loss, loss of good will, overtime etc. These losses may not be covered by the insurance company.
Studies from Health and Safety Executive (UK) show that indirect or hidden cost can be up to 36 times greater than the direct cost of the accident.(Phil Hughes and Ed Ferrett 2012). Hence proving health and safety in today's dynamic and ever-changing construction projects is not only imperative for the protection of the workers well being but also for the compeletion of projects in scheduled time and the allocated budget.

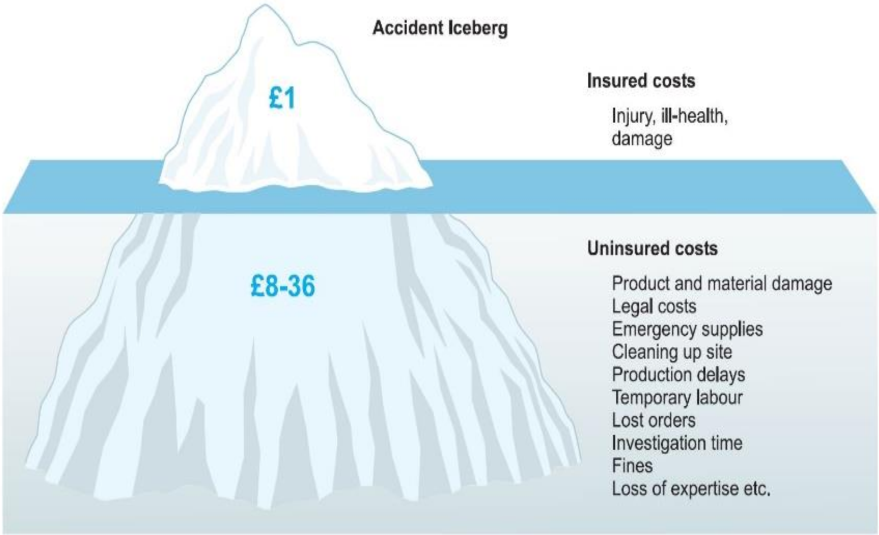

Figure 1 Accident cost iceberg

2.3 Factors compromising health and safety:

(T Michael Toole 2002) identified eight root causes that compromise the health and safety of workers on a construction site:

1. Lack of proper training
2. Safe equipment not provided
3. Unsafe site conditions
4. Poor attitude towards safety
5. Deficient enforcement of safety
6. Unsafe methods or sequencing
7. Not using provided safety equiment
8. Isolated or sudden events

Out of above eight factors, five underlined factors can be monitored by just observation using Unmanned Aerial System (UAS). Hence to improve safety at the construction site, safety managers need a way to efficiently observe and correct practice or action that may compromise the safety of the people working at the jobsite.
(Javier Irizarry, Masoud Gheisari et al. 2012) explains that observation has three main characteristics

1. Being Frequent
2. Directly Observing
3. Direct Interaction with workers

Hence this is done by safety manager, who would inspect the job site on a regular basis and assess the safety of the site by observing and interacting with workers. But in construction industries the project are generally large and complex, this would mean that much more time is taken and the inspection becomes even more complex (Javier Irizarry, Masoud Gheisari et al. 2012). (Atieh Sadat Borhani 2016).

2.4 Use of Unmanned Aerial Systems for health and safety:

Advances in technology can provide great benefits in important areas of construction such as health and safety.(Javier Irizarry, Masoud Gheisari et al. 2012). UAS (Unmanned Aerial System) is the complete package needed for the observation of

construction site. UAS includes the Unmanned Aerial Vehicle (UAV) commonly known as a drone, the ground control system, the controller, camera, GPS and all tool and software required for the working and maintenance of the UAS (ICAO 2018). UAS hold the best prospect to taking a step towards a zero accident and safer construction sites. UAS has low cost, high mobility, safety support, high speed visual assets acquisition and data transfer. UAS is also a key contributor to the automation for its ability to give real time and accurate observation of the construction sites work safety, cost-effectiveness and carbon emission reduction, while there are possible adverse impacts on the basis of current limitations of Unmanned Aerial Systems however, it can be predicted that the usefulness of drones will continue to increase in the future of the construction industry (Yan Li and Chunlu Liu 2018). In developing countries like Pakistan where both construction industry and the academia has failed to form a standard mechanism for monitoring and enforcing safety for the health and wellbeing of all worker at the construction jobsite, new technologies such as UAS should be applied to provide a better mechanism of safety.

The real life application varies from project to project e.g. building project or a road project. In a construction project a skilled safety manager with knowledge of handling drones can stay on site in his office and hover the drone over construction site for inspection. With real time data available to safety manager, he can take proactive decisions and take measures before the occurrence of accident or notify about the presence of hazards to the stakeholders. Every type of project will have its own sets of rules e.g. frequency of visits made by drones, quality of image taken by drone, different point of interests etc.

Thus, it is necessary to identify the barriers that inhibit the adoption of this technology in Pakistan’s construction industry.

3. METHODOLOGY:

The study was done using a mixed research method in which both quantitative and qualitative data were acquired through web-based questionnaire and semi structured interviews. From all the data obtained, its analysis led to development of a framework was developed that can help in adoption of UAS technology in construction industry.

3.1 Survey:

A pilot survey was conducted to ensure the validity of questionnaire by stakeholders. Responses from pilot survey were incorporated in questionnaire.

The online questionnaire for survey consisted of 32 questions and 62 responses were received. The introductory question requested the information about the respondent, the responses showed that majority of our respondent were contractors (40%) followed by clients (24%) and consultants (21%).

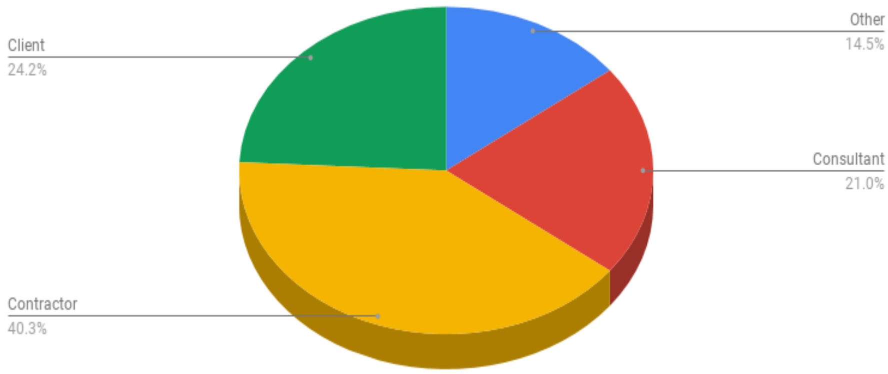

Figure 2 Respondent percentage

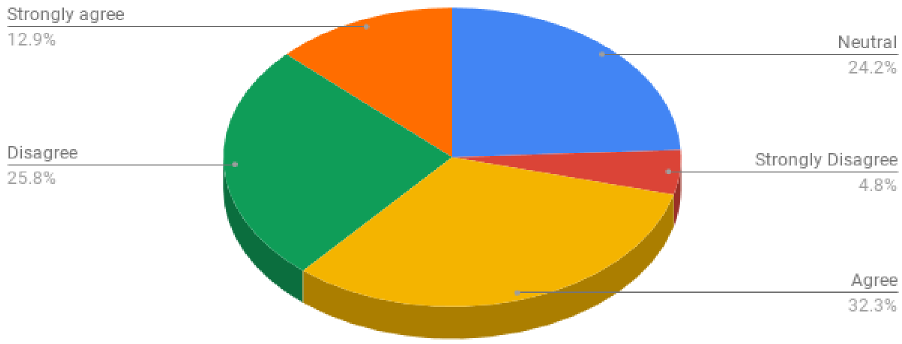

Figure 3 Response to implement of UAS

The second section of the survey was used to collect data about the perception of using UAS for safety and its benefits. 21% of the respondents strongly agreed and 35.5% agreed that UAS can be very beneficial for safety in their work.

The third section of the survey collected data about the advantages that UAS can give to the construction firms. The respondents agreed that using UAS would give a competitive advantage to the firms, the respondents also identified that there was a need for technology training and increase in quantity of skilled labour which could only be brought about by improvement of attitude and commitment from higher management of the construction firms.

The fourth section of the survey collected data about the lack of incentives and motivation from the regulating agencies such as Pakistan Engineering Council. Majority of our respondent affirmed that they do not receive any incentive for applying new methods of safety on the construction sites.

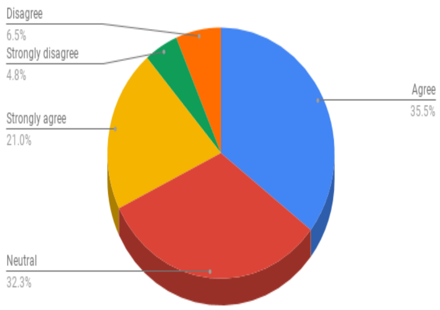

Figure 4: Response to benefits of UAS

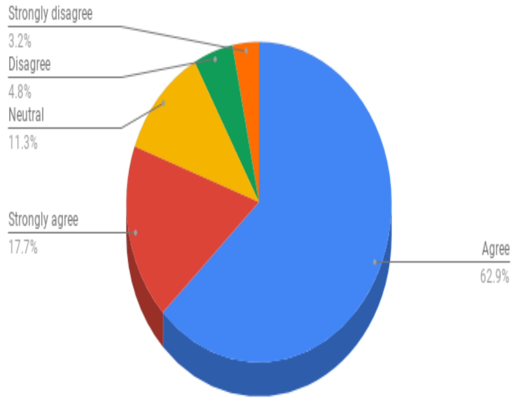

Figure 5: Response to UAS as competitive advantage

The last section of the survey was used to affirm that regulation and acts would be required for the use UAS on construction sites.

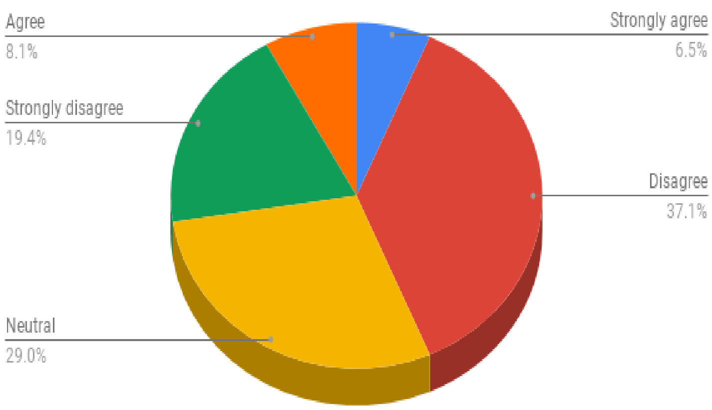

Figure 6: Response regarding incentive to use technology for safety.

4. RESULTS:

Data acquired from the survey was analysed using statistical software SPSS. From the results of statistical analysis factors like awareness, perception, inhibitors of technology, initial investment for technology, culture, attitude of the management, uncertainty in technology, privacy of stakeholders etc. were analysed. More factors from literature review were extracted. Using these factors a cyclic conceptual framework for the adoption of UAS in the Pakistan's construction industry for health and safety was developed. Due to lack of experts opinion or real case study conceptual framework was validated by the opinions of the experts of technology adoption in automation.

4.1 Conceptual framework:

This cyclic framework is developed for technology adoption for health and safety in construction industry of Pakistan. The first step in adoption of any innovative technology such as unmanned aerial system begins with awareness to the stakeholders. Awareness changes the attitude of the stakeholders. If awareness is not created it, due to cyclic process further awareness needs to be created. With enough awareness, when stakeholders are willing to accept the existence of technology and adopt it for their benefit. Proper training and education are required to create and increase the quantity of skilled labours. Initial investment requires motivation from higher authorities in the form of incentive for adoption of technology like tax levy etc. With enough proper rules and SOPs, UAS can be effectively used in construction industry.

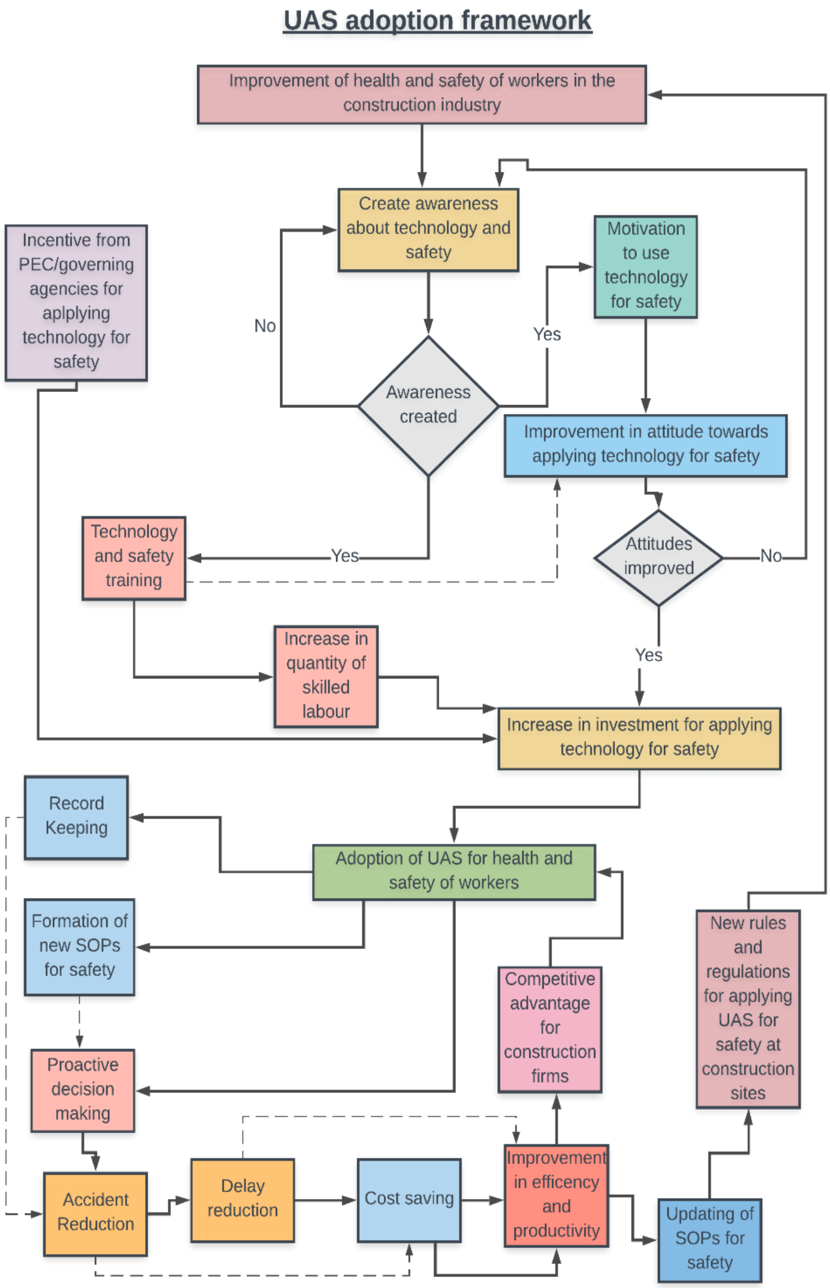

Figure 7: Conceptual framework for adoption of UAS

First cycle of the technology adoption will reveal the unseen factors which may occur during adoption of the technology and can used to improve rules and regulations regarding usage of drone which further improve the health and safety of workers in the construction industry.

5. CONCLUSION:

The research conducted considered areas of occupational health and safety, behaviour and attitudes towards technology adoption for safety in construction industry. Construction industries of underdeveloped countries rely on traditional approaches of providing safety by responding reactively rather than proactive approach and SOPs which is necessary to reduce time delays and increase productivity of labours. Our research suggested, use of UAS for health and safety. Some of the major factors were identified which inhibit the adoption of technology (i.e. UAS) for providing health and safety and prevent accidents and hazards from occurring in the construction industry.
From this research, the main factors identified which inhibiting the adoption of Unmanned Aerial System (UAS) in the construction industry of Pakistan are lack of awareness about the technology and its benefits in applying this technology in the construction industry. The poor attitude towards health and safety of worker in the construction industry also plays a major part.
A conceptual framework was developed for technology adoption of UAS for the purpose of improving health and safety in construction industry of Pakistan. Future research should validate this proposed framework by translating into practice and to modify it according to the new factors that may emerge while applying Unmanned Aerial System (UAS) in construction industry.

6. LIMITATIONS:

Our study had few limitation the first being a small sample size due lack of responses to questionnaire, secondly lack of any case study that was available, third that there are only a handful of technology adoption experts that have the expertise to help improve the framework, fourth there is no reliable data source about the accidents on construction sites.
This study was limited to only a theoretical study due the unavailability of an Unmanned Aerial System (UAS).

ACKNOWLEDGEMENTS:

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Adoption and Awareness of Building Information Modelling (BIM) in Pakistan

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Abstract

During 2016-2017, a higher growth rate of 9.05 % is achieved by Pakistan construction industry. It is observed that the construction industry has a poor reputation of accomplishing the projects in required time, cost, and quality. It is because of using the traditional management techniques that can be easily minimized with the implementation of new technologies like Building Information Modeling (BIM). In this study, a questionnaire survey is performed, which helps in finding the present state of BIM in Pakistan and also highlights the barriers in the successful adoption of BIM in Pakistan. Out of 105 responses received, 68 % of Architecture, Engineering, and Construction (AEC) professionals are aware of BIM and among them, only 12% had utilized BIM in their projects, which is a very low percentage and has to be increased. This research helps in the awareness of BIM in Pakistan as 83 % of the respondents responded that it helps us in clearing our minds about BIM.

Key Words: Building Information Modeling (BIM), 2D CAD, BIM in Pakistan, Barriers in Adopting BIM in Pakistan.

1. INTRODUCTION

Construction industry is an important economic development sector in Pakistan. It produces the largest ratio of employment and also plays a vital role in economic development (Maqsoom et al., 2013). The construction industry has an annual growth rate of 9.05% during the period of 2016-2017 (Pakistan Economic Survey 2017). The construction Industry in Pakistan, like other developing countries, has a poor history in terms of completing projects in required time, cost and quality (Gardezi et al., 2014, Maqsoom and Charoenngam, 2014).Moreover, many projects fail to accomplish their objectives due to cost and time overruns; hence, the Pakistan construction industry is unable to deliver as per government plan of progressive development (Maqsoom and Charoenngam, 2014).So through usage of modern tool like BIM to accomplish the project successfully. In this study, problems are highlighted which hinders for successful completion of a project; a communication gap within an organization, lack of government policies, lack of awareness among the stakeholders etc.

The objective of this research is to scrutinize BIM applications in designing, coordinating, managing and execution of construction projects and to evaluate the potential of BIM for its use in Pakistan construction industries.

2. BUILDING INFORMATION MODELING (BIM)

2D computer aided design (CAD) was first introduced by the Autodesk company in December 1982. It helps in better documentation, accurate drafting and save time. Due to easiness of using AutoCAD, the architects shifted towards AutoCAD (Yan and Demian, 2008).

The concept of BIM was first introduced by Chuck Eastman and Robert Aish in 1970. For more than twenty years, this system of modeling was used under different names such as virtual building, intelligent object and product model (Ozorhon and Karahan, 2016). This edge of BIM over traditional CAD model is that the model generated by BIM can be effectively used for planning, design, construction, and operation of the facility (Azhar, 2011). Architecture, Engineering, and Construction (AEC) professionals prefer BIM for effective and efficient design and construction management (Charlesraj, 2014). BIM is not only a software but a complete process as well (Azhar, 2011). BIM as an official term was used in 1992 and later on in 2000 software called "Revit Software". After that the same software was sold to Autodesk and they brought many new changes in 2004. Currently, that software is known as Autodesk Revit. BIM has brought revolutionary change in the field of building design to some high level of extent (Haron et al., 2009).

BIM can effectively help in minimizing these problems. BIM may be defined as the process of creation and implementation of a computer based model to integrate the planning stage, design stage, the execution phase and operation of a project (Masood et al., 2014). Autodesk360 cloud (A360) provides a central workspace in the cloud for project content and the people working on a project (members of a project).A360 cloud is a collaborative tool through which all the team members are connected and they share their designs etc. to A360 drive as shown in figure 1 . Members of the project team can easily access and view all the data formats (DWG, PDF, DWT, RVT etc.) shared in the drive from any device.

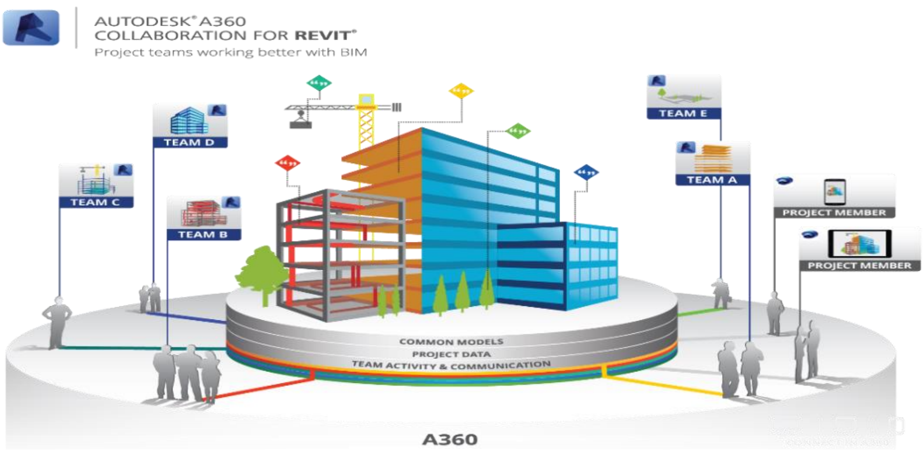


Figure 1:A360 Cloud

3. DIFFERENCE BETWEEN BIM AND 2D CAD

BIM and 2D CAD differ from each other in such a way that 2D CAD explains a building by isolated 2D views such as plans, sections and elevations. If changes are to be made, changes in all views must be done, a tedious process which is more prone to errors and leads to improper documentation. Moreover, 2D drawings show data as graphical entities only, such as lines, arcs and circles, as compare to the intelligent background semantic of BIM models, in which objects are defined in terms of building elements and systems such as spaces, walls, beams and columns. A BIM model include all data needed, which is related to the structure, including its aesthetics, functional properties and information related to project life cycle, in addition of “smart objects”. For example, in BIM any heating unit also store information related to its supplier, operation and maintenance procedures, flow rates and clearance requirements (Innovation, 2007). The processes of BIM helps in development and use of the computer generated n-dimensional (n-D) models to simulate the planning, design, construction and operation of a facility. It helps AEC professionals to overlook what is to be built on the site and to identify effective design, construction or operational clashes and any issues which would disturb the project execution as planned (Azhar, 2011).

4. BIM IMPLEMENTATION GLOBALLY

A wide research is ongoing to realize the issues for the implementation of BIM in the construction industry and to communicate the benefits of BIM implementation to the construction industry. It is reported that globally BIM implementation was 26% in 2007 and over doubled in less than a decade, reaching 57% in 2016 (Bhatti et al., 2018).
The governments of the US, UK, Germany, Canada and other developed countries have made the BIM implementation compulsory for their industries (Wang, 2014). A research shows that the lower percentage of BIM adoption in Malaysia is due to the lack of national BIM policy, poor holistic readiness , software integration, competition and unwillingness to share knowledge amongst industries (Bin Zakaria et al., 2013).
BIM implementation in the top ten largest international construction markets (working on level 2 of BIM) are as shown in Table 1.

Table 1: BIM Adoption percentages in the world (Bin Zakaria et al., 2013).

Countries	Adoption percentages	Countries	Adoption percentages
Germany	90%	France	65%
United States	72%	United Kingdom	54%
Canada	67%	India	22%

5. BIM IN PAKISTAN

When it comes to development in infrastructure, Pakistan lags far behind due to illicit practices in the construction sector (Maqsoom et al., 2013). The critical risks in Pakistan’s construction industry include poor quality, lack of planning, alteration in scope and design of a project, corruption, claims and disputes, inadequate design, and quantity changes (Shabbar et al., 2017). Similarly, one the main reason for major delays and cost overruns have been reported as mistakes and error in design, variations, delays in preparation and approval of drawings, conflicts between drawings and specifications, unrealistic time and cost estimate, improper planning, poor coordination between project stakeholders, and poor contract management (Gardezi et al., 2014). In Pakistan only 11% of related industry has implemented BIM and only to generate 3D models which is very limited part of BIM (Bhatti et al., 2018) and has to be increased. Measurements should be done for the adoption of BIM in Pakistan also national policies should be made to control the problems facing by the construction industry in Pakistan (Ali et al.).

6. RESULTS AND DISCUSSIONS

6.1. Survey

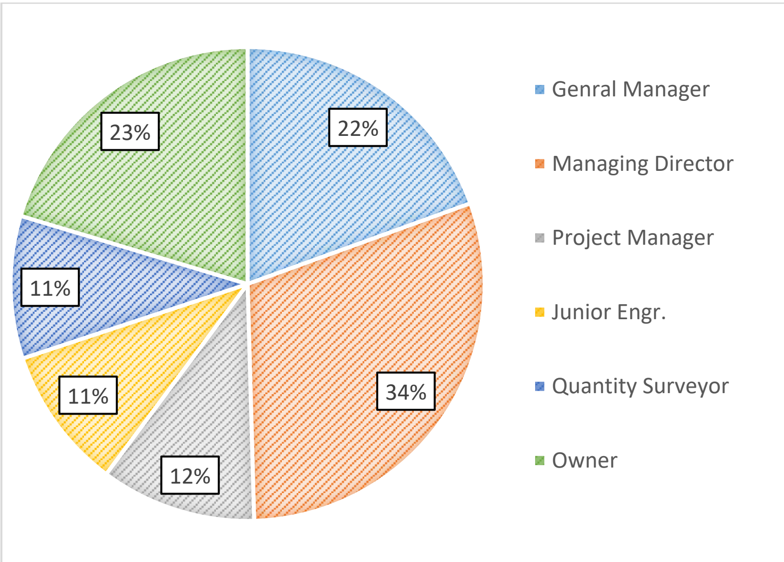
A questionnaire document is any written set of questions, while a survey is both the set of questions and the process of gathering, combining, and examining the responses from those questions.

After going through the literature review, an online questionnaire was developed through google documents and shared for collecting information regarding the BIM implementation in Pakistan's construction industry. It consists of three sections, section A is a demographic section and includes the demographic profile of respondents. Section B consists of certain questions about the implementation of BIM. Whereas in section C questions about the barriers in the successful implementation of BIM was asked.

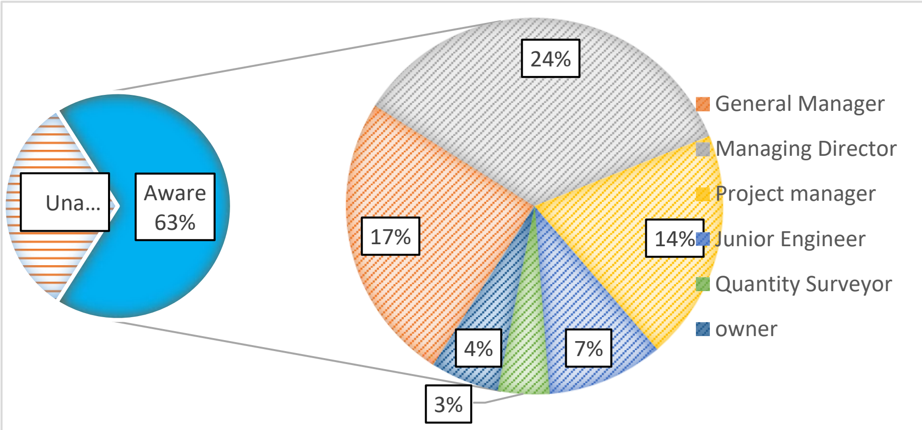
The questionnaire was distributed via email amongst the contracting companies, consultants, architectural firms and engineers. The email addresses of the firms was acquired from the Pakistan Engineering Council (PEC) website and through personal contacts.

6.1.1 Demographic section

Figure 2a shows the respondent profiles and Figure 2b shows their awareness about BIM. From the survey performed,105 responses was received and analysis were done which shows that 63% of the professionals in the construction industry of Pakistan are aware of BIM and the remaining 37 % are still unaware of BIM and its applications.



(a)

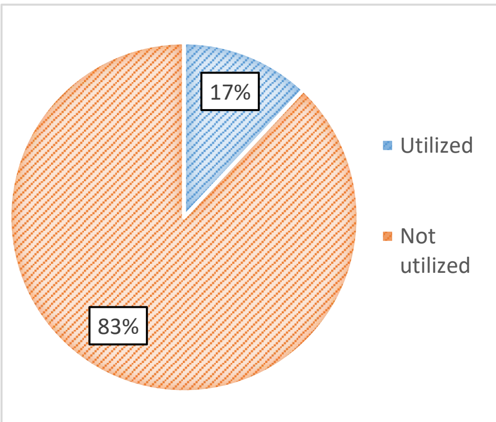


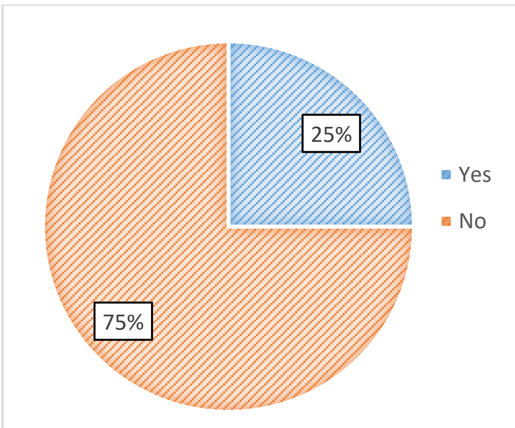
(b)

Figure 2: Demographic profile of Respondents; a) Job Position, b) Awareness about BIM

6.1.2 BIM Implementation:

In Figure 3a it is shown that among 63 % of the respondents who were aware of BIM only 17% of them had utilized BIM in their projects, which is a very low percentage. Whereas in Figure 3b, it is shown that only 25% of organizations have BIM policies and protocols. A question was asked “Do you forecast companies will be left behind if they don’t adopt BIM quickly enough” in which 82 % of the respondents thinks that companies will be left behind if they do not adopt BIM in their projects quickly. So, according to this survey results, the companies should adopt BIM quickly for their survival in the upcoming decades.





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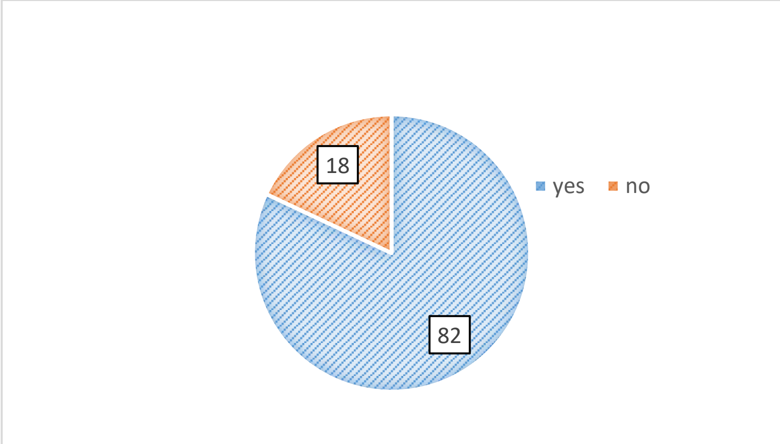

c.

Figure 3: BIM Implementation, a) Utilization of BIM in projects, b) The availability of BIM Policies in organization, c) Prediction about BIM adoption in near future.

7. BARRIERS/ISSUES RELATED TO ADOPTING BIM IN PAKISTAN

Table 2 shows the barriers and their ratings in the successful adoption of BIM in Pakistan's construction industry. Respondents are asked about different barriers of BIM adoption to rate it on a Likert scale from 0-5.The responses are then analyzed and the average ratings for each barrier is obtained.

Table 2: Barriers in adoption of BIM

Barriers	Rating
Lack of Government regulation about BIM	78%
Lack of seminars on new technologies like BIM by firms	77%
BIM adoption requires organizational restructure	72%
Lack of BIM professionals	70%
Lack of ability to manage projects through BIM	64%
Searches of firm for a better consultant coordination process	62%
Initiatives encouragements of firm in developing new solutions	62%
Top management don't support change	62%
Communication gap in an organization	60 %
High initial cost of BIM	52%
Current practices are serving good	35 %
Firms provides software training to their employees	55%

8. AWARENESS OF BIM:

For the awareness of BIM amongst the AEC professionals, a video about BIM and its applications was shared and a question was asked whether the video is helpful in clearing their minds about BIM or not. 91% of the respondents responded that the video was helpful in clearing their minds about BIM.

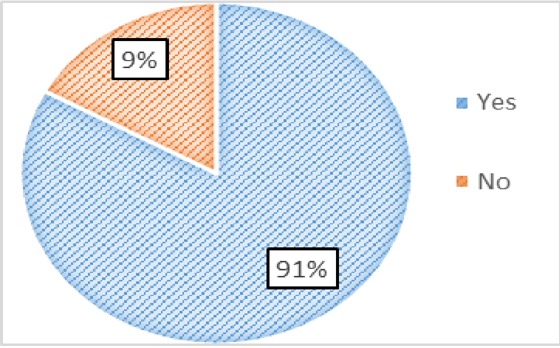

Figure 4: Was this video helpful or not?

9. CONCLUSION:

Construction industry of Pakistan is facing problems of delays and miscommunications in all stages of the project. The construction industry are not yet convinced to adopt the modern management approaches such as BIM. Through this case study, the present state and barriers of BIM implementation are highlighted. The study recommends BIM to be beneficial for the construction sector. The topmost barriers of BIM adoption in Pakistan construction industry are identified as the present structure of organizations and lack of government regulation about BIM. For the awareness of BIM in Pakistan's construction industry a video was shared with the AEC professionals and in the near future seminars will be arranged to show the advantages of BIM with the help of practical work.

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To Investigate Utility of Building Information Modelling (BIM) to Improve Productivity of the Construction Industry

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Abstract

Productivity of construction industry is lower in contrast to other industries. The industry is also lacking behind to adopt technology as equivalent to other industries. Building Information Modelling (BIM) is a revolutionary development in the Architecture, Engineering and Construction (AEC) industry. There are numerous factors that contribute to reduce efficiency of the industry. This study aims to investigate the potential of BIM to solve issues that can reduce productivity of the construction industry. A questionnaire survey is conducted for this purpose from the practitioners of the construction industry of Pakistan. The results of 171 valid responses greatly support the hypothesis that BIM has the potential to solve issues that can cause loss of productivity of the sector. Linear regression analysis is also conducted. The analysis shows that collaboration, elimination of rework and conflicts are significant factors. This research will be beneficial for academics, as well as, industry to enhance productivity of the construction industry by identifying factors and mitigating them with the help of BIM.

Keywords: Construction Industry, Productivity, BIM, AEC

1. INTRODUCTION:

According to Global Construction 2030 (2015) the extent of construction production will increase by 85% to $15.5 trillion globally by 2030 due to China, India and USA. These countries are leading the way and rate for 57% of over-all global growth. Construction sector is the second largest contributor to Pakistan's economy. Around 30-35% of employment is directly or indirectly linked to the construction industry. The industry has played a significant function in creating jobs and facilitating the increase of the economy (Rizwan *et al.*, 2008). According to estimates published by the Economic Survey of Pakistan (2018), the industry grew via 9.13% in the past year and contributed 2.82% to a country’s Gross Domestic Product (GDP). Regardless of its remarkable contribution to the GDP, Pakistan’s development industry is one of the most unnoticed and unorganized sectors.

Productivity in the construction industry is usually defined as output per labor hour. As labor comprises a significant proportion of a construction value and the quantity of labor hours in accomplishing a task in the industry is greater inclined to the influence of administration than are materials, this productivity rate is frequently referred to as 'labor productivity' (David, 1994). Literature shows that a number of factors affect productivity, however, there are unidentified factors that need to be studied even in advanced nations (Makulsawatudom and Emsley, 2002).

Certain policies to improve productivity of the construction industry are not usually comparable in every country. It is found that there are some special factors that affect labor productivity and are bracketed together according to their attributes such as design, execution plan, material, equipment, labor, fitness and safety, management, work-time, tasks, collegiality and coordination, owner/consultant, and some external factors (Arditi, 2005).

BIM represents physical and functional chrematistics of a facility digitally. It's a revolutionary development in the Architecture, Engineering, and Construction (AEC) industries. It simulates a construction project virtually at the outset of a project (Eastman *et al.*, 2008). Building Information Model is an intelligent parametric virtual representation of a building that provides all the necessary data required by various users for analysis and to generate information that can be utilized in different decision-making processes (AGC, 2005 in Azhar *et al.*, 2008, p.436).

Many studies have been conducted on the issue of productivity in the construction industry, such as, Latham Report and Egan Report. However, BIM wasn't fully developed or adopted back then. The present study is focused to find out if the latest ICT development i.e. BIM can be utilized to improve productivity of the construction industry.

2. RESEARCH METHODOLOGY:

Literature review is conducted regarding productivity of the construction industry, factors that can reduce productivity, BIM and its potential benefits. A questionnaire survey is prepared to conduct survey from the industry to find out if BIM can improve productivity of the construction industry. A total of 180 responses were obtained but 171 responses were valid. The results of the surveys are analyzed and shown graphically. Additionally, linear regression is also employed to find out significance contribution of factors. The methodology adopted is shown below:

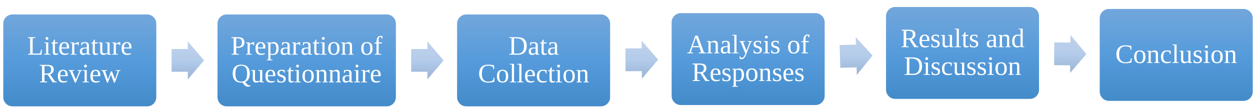

Figure 1: Steps of Methodology

3. RESULTS AND DISCUSSION:

The results of the survey are given in the tabular form. The responses of questionnaire from the industry shows a spectrum between strong agreement and strong disagreement as shown in the table below. However, percentages of agreement for each question is more than percentage of strong disagreement.

RESPONSES	QUESTIONS				
	1	2	3	4	5
STRONGLY AGREE	29%	20%	27%	31%	31%
STRONGLY DISAGREE	2%	2%	5%	1%	0%

3.1 Summary of SPSS Analysis:

- **Predictors:** (Constants), Digitalization, Collaboration, Rework, Conflicts.
- **Dependent Variable:** Intelligent Parametric 3D Model.

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	0.599	0.358	0.343	0.906

MODEL	UNSTANDARDIZED COEFFICIENTS		STANDARDIZED COEFFICIENTS	T	SIG.
	B	STD. ERROR			
(Constant)	0.393	0.537		0.732	0.465
(Collaboration)	-0.252	0.121	-0.175	-2.072	0.040
(Rework)	0.634	0.093	0.459	6.808	0.000
(Conflicts)	0.490	0.086	0.382	5.669	0.000
(Digitalizing)	-0.026	0.117	-0.017	-0.224	0.823

$$[Y = 0.393 – 0.252*B_1 + 0.634*B_2 + 0.490*B_3 – 0.026*B_5]$$

B_1 (improving co-ordination using shared model) is a significant factor because lack of co-ordination can result in poor design and conflicts in the design that would lead to rework, which means loss of man-hours spent, as well as, materials used. Additionally, access to updated information would be readily available through a shared BIM model. B_2 (avoiding rework by detecting clashes in design phase) is a significant factor because rework results in cost and time overruns. Since BIM uses parametric 3D model technology, it can detect clashes during design phase. B_3 (conflicts) is a significant factor since by using BIM designing and constructing a facility would be more efficient and information is available through shared model. As all information can be integrated into a single model so there is no or less ambiguity for a consultant, a contactor or a client and disputes can be easily tackled. B_5 (Insignificant - Improving design by constructing/deconstructing project digitally) is insignificant since number of factors considered for this study are few therefore this might be the main reason of insignificancy of this point, as well as, R square value. Its significance and value of R square might increase in future by adding more factors to the survey.

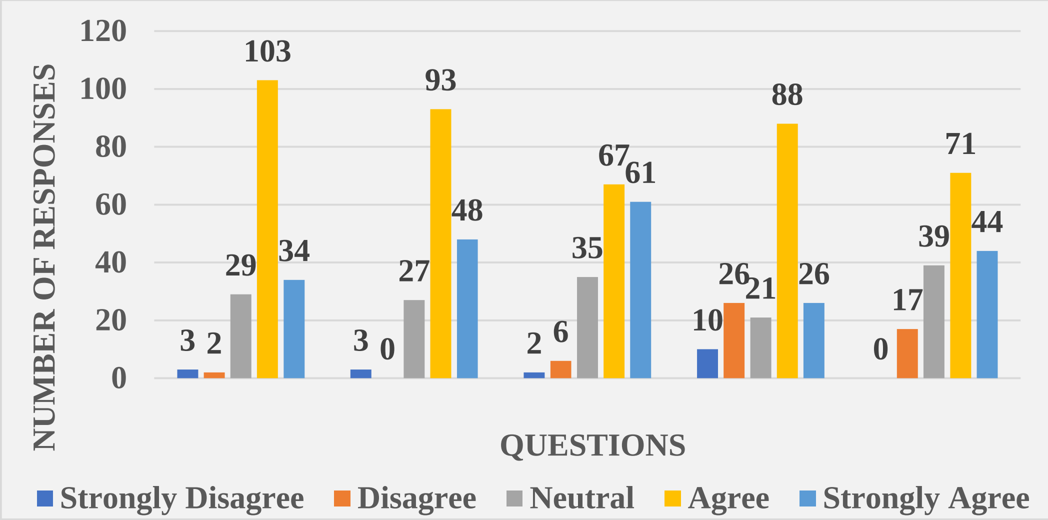

Figure 2: Survey Results

Figure 2 shows an overview of the responses of the respondents. It can be seen that a major portion of the responses show greater acceptance. Some respondents stayed neutral probably due to lack of information about the utility of BIM at the moment. However, a small portion of respondents did not agree that BIM can increase productivity in some cases as evident from the above responses. When asked whether BIM can improve design of a facility, a great number of respondents 103 (60%) agreed while only 3 respondents (2%) strongly disagreed to this statement. Similarly, 93 respondents (54%) agreed that collaboration can be improved by incorporating BIM in construction projects while 3 (2%) of them strongly disagreed. According to 67 (39%) respondents, clashes could be minimized with the help of BIM but 20% (35 respondents) stayed neutral and 6 of them (4%) disagreed. When enquired if productivity can be increased using 3D model instead of error prone 2D drawings, 88 respondents (51%) agreed while 26 of them (15%) disapproved. And finally, when respondents were queried about how utility of BIM can mitigate conflicts, a large portion agreed i.e. 42% (71 respondents) while 17 (10%) respondents disapproved.
Using a 3D model, ambiguity in the design can be minimized. In a case study carried out by Darius Migilinskasa (2013) accessibility to use single prepared 3D frame model for visualization and structure analysis, it saved almost 20% of time for inspecting and redrawing a 2D drawing with errors correction when deviations occur. The 3D model was used for approximation of the bill of quantities for the work packages, which made the negotiation procedure easier with subcontractors and contractors (Darius Migilinskasa *et al.*, 2013). The BIM 3D technology is very helpful throughout the design as a key tool for construction drawings & coordination during the project as it permits all the members of construction project to stay in touch with the design at once. Receiving the maximum aids from the BIM technology is directly associated to the ability to maximize collaboration in project. No matter who is leading, all key members have the access to a shared data base which in turn offers most benefits for the project team in terms of collaboration. It also empowers to classify clashes and disputes which in turn prevent reworks during the project (Popov *et al.*, 2010).

4. CONCLUSION:

The results of the survey show that a wider majority of practitioners of the construction industry of Pakistan believe that BIM has potential to improve productivity of the construction industry. Most of the respondents agree with potential strengths of BIM, such as, collaboration, elimination of rework and conflicts, to solve issues that can lead to loss of productivity. Collaboration among project participants is significant for productivity, which is also the prime objective of BIM. Furthermore, lack of rework due to clarity in scope of work in the design phase would yield better progress during execution phase of a project. Moreover, mitigation of conflicts because of co-ordination plays a significant in improving productivity.

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To Investigate Potential of Building Information Modelling (BIM) to Mitigate Disputes in the Construction Industry

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Abstract

Disputes are part and parcel of construction industry due to the complex nature of the industry, as well as, the uniqueness of construction projects. These disputes not only damage the industry monetarily, but also waste time and strain working relationships. There are alternative dispute resolution techniques to solve disputes rather than to avoid disputes at the first place. Additionally, the industry is also slow in adopting technology when compared with other sectors. Building Information Modelling (BIM) is recognised as a “revolutionary development” and “game-changer” for Architecture, Engineering and Construction (AEC) industry. The purpose of the present study is to investigate if BIM has potential to mitigate disputes in the construction industry. A questionnaire survey is conducted from the practitioners of the construction industry of Pakistan; 190 valid responses were collected and analysed. The majority of the respondents agree with the potential benefits of BIM to mitigate disputes. Additionally, linear regression analysis also showed that co-ordination among project participants, collaboration to clear scope of work at the outset of the project and availability of accurate design drawings would mitigate disputes among the stakeholders. The present study would yield benefits to academia for further investigation, as well as, to the industry to avoid potential causes of disputes by utilizing BIM.

Keywords: Disputes, BIM, construction industry, AEC

1. INTRODUCTION:

The construction industry makes the world go around. World Economic Forum (2016) states that the construction industry greatly influences economy, society and environment. The industry, on average, contributes 6% in global Gross Domestic Product (GDP); to be specific, 5% in developed economies and more than 8% in developing economies. Additionally, it is expected that the construction industry would be generating a revenue of $15 trillion by 2025. Furthermore, constructed facilities accounts for 25-40% of carbon emissions and employs greater than 100 million people throughout the globe. According to the International Monetary Fund (2014) if developed countries spend additional 1% of their GDP on the construction of infrastructure, they can achieve 1.5% increase in GDP in a period of 04-years. Hence, the significance of the industry for the inhabitants of the Earth is clearly profound.

The industry is one of its kind as compared to any other industry in the world because of its unique nature: projectized, fragmented, uncertain, work on site, and influences such as political, economic, social, technical, legal and environmental. These factors increase complexity for the construction industry, which is difficult and/or impossible to deal with sometimes, thus raise concerns regarding the performance of the industry. In addition to these complexities, the unique nature of projects also plays its role, which results in conflicts, disputes and delays in construction projects (Pena-Mora, 2003). These issues occur due to the intrinsic complex nature of the industry and are crucial for the industry, as well as, the wider economy as evident from the statement: "In the United States alone, $60 billion are spent every year on lawsuits, of which the construction industry accounts for nearly $5 billion. The fact that these construction litigation expenditures have increased at an average rate of 10% per year for the past ten years…" (Pena-Mora, 2003, pg. V).

Research	Sources of dispute	
Blake Dawson Waldron (2006)	1. Changes of conditions	5. Law
	2. Interpretation	6. Access to construction site
	3. Workplace conditions	7. Access to materials
	4. Communications	
Cheung & Yiu (2006)	1. Management	3. People
	2. Communication	4. Contract documents
Yiu & Cheung (2007)	1. Delay	2. Unrealistic expectations
Killian (2003)	1. Change Orders	3. Pre-Construction
	2. Pre-Award Design	4. Quality Assurance
Mitropoulos & Howell (2001)	1. Uncertainty	3. Opportunistic behavior
	2. Contractual problems	
Kumaraswamy (1997)	1. Changes of conditions	6. Management
	2. Changes of scope	7. Delay
	3. Design	8. Communications
	4. Unpredictability	9. Unrealistic expectations
	5. Contract documents	
Colin *et al.* (1996)	1. Payment	4. Negligence
	2. Performance	5. Quality
	3. Delay	6. Administration
Sykes (1996)	1. Misunderstanding	2. Unpredictability
Bristow & Vasilopoulos (1995)	1. Unrealistic expectations	4. Lack of team spirit
	2. Contract documents	5. Changes
	3. Communications	
Dickmann *et al.* (1994)	1. People	3. Product
	2. Process	
Heath *et al.* (1994)	1. Change of scope	4. Distribution
	2. Change in conditions	5. Acceleration
	3. Delay	6. Termination
Rhys-Jones (1994)	1. Management	6. Tendering pressures
	2. Culture	7. Law
	3. Communication	8. Unrealistic expectations
	4. Design	9. Contracts
	5. Economics	10. Workmanship
Sample *et al.* (1994)	1. Acceleration	3. Whether
	2. Access	4. Changes
Watts & Scrivener (1992)	1. Change	3. Delay
	2. Law	
Hawitt (1991)	1. Change of scope	4. Delay
	2. Change of condition	5. Disruption
	3. Acceleration	6. termination

Figure 4: Sources of Potential Disputes (Mitkus and Mitkus, 2014)

Conflicts and disputes occur due to the inherent nature of the industry and uniqueness of the construction projects (Pena-Mora, 2003; Gardezi et al., 2013). As stakes are high in construction projects, the variable interests of the stakeholders involved in a project create conflicts and disputes (Cheung & Yiu, 2006). Gardezi et al., (2013) argued that literature is neglecting the fact that it is human nature, not to be entirely satisfied in all aspects even after cutting edge technology and rigorous processes, so the construction industry isn't an exception. Mitkus and Mitkus (2014) after literature review produced factors that can cause disputes in the construction projects, given in Figure-1. Nevertheless, Mitkus and Mitkus (2014) presented a different perspective as a primary root cause of conflicts and disputes in the construction industry, which is the lack of effective communication.

Due to the very nature of construction projects, it appears that conflicts and disputes are inevitable (Whitfield, 1994). However, the construction industry has been lagging behind to adopt innovative technology as compared to other industries (World Economic Forum, 2016). Building Information Modelling (BIM) is recognised as a revolutionary and game changing Information and Communication Technology (ICT) in Architecture, Engineering and Construction (AEC) industry. Nonetheless, it not only a technology but a socio-technical system (Tariq, 2014; Tariq, 2017). The purpose of this study is to investigate if BIM has the potential to mitigate disputes in the construction industry.

2. METHODOLOGY:

The methodology undertaken to do this study is based on primary and secondary data. The secondary data is literature review regarding the significance of the construction industry, potential causes that create conflicts and disputes, BIM and its potential advantages. A questionnaire survey is prepared based on the literature review. The survey is conducted from the construction industry of Pakistan to find out if the practitioners believe that BIM has the potential to mitigate disputes in the industry. The multiple linear regression analysis is carried out in SPSS to assess the significant factors in this regard. The breakdown of the methodological steps is given below:

Figure 2: Methodology of the Study

3. RESULTS and DISCUSSION:

The results of the primary data collected through surveys from the construction industry of Pakistan are presented in the form of bar charts below. There are 190 valid responses. The responses from the industry show that the majority of the respondents agree, ranging from 49% to 58% for individual questions, with the potential benefits of BIM to mitigate disputes and consequent delays. Additionally, the percentage of strong agreement ranges 30.5%-37%. The percentage range of respondents who remain neutral is 11%-16%. A negligible percentage of respondents disagrees with any statement and none of the respondent strongly disagree with any factor. The result of responses for each particular question is also shown in figures 3-8.

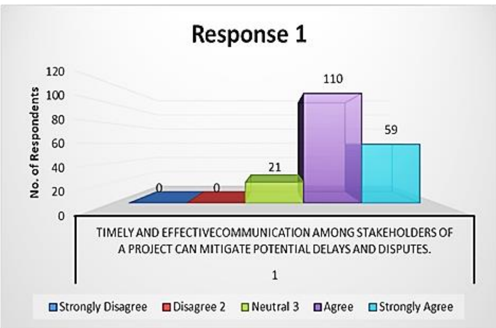

Figure 3: Responses to Survey Question No. 1

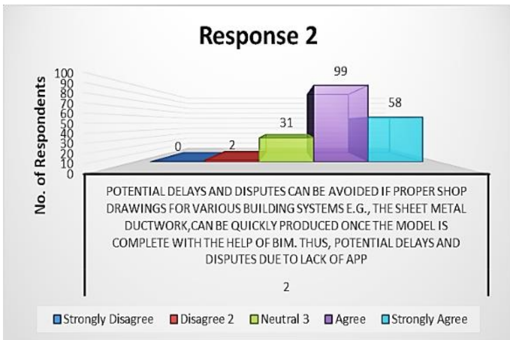

Figure 4: Responses to Survey Question No. 2

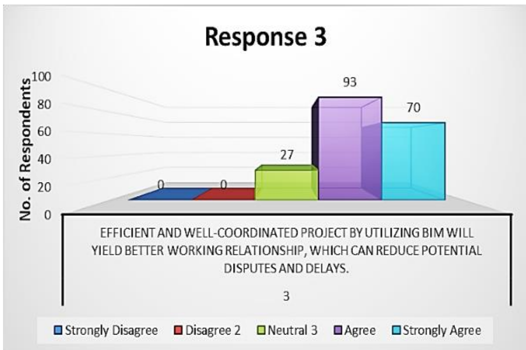

Figure 5: Responses to Survey Question No. 3

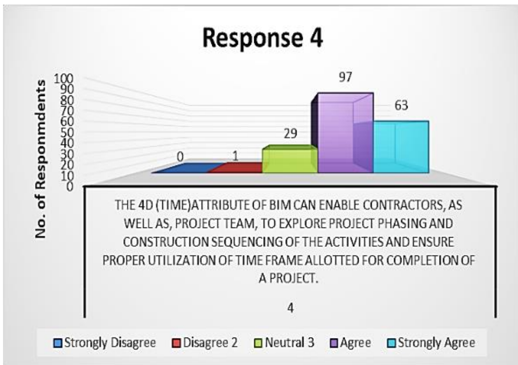

Figure 6: Responses to Survey Question No. 4

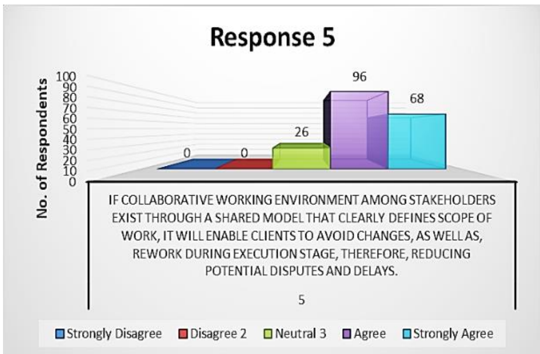

Figure 7: Responses to Survey Question No. 5

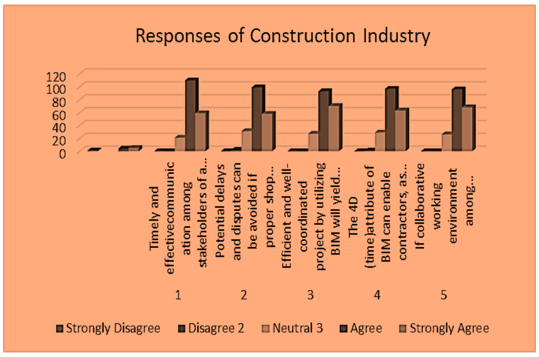

Figure 8: Responses to All Questions of the

3.1 Regression Analysis:

The authors employed multiple linear regression analysis technique to find out significant factors. Out of the five factors, three factors came out to be significant, which are shown in Table.1. The regression model is also given below, which is developed from the coefficient tables. Significance of factors is controlled by comparison of 'sig' term in Table 1 with significance level =0.05. It is clear from the Table1 that all three factors have 'sig' term less than 0.05 so they can be regarded as significant factors. Only the significant factors are taken into consideration and embedded in the regression model. Similarly, the value of R square is 0.23 (shown in Table: 2). In this research, three factors are significant; more factors are required to be assessed for significance which might raise the value of R-square. Different other factors would be taken into account to carryout research in the future. Nevertheless, the model is significant as evident from Table:3.

Table 1: Co-efficients Table

Model	Unstandardized Coefficients		Standardized Coefficients	T	Sig.
	B	Std. Error	Beta		
1 (Constant)	1.623	.337		4.820	.000
X1 Efficient and well-coordinated project by utilizing BIM will yield better working relationship, which can reduce potential disputes and delays.	.223	.053	.276	4.245	.000
X2 If collaborative working environment among stakeholders exist through a shared model that clearly defines the scope of work, it will enable clients to avoid changes, as well as, rework during the execution stage, therefore, reducing potential disputes and delays	.251	.063	.269	4.000	.000
X3 Potential delays and disputes can be avoided if proper shop drawings for various building systems e.g., the sheet metal ductwork, can be quickly produced once the model is complete with the help of BIM. Thus, potential delays and disputes due to lack of appropriate drawing can be solved.	.136	.053	.171	2.562	.011

$$\mathbf{Y=1.623+0.223X_1+0.251X_2+0.136X_3}$$

Table 2: Model Summary of Linear Regression

Model	**R**	**R Square**	**Adjusted R Square**	**Std. Error of the Estimate**
1	.482[a]	.233	.220	.518

Table 3: ANOVA

Model	Sum of Squares	Df	Mean Square	F	Sig.
1	15.120	3	5.040	18.805	.000[b]
Regression	49.849	186	.268		
	64.968	189			
Residual					
Total					

Three factors, as shown in Table-3, are significant because: X1-coordination among stakeholders of a project with the X3-availability of accurate drawings of the design and clarity of scope of work will mitigate disputes among the parties involved. Furthermore, X2-collaboration in the context of clear scope of work will enable stakeholders to do the needful without any rework that causes time delays and cost-overruns. Thus, improving the working relationship of the parties involved and their productivity while reducing conflicts and disputes that result in delays of construction projects.

4. CONCLUSIONS:

It is clearly evident from the responses given above that the practitioners of construction industry of Pakistan believe in the strengths of BIM to mitigate conflicts and disputes. The liner regression analysis also showed that collaboration among stakeholders to clear the scope of work at the outset of a project, co-ordination between project participants and availability of accurate drawings are significant factors to avoid disputes.

5. RECOMMENDATION:

The scope of the present study is limited and thus few factors were investigated, which is limitation of the present work. The future research work in this domain should be carried out with more factors, which might yield better R square value. Furthermore, a formulation of a new approach utilizing BIM to shift the paradigm of reactive dispute resolution to a proactive dispute resolution would be a great contribution.

ACKNOWLEDGEMENTS:

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Incident Reporting Tool

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Abstract

Safety is one of the effective factors on the operation of construction projects and plays a key role in success of a project. Incident reports aid in identification of problems and accidents can be reduced based on corrective actions taken, thus improving the safety. Incident reporting systems are a good source to analyse, track and document all incidents that have taken place on a job site. Unfortunately, most incident reports are generally unstructured, providing little or no guidance to the reporter. Therefore, most reports contain information only about what happened, as opposed to why an incident happened, making identification of possible hazards and prevention strategies extremely difficult. The study tries to address that the complexity of processes involved in construction can affect safety because of unpreventable workplace injuries. By understanding the situational elements of the prior incidents through incident reporting systems helps in developing preventative safety procedures. This paper aims to achieve the objectives of safe and transparent practice in our industry by developing a framework Safety Reporting Tool (SRT) which will help in apposite reporting as well as management of accidents. This data can be further compiled and can be used for developing an organizational safety plan as a proactive approach to prevent recurrence of unwanted incidents and also continuous improvement of safety operation of projects.

Keywords: Construction Safety, Incident Reporting, Reporting Framework, Reporting Tool.

1. INTRODUCTION:

Construction industry is one of the largest industries in the world. Construction along with its processes makes a complex system with different nodes linked and interacting with each other. Baccarini (Baccarini, 1996) proposes a definition of complexity of construction projects as “consisting of many varied interrelated parts and can be operationalized in terms of differentiation and interdependency”. According to the statistics, the percentage of accidents is very high in construction industry than any other industry due to its complex, unpredictable and diversified nature (Ahmed, 2000). Therefore, improvement of safety in construction projects is the most significant concern. In order to minimise this rate, various efforts have been put in place including safe practices (Chan, 2008) . This resulted in devising various schemes which helped a lot in achieving a lower rate during last 20 years (Choudhry, 2008). Contrary to this, common practice seen in construction industry of Pakistan which is gravely focused on achieving the desired outcomes; cost and time reduction, neglecting the safety perspective. Ineffective reporting mechanism is observed by the contractors who are reluctant to share the statistics mainly due to fact that it weakens their core competency and reputation in the market. (Farooqui, 2007) ; (Ali, 2006). In the view of gap observed, there is a dire need of a reporting mechanism. This paper aims to achieve the objectives of safe and transparent practice in our industry by developing a framework Safety Reporting Tool (SRT) which will help in apposite reporting as well as management of accidents. This data can be further compiled and can be used for developing an organizational safety plan as a proactive approach to prevent recurrence of unwanted incidents and also continuous improvement of safety operation of projects.

2. PROCEDURES AND METHODOLOGY:

2.1 Existed Tools Review and Framework:

An overview and analysis of twenty-five (25) existing tools to support and facilitate the design process and prepared the ground for the main study; some of them as an example, can be viewed in Table 1.

Table 1: Review of Existing Softwares

Type of Software	Software Name	Type, Developer	Software Name
1. Reporting	Techopedia	4. Management, ManageEngine	ServiceDesk Plus
2. Reporting	Intelex Safety	5. Management, Plan Brother	Incy.io
3. Reporting	Zendesk	6. Management, Hund	Hund

2.2 Safety Reporting Form:

Collecting evidence and accurate data of the incident is an important and tedious task as it requires systematic mechanism. To serve this purpose, a concise form is developed; Figure 1 shows a part of form to be filled by Supervisor, Safety Manager, Witness followed by the input from Investigator making it transparent and indubitable process.

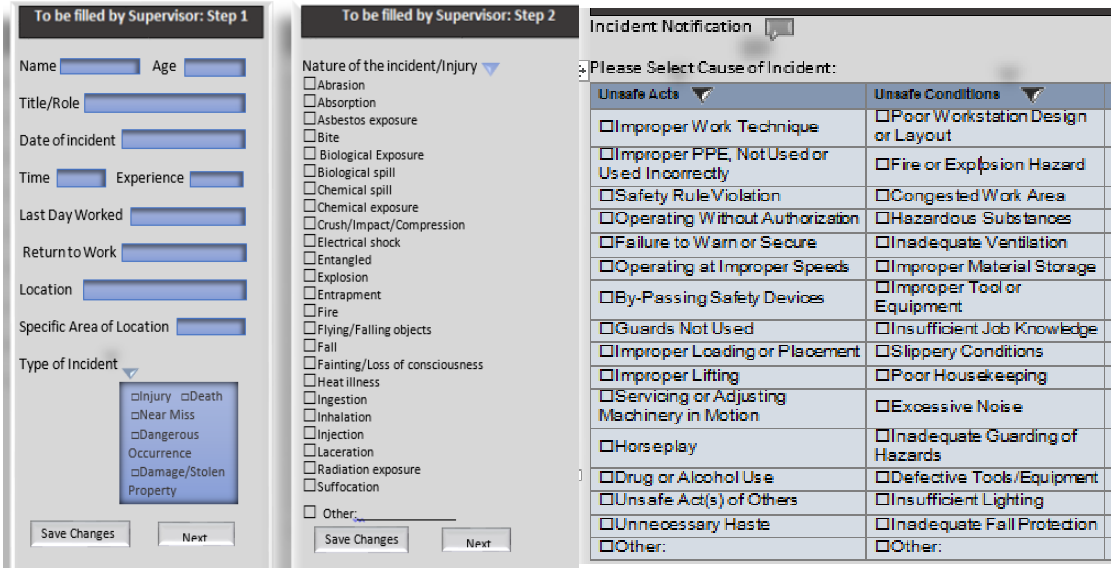

Figure 1: Safety Reporting Form

2.3 Investigation Process:

With the data inferred from the Safety Form, a Root Cause Analysis investigation identifies all the root causes associated with a problem to ensure there is no recurrence of the problem. Once recognised the causes will often be the result of physical factors, human factors, organisational factors.

2.3.1 Categorization:

To facilitate the investigation process, following levels of investigation are suggested and compiled in Table 2 ensuring the transparency of the process.

Table 2: Categories of Investigation

	Level 1: Concise	**Level 2: Comprehensive**	**Level 3: Independent**
When should it be used?	Incidents which resulted in no, low or moderate harm	Events which resulted in reportable incidents	Incidents with high level of media interest or mental health, homicides
Who should investigate?	Conducted by local staff, should add a person with knowledge of RCA	Conducted by an RCA experienced team not involved in incident.	Conducted by people who are independent to the provider, service or organization.
Analysis	5 Whys, Fish Tail analysis	5 Whys, Fish Tail analysis, High level of detail, Full use of analytical tools	High level of detail, Full use of analytical tools
Report	Often released as summary document and includes plans for shared learning locally and/or nationally	Full report with summary including recommendations for sharing locally and nationally	Full report with summary including recommendations for sharing locally and nationally

2.3.2 Evidence and Data:

Establishing the following facts from the form filled by Supervisor, Safety Manager and Witnesses in Step 2.2:
What: is the injury? Was the task assigned? Was the work process? Machinery/ plant/ equipment were in use? Safety rules were violated? Safe systems of work, permits to work, isolation procedures were in place? Training had been given?

2.3.3 Root Cause Analysis (RCA):

Based on the facts established above, there are certain steps that should be followed to facilitate identification of the root cause of the incident. The steps outlined below are the minimum requirements for completion of an RCA:

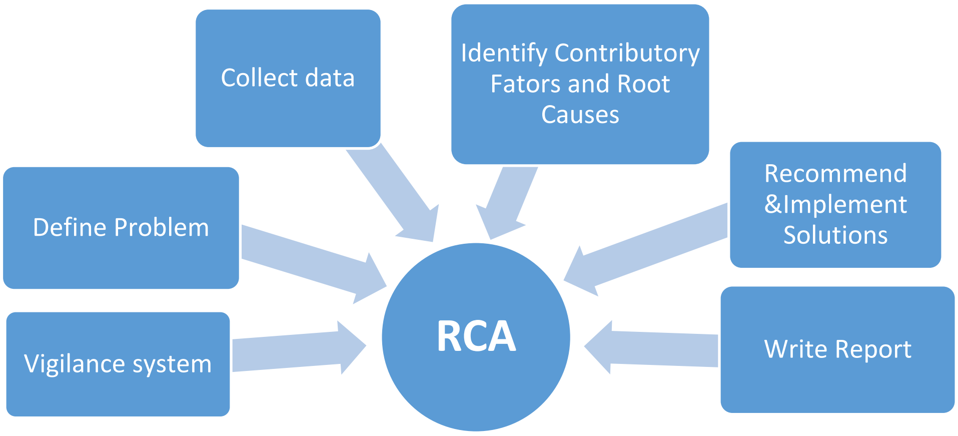

Figure 2: Overview of Root Cause Analysis

2.3.4 Report:

The main purpose of report formulation is to provide a formal record of the investigation process and share learning from the specific incident case to the third party.

2.4 Penalties:

Organization should take defensible disciplinary actions against an employee who violates a safety rule. Management should provide thorough safety training to all employees, and be sure that all safety rules are specific, clear, and follow OSHA or other safety guidelines.
Penalties are determined on a case-by-case basis. It also depends on the offence and type of duty holder the offender is. Some of the factors that are considered in deciding a penalty are severity of the contravention, risk of harm, compliance history of the work site party, including: orders, violation notices etc., also see if there is a commitment to health and safety.

2.5 Contractor Scoring:

This will achieve the secondary objective of the paper; introducing a transparent process of Contractor scoring based on safety performance (Table 3). This will prevent awarding of contracts to the contractors who are practicing unethical but lucrative methods.

Table 3: Contractor safety scoring during project execution

Category	Criteria	Score
Personnel	Knowledge and experience of Occupational Health and Safety With exceptional Performance; well above the acceptable standard	6
	Occupational Health &Safety representative's presence in critical on-site operations	7
Preparation	Safety Plan and provisions of safety in policy	10
	Submission of finalized safety plan earlier than the time required by contract	7
	Effective Hazard Analysis and Risk management	10
Implementation	Excellent safety induction/training program.	7
	Provision of safe conditions and proper PPEs	10
	Excellent safety performance in terms of KPIs	10
	Standard of monthly OH&S reports is excellent and submitted on time	5
	Monitoring and inspection	10
	Unbiased Internal audit culture	5
	Reporting of all incidents and accidents and prompt actions taken	7
	No repetition of the same non-conformance	6
	Total	100

2.6 Heat Map

Construction projects always encounter frequent safety issues such as falling from heights, being struck by/against objects, collapse, explosion, fire, cuts, and electrocution (Farooqui R. , 2011) (Zhou & Irizarry, 2016). Heat map is a powerful tool used to visualize the results of risk assessment process in a meaningful and comprehensive way. Heat maps provide a holistic view while making strategic decision. A brief overview of the most common types of heat maps used to visualize incident and accident data is as follows.

2.6.1 Trade Heat Map:

The trade heat map indicates the proportion of accidents in relation to different trades included in construction projects e.g. welding, electrification, plumbing etc. (Table 4) The increasing intensity of red colour indicates the high frequency of accidents, pale green colour indicates less number of accidents

Table 4: Injuries according to Trades

Trade	Fatalities	LTI	RWC	MTC
Labour				
Mason				
Steel Fixer				
Plumber				
Welder				
Electrician				
Carpenter				
Heavy Equipment Operator				

LTI	Lost Time Incident
RWC	Restricted Work Case
MTC	Medical Treatment Case

2.6.2 Incident Type Heat Map:

This heat map highlights the frequency of occurrence of various types of incidents on construction sites. An illustration of the incident type heat map is given below. The increase in red color indicates higher number of incidents while increase in blue color indicates low number of accidents.

Table 5: Incidents on site

Incident Type	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Fall from a higher level												
Fall on same floor level												
Struck; by objects												
Electric Shock												
Struck by falling objects												

2.7 Graphs:

After getting improved data from investigation, following graphs can be generated: Age of Workers, Trade, Experience of Workers Location vs. Frequency of Accidents

3. RESULTS AND DISCUSSION:

A framework SRT has been formulated as it provides a structure for mapping out, defining, and analysing the process, which is the primary objective of this paper. Absence of comprehensive knowledge about reporting of incidents, results in safety issues and ultimately affecting the KPI's of projects.

This study tried to address the importance of a well-structured reporting mechanism and presented a model in the form of a Safety Reporting Tool (SRT) which will prove to be beneficial in guiding and management of incidents if adopted and accepted religiously within the organization.

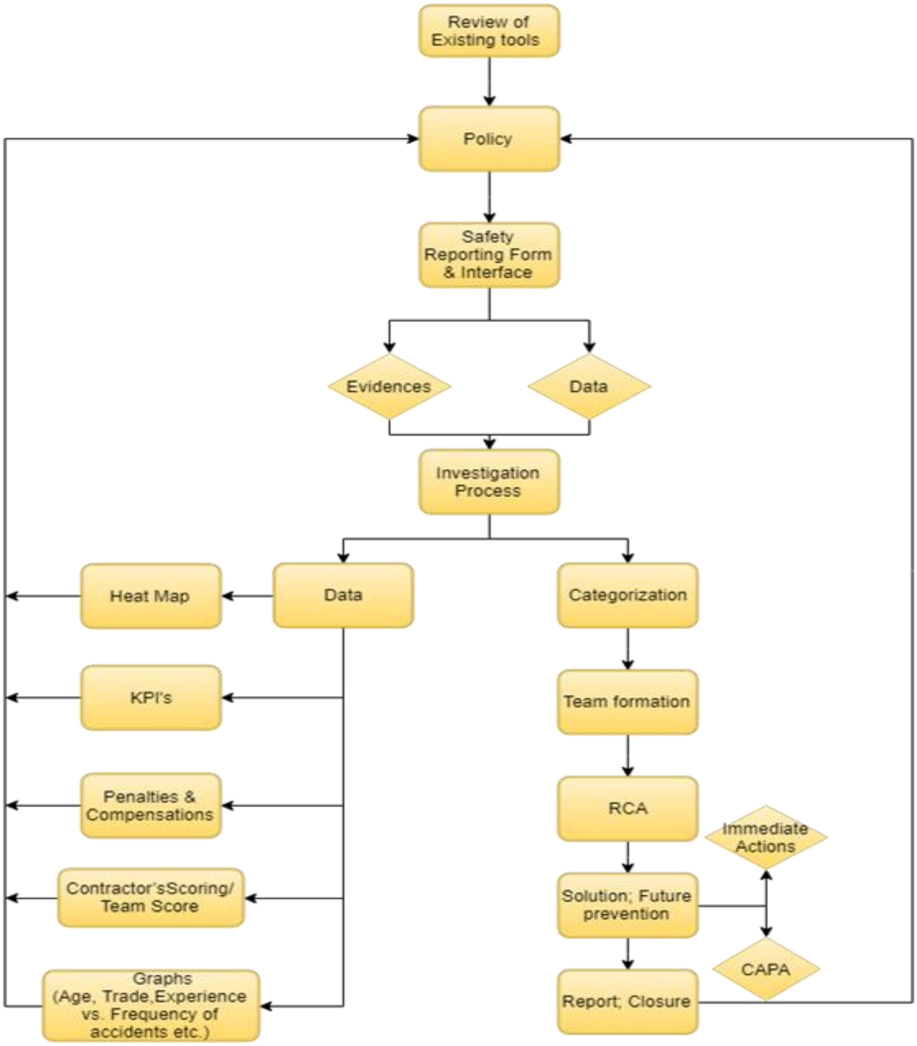

Figure 3: Framework of Incident Reporting Tool

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Up-gradation of Conventional Buildings into Green Buildings [Case Study of Civil Engineering Department's Buildings in University of Central Punjab]

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Abstract

Buildings are one of the major sources of greenhouse gas emissions and they are contributing to climate change. Buildings not only consume natural resources in the form of construction materials during their construction phase but they also consume other resources in the form of water and energy during their operation and maintenance phase. If the building is not designed keeping in view its sustainability, it will not only be uneconomical but also socially and environmentally problematic. We have a massive built environment which is being constructed without considering its social and environmental consequences, even its long term economic aspects. While keeping in view triple bottom line, deconstruction of conventional buildings might not be an option. So their up-gradation in operations and maintenance phase might be an only practical and acceptable option in most of the cases. In this study two Civil Engineering Department's (CED's) Buildings in University of Central Punjab (UCP) are analyzed on the basis of LEED standards. At the end, recommendations are given to improve their performance and make them green and sustainable.

Keywords: Sustainable development, Green Buildings, LEED, Operation and maintenance

1. INTRODUCTION:

Civil engineering is arguably considered as the oldest engineering profession. As the ancient man searched and constructed shelter to protect himself, he must have got a very basic and simple unit. As the human civilization evolved, construction processes also evolved. Now, the modern units are supposed to provide basic needs of human being, which includes water supply system, sanitary system, gas, electricity etc. But, the problem is, conventional construction occurs at the cost of natural resources without considering their impact, future availability and fate. Built environment not only consumes resources during construction phase but they also consume resources during their operation and maintenance phase. Most of the natural resources used in conventional buildings are limited in nature. On the other hand, built environment contributes to more than two third of the total greenhouse gas emissions (EIA, 2008). These greenhouse gases are causing global warming and climate change. The rapid increase in population and living standards are resulting in high demand of built environment, which is further exacerbating the problem.

To cater the needs of ever increasing population, it is a high time to build green. Green building is a concept by which all the factors are taken into account to produce an economic, social and environment responsible structure. In it, negative effects of buildings are reduced as far as possible. Research shows that occupants of green buildings are 2% to 16% more productive (Vivian, L et al., 2005) because of enhanced indoor environmental quality due to the provision of natural lighting, ventilation, vegetation and their passive design. It also results in on average 24% less energy consumption as compared to typical buildings (Turner, 2008). The average marginal cost of green buildings is less than 2% (Gregory H. Kats, 2008) which are offset by savings over time.

In this work, two buildings of Civil Engineering Department, University of Central Punjab are studied. They are analyzed on the basis of LEED (Leadership in Energy and Environmental Design) which is the most widely used Green Building rating system in the world. On the basis of analysis, recommendations are given at the end to improve the efficiency of buildings. It will help in improving their energy efficiency, water efficiency and indoor environmental quality. They can improve health and productivity of its occupants while reducing their carbon footprint and expenses.

2. RESEARCH SIGNIFICANCE AND OBJECTIVES

It is recommended to think and design green from the start of project to get maximum out of it. But, unfortunately we have a massive conventionally built environment which is impacting the environment in its worst manner and which needs to be green. Demolishing them and reconstruction is not a feasible option; hence the unique solution is retrofitting to make them green. The objectives are: to make them suitable for the ecosystem, to improve their energy and water efficiency, to enhance indoor environmental quality for the better health and performance of its occupants by reducing waste and pollution generating through them and to make them economical.

3. METHODOLOGY AND DISCUSSION

There are many rating systems to gauge Green Buildings. We have used Leadership in Energy and Environmental Design (LEED) to conduct this study. It was first established in 1999 by US Green Building Council (USGBC) which was formed in 1992 to build green systems. LEED rating systems are used in this study to gauge two Civil Engineering Department's buildings present in University of Central Punjab, Lahore. Their aerial view captured from Google Earth is shown in fig. 1. The total covered area of buildings is 27,219sq.ft (or 2529 sq. m). It has 94 FTE (full time equivalent) occupants.

Figure 1: Aerial view of buildings

4. SUSTAINABLE SITE

The purpose of sustainable site is to protect the ecosystem. The maintenance of the exterior should be easy. Minimum energy, water and manpower should be required for its management. Hazardous chemicals (in the form of paints) should not be used for aesthetic purposes as during rainfall they can be added into runoff. Provide pervious surface outside the building. Pervious surface allows infiltration of storm water into ground, which conserves water table and hydrology. Roof should be planted with native or adaptive vegetation to protect open habitat and to reduce heat island effect. The exterior of the buildings in our case is already in good condition and no paints are used on their exterior as shown in fig. 2. But, there must be a pest management plan to prevent pests entering into the building.

Figure 2: Front view of building

Fuel consumption during transportation produces greenhouse gases. Alternative transportation e.g. university's transportation, public transportation and carpooling should be encouraged by giving incentives. 26 buses are dedicated to University and public transportation is also available as shown in fig. 3 and fig. 4 respectively. University should start awareness campaign to promote them.

Figure3: University's transportation

Figure 4: Public Transportation

5. WATER EFFICIENCY

Plumbing fittings must be efficient and should be designed according to the codes. Any leakage must be repaired on time. The consumption of water must not be greater than the baseline. Proper metering should be done in the building to track the consumption and to predict the leakages in case of abnormal readings. Baseline in our case is 626 gallons per day (or 2.37 m^3 per day). Water metering system is not provided in the buildings so their consumption cannot be metered. Sensor water taps are recommended to save water. Provision of grey water usage should also be given to reduce the consumption of potable water.

6. ENERGY AND ATMOSPHERE

Install energy meters throughout the building to track consumption. Conduct ASHRAE Level- I walk through analysis for the building's assessment, to plan and maintain energy efficient strategies. ASHRAE (American Society of Heating, Refrigeration and Air conditioning Engineers) is devoted to the advancement of indoor environment control technology in the HVAC industry (ASHRAE, 1973). ASHRAE level 1 walk through analysis involves interviews of facility staff, review of utility bills, review of other operating data and walkthrough the facility to identify areas of energy inefficiency. Ensure that building is fulfilling minimum energy performance requirements. ENERGY STAR Portfolio Manager can be used for this purpose. If the building is not fulfilling minimum energy performance requirements, conduct ASHRAE Level II energy audit and optimize energy efficiency performance (Cottrell, 2011). ASHRAE Level II energy audit includes detailed calculations of energy inefficiencies and financial losses.

Prohibit CFC-based refrigerants because they deplete ozone layer. Develop plan for operations and up gradations. Also train the staff accordingly for operations and minor repairs to maintain and optimization of energy performance. Identify opportunities through which energy performance can be further enhanced. Renewable energy should be used to decrease the negative impacts of non- renewable energy usage.

In our case study area, energy meters are not installed in the buildings. Energy meters are the backbone of energy performance calculations and they are mandatory for green buildings. The roof of buildings is already installed with solar panels as shown in fig. 5 but again energy meters are not installed with them so we do not have any data about their electricity generation and consumption through renewable means.

Figure 5: Solar panels installed on roof.

7. MATERIALS AND RESOURCES

Make a sustainable purchasing and solid waste management policy to reduce environmental impacts of materials. Purchasing of post-consumer, post-industrial and rapidly renewable materials should be preferred. The material and products used in the building should be harvested and processed within 500 miles (or 804.7 km) of the project. Conduct audit of waste to establish baseline to increase recycling and waste diversion. Divert waste from incineration and landfill and maximize their reuse and recycling.

8. INDOOR ENVIRONMENTAL QUALITY

The building should comply with ASHRAE 62.1-2007 standards for ventilation. Smoking should be prohibited inside the building and provide smoking area 25ft away from the entrance of building and outdoor air intakes. Install MERV13 or better air filters on all air intakes and they must be maintained and replaced on regular basis to reduce particulates in air. Make green cleaning policy for the purchase of cleaning products and equipment. Guidelines should be made for the handling and storage of cleaning chemicals and staff should be trained accordingly. Feedback from the occupants should be taken for the continuous improvement in policy. Install permanent monitoring CO_2 sensors. ASHRAE STANDARD 55-2004 should be observed for the acoustic, thermal and other comforts of occupants. Occupants must have an access to lightings control for their comfort and productivity. Daylight and outside views should be introduced to regularly occupied areas, to make connection of occupants with outdoor environment.

9. CONCLUSIONS:

On the basis of analysis it is concluded that with some minor and major changes energy efficiency, water efficiency and indoor environmental quality of the buildings under consideration can be improved and their carbon foot print can be reduced significantly. In the section below some improvements are recommended to improve the performance of buildings.

10. RECOMMENDATIONS:

Following are the recommendations based on the analysis made in this study for the social, environmental and economic improvements of buildings under consideration, in their Operations and Maintenance phase.
Provide impervious surface outside the building to promote infiltration of storm water into the ground to conserve water table and hydrology

- Provide native or adaptive vegetation over the roof to protect open habitat and to reduce heat island effect
- Encourage occupants to use university's, public and shared transport by giving incentives to reduce emissions due to commute
- Install water and energy meters to track losses, performances and efficiencies of the
- systems
- Give provision of grey water usage in toilets to reduce the consumption of potable water
- Make a sustainable purchasing and solid waste management policy to reduce environmental impacts of materials
- Install MERV13 or better air filters on all air intakes

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Analysis of Lahore Development Authority (LDA) Construction By-Laws

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Abstract

Currently, Pakistan is facing water scarcity and energy deficiency. In future, the problem can further escalate with increase in its population and development of infrastructure. So, it is a need of an hour to revise its construction by-laws, to improve them to have a sustainable development. For this purpose, an attempt has been made to analyze the construction bye-laws of LDA. In this study, LDA bye-laws are analyzed on the basis of LEED (Leadership in Energy & Environmental Design), which is a widely used green building rating system in the whole world. It is concluded that LDA bye-laws are lacking in clauses to generate water efficiency, energy efficiency and indoor environmental quality in the buildings. At the end, some suggestions are made which can be incorporated in LDA bye-laws to make them green up to some extent.

Keywords: Infrastructure, Construction By-laws, LEED, LDA By-laws, Green Building.

1. INTRODUCTION:

In present days, Pakistan is facing the problems of water scarcity and energy deficiency. Pakistan is one of the 36 most water-stressed countries in the world (Beham, 2018). Pakistan crossed the water scarcity line in 2005, it would run out of water by 2025 and more than 80% of water in Pakistan is considered unsafe (Shahmeer, 2018). Besides it, Pakistan is facing energy (electricity) deficiency as well. Present energy demand in Pakistan is almost 25,000MW and is facing the shortfall of 9,000 MW (Zafar Bhutta, 2018). In the future, these problems can further escalate if these problems are not addressed at present.

These major problems can be solved by sustainable development. Sustainable development is the development that meets the needs of present without compromising the abilities of future generations to meet their own needs (Brundtland, 1987). Sustainable development can be addressed by green buildings. A green building is a building that, in its design, construction and operation, reduces or eliminates negative impacts, and can create positive impacts, on our climate and natural environment. Green buildings preserve precious natural resources and improve our quality of life (WGBC, 2016). Green buildings promote sustainability without disturbing natural habitats. Green buildings reduce energy consumption, they save money and play a vital role in restoration of environment and habitat.

Lahore is the provincial capital of the Punjab, which is the most populous province of Pakistan. It is also the second largest city in the country. Lahore is rapidly growing in all its directions and creating environmental problems in all of its dimensions, which needs a huge infrastructure to cater the needs of its ever-increasing population.

Unfortunately, it is happening at the cost of depletion of natural resources. The depletion of these resources can be excessively reduced by improving Lahore Development Authority (LDA) by-laws. LDA Housing Authority updates them on yearly basis. It is responsible for development in Lahore, Pakistan. It also regulates and issues permits for new construction projects.

To analyze LDA by-laws Leadership in Energy and Environmental design (LEED) is used in this study. LEED is most widely used green building rating system in the world. LEED has developed its own rating systems. The rating systems are categorized in five basic areas: Sustainable Sites, Water Efficiency, Energy and Atmosphere, Materials and Resources, and Indoor Environmental Quality. But we are only focusing on the Water Efficiency and Energy and Atmosphere. LEED is flexible enough to apply on all types of the building but in this study, we will focus on residential house. The significance of this study is by updating the LDA construction by-laws, sustainability and restoration of natural habitat is introduced in Pakistan.

2. OBJECTIVES:

The construction By-Laws of LDA are analyzed on the basis of LEED. The objective of this study is to analyze LDA by-laws to make them water efficient and energy efficient. As they are among the major problems of Pakistan. Lahore Development Authority (LDA) bye-laws are lacking in clauses to generate water efficiency, energy efficiency and indoor environmental quality in the buildings. At the end of this study economical, environmentally friendly and easy to implement suggestions are given which can be incorporated in the clauses of LDA by laws.

3. LITERATURE REVIEW:

LEED focuses on five main components of green buildings. They are Sustainable Sites, Water Efficiency, Energy and Atmosphere, Material and Resources and Indoor Environmental Quality. In this study, we will analyze LDA by-laws for water efficiency and energy efficiency as they are among the major problems of Pakistan.

As energy and water efficiency increases, they will help in preserving precious natural resources, which will otherwise deplete at the cost of abilities of the coming generation. The overuse of resources can also damage the habitat and eco-system. As the energy efficient buildings maintain favorable temperature, they can enhance the comfortability and efficiency of their occupants. They also help in reducing the operational cost of the building. The environmental friendly performance of green building can also not be underestimated as they emit greenhouse gases at considerably lower level as compared to conventional buildings because of reduced usage of cooling and heating appliances.

3.1. Water Efficiency:

Water efficiency is important component of green building. Its purpose is to reduce the consumption of water, its efficient use, reuse and recycling. Following are the strategies from LEED v4 homes which can help in achieving the goal up to some extent.

3.1.1. Water Metering

Water metering is an important tool to track water consumption and leakages. If water is billed then it can largely reduce water consumption. The consumer will be motivated to follow different strategies to reduce water consumption. For example, by applying efficient fixtures and repairing any leakages on time etc. Water metering is prerequisite of LEED.

3.1.2. Storm Water Quality Control

Storm water runoff can be controlled on site by increasing pervious areas which will help on site infiltration to recharge ground water table. It will also help to reduce the contamination of storm water.

3.1.3. Water Reuse

There are many different strategies by which water can be reused. For example, by the provision of rainwater harvesting and gray water use they can be encouraged in LDA by-laws. If the water is recycled at the city level and is charged by the wastewater producers then the consumption of wastewater can be reduced effectively.

3.2. Energy Efficiency:

Energy consumption can be reduced at national level by building energy efficient infrastructure. It can be a giant step towards the reduction of greenhouse gases emission which are major contributors of climate change. Following are the strategies addressed in LEED which can be incorporated in LDA by-laws to produced energy efficient infrastructure.

3.2.1. Building Orientation

Building orientation is an important factor by which solar energy can be harvested in winter season and building can be saved from direct heat in summer season. As Lahore lies in the region where summer and winter both are intense but summer season lasts longer as compared to winter season.
The building orientation should be so that the sunlight should not enter in the building directly. Otherwise the energy consumption of the buildings will increase because of the usage of cooling systems, which will also result in emission of greenhouse gases.

3.2.2. Insulation

Its purpose is to minimize heat transfer with the proper design and installation of insulation material which can help in reducing the use of cooling system in summer season and heating systems in winter season.

3.2.3. Reduce Heat Island Effect

Heat island effect can be reduced by encouraging plantation on site. By the shade of trees direct exposure of walls to the sun can be reduced. Similarly, by roof vegetation or by implementing the material on roof having high Solar Reflective Index (SRI) the heat island effect can be reduced.

3.2.4. Windows

Windows are the source of light and ventilation inside the buildings. Windows sizes should be addressed in by-laws depending upon the climatic zone and floor area ratio of the buildings and which will fulfill the requirements of Energy Star, version 3 pathway.

4. ANALYSIS:

Table 1: Comparison between LEED credit numbers and LDA clauses

Components	LEED Credit Numbers	Corresponding LDA Regulations
Water Efficiency	**WE Prerequisite:** Water metering is mandatory	---------
	SS Credit: Storm water quality control increase pervious cover and on-site infiltration to reduce contamination of runoff and disruption of natural hydrology.	---------
	WE Credit # 1.1: Rainwater management system will help to use the rainwater for indoor and outdoor purposes. For indoor use, design the storage tank to hold at least 50 percent of the roof area	Regulation 6.6.6: All buildings to be constructed in future in Lahore should have provision for roof top rainwater harvesting commensurate with its plinth area.
Energy Efficiency	**EA Credit:** Suitable Building orientation can help in controlling the inside temperature of the building and can save the energy by preventing the usage of energy consuming appliances like lights and bulbs	---------
	EA Credit # 2: Insulation should be according to the Energy Star, version 3 requirements.	---------
	EA Credit # 3: Heat Island Reduction, applies to home. Describes the area for vegetation in homes and the plantation of trees. Which helps to maintain the temperature of the building and shading for hardscape.	Regulation 2.2.3: It only describes the ground coverage and floor area ratio (FAR).

EA Credit # 4: Windows dimensions should be decided based on climatic zone and window to floor area (WFA) ratio.	---------

6. CONCLUSIONS:

Following conclusions can be drawn from the conducted study:

- In LDA regulations water metering should be mandatory for the buildings which is not mentioned in current by-laws.
- The LDA by-laws regulation should restrict the pervious areas to recharge ground water and to reduce storm water runoff.
- LDA regulations are lacking in addressing building orientation which should be mentioned in them.
- No regulations related to insulation are mentioned in LDA by-laws. They should also be addressed in by-laws to increase energy efficiency of buildings.
- Regulation 2.2.3 mentions only ground coverage and Floor Area Ratio. Vegetation should be encouraged on open areas.
- Dimensions of windows are not being discussed in LDA by-laws. They should be suggested in by-laws on the basis of climatic zone and Window to Floor Area ratio.

In the next step rules should be defined to restrict the sizes of on-site pervious areas for infiltration, open spaces for vegetation, size of windows and orientation of buildings.

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Exploring Building Information Modelling (BIM) Readiness in Islamabad Capital Territory (ICT) of Pakistan

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Abstract

Building Information Modelling (BIM) is a significant development in Architecture, Engineering and Construction (AEC) industry. It is a collaborative way of working among stakeholders of a project underpinned through technology, i.e. shared data rich virtual model of a construction facility. Governments, round the world, are steering the process of BIM adoption to achieve value for public money spend on infrastructure. Different studies are carried out, throughout the world, on different focus groups to find out their readiness of BIM. The present study is the first ever study to explore an overall awareness and readiness of BIM in the construction sector operating within the vicinity of Islamabad Capital Territory (ICT), Pakistan. A survey is conducted in the study area; 190 valid responses are obtained. The results show that 59% of the respondents are aware of BIM, however, only 39% are currently utilizing BIM. Nevertheless, 62% respondents are willing to adopt BIM in the near future. Therefore, awareness and the willingness to adopt BIM is on an upward trend, while, the adoption of BIM is a bit low at present. The government should steer the process of BIM adoption, like the United Kingdom (UK) Government, to achieve value for public money in developing infrastructure.

Keywords: BIM, BIM awareness, BIM readiness, BIM adoption

1. INTRODUCTION:

BIM is documented as a ‘revolutionary’ development and a ‘game changer’ for AEC industry (UK Cabinet Office, 2011; Ullah *et al.*, 2019). There is no single universally agreed a definition of BIM as it is a huge concept to comprehend. Nevertheless, in a nutshell it is a precise data rich virtual representation of a construction facility and a collaborative way of working (Eastman *et al.*, 2011).

The significance of the BIM is evident as the government of the UK initiated a task group with the help of the UK’s construction industry in 2011 for its implementation. The cut-off date to adopt BIM Level-2 for all public projects in the UK was 2016 (UK

Cabinet Office, 2011) and BIM Level-3 would be implemented from 2016 to 2025. Furthermore, BIM plays a key role in achieving set targets in Construction 2025 (HM Government, 2013).

Numerous benefits of BIM have been reported in the literature. Zheng *et al.*, (2019) identified that BIM can be beneficial throughout a project's life-cycle: design, execution, operation, as well as, to improve the living environment of a facility. Additionally, it can improve safety (Li *et al.*, 2017) and productivity (Arayici *et al.*, 2011), which will reduce the delays and cost (Azhar, 2011).

However, Ullah *et al.*, (2019) state, based on the literature review, that despite the benefits of the BIM, there are many barriers to adopt it, such as: lack of awareness, the cost involved for software, hardware and training, lack of interest from clients/contractors/sub-contractors, legal and contractual issues.

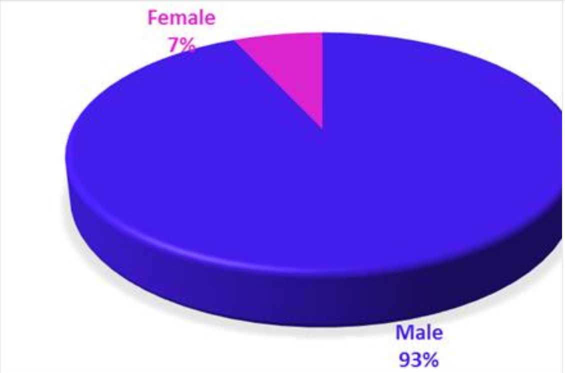
Several studies in different parts of the world on BIM readiness targeted towards specific groups have been conducted in the literature to assess the awareness of the subject matter and its readiness (Zhou, 2012; Kugbeadjor, 2015; Ghaffarianhoseini *et al.*, 2016; Shen, 2016; Yusuf, 2017). However, no such study is undertaken in Pakistan, thus, for the first-time authors conducted this study for the construction industry of Pakistan keeping in view of significance and the growing demand of BIM internationally (Ismail, 2017).The present study is aimed at to explore an overall awareness of BIM among practitioners of in the construction industry within the premises of ICT, Pakistan. Furthermore, the authors are interested in to find out if the practitioners of the industry are willing to adopt it in the near future.

2. METHODOLOGY:

The methodology of this exploratory study is based on literature review and questionnaire survey data collected from stakeholders involved in construction projects within the ICT, Pakistan. A questionnaire survey was carried out to explore awareness, usability and the willingness of stakeholders to adopt BIM in the near future. The aim of the research is to explore an overall awareness of BIM in the construction industry, not just the top management of an organization, thus, a wider spectrum of the participants is targeted as unit of analysis. Awareness regarding BIM of the wider workforce of the industry i.e. individuals working in the sector, is utmost significant to find out prevailing conditions of the industry. Additionally, as the industry is comprised of different roles: clients, contractors and consultants, thus investigating the readiness of BIM from clients, contractors and consultants is also very important for better understanding the latest scenario of overall the industry. Therefore, 190 valid responses from individual working as clients, consultants and contractors are gathered and analyzed. The results of the survey are presented graphically and critically analyzed in the results and discussion section.

3. RESULTS AND DISCUSSION:

The results of the survey conducted within the ICT, Pakistan regarding awareness of BIM and its readiness are given below. A total of 190 respondents from different stakeholders of construction industry participated in the survey.



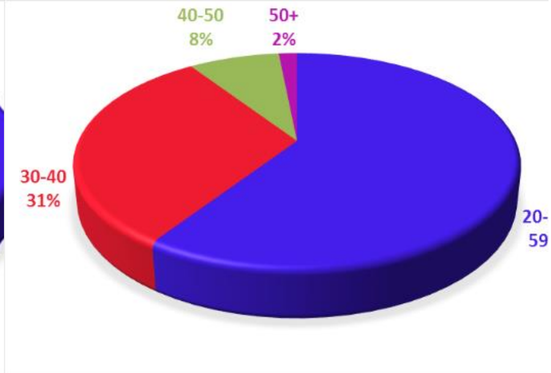
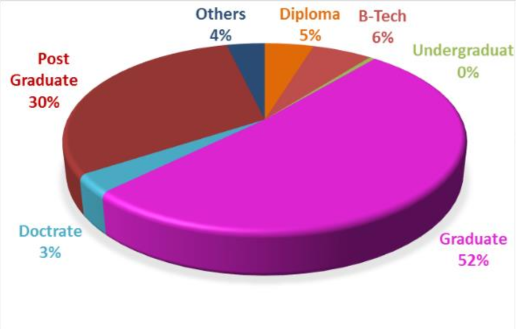


Figure 1: Gender of the Respondents

Figure 2: Age Group of Respondents

The study reveals male dominancy in the construction industry of the ICT, with the figure of 93 and 7% of male and female respondents respectively as shown in figure 1. However, youngsters between the age of 20-30 years make a simple majority of 59% in this field, whereas, 31, 8 and 2% of the respondents are between the age group of 30-40, 40-50 and 50+ years respectively.



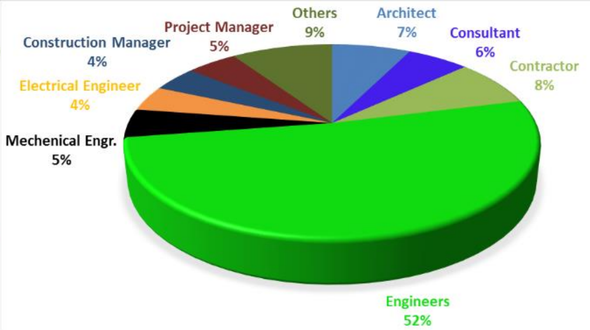
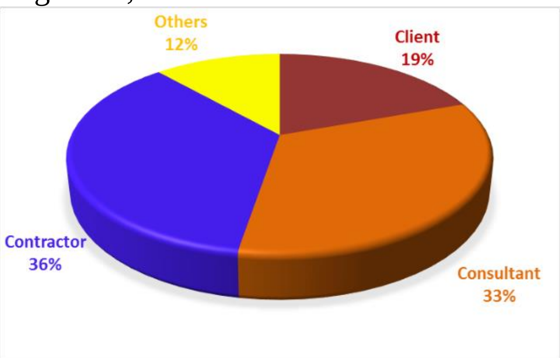


Figure 3: Qualification of the Respondents

Figure 4: Profession of the Respondents

The qualification demographics of respondents are shown in figure 3; 52% are graduates and 30% are postgraduates. Additionally, 3% of the respondents hold PhD degree as well. But the majority of the respondents have a four years degree and above qualification. The current profession of the respondents is shown in figure 4. The respondents displayed an array of profession, but the majority of them are Civil Engineers, i.e. 52%.



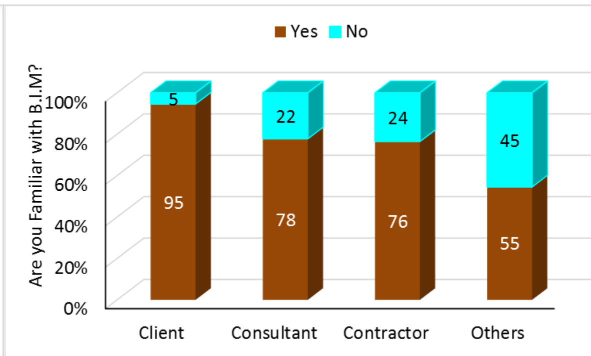


Figure 5: Role Assumed by the Organization

Figure 6: Familiarity with BIM

The statistics of the respondents working as a client, consultant or contractor are shown in figure 5. The majority of the respondents taken this survey are contractors and consultants with the values of 44 and 32% respectively, whereas, 15% respondents of the survey categorized their organization as a client. In response to a question about BIM awareness the percentage of the respondents who are aware of the BIM is 59% while 41% are ignorant of this development. Furthermore, the results show in figure6 that maximum of 95% clients are aware of BIM as compared to consultants and contractors. Keeping in view that Pakistan is a developing country and challenges faced by the construction industry, 59% respondents are aware of BIM, which seems appropriate to begin with. Nevertheless, to keep up with the pace of the global construction sector, BIM awareness of the construction industry of Pakistan should be increased through workshops, seminars, and conferences. Furthermore, awareness regarding different levels of BIM should be imparted, which was out of the scope of the present study.

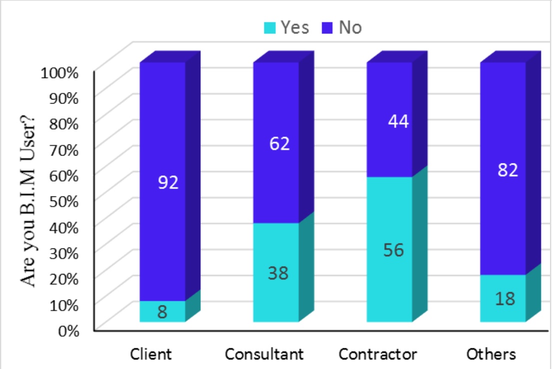


Figure 7: BIM Users in ICT Pakistan

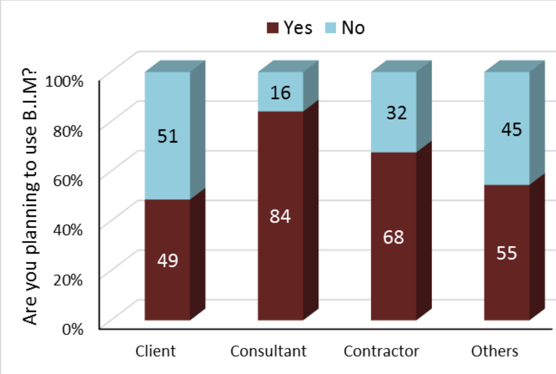


Figure 8: Planning to Use BIM

The overall percentage of the respondent currently using BIM is 39%, whereas, 64% aren't currently using it. Figure 7 shows that currently a maximum of 56% of contractors are using BIM, whereas, a minimum of 8% of clients are utilizing BIM. Again, the levels of BIM usage, i.e., level 0-3, isn't the scope of the present study. Nevertheless, the examination results clearly show that the awareness is at a higher end as compared to adoption of BIM in the study area. Thus, the adoption of BIM is at an inception stage. The barriers to adopt BIM are cost of training, software and equipment, lack of expertise and training (Shen *et al.*, 2016). Nevertheless, the respondents are asked if they are planning to use BIM in the future; 62% of the respondents gave positive response to adopt BIM as compared to 38% respondents who aren't yet planning to adopt it. Figure 8 shows that the maximum and minimum values observed are for consultants (84%) and contractors (49%) respectively. But the level of BIM adoption is yet to be investigated.

4. CONCLUSION AND RECOMMENDATION:

The first ever study to explore awareness of BIM and readiness to adopt it is undertaken in ICT, Pakistan. Questionnaire survey is conducted and 190 respondents participated in the surveys having different gender, age group, qualification, and roles in the construction industry. Overall, it is explicit from the results that practitioners of the construction industry, clients, consultants and contractors, of the study area are aware of BIM i.e. 59%, however, only 32% has adopted BIM at present. On the other hand, 62% respondents stated that they are planning to use BIM in the future, which shows a positive attitude towards BIM adoption. Therefore, willingness and readiness to adopt BIM is on an upward trend even though the adoption is low currently.

It is recommended that awareness, readiness and adoption based on different levels of BIM can be further explored in the near future.

ACKNOWLEDGEMENT:

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Environmental Engineering and Water Resources

Paper ID: 401

Reduction in Fresh Water Consumption by Grey Water Reuse for Flushing and Irrigation - A Case Study of a Multistorey Hotel Building

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Abstract

Over time the concern of amount of water consumption has gained strength with depleting water resources each day all over the world. Saudi Arabia with already minimal surface and sub-surface water resources is on alert for future planning and management of water use in the Kingdom. Large scale water conservation projects are in line for future constructions. But as a matter of fact, the old buildings must also be rehabilitated according to the sustainability standards. This study has been taken up for an operational hotel building where there are no water conservation techniques in practice. Grey water recycling being the most common method for water reuse has been studied for its feasibility in this building. Various calculations were performed for water usage facilities including water required for flushing and irrigation as compared to the volume of grey water being generated by the building. It was found that the hotel building releases almost 75% of grey water daily of its average daily water requirement whereas the requirement of water for flushing and irrigating green areas came out to be 26%. A grey water treatment unit of 12,000 gallons per day (gpd) capacity was recommended to be installed at the hotel which shall replenish the water for flushing and irrigation resulting in a reduction of fresh water usage by 26% in addition to various other economic and environmental benefits. This research shall be useful for the construction industry and shall motivate the concerned authorities to rehabilitate the older buildings to make them sustainable along with new construction.

Keywords: grey water recycling; sustainability; water conservation; water reuse; flushing; irrigation

1. INTRODUCTION:

The available water resources all over the world have already rung the emergency alarm. It is due to the fact that people have been using water abundantly without even realizing that the amount of usable water for domestic purpose is in fact very limited as compared to the total amount of water available on Earth. Overtime with the increase in population, the water demand kept on increasing which resulted in decrease of surface water as well as groundwater resources. The time this reality has surfaced, the researchers and scientists have done extensive work for creating awareness about saving water as one part and developing methods to conserve this precious resource as the other part. One of the approaches for conservation of water is to reuse the used water and rely less on the fresh water use.

Grey water has been identified as one of the common sources of water which can be reused for domestic purposes after specified treatment. According to definition grey water is the used water which has not been in contact with the water coming out of water closets or urinals. This grey water is collected into a common storage facility from where the required amount of grey water is introduced into the recycling system.

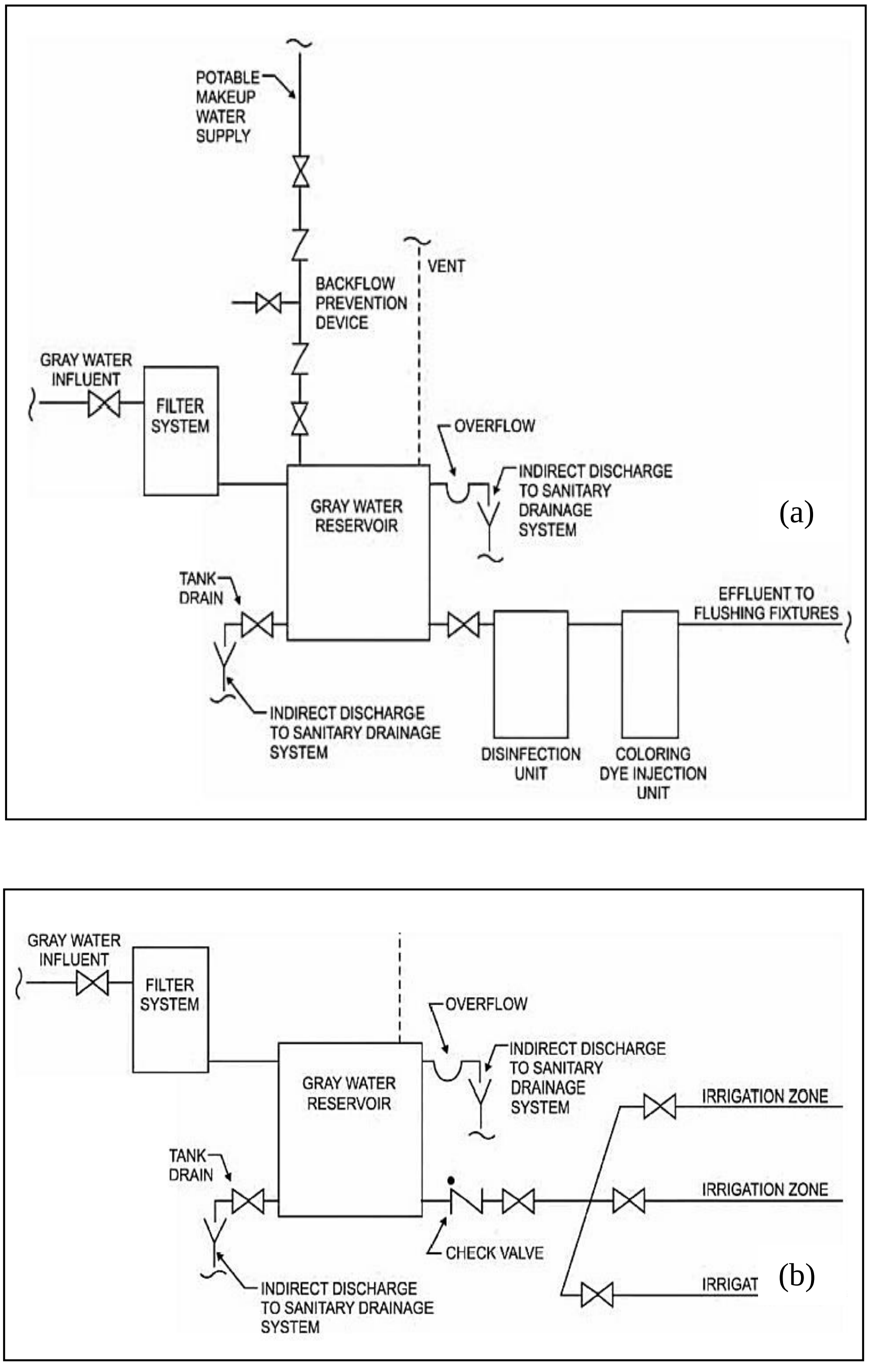

Figure 1: Typical Grey Water Recycling System and Reuse
(a) for Flushing, (b) for Irrigation (IPC, 2011)

The recycling process of grey water consists of a series of treatments including physical, chemical and biological treatment methods by passing it through membranes, disinfecting with chemicals and exposing it to ultraviolet radiations respectively. The reuse of recycled grey water offers many benefits including lowering the demand of fresh water, reduction of load on sewer system, energy conservation and good plant

growth resulting in reduction of overall expenses. Although the treated grey water can be used for cooling towers, chillers, industrial processes and construction activities. However, the most common use of treated grey water is supplying it for flushing toilets and irrigation of green areas (Chen et al., 2013). Common components of a grey water recycling system for the purpose of flushing toilets and irrigation are shown in Figure 1.
Having all these talks going around the world, Saudi Arabia has also started taking keen interest in water conservation and related technology. With a minimal amount of annual rainfall and having negligible surface and sub-surface water resources, the country has to rely wholly on the sea water available at its Eastern and Western coasts. To make use of seawater, Saudi Arabia has to put a lot of economic and energy resources for the process of desalination which is becoming a huge burden over the country's economy with each passing year. The water has been identified as one of the major parameter to be considered for sustainable construction activities in Saudi Arabia (Shaawat and Jamil, 2014) as well as a sustainable component for new buildings under environment criteria (Shaawat et al., 2018). Nolde (1999) shared his over a decade's experience in installing grey water recycling systems for toilet flushing in multistorey buildings in Germany. He experienced with various types of treatment units and concluded that good amount of water can be obtained after treatment which is sufficient for flushing of toilets without any hygienic risk or loss of comfort. In a similar kind of research studies, Gabarró et al. (2013) used grey water recycled water for irrigation in a sports centre. The sports centre had large green areas consisting of natural and artificial grass sections which needed to be irrigated on regular basis. The results obtained by them were good and promising quality of recycled grey water was produced by the treatment units.
It is the need of the time that water conservation measures be adopted for new construction and rehabilitation be done for the already constructed buildings to convert them to sustainable buildings. This research has been taken up with the same approach. Major aim of this research is to study the possibilities of converting old buildings into sustainable buildings in terms of water use. Water consumption is maximum in residential apartment and hotel buildings which also results in maximum grey water discharge which, if reused again, could save a huge amount of money as well as water resources collectively. This research would prove to be beneficial for the sustainable construction industry to motivate the concerned for providing recycling systems in new buildings and rehabilitate the older ones for the same.

2. EXPERIMENTAL PROCEDURES:

2.1 Data Brief:

The building selected for the case study is a multistorey hotel building located in Al Khobar, Saudi Arabia at 26° 19' 20" N and 50° 13' 00" E. The building comprises of 12 typical floors in addition to ground and first floor reserved for services and amenities. Each typical floor has a covered area of 9,150ft^2 and has 24 guest rooms. Figure 2 shows the 3D model of the hotel building created in Revit.
The hotel building also has a good amount of grass cover and irrigable area within the site boundary including some of it designated on roof terraces for aesthetics. Currently there is no grey water recirculation system installed at the building and the whole water requirement of the building is being met by using fresh water obtained from the seawater desalination plant located at the eastern coast of Saudi Arabia in Al Khobar

city.

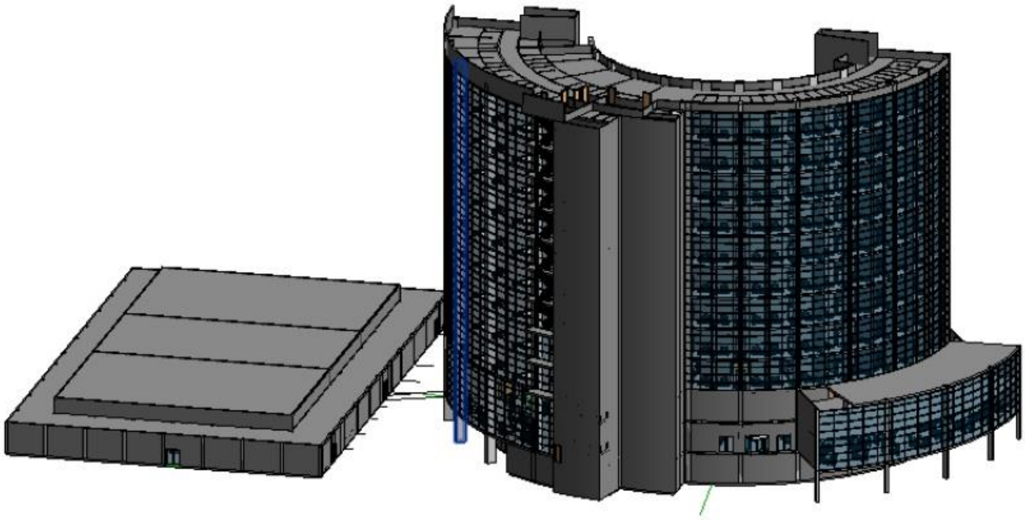

Figure 2: 3D Revit Model of Hotel Building

2.2 Research Work Methodology:

The work methodology consisted of a stepwise procedure. Figure 3 shows the workflow and the parameters required for calculation in each step. In the first step the total volume required for the building was calculated which included the demand for toilets, kitchen, laundry and cleaning etc. This demand was based on occupancy load of the building which was calculated to be 650 people maximum including staff and guests taken as at least two per room. Per capita mean water consumption was considered to be 58.6gpcd for indoor use (DeOreo et al., 2016). It is important to be noted here that this average water consumption does not include the water required for the purpose of fire fighting which is a non-recurring water demand and needs to be stored at a permanent storage facility.

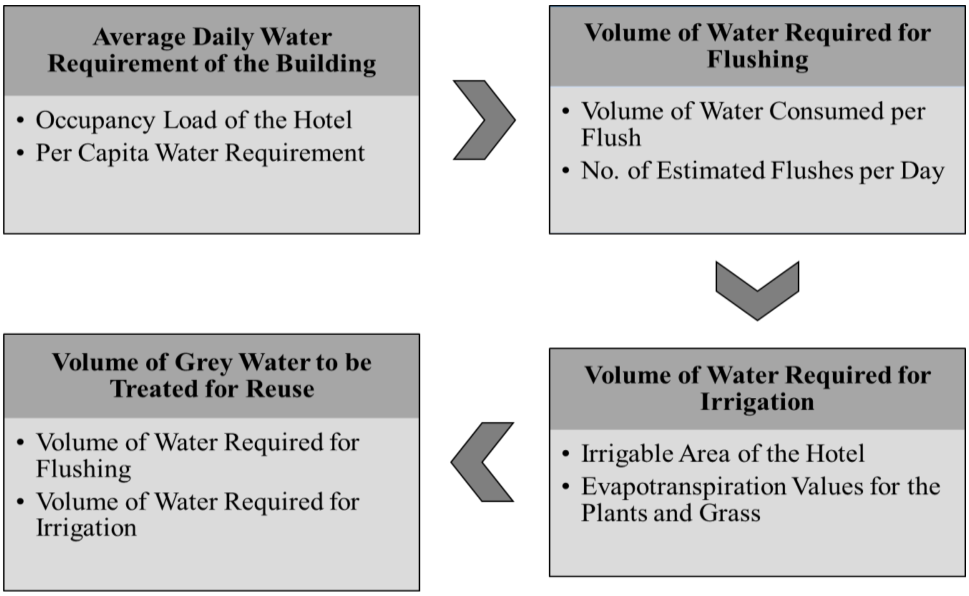

Figure 3: Methodology of Research Work along with Required Parameters

The first step of workflow was followed by the calculations for the volume of water required for flushing. For the purpose of conservation of water for the building, water saving equipments were considered to be used which consume the minimum amount of water required for the purpose. Hence for flushing, water closets with dual flush system were selected. A dual flush WC offers two options of flush volumes as required, the larger one delivering 1.6gal and the smaller providing 0.8gal per flush (IPC, 2017). An average number of flushes per day per person is found to be 5 out of which the large volume flush is found to be 1.5 on average (DeOre et al., 2016).
The third step consists of the calculation of water required for the irrigation of plants and lawn area of the building. This amount of water is not included in the indoor use of the building and it is considered that this volume of water is in addition to the average daily water requirement of the building. The total irrigable area for the building under study was found to be approximately 35,000ft^2. Potential evapotranspiration (PET) value of 0.3in/day was considered for natural grass and hot humid climate of the study area (Allen et al., 1998). The final step of the methodology was related to the overall calculation of water required and the amount of grey water which is needed to be treated and recycled for reuse. The amount of water available for recycling was also calculated in terms of percent of the total water requirement of the building.

3. RESULTS:

3.1 Amount of Water for Various Facilities

With a mean water consumption of 58.6gpcd the average daily water requirement of the hotel building was found to be 38,090gallons for an occupancy load of 650 people for indoor use only. Whereas flushing water requirement for the building was calculated as 3,380 gallons per day as shown in Table 1.

Table 1: Calculation of Water Required for Flushing

Parameter	Larger Flush	Smaller Flush
Occupancy Load	650	650
Flush Volume	1.6gpf	0.8gpf
No. of Flushes/day/person	1.5	3.5
Flush Volume	1,560gallons	1,820gallons

It can be observed from the above calculations that the amount of water required for flushing WCs makes up 9% of the average daily water requirement of the building per day. According to the report published by Water Research Foundation (DeOreo et al., 2016) the leakage losses/wastage and unaccounted for water can be taken as 12% and 4% respectively of the average daily water requirement. In the next step, by using the criteria of irrigation water demands, the volume of water required to irrigate the 35,000ft^2 lawn and green area of the hotel building was calculated to be 6,550gallons making it 17% of the average daily water requirement.

3.2 Calculation of Grey Water Reuse

By putting all statistics together in Table 2, we can determine the amount of grey water being released daily by the hotel building. It is seen that almost 75% of the average daily water requirement of the building is being released as grey water. This amount includes used water coming out from dishwashers, kitchen sinks, laundries, bathtubs, lavatories and cleaning floors etc. As already discussed that the reuse of grey water shall be considered for flushing toilets and irrigation for this research.

Table 2: Calculation for Grey Water Reuse

Source	Volume of Water	Percent of Average Daily Water Requirement
Average Daily Water Requirement/day	38,090gallons	
• Water for Flushing WCs/day	3,380gallons	9%
• Leakage Losses/Unaccounted for Water per day	6,100gallons	16%
• Grey Water Release/day	28,610gallons	75%

These two water facilities need a total of 26% of average daily water requirement equalling to 9,930gallons of water per day. Hence if a grey water recycling plant of capacity 12,000gpd is installed having a mean yield efficiency of 0.85 (Abdel-Kader, 2013) at the hotel building, that would reduce the consumption of water by 26% resulting in conservation of water and economic resources. The excess water, in the form of effluent, shall be discharged from the plant and connected to the sewer system around the building. This approach of recycling and resupplying the water to the designated usage is represented as a schematic diagram in Figure 4.

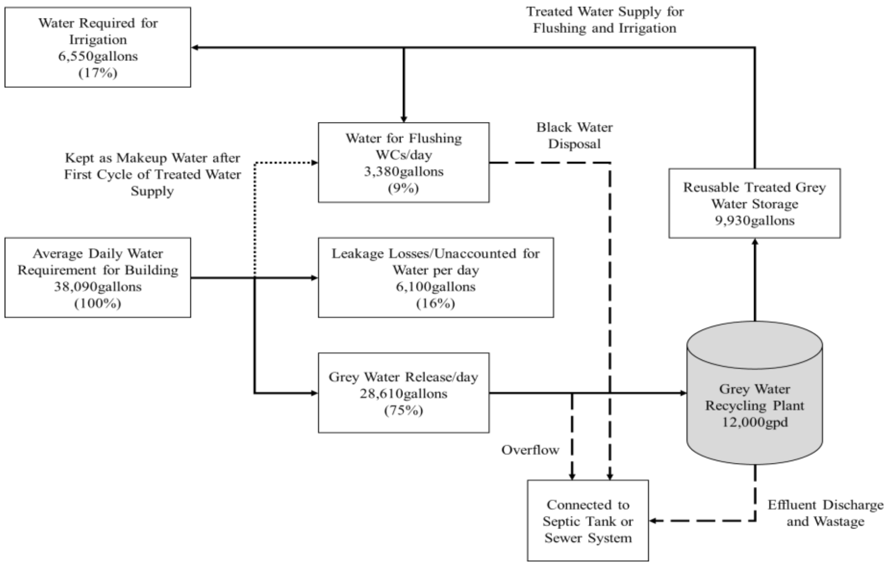

Figure 4: Schematic Diagram for Grey Water Recirculation System and Amount of Water Saved for Hotel Building

4. CONCLUSIONS:

A multistorey hotel building was put to test to study the feasibility of installing a grey water recycling system for the purpose of saving water in order to play a role towards sustainability. Various calculations were performed regarding volume of water required for flushing and irrigation in addition to average daily indoor water requirement of the whole building. Expected volume of grey water released from the building was calculated based on the preliminary calculations which helped in obtaining the required amount of grey water to be treated. It was observed that almost 75% of the used water released by the building is grey water amounting to 28,610gallons, whereas only 26% was required to be treated for reuse in flushing and irrigation processes. A grey water recycling plant of capacity 12,000gpd was proposed for the hotel building. After the first cycle of the treatment the building would be able to save 26% of fresh water which would help them in saving expenses as well. It is also recommended to the hotel authorities to extend the recycling plant by adding specific equipment and prevent the remaining part of grey water going to waste. It shall produce additional treated water which could be used for cooling towers and chillers resulting in further reduction of cost and freshwater consumption.

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Paper ID: 503

Analysis of Existing Hydraulic Structures with Crack and Pore Water Pressure using Reliability Methods

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Abstract

The safety and reliability of German hydraulic structures, like ship locks, weirs, etc., have to be verified from time to time or for the decision of rehabilitation or renewal, This contribution presents a probabilistic methodology for the uncertainty quantification of relevant parameters and failure mechanisms for the evaluation of reliability levels of existing hydraulic structures. For a ship lock chamber wall, a gravity wall construction, a dominant action crack and pore water pressure is considered and its influence on the structural reliability is discussed. The case study is benchmarked for requirements posed by the latest European standards and German guidelines. For the application of the proposed methodology a typical structural geometry, material and load system for a ship lock wall is considered. Overturning and compressive strength have been investigated as the exemplary limit states. The results indicate a decrease in reliability levels in case of crack and pore-water pressure as an externally applied force. The differences are mostly influenced by the difference of levels of water within the chamber structure. Additional reliability based sensitivity analysis indicates that friction angle and concrete weight have the highest impact on the reliability for the considered parameter uncertainties and the limit states. The investigations are part of a research project in which probabilistic analyses for existing hydraulic structures shall promote a decision instrument so as to rehabilitate existing hydraulic structures and to support sustainability aspects in structural engineering.

Keywords: Hydraulic structures, Reliability analysis, Probabilistic modeling, Structural analysis, crack-pore water pressures.

1. INTRODUCTION:

Three main progressions are seen in structural engineering design for the evaluation of reliability and safety. These methods include allowable stress design (ASD), semi-probabilistic design (partial safety factors) and the latest full probabilistic design (fib 2016). Reliability can be regarded as a key element of the sustainability of a structure. While assessing concrete hydraulic structures, it is often noticed that previous structural designs and verifications did not consider the crack and pore water pressure as an externally applied force (DIN 19702 2013). These forces can be considerable when global structural safety is considered in a critical section. This contribution reviews and applies an analytical formulation recommended by the German standard for solid hydraulic structures (DIN 19702, 2013) and guideline for the verification of existing hydraulic structures (BAW 2016) for a typical geometry and load combination on a ship lock structure and performs a probabilistic analysis. The relevant codes and standards recommend a minimum target reliability level of β_T =3.8 for Ultimate Limit State (ULS) and β_T =1.5 for Serviceability Limit State (SLS) which must be achieved for the assertion of safety and reliability for at least 50 years for buildings (EN 1990, 2010) and at least for 100 years for hydraulic structures (DIN 19702 2013). Although several Limit State Function (LSF) need to be fulfilled for a concrete hydraulic structure (Tahir et al 2016) in the first part of the research we have selected only two Ultimate Limit States (ULS) and two Serviceability Limit States (SLS) for the proof of the concept. First a LSF uncertainty quantification of parameters is conducted which serves as input into reliability analysis methods. Additionally, the analysis was conducted considering crack and pore water pressure formulations. To demonstrate the proposed methodology a case study was performed for a typical geometry and loading system for a ship lock water component for a predetermined cross section.

2. METHODS:

2.1 Structural design analysis of hydraulic structures:

For the evaluation of structural safety and reliability of existing hydraulic structures several different verification and limit state functions need to be fulfilled. The Eurocodes as European standards (i.e. EN1990 2010) classifies the limit state function (LSFs) into Ultimate Limit States (ULS) and Serviceability Limit States (SLS). These LSFs consider several possible failure modes are varying from stability, geotechnical to structure's internal stresses. This contribution considers only four of the recommended verifications in the guidelines (BAW 2016) for existing un-reinforced/lightly reinforced concrete hydraulic structure subjected to several loads and load combinations. These include overturning failure as a global safety measure (ULS), compressive failure for internal stress stability (ULS) as well as a joint opening stability where the joint opening must not exceed more than 50 % of base length (SLS), and see (BAW 2016). The figure 1 (left) indicates the LSFs for each ULS and SLS whereas the figure 1 (right) shows the force system for a typical ship lock gravity wall with earth pressure, groundwater and chamber water pressures. Section A-A was considered for structural and reliability analysis.

2.2 Crack and pore water pressure:

Components of the hydraulic structure remain in contact with water and are hence subjected to crack water pressures in cracks and open joints and pore-water pressures in the internal sections. The resulting additional forces and stresses from these water pressures have not been considered in previous design up to about 1980, therefore their inclusion is essential to update the safety levels of existing structures and to ensure their reliability for the remaining service life.

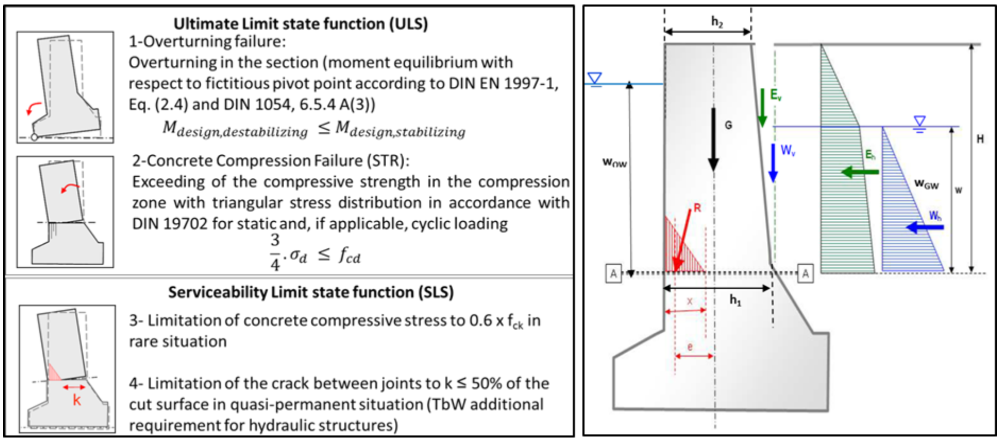
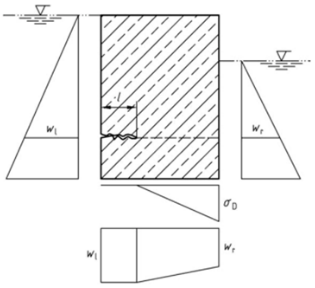


Figure 1: (Left) considered Limit state functions (ULS & SLS); (Right) Typical geometry and load system for a ship lock wall structure (adapted

Figure 2 and the following equation indicate the stress distribution of crack and pore water pressures within a hydraulic structure with varying water levels on both sides of the section.



System sketch for internal water pressure of components in accordance with [DIN 19702:2013-02].
Legend:
l Crack depth
w_l Water pressure left (at crack level)
w_r Water pressure right (at crack level)
σ_D Compressive stress

Figure 2: Stress distribution for water pressures in internal cross section, (DIN 19702 2013)

Equation (5) of DIN 19702 provides the depth of the pressure zone considering the internal water pressure, where x_{wd} is the depth of the cracked section equation (1) and e_d is the external eccentricity (Moment/Normal forces), h is section width and $\overline{\sigma}_{wd}$ is the ratio of the water pressure to the design normal force (drawn water pressure).

$$\frac{x_{wd}}{h} = 3 \cdot \left(\frac{1}{2} - \frac{\frac{e_d}{h}}{1 - \overline{\sigma_{wd}}} \right) \qquad (1)$$

Equations (6) and (7) of DIN 19702 provides the resulting modification factor for design internal forces considering the internal water pressure, for modified design normal force (N_{wd}) equation (2) and modified design moment (M_{wd}) across a section equation (3).

$$N_{wd} = \left(1 - \overline{\sigma_{wd}} \cdot \left(1 - \frac{1}{2} \cdot \frac{x_{wd}}{h} \right) \right) \cdot N_d \qquad (2)$$

$$M_{wd} = \frac{1}{1 - \overline{\sigma_{wd}}} \cdot \frac{N_{wd}}{N_d} \cdot M_d \qquad (3)$$

2.3 Reliability analysis:

The vital part of a reliability assessment is the calculation of the probability of failure P_f related to a specific loading situation and a limit state function equation (4) It is defined as

$$P_f = \Pr[g(\underline{\mathbf{X}}) \leq 0] = \int_{g(\underline{\mathbf{X}})\leq 0} f_{\underline{\mathbf{X}}}(\underline{\mathbf{x}})d\underline{\mathbf{x}} \qquad (4)$$

where *X* is a random vector of input parameters with joint probability density function $f_X(x)$ and *g()* is the limit state function (LSF). Probabilistic modelling of important parameters in a limit state is an essential part of the evaluation of the reliability levels. Following probabilistic models for parameters were considered using literature indication for the case study.

Table 1: Probabilistic models of random parameters for LSFs

	Parameter	Distribution	Mean (μ)	Coefficient of variation	Standard Deviation (σ)	Source
1	Concrete strength (f_{ck})	Lognormal	12	15%	1.80	(JCSS 2001)
2	Concrete weights (γ_B)	Normal	23	5%	1.15	(JCSS 2001)
3	Soil weights (γ_{BG})	Normal	20	5%	1.00	(JCSS 2001)
4	Friction angle (φ)	Lognormal	35	8%	2.8	(JCSS 2001)
5	Groundwater level (GWL)	Lognormal	5	10%	0.5	Field Data

First Order Reliability Method (FORM)

Currently, several methods are used for the evaluation of the reliability which could be classified into approximated methods and Monte-Carlo based simulation methods. This contribution will employ the approximation based method i.e. the First Order Reliability Method (FORM), (Rackwitz and Fiessler 1978), also known as first-order second moment approximation. The two main advantages of employing FORM is its numerical robustness and the allocation of reliability based sensitivity analysis as a byproduct without additional calculation. Essentially FORM defines a Taylor series expansion of the limit state function g(x) to the first order at the design point. This enables an efficient solution for estimating the reliability index β. This could be visualized in figure 3 (a) below, where the joint probability distribution of two random variables and the limit state function g(x) = R-E define the safe and the failure regions. Figure 3 (b) indicates the process of searching the failure point and hence the reliability index (β).

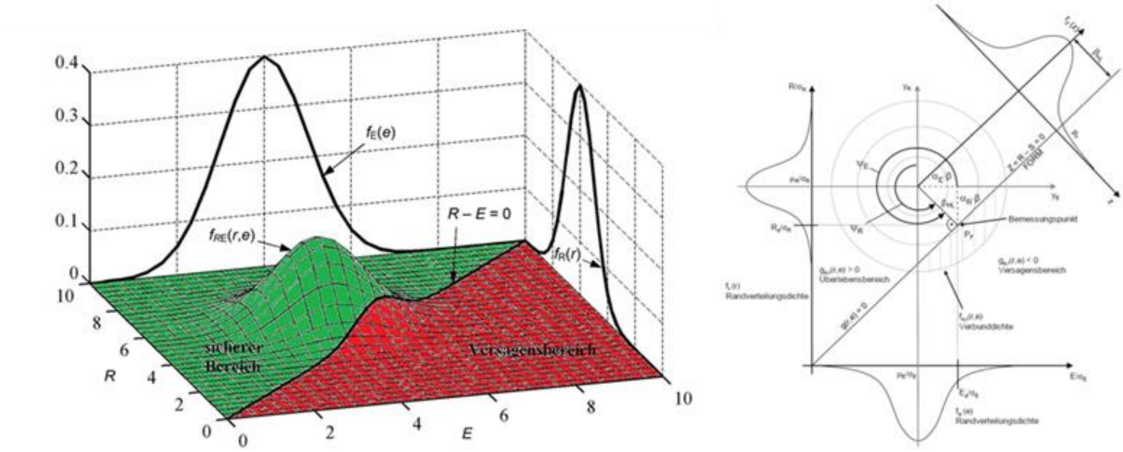


Figure 3: (Left) Representation of the Joint PDF with limit state function R - E = 0 and failure region (red) (Hausmann 2007) (Right) Design point, reliability index β_{HL} and Joint probability density of E and R in the standard normal space (Braml 2010).

3. CASE STUDY AND RESULTS:

As discussed in the sections earlier the reliability analysis using FORM method was conducted for a typical unreinforced (plain) concrete ship lock chamber wall , loads and limits states (ULS and SLS) as indicated in Figure 1. The analysis was conducted for a wall thickness of 3m, a wall height of 8 m, a groundwater level at 5 m height, varying water levels in the chamber and for two major load cases, with and without considering crack and pore water pressures, see figure 4. For Ultimate Limit State (ULS) it was observed that reliability levels are (nearly) achieved without crack and pore water pressures (PWP), considering the target safety level ($\beta_T = 3.8$) by (DINEN1990 2010), indicated in Figure 4 (A), whereas a decrease is seen when PWP was considered in the cross-sectional analysis. The difference varies with changing the water level in the chamber and the limit state function as indicated in Figure 4 (C). A difference of 100 % is seen since cases with chamber water levels less than 5 m show failure for gaping joint SLS verification. Regardless of ULS or SLS verifications an exponential increase of reliability is seen for cases where the water level is greater than 5m. This could be contributed to the fact that considered groundwater level is at 5m and since water in the chamber is a stabilizing force, the load cases with higher chamber water level have significantly higher structural safety. The same reasoning could be applied to the reduction in the difference in reliability levels for cases with and without PWP as indicated in Figure 4 (C). Further investigation shows that this point could be due to the limitation of the analytical method prescribed by the existing crack and pore water methodology in codes and guidelines.

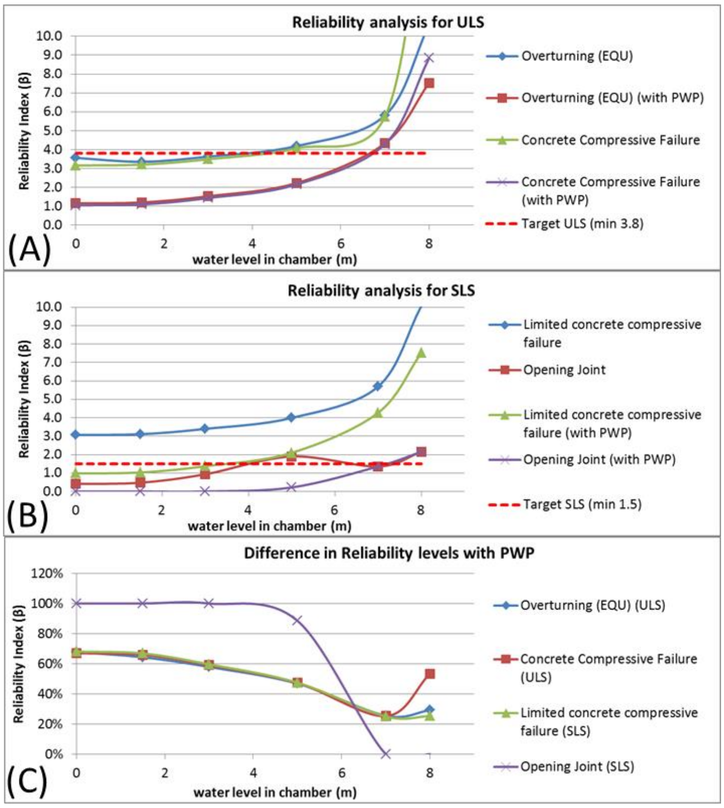


Figure 4: Results of Reliability analysis for ULS and SLS, with/without PWP

One of the most significant advantages of the FORM method is the ability to provide sensitivity analysis without any additional computational effort. The so called alpha values, indicate the extent of the sensitivity of the parameter on the reliability. The performed sensitivity analysis for the limit states “Overturning” indicates friction angle, concrete weight whereas and groundwater are the critical parameters as shown in Figure 5.

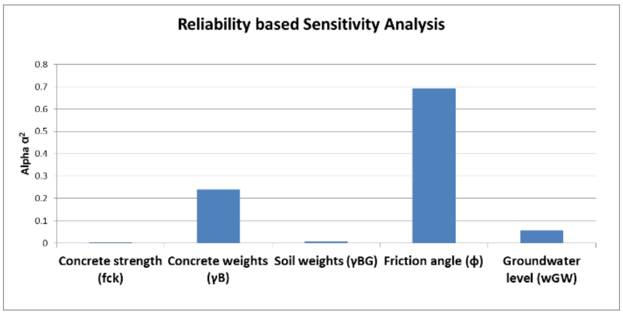


Figure 5: Reliability-based sensitivity analysis of Overturning (EQU) as ULS

4. CONCLUSIONS:

The contribution presented a methodology to incorporate crack and pore-water pressures, a hydraulic structure specific load into reliability assessment framework. An analytical method developed by BAW was employed and the effect on reliability levels of ULS and SLS were investigated using FORM method. It could be concluded that consideration of crack-pore water pressures decreases the overall reliability of the structure. The difference in reliability levels with and without inclusion of pore pressures decreases with decrease in difference of water levels in ground and chamber. FORM based sensitivity analysis indicated friction angle, concrete weight and groundwater level have the most influence on reliability for overturning ULS. A certain asymptotic relationship is seen in reliability levels after the water levels in chamber and ground are equal. This trend requires more research, for which the authors suggest a validation through non-linear iteration based method rather than analytical approach. The current study presented a workflow and case study for an idealized system, application to actual structures is expected in future work with additional limit state functions and field data.

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Impact of Raising Mangla Hydropower Dam on Rural Land and Crop Production Pattern

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Abstract

Dam induced displacements causes several damages to the life of inhabitants in different areas of the world. Many people lose the ability of returning towards their lives. It is necessary to establish mandatory policies for the dam induced affected people. This study presents the consequences of raising Mangla Dam, mainly the after effects on land and crop production pattern. In order to attain the required information, different approaches were employed like questionnaire, in-depth interviews, focused group discussions and direct and indirect observations. After going through analysis of the conducted field work and collected data, the obtained results indicate that around 803 acres of residential land, 4,358 acres of agricultural land, and 4,299 acres of barren land was affected due to this project. The crop production also decreased due to the land becoming less fertile and the people started preferring to grow different kinds of vegetables more than the congenital crops like millet, wheat and maize.

Keywords Resettlement- Mangla Dam- Dam Induced Displacement- Raising of Dam- Rural livelihoods- Population- River Jhelum- Pakistan

1. INTRODUCTION:

Pakistan has vast resources of water. A huge belt of mountainous region is found in north east of Pakistan. The two main rivers of Pakistan are the Indus and the Jhelum. These two rivers forms a huge network of irrigation system to fulfill the water requirement for drinking purpose and crop production. In order to accomplish the demand of water in low flow season, storage structures are required. Mangla and Tarbela are the two main multipurpose dams in Pakistan. With the passage of time these water storage structures reduced their capacity to store water due to the huge amount of silt deposited. Hence the construction of new dam or raising of existing dam is required to increase the volume of storage structure in order to attain the maximum amount of water. When raising of the dam takes place, more land is acquired around the existing dam. In this way, the people of nearby area gets affected and they forced to displace from their land to another place which is called dam induced displacement. This matter is of great humanitarian concern which displace thousands of people, directly affects their houses, agricultural land and sources of income etc.

Mangla dam was completed in 1967 having storage capacity of 5.2 MAF. In 2004, the raising project was started to prevail over the required amount of water for irrigation and energy to generate electricity.(A. Ahmed, 2007). The central source of depletion in storage is the deposition of sediments in dam which reduced 80% of its storage capacity estimated in 2005 by using hydrographical survey (Abbas, 2007).Therefore it is mandatory to built new storage structure or raise the existing dam. Due to fast development in dam sector, many impulsive displacements takes place in present era. It was estimated that around 39.5 to 79.5 million people move from one place to another (Icold Ahead, 2013). The same case study is presented here regarding the resettlement aspects of dam raising.

Resettlement due to construction of dams displaced millions of people around the world. It was estimated by World Bank that 10 million people were displaced due to these kind of development projects in last decade of 19th century (Cernea, 1997). Dam induced displacement contributes in the displacement to the largest number of people (approximately 40%) as compared to other infrastructural development projects. Several studies (Cernea, 2003) shows that DIDR negatively affect on the living values of the displaced communities. Many people loss their valuable assets, move towards economic poverty, their social and cultural ethics and may never be able to return towards their lives (Maldonado, 2012). In Dam induced displacement and resettlement only few projects had positive outcomes for the peoples (Ty, 2015).Dam induced displacement may provide a positive way to move forward by providing better facilities and compensations of the loss of the affected people.

The results of this study have several implications for theorist and practitioners. This research has found results which support the need for improvement in the planning, design and execution of resettlement policies. Elements such as active participation in decision-making should be underlined in future policy reformation.

2. STUDY AREA

Jhelum river is the second largest river in Pakistan. Mangla watershed region lies at 73°3' to 75°35' E and 32°59' to 35°10' N. It consist of mountainous region having snow cover, agricultural lands, barren land, grassy lands and forests. The storage capacity of Mangla dam reduced to only 5764.31 million cubic meter at the time of raising (M. J. Butt, 2011).

The tributaries of Jhelum river such as Poonch, Neelum and Kunhar plays a vital role to draw silt in various pockets of Mangla dam. Water and Power Development Authority collects the flow data at Upper Jhelum, Lower Jhelum, Jari, Main and Kanshi. Administrative authority of Pakistan for dam displacements proposed a plan using the State Resettlement Policy 2000 in collaboration with International Finance Corporation. Resettlement Action Plan of Mangla consisted of five sectors of Azad Kashmir, New Mirpur city and four towns i.eIslamgarh, Chaksawri, Punjab and Dudial. These five sectors are presented in Figure 1.

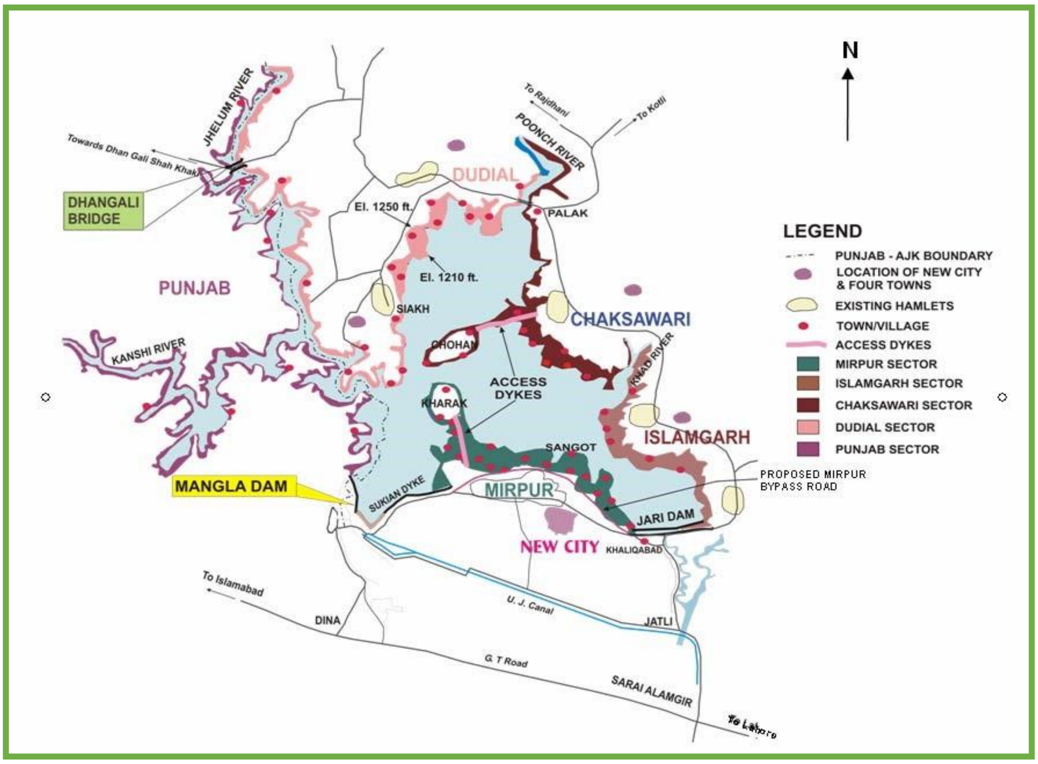

Figure1: Sector-wise Location of Affected Area (Chaudry, 2007)

3. METHODOLOGY

This study is based on questionnaire and oral opinions of the people of the five sectors of Mirpur Azad Kashmir settled in new cities and towns after resettlement program of Mangla dam raising. Complementary approach is used by combining all methods in order to obtain precise results. Preliminary survey of villages was conducted to visualize the location. A total of 100 households were questioned in detail by using questionnaire, in which 30 households were located in Mirpur and 70 of them were questioned in small villages of Islamgarh, Dudial, Chaksawri and Punjab. The questionnaire focused on the surroundings of the displaced people such as community characteristics before and after resettlement. Information is also collected through interviews to understand the feelings and insight thoughts of the project affected persons. Interviews were also held with the community leaders, government officials and the concerning hydropower development authorities to gain in-depth knowledge on the implemented resettlement plan. Last but not the least statistics papers and reports on resettlement action plan were collected from the consultants and contractors of the project.

The collected data was analysed in connection with objectives of the study and the resulting analysis is presented in forms of charts and tables in the following section.

4. RESULTS AND DISCUSSIONS

The study of factors involved in the process of resettlement greatly contributes for reader to understand the planning and execution of mega structures causing mass displacement of people. The process of resettlement has different phases in which every person of Mirpur, Islamgarh, Dudial, Punjab and Chaksawri was directly or indirectly involved. The information from the verbal sources of the resettled area, the village leaders, the community and the experience of the local people helps to understand the development and implementation of the whole resettlement process. In this study only impact on land and the crop production pattern before and after resettlement has been discussed.

4.1 Impact on Land

On the basis of field survey and other parameters such as interviews from concerned authorities, local people, government officials and historical records, it was estimated that more than 9,000 acres of the land was affected on the periphery of the dam out of which 4,299 acres of barren land is also included. The most affected land was residential and agricultural land (Chaksawri and Dudial) having area of 803 acres and 4,358 acres, respectively (Table 1). The graphical presentation of type of land affected in different sectors is shown in Figure 2.

Table 1: Raising Impact on Land (Source: Fieldwork Mangla, 2017)

Sector	**Mirpur**	**Islamgarh**	**Chaksawri**	**Dudial**	**Punjab**	**Total Area (Acres)**
Residential Land (Acres)	240	149	242	153	19	803
Agricultural Land (Acres)	860	617	1151	1354	376	4358
Barren Land (Acres)	2173	1533	1418	2173	2955	4299

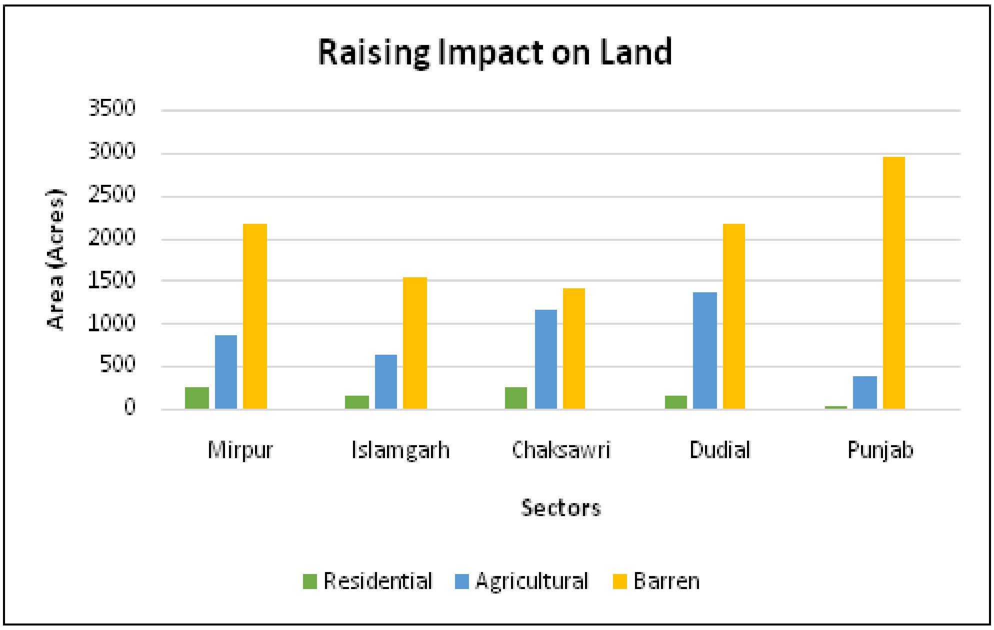

Figure 2: Type of land affected in different sectors

4.2 Impact on Crop Production Pattern

On the basis of field survey and other parameters, analysis shows that the people of affected areas cultivated millet, fruits, wheat and various types of vegetables on large tracts of productive land to meet their need of food and nutrients. 54% of the villagers of each sector responded that their food requirement from the previous land satisfied their needs. After resettlement, they responded that the existing land doesn't fulfil their food requirements due to infertility and other multiple changes in resettled area. In order to generate income and sustain their family expenses, the people sold their own part of food production which they used at their homes.

The households of these villages grew different type of crops like wheat, pulses, millet, maize, rice and vegetables out of which 28% of villagers grew wheat and 12% maize before resettlement. This production reduced to 23% for wheat and 6% maize after resettlement. Only the increase in production of vegetables from 11% to 20% was estimated after resettlement (Table 2). This indicates that the sources of income from crop production decreased after resettlement and also the change in crop production pattern. This change in crop production pattern is graphically presented in Figure 3.

Table 2: Production of crops before & after Resettlement

Crops	**Respondents in % who grew crops Before Resettlement**	**Respondents in % who grew crops After Resettlement**
Wheat	28	23
Maize	12	6
Millet (Bajra)	15	7
Pulses (Beans)	26	18
Rice	8	2
Vegetables	11	20

A major shift towards increased interest in growing vegetables after resettlement was observed in the cropping pattern.

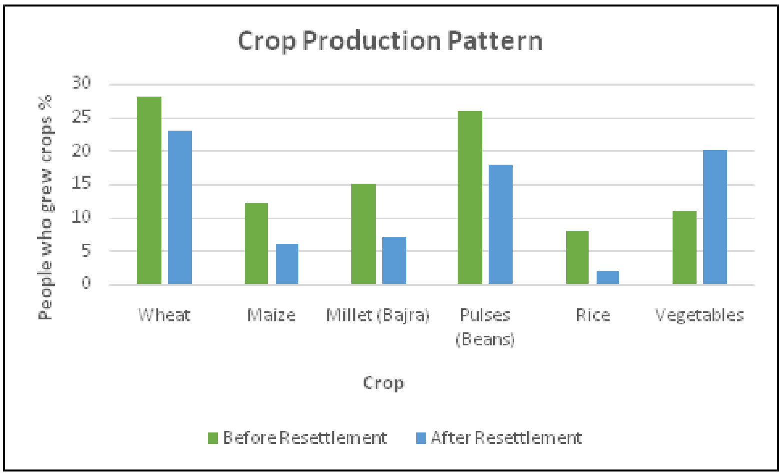

Figure 3: Crop production pattern before and after resettlement

The above presented analysis of the results reveals that the raising of Mangla Dam significantly affected the life of people residing in the area. A large number of population had to leave their native towns and move to new places. The process not only changed the cropping policy but also affected the land use pattern in the area. Since agriculture had been the major source of income for the residents, this change also had impact on social structure and other life factors of the displaced people. Future studies are encouraged to analyse the impact of the raising process on these social factors.

5. CONCLUSIONS

In this study, two main social aspects of the resettlement program of Mangla dam raising has been analysed. On the basis of the results obtained from the conducted study, following conclusions can be drawn:

- 9000 acres of land was affected including residential, agricultural and barren land.
- Due to the change in crop production pattern the sources of income was also affected. As the soil was not so fertile as compared to the previous one, the crop production decreased and the people started to grow different kinds of vegetables more than millet, wheat and maize.

6. RECOMMENDATIONS

This type of dam induced displacements badly affects the living ways of the inhabitants of these small areas due to the negligence and mismanagement of such kind of resettlement policies. There is a need of time to improve the planning, designing and implementation of these policies according to the "National Resettlement Policy of Pakistan 2000". Similarly active participation of local leaders in decision making can play a vital role for the improvement in these policies. Future research may focus on the application of adaptation of new strategies for the resettled communities. The available resources like Government authorities and NGO's must be used pro actively to protect people from poverty by helping them in some way or the other. Training must be provided to the people on former resettlements and experiences must be shared to create successful adaptation and to save people from impoverishment.

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Evaluating the Change of Water Table Position for Sustainable Development: "A Case Study of Ghazi Barotha Hydropower Project"

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Abstract

The primary objectives of this study was to evaluate the effects of the construction and subsequent operationalization of 1450 MW power generation facility "Ghazi Barotha Hydropower Project", on water table position in the surrounding areas. To assess the effects, the data comprising of elevation values of water column in open wells set up at different locations in vicinity of the facility have been collected. For a precise and comprehensive analysis, the study area was divided into 12 clusters. The hydrograph of each cluster between average water column elevation values and years (from 1997 to 2008) have been analysed. By comparing the values, before and after building the facility, it has been concluded that there are significant effects on positions of water table on some area, whereas in other areas, no substantial effects have been observed.

Keywords: Ground water, Ghazi Barotha Hydropower Project, Hydropower, Water depth, Dam, Water Table.

1. INTRODUCTION:

Hydropower is the dominant source of renewable energy generation globally which is producing 71% of all renewable generation with having total installed capacity 1064 GW in year 2016 (WEC,2016). It is 16.4% of total electricity produced in the world from all the sources. At the end of the 20th century, about 45000 large dams (above than 15 m) with a total reservoir area of about 500,000 Square km have been constructed to produce hydroelectricity, irrigation purposes, and for drinking water storages (Gleick, 1998; WCD, 2000). These dams contribute a major source of economic growth of industry with addition of progress of rural countries, and are considered as "sustainable hydropower" or "green hydropower" (Truffer et al., 2003). The sustainable practices in development of hydropower projects have significant advances. These elements have added to enhance the willingness and acceptance for financial donor and policymakers to encourage the development of hydropower by providing the investment and giving suitable atmosphere by policymakers. Research have been extended widely to access the effects of dams or hydropower projects on water table position of their adjacent area. Some of the researchers have focused on positive effects of micro dams on water table encashment, whereas, some have studied effects of dams or reservoir on groundwater quality. The possible effects have been explained as follows;

1.1 Impacts of hydropower projects on groundwater

The dams and hydropower may change groundwater quantity, its flow pattern or its quality. The building and operationalization of hydropower project is connected with a serious environmental issues such as diversions of flows, interruption to migration of fisheries, hydro-cresting, flushing of reservoirs and flooding of lands and changes in bio-geochemical cycling (Friedl et al., 2002). Seepage has been observed the main impact on the adjacent areas of the reservoir. Consequently, the hydraulic grade line of the area goes up, considerably, imposing acute impacts on crop growing, drinking & irrigation water demand and hydraulic structure's foundations. The groundwater levels before and after the construction of dam is best indicator to understand the effects of dams on groundwater. It has been observed that groundwater levels are increased after the impounding of reservoir. About 1.5 m increase in level of groundwater in North of Chennai has been observed after the completion of the check dam (Parimala Renganayaki et al., 2013). The recharge of aquifers boost ups due to seepage and elevated rate of infiltration. Moreover, the zones, which primarily was infertile, get their volume of recharge and becomes fertile. Similarly, over-recharge has also observed lead to water logging, the observed phenomenon in the vicinity of Tarbela dam (Tariq, 1993).The water table of the surrounding area of Khanpur dam is 75 feet deep and it goes to 150 feet as go away from dam site. Due to khanpur dam, the groundwater aquifer of the close area is sustained (Naeem et al., 2012). These dam structures can facilitate the recharge of ground water aquifer above 80% under the variable conditions of flood events, magnitude of storage and specific conditions of site. Further, some of the studies have also revealed that aquifer re-charge can be attained up to ninety five percent (Haimerl, G. 2000). It has been observed that groundwater quality is increased by increasing the levels of reservoir. The ground water inflow and outflow quantity was 2.5 times greater of Budush dam when reservoir level was 307 m.a.s.l. as compared when the reservoir stage of 245.5 m a.s.l. (A. Adili., 2014).

1.2 Impacts on groundwater flow pattern

The hydraulic head is caused to flow groundwater upward, downward or horizontally. The rivers, streams, precipitation and reservoirs are main source of groundwater recharge. With increase in infiltration the groundwater moves within soil having low permeability region. The excess seepage from dams not only increase the level of groundwater but also it caused the reason of change of flow pattern of groundwater. The variation in flow pattern is mainly due to the upsurge of water elevation of the reservoir, which in fact, creates the raised difference in hydraulic gradient amid the storage and the neighbouring aquifer. The groundwater direction was towards the river side before the construction of Budush Iraq dam and after the construction of dam, the reservoir gained net recharge to the aquifer (A. Adili, 2014). The construction of any structure that changes the groundwater regime can have potentially un-expectable results to the structure itself and the local hydrology (Milanovic, 2002).

1.3 Impacts on groundwater quality

Dams and hydropower also create impact on groundwater quality by infiltration of reservoir water into groundwater. The quality may be evaluated based on the values of total dissolved solids quantum or ionic concentrations of the ground water (Kunkle, G. R. 1965).The infiltrated water from the dams contain 0.4 to 47 mg per litre of zinc (Zn), 1100 to 1800 mg per litre of sulphates and up to 7.4 mg per litre of lead (Pb) (Adamczyk el al., 1994). The process of Eutrophication occurred in reservoir water and that contaminated water infiltrated to the groundwater. The pore water have found containing contaminants fractions and dissolved organic carbon infiltrated to the ground water (Wildi et al., 2004).

Literature review reveals that there are long term effects of change in water table position on lives of people in the area of the project, their living & housing, agriculture, livestock, social & health, ecology, environment and communication. Therefore, it is important to evaluate the change in water table position in the "Ghazi Barotha Hydropower Project (GBHP)" vicinity areas for sustainable development.

This case study may facilitate in future, at planning and designing phases of similar kind of hydropower projects, in early stage assessment of impacts, keeping in view the impounding effects on the position of ground water table. This may be helpful to mitigate the possible adverse impacts on the ecology of the area. Further, it may also help to add remedial measures against the possible impacts at the planning phase in the projects.

2. SCHEME OF STUDY:

2.1 General Scheme:

The GBHP utilizes the available head of mighty Indus in the middle of the downstream of Tarbela Dam Hydropower Station and the joining point of the River Indus with River Haro. The facility is of 1,450 MW installed generating capacity, comprising of 5x290 MW generating units.
There are three major components of the Project.

a) **The Barrage;** Located near Ghazi in the downstream of the Tarbela Reservoir at distance of 7 km, regulates the discharge from Tarbela.

b) **RCC Power Channel;** measuring about 52 km long intakes discharge from Head Regulator at barrage of 1600 cumecs.

c) **The Power Complex;** Located near Barotha, Operates under the discharge from Tail Regulator.

2.2 Methodology:

The groundwater was extracted from open wells manually and by Persian Wheel operated through diesel pumps . The Unconfined Aquifers were the Principal aquifers and variation in hydraulic grade line i.e water table position (depth) was taken as an indication for the study of the impacts. The depths of water table was recorded on a regular basis, the data was comprised of water column depths. The change in water table position was identified through the aforementioned data by comparison of levels of wells before and after the construction and operationalization of the Project.

2.3 Wells Inventory:

Identification of a number of wells was made with the help of authorities at the Project. Out of total of 127 wells, 37 and 39 were located along the left and right bank of Indus River respectively. The remaining were near power channel/complex and at barrage. The following information was collected from the wells;

a) Well ID No.
b) Well Location Coordinates
c) Well Type

2.4 Mapping of wells and Data Collection:

Mapping of the wells was carried out to get an overview of the area. The mapping done on base map of the project is as shown in Figure 1 & 2. The average water column of each well was calculated from 1997 to 2008 for each season i.e. summer and winter season of a year. The hydrograph was then plotted between the average water column depth of wells and time series on seasonal basis from 1997 to 2008 .The average rainfall intensity recorded in mm was also added on secondary axis in the hydrograph to observe the results that how much changes in water columns depth in wells were varied with respect to rainfall. The changes in groundwater depth before and after the construction of project can be accessed by variation in hydrograph. The color variation in hydrograph was used to distinguish the pre and post construction periods; grey color is used for 1997 to 2002 period whereas blue color was used in hydrograph for year 2003 to upward which is pounding year of the project. The wells were categorized into different clusters (groups) to get the better understanding of the effects of project on surrounding plain.

The clusters arrangement is designated in Figure 2.

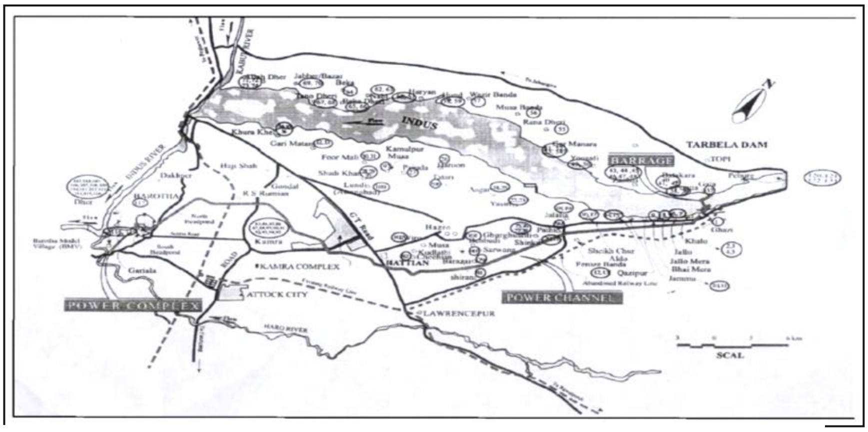


Figure 1: Mapping of the Wells

The detail of clusters is as under:

Sr. No	Location	Cluster Name	Area	Quantity of Wells
1	**Indus River** (Left Bank)	Left Bank-01	Ghaziabad to Qazipura	14
		Left Bank-02	Aladowa to Asgharabad	12
		Left Bank-03	Shaaddi khan to Haroonabad	11
2	**Indus River** (Right Bank)	Right Bank-01	Gallaya and battakara	06
		Right Bank-02	Zaryabi and Hyndalla	18
		Right Bank-03	Hyriana and Allah Dheira	15
3	**Power Channel** (Right /Left Bank)	Power Channel-01	Ghurghushti to Chechian	08
		Power Channel-02	Shadi khan to Wiro&Barotha	10
4	**Barrage Zone**	Barrage Zone-01	Pehoore hammleyt	04
5	**Power Complex** (Right & Left Bank)	Power Complex-01	Dher Wells	06
		Power Complex-02	Dher special wells	08

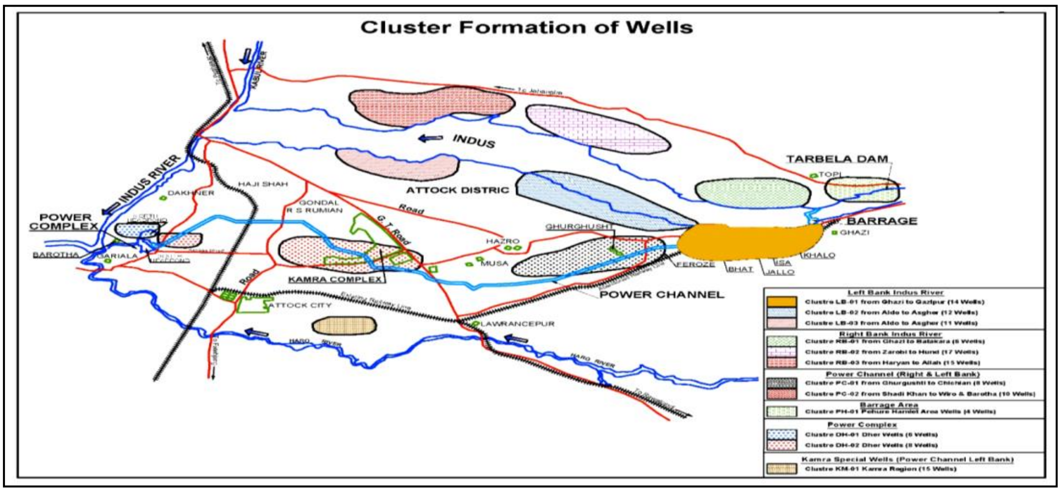
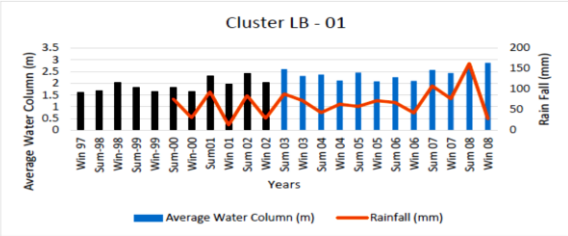
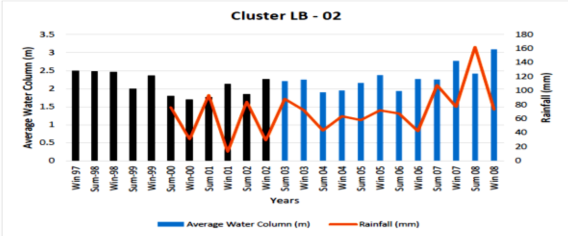
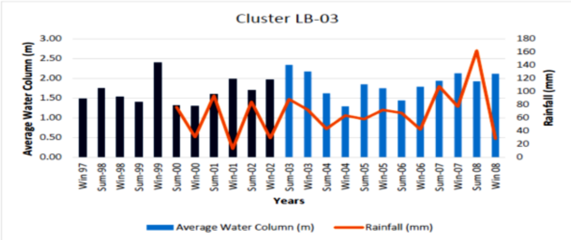
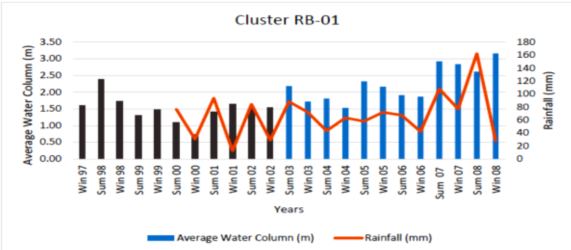
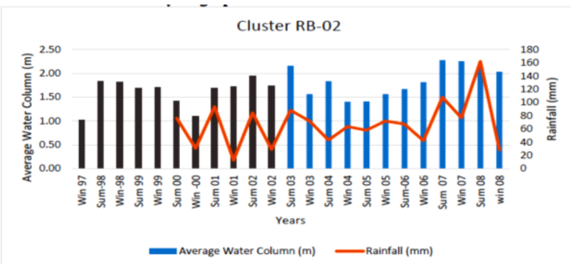
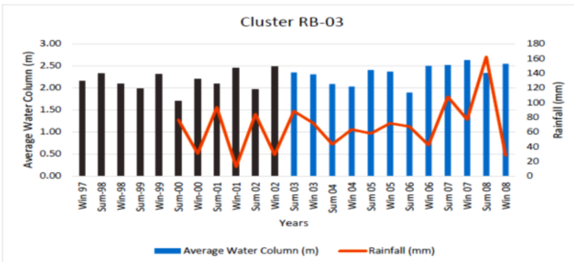


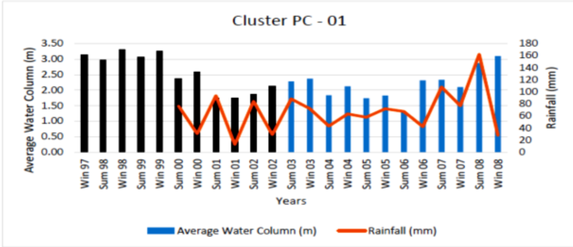
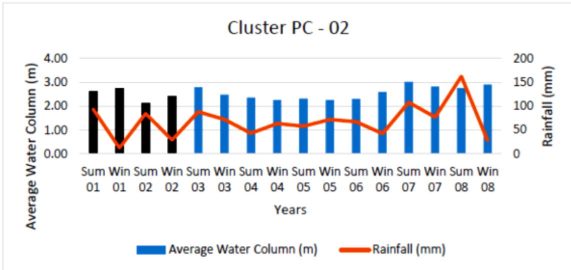
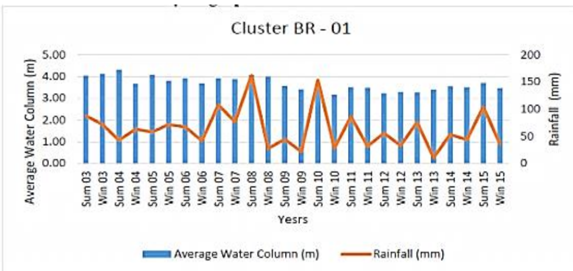
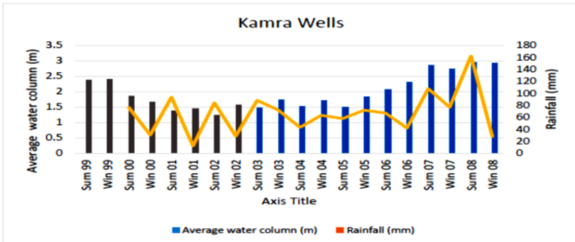
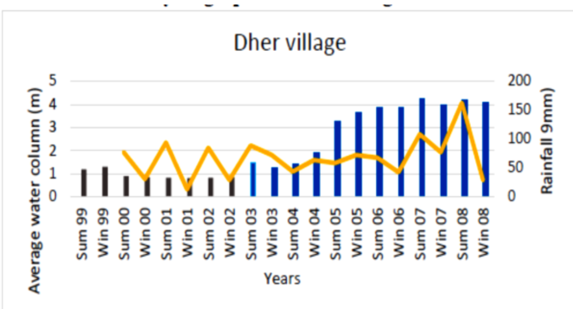
Figure 2: Layout of Clusters

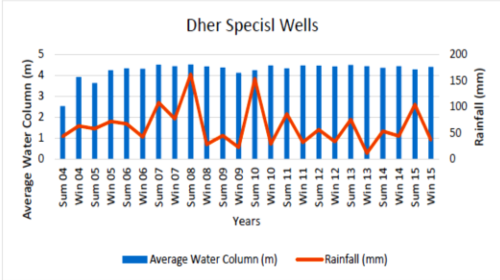
3. RESULTS:

3.1 Results and Discussion:

Average seasonal water column depth of all wells in a Cluster is calculated and then graph is plotted against time series to determine the impact of GBHP on water table depth. The observations are tabulated here along with the graphs for the presentation of status of the water column in each cluster.

Cluster	Observation	Graphical Presentation of Results
Left Bank-01	Significant Rise in the water column after construction of GBHP. The rise may be mainly due to increased rainfall and seepage effect.	
Left Bank-02	A minor change in water table position has been observed before and after construction of GBHP.	
Left Bank-03	A very little effect on position of water table observed in this area after completion of GBHP.	
Right Bank-01	Water table of the area has been observed slightly raised after the completion of the project.	
Right Bank-02	No significant effect has been measured on groundwater level.	
Right Bank-03	No significant effect has been measured on water table position in this area.	

Power Channel-01	No significant effect has been measured on water table position in this area.	
Power Channel-02	No significant effect has been measured on water table position in this area.	
Barrage Zone-01	Season by season fluctuation have been observed in conjunction with barrage level. The position of water table increases as the level in barrage increases.	
Power Complex-01	Constant increasing trend in water table position due to seasonal effects have been observed.	
Power Complex-02	Significant rise in the water table position of the area has been observed.	

Power Channel-01	Minimum fluctuation has been observed.	

4. CONCLUSIONS:

Based on the analysis of the graphs presented above, it may be concluded that there are different effects for different portions of the project vicinity areas, on water table position. On some portions, the water table of the area has raised significantly which may be mainly associated with the better monsoon and seepage effects. Whereas, on other portions there is minor or no effect. Some portions like the Cluster PC-01 keeps the highest water table position in comparison to other cluster which may be due to continuous impounding effect. Ghazi, Kkalo, Barrage areas of Project, Dher and Kamra Village are the most affected areas, where water table depth rises after the construction of project due to increased seepage effect.

ACKNOWLEDGEMENTS:

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Transportation Engineering

Paper ID: 605

Determining Optimum Proportion of Fly Ash as Partial Replacement of Asphalt for Flexible Pavements

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Abstract

Fly ash is a by-product obtained from the combustion of coal. It is a pozzolanic material and has been used as a binder for different types of construction application such as concrete mix, and soil stabilization. However, the studies related to exploring its behaviour when it is used in combination with bitumen are scarcely found. The focus of this research was to determine the optimum proportion of fly ash to be used as partial replacement of bitumen in flexible pavement design. The evaluation is made based on Marshall Stability and flow of Hot Mix Asphalt samples. Specimens were prepared using bitumen with and without fly ash. Varying percentages of fly ash were used as a partial replacement of bitumen. Based on the experimental work carried out during this study, it was observed that flow decreased with the increase in proportion of fly ash. On the other hand, stability was found to be maximum when the proportion of fly ash was at 10% by weight of bitumen. The result has shown that 10% replacement of fly ash with bitumen resulted in the highest stability and acceptable value of flow.

Keywords: Fly ash, hot mix asphalt, Marshall Stability, flow

1. Introduction:

Fly ash is produced as a by-product in power plants and manufacturing industries energized by coal combustion. Properties of fly ash resemble with other cementitious materials, but it is more economical being an industrial by-product. Use of fly ash has been reported to give more strength, and durability than other cementitious materials (Jala and Goyal, 2006). At the same time, its usage in construction projects is deemed environment friendly because its disposal on ground or in water can prove to be hazardous. Primarily, two classes of fly ash are used in civil engineering projects i.e. class C and F. The classification is based upon chemical composition of these types which is a result of their combustion processes. Class F fly ash is most commonly used as a replacement of Portland cement in construction applications because of its high silicon content (Xu, 1997; Oscar, 1999). According to Cooley et al, (2001) & Mistry and Roy (2016); fly ash is also being used in Hot Mix Asphalt (HMA) for many years as mineral filler and has been reported to provide positive results.

However, the review of literature shows that the effect of using fly ash as replacement of bitumen in HMA is further to be investigated. Hence, the objective of this research was set to determine the effects of fly ash, as a replacement of bitumen, on stability and flow of HMA. Class F fly ash has been used in this study because of its more popular application in concrete materials. It has further led to the recommendation about the optimum proportion of fly ash to be used in this regard. The results of this study would be useful for field engineers for optimum design of high-performance asphalt with addition of fly ash. Its importance would increase with the initiation of coal-based power generation projects which would result in ample supply of fly ash.

1.1. Fly Ash in Concrete

Use of fly ash in concrete dates back to six decades. This trend has grown due to the increase in construction activities as well as coal combustion for energy production (Thomas et al, 1999). The use of fly ash has been reported to increase the workability of cement concrete without significantly affecting its strength. Moreover, it has also been found beneficial for increasing the durability of concrete structures (Thomas et al, 1999; Rafieizonooz et al, 2016).

1.2. Fly Ash in Asphalt Pavement a Mineral Filler

Fly ash meets the specifications of mineral filler in HMA which includes its gradation, organic impurities, plasticity and hydrophobic nature. In addition to that, it also reduces the potential for asphalt stripping. A comparison of traditional and fly ash modified HMA was done by Modarres, and Rahmanzadeh (2014). The results of this experimental program showed that the use of fly ash resulted in higher stability and resilient modulus. It also enhanced the water sensitivity of mixes. Results indicated that the fly ash modified HMA exhibited more flexible behavior than the traditional mix.

1.3. Fly Ash in Stabilized Base Course

Fraay et al. (1990) described that stabilized base courses for pavements can be prepared in a cost-effective manner using proportioned mixtures of fly ash, aggregate, and an activator (cement or lime). These mixes are reported to produce strong and durable base course for pavement. These stabilized courses give comparable strength and stability as compared to cement treated aggregate layers at lower cost.

1.4. Fly Ash in Embankment

Kim et al. (2005) conducted an evaluation for use of fly ash in highway embankments. They observed that fly ash embankments give lower maximum dry density and well-defined moisture-density curves. Moreover, they also reported that hydraulic conductivity for fly ash embankments was lower than traditional materials. In terms of mechanical properties, such as shear strength, and compressibility, fly ash embankments were found to be similar to typical sandy soils. Hence, it can be said that use of fly ash can provide higher resistance to moisture while maintaining the same mechanical characteristics as traditional material.

1.5. Fly Ash a Replacement of Bitumen in HMA

Sobolev et al. (2013) investigated the effects of fly ash in HMA and its effects on workability, resistance to cracking and oxidative aging. This research reported that

addition of fly ash shows a positive impact on these properties. Another study was conducted by (Vasudevan 2013), which explored the stability of HMA while using different percentages of fly ash (by weight of bitumen) by performing Marshall Test.
He blends asphalt and fly ash at the ratio of 4% and 1%, 4.5% and 2%, 5% and 3%, 5.5% and 4%, 6% and 5% respectively (Vasudevan 2013), concluded that these percentages of fly ash cannot achieve the suitable results for the stability, flow and air voids. But they suggested to increase the percentages of fly ash so that better results may be achieved due to high binding properties.
The above studies indicated the need to do a more comprehensive study for testing the effects of using fly ash in terms of strength of HMA, i.e. stability and flow. The above studies have also recommended the use of fly ash in proportion of 6% or higher as per weight of bitumen with smaller increments. Both these issues have been addressed in this study.

2. MATERIAL AND METHODOLOGY

2.1. Aggregate Gradation

Aggregates used for this research were taken from a single source; i.e. quarries of Hub Chowki. This source is commonly used for acquiring aggregates for construction works in Karachi. The gradation selected for the performance testing is specified by National Highway Authority (NHA) Pakistan for wearing course with maximum aggregate size of 12.5mm. NHA is the prime administrative authority of inter-city highway network of Pakistan. More details about this gradation are shown in Figure. 1 and Table 1.

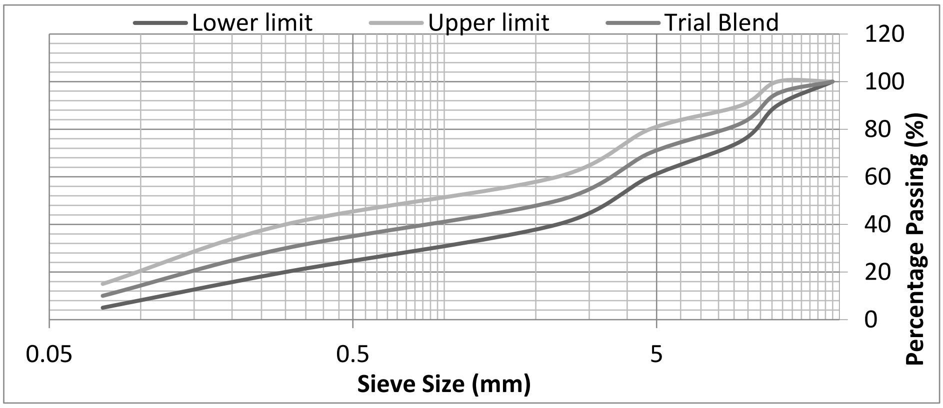

Figure 1. Gradation Chart

Table 1. Gradation

Sieve Sizes mm (inch	Standard (%age Passing)	Trial Blend (%age Passing)
19 mm (3/4 inch)	100	100
12.5 mm (1/2 inch)	90-100	95
9.5mm (3/8 inch)	75-89	82.5
4.75mm (No.4)	60-80	70
2.36mm (No.8)	40-60	50
0.3mm (No.50)	20-40	30
0.075mm (No.200)	5-15	10
Pan	3-8	5

2.2. Bitumen Tests

Bitumen has been used in engineering projects for many decades. This trend has led to establishment of certain test procedures to evaluate performance characteristics of bitumen. These test procedures also involve empirical rules that have been incorporated through continuous refinement by experts of this field. These tests are mainly concerned with the ductility and grade of asphalt which determines its performance under repeated loading and extreme environmental conditions. The tests for these properties include penetration, softening point and ductility test (Papagiannakis and Masad, 2008). The results of these tests are given in Table 2. They are hereby reported for reference of researchers who intend to do further research on interaction of bitumen quality with fly ash.

Table 2. Bitumen Test Result

Test	**Result**	**Test Procedure**
Penetration grade	(60-70 grade achieved)	ASTM D5
Ductility	34.6 cms. Distance of elongate	ASTM D113
Softening point	The average value of softening point of sample was 45^{0}C	ASTM D36

2.3. Marshall Mix Design

Bruce Marshall of the Mississippi Highway Department developed the basic concepts of the Marshall Mix Design Method around 1939 which were further refined by the U.S. Army. Currently, this method is used in Pakistan for mix design of HMA, although other methods are also available. The Marshall method is aimed at determination of Optimum Bitumen Content (OBC) that gives highest possible density while satisfying the minimum stability and range of flow values.

In this study, specimens having air void ratio closest to standard 4% value were considered to be having the OBC. Then, three (03) specimens containing the OBC at each proportion of fly ash were tested for their stability and flow. According to standard procedure of this method, each compacted specimen is 2.5$\pm$0.05inch (63.5$\pm$1.27mm) in height and approximately 1200gms in weight. Each parameter is evaluated for at least three identical specimens and the average of these specimens is considered for analysis (White, 1985). Complete procedural details about Marshall Mix Design Method can be found in ASTM standard D1559.

2.4. Air Voids (V_a)

Proportion of air voids is used to find the degree of compactness of HMA sample. Volume of air voids in a sample can be calculated as the difference of bulk specific gravity (G_{mb}) and theoretical maximum specific gravity (G_{mm}) of test specimen.

The test for determination of G_{mb} involves measuring HMA sample's weights under three different conditions, namely; dry, saturated surface dry and submerged in water. Equation 1 can be used to calculate G_{mb} (Hinrichsen and Heggen, 1996; Buchanan, 2000).

$$Bulk\ Specific\ Gravity = G_{mb} = A/(B - C) \qquad (1)$$

Where,

A = mass of dry HMA sample (g)

B = mass of saturated surface dry HMA sample (g)

C = mass of HMA sample in water (g)

G_{mm} of a HMA mixture is the maximum possible specific gravity which will be achieved at absolute absence of air voids in the sample. Equation 2 was used to calculate maximum specific gravity. The standard theoretical maximum specific gravity test is defined by the AASHTO standard T 209 and ASTMD 2041.

$$Theoritical\ Maximum\ Specific\ Gravity = G_{mm} = A/(A + D - E) \quad (2)$$

Where,
A = mass of dry HMA sample (g)
D = mass of flask completely filled with water (g)
E = mass of flask filled with HMA sample and water

3. RESULTS AND DISCUSSION

3.1. Marshall Stability

Results of the Marshall Stability tests are show in Figure. 2 and Table 3, which shows the average stability of samples at optimum bitumen content. It illustrates that Marshall stability will increase due to increase in bitumen replacement by fly ash from 0 to 10%. Replacement beyond 10% results in reversion of this trend and the Marshall Stability values decrease with an increase in percentage of fly ash. At 10% fly ash, the Marshall Stability value increases by 8% compared to the controlled specimens.

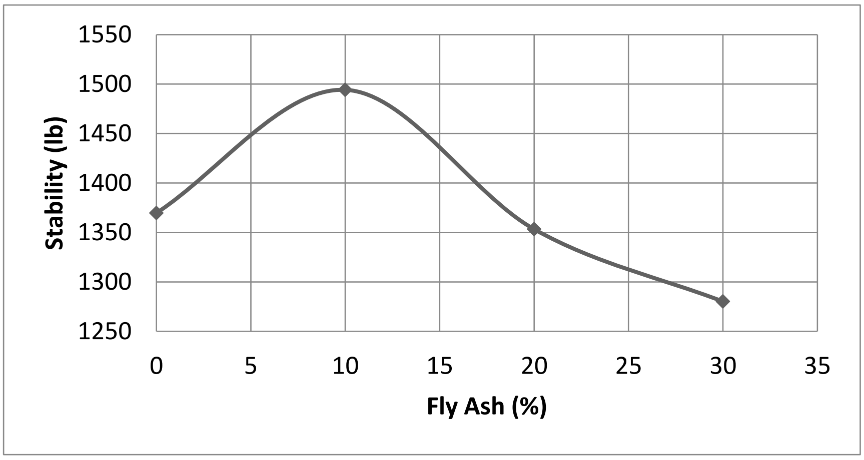

Figure 2. Marshall Stability

3.2. Marshall Flow

Marshall Flow tests were carried out and the results are shown in Table 3 and are graphically presented in Figure 3 for specimens with OBC. From this figure there is an increase in flow values with increase in percentage of fly ash. This means that mixtures with higher fly ash content are more susceptible to rutting. However, the flow value is within acceptable range given by AASHTO at 10% fly ash content (Hınıslıoğlu ans Ağar, 2004).

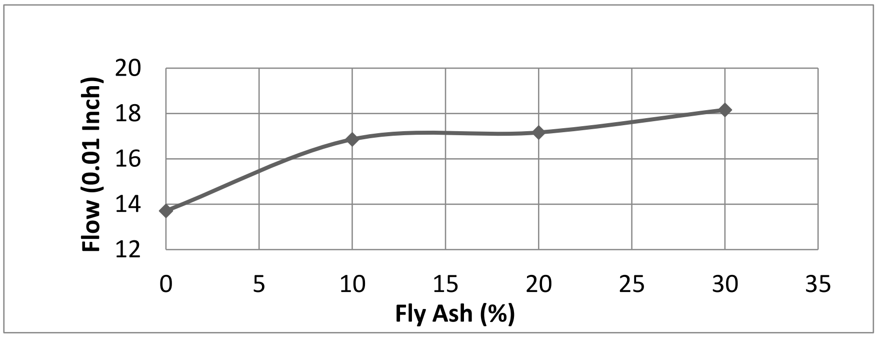

Figure 3. Marshall Flow

3.3. Optimum Bitumen Content (OBC)

The OBC values against different proportion of fly ash as replacement of bitumen are shown in Table 3. It shows that OBC at 10% replacement is almost equal to the control sample. These values decrease significantly as the proportion of fly ash is increased. The possible reason behind this trend could be that fly ash, initially, improves the void ratio in the mixture which provides more stability. On the other hand, it decreases the adhesion within the mix due to its fine particles. Hence, it can be said that 10% replacement of bitumen with fly ash in HMA increases its stability by improving its specific gravity/air voids ratio which is the measure of its compactness. But more than 10% replacement of bitumen the effect of loss of adhesion becomes prominent resulting in reduction of stability. The same reason could be stated for the continuous increase in flow value.

Table 3. Marshall Stability and Flow Test

Fly Ash (%)	OBC (%)	Stability (lb)	Flow (0.01in)
0	5.5	1369.6	13.71
10	5.4	1494.2	16.86
20	4.6	1353.3	17.162
30	4.25	1280.20	18.16

4. CONCLUSIONS AND RECOMMENDATIONS

This study was aimed at testing stability and flow characteristics of HMA with the proportion of fly ash 0, 10, 20, and 30% with respect to weight of bitumen. Highest stability was achieved when 10% of bitumen was replaced with fly ash. For these specimens, the average increase in stability value was about 8% compared to control specimens. The flow value gradually increases from 0.1371 to 0.1816 inch as the fly ash content is increased from 0 to 30%. However, the flow value at 10% replacement of bitumen is within the recommended specification of AASHTO. Hence, it can be concluded that 10% is the optimum proportion for replacement of bitumen by fly ash for gaining maximum strength of HMA without compromising its durability.

The results of this study provide an optimum mix design of modified asphalt with maximum stability and acceptable flow/rutting resistance. The utilization of such waste material would help in sustainable mix design.

The possible future directions of research in this field may be evaluation of mechanical properties of HMA at 10% replacement of bitumen by fly ash. Moreover, study of interactive effects of grade of asphalt at different proportions of bitumen replacement by fly ash is also another possible avenue to be explored.

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Performance of Cement Treated Base Course in Composite Pavement

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Abstract

The importance of highway in modern transportation systems cannot be overstated. Pavements are one of the major subsystem of the highway system. Today, pavement management system has been increasingly employed by many state agencies to assist in highway pavement management. One of the key component of the management system, which is also the most challenging part, is the pavement deterioration prediction models. The development of any country depends upon the transportation system. In Islamabad when precipitation occurs and proper drainage system is not available then water precipitates into the pavement layers and finally reach to the sub grade. Due to that settlement of sub grade take place that fails the pavement. The base course of asphalt pavement effects the durability of wearing surface. The variation in thickness of base course change the strength of pavement. The use of CTB (Cement Treated Base) under the wearing surface protects the asphalt pavement better than normal base course. But one issue arises when CTB is used that is the cracks produced in CTB. The main focus of our research is to reduce these cracks by introducing the layer of chip stones or open graded asphalt or crushed aggregates between CTB and wearing surface and choose best suitable option for future recommendations. The CTB provides best design of composite pavement for HTV. This design will enhance the durability and design life of pavement structure. It is more resistant against the heavy traffic loads. This composite pavement design is best suitable where subgrade soil strata have insufficient capability to bear the traffic loads.

Keywords: Design & analysis, Composite pavement, Cement treated base course

1. INTRODUCTION:

Composite pavements are comprising of different kinds of layers. Usually, asphaltic layer on the upper of a concrete pavement or a concrete layer on concrete pavement is used. For second type, commonly the lower layers contain aggregates or recycled concrete while the upper porous layer comprises high-quality smaller size aggregates (Rajib B. Mallick, 2013). The Major drawback is that the reflective cracks in the asphalt is produced by movement in the joints of concrete layers. To reduce these cracks, further one layer among these; chip-seal stones, open-graded asphalt & crush aggregates will added (Moghadas Nejad, Noory, Toolabi, & Fallah, 2014).

The main focus of this research is to find the solution of that problem by introducing the (CTB). CTB stands for cement treated base course that is provided in pavement as a base course. Some problems generated when we use CTB because CTB is made of concrete and concrete is a brittle material when load comes on CTB some cracks develop in the CTB. To overcome that crack problem, we use CTB with three different combinations of materials such as open graded asphalt, crushed aggregates and chip seal stones as an additional layer between CTB and wearing course. This central layer cannot reflect the cracks into the wearing surface. The main advantage of CTB is that, if water penetrates into the pavement then CTB acts as an insulator it does not allow water to move into the sub grade. So, finally subgrade protects from water and main cause of failure of pavement reduces.
Islamabad is currently facing problems with regards to the management and deterioration of pavement on arterial roads. The major problem in Islamabad is climate variation that effects drainage system of arterial roads. As the major portion of Islamabad consists of sloping areas and when precipitation occurs in different forms then water does not drain out properly from lower areas. So, water penetrates into asphalt pavement and finally reaches to sub grade. Water changes the favorable soil properties and reduces the bearing capacity of sub grade that leads to settlement. As sub grade of pavement fails the other layers such as sub base, base and wearing surface are also badly affected. At the end the wearing surface of pavement deteriorates. Cement treated base course was used because this provides stiffer and stronger base than an unbound granular base. A stiffer base reduces deflections due to traffic loads. CTB thicknesses are less than those required for granular bases carrying the same traffic because the loads are distributed over a large area. Cement stabilized bases resist consolidation and movement, thus eliminating rutting in all surface but the asphaltic layer.

2. EXPERIMENTAL PROCEDURES:

CTB is cement treated base course which is a modern technique used by NHA in the pavement to prevent rutting phenomenon and settlement of pavement under heavy traffic. Similar to concrete, CTB continues to gain strength with age. This is especially important when considering that many pavements experience greater traffic loads and volume throughout their service life.

Three main part of CTB;

1. First part is the base slab of concrete,
2. Second part is the intermediate layer and it is of three materials; Open grade asphalt, chip seal stones and crushed aggregates
3. Third part is the wearing surface of pavement.
   The following different conventional as well as advanced testing techniques were chosen for design and analysis of cement treated basecourse (CTB). These include conventional tests i.e. Softening point, Flash and fire point, Penetration, Ductility and Specific gravity and advanced tests. Following tests were performed to check the performance of CTB.

Table 1: Conventional Testing Outcomes of Asphalt

STANDARD	TEST NAME	RESULT	REMARKS
ASTM D36 / **AASHTO T53**	Softening point	71°C	Standard range for softening point is 40 to 80° C , our result is close to standard value.
ASTM D92 / **AASHTO T48**	Flash and fire point	Flash pt. = 260°C / Fire pt. = 268°C	Standard range for flash and fire point is 232°C to 400°C, our result is close to standard value.
ASTM D3142 / **AASHTT166**	Specific gravity	1.19	Standard range for specific gravity lies between 0.97 to 1.02, our specimen S.G is 1.19.
ASTM D5 / **AASHTO T49**	Penetratio n test	46	Standard value of penetration at standard conditions is 0 to 49mm. Our value is in between this range.
ASTM D113 / **AASHTO T51**	Ductility	6.0 cm	The ductility value ranges from 5cm-100cm. Our value is within the range.

Table 2: Design Mechanism Of CTB Pavement

Base course		
Materials	**Ratio /Thickness**	**Remarks**
Sand	1:1.5:3	Pass of 19 mm sieve
Cement		Fine
Aggregates		Pass of ¾ and retained of #4 sieve
Intermediate layers		
Crushed aggregates	2”	Retained of ¾ sieve
Open graded asphalt	2”	OK
Chips seal stones	1”	OK
Wearing surface		
JMF	2”	Class A

3. RESULTS:

In this section we check the performance ability of sample under different condition and different temperature. Any change in the samples under different loadings will also considered. Any pavement design based on two major test that is rutting and resilience modulus. Our mission is to analysis the rutting phenomenon and resilience modulus of the samples and discuss the behaviour of samples and to select the best design of CTB pavement.

3.1 DESIGN 1

In design (1) we use 2” thick layer of each material is used respectively (wearing surface, open graded asphalt, cement treated base course).
Modulus of resilience (AASHTO T 342)

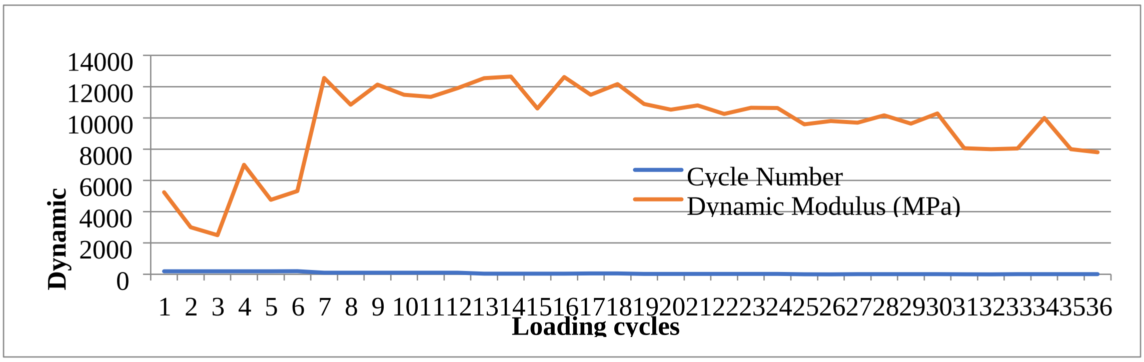

Figure 1: Changing behavior of modulus resilience under different loadings

Figure shows the relation between loading cycles and dynamic modulus. The above graph shows that when No. of passes increases the value of dynamic modulus fluctuates at a certain limit and then shows a constant behavior. The maximum value of dynamic modulus is 12,500MPa.

3.1.2 RUT DEPTH (AASHTO T 324)

Rutting Depth

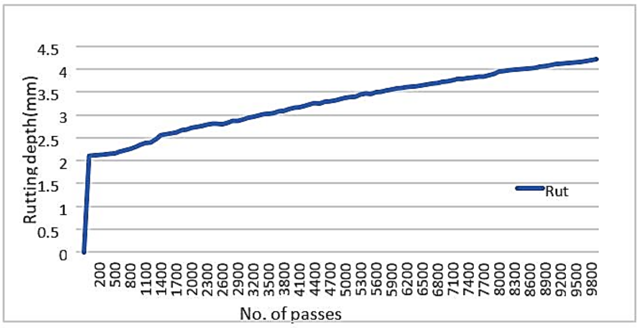

figure 2 : Analysis of rutting under different

Table 3: Comparison of Design 1

Comparison Between Modulus Of Resilience And Rut Depth		
	Modulus of resilience	**Rut depth**
Maximum	12500 MPA	4.1 mm
Minimum	8200 MPA	2.1 mm
Average	10350 MPA	3.16 mm

3.2 DESIGN 2

In design (2) we use 2" thick layer of each material is used respectively (wearing surface, crushed aggregates, cement treated base course).

Modulus of resilience (AASHTO T 342)

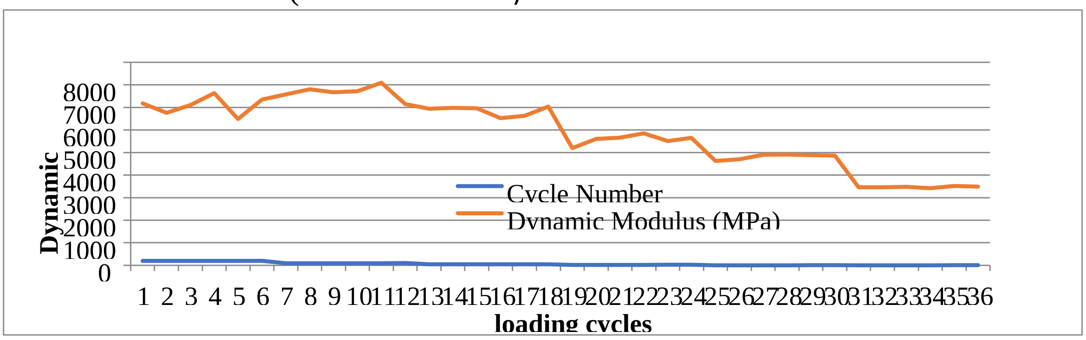

Figure 3: Changing behavior of modulus resilience under different loadings

The above graph shows that when No. of passes increases the value of dynamic modulus fluctuates at a certain limit and then shows a constant behavior. The maximum value of dynamic modulus is 8100MPa. (it is less than open graded asphalt).

RUT DEPTH (AASHTO T 324)

Table 4: Comparison of Design 2

Rutting Depth

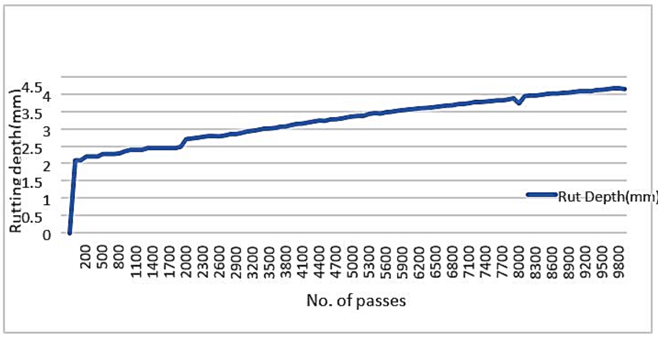

figure 4: Analysis of rutting under different passes

Comparison Between Modulus Of Resilience And Rut Depth		
	Modulus of resilience	Rut depth
Maximum	8100 MPA	4.18 mm
Minimum	3800 MPA	2.2 mm
Average	5950 MPA	3.19 mm

3.3 DESIGN 3

In design (3) we use 2" thick layer of wearing surface, 1" thick layer of chip seal stones and 3" thick layer of cement treated base course respectively.
Modulus of resilience (AASHTO T 342

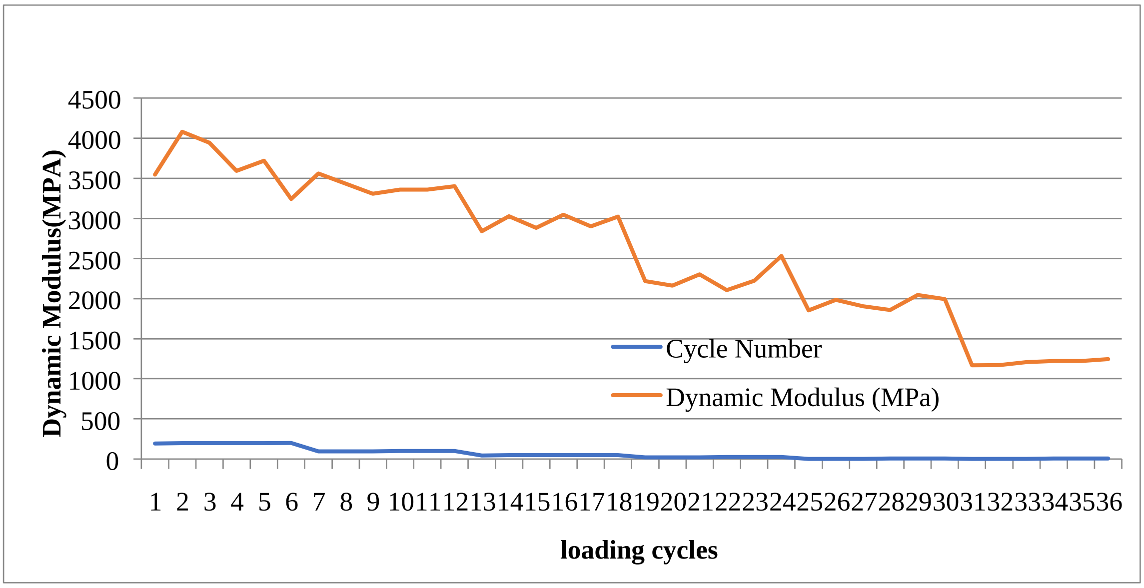

Figure 5: Changing behaviour of modulus resilience under different loadings

The above graph shows that when No. of passes increases the value of dynamic modulus fluctuates at a certain limit and then shows a constant behavior. The graph shows that maximum value of dynamic modulus is 4100MPa. (it is less than open graded asphalt and crushed aggregate).

RUT Depth (AASHTO T 324)

Rutting Depth

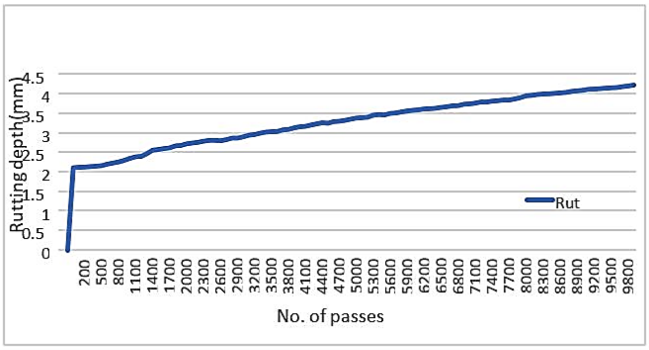

figure 6 :Analysis of rutting under different

Table 5: Comparison of Design 3

Comparison Between Modulus Of Resilience And Rut Depth		
	Modulus of resilience	Rut depth
Maximum	4100 MPA	4.22 mm
Minimum	1800 MPA	2.2 mm
Average	2950 MPA	3.21 mm

4. CONCLUSIONS:

Following conclusions can be drawn from the conducted study:

- According to upper discussion the design (1) has maximum value of M_R is almost 12,500 MPa & minimum value of M_R is 8,200 MPa. The maximum rut depth occurs in the design (1) is 4.1 mm & minimum rut depth is 2.1 mm. Hence the design (1) is best suitable among these three designs for CTB pavements as per AASHTO T 342.
- It is best suitable for those areas where fluctuations of ground water table occur because CTB pavement prevents the water to penetrate into the pavement as base course acts as barrier between subbase and wearing surface. Base course prevents the capillary movement of ground water table into wearing surface. Hence pavement remains safe from deteoration problems.
- This CTB design has resistance against heavy traffic loads as it has greater strength of M_R and low value of rutting, so this design can applicable for HTV.
- CTB pavement has several cost saving advantages over rigid pavement as it is long lasting life span.

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Operational Performance Analysis of Signalized Intersections: A Case Study of Lahore

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Abstract

Owing to ever-increasing motor vehicle population, congestion at intersections is a major problem. Effective control mechanism at intersections can help manage traffic and reduce its adverse effects significantly. City of Lahore, having the highest number of vehicles in Punjab, Pakistan faces enormous congestion issue. Consequently, vehicles face delay and enhanced travel times at signalized intersections. In this regard, this study is carried out to evaluate operational performance of two major signalized intersections in urban centre of Lahore. SIDRA Intersection, a traffic analysis tool widely used for traffic performance evaluation studies, is used for analysis.

Keywords: Operational Performance, Signalized Intersection, SIDRA intersection, Delay.

1. INTRODUCTION:

Intersections are amongst the primary components of road infrastructure that significantly affect traffic flow, environment and road safety especially in urban areas. Accordingly, effective control of these intersections is important as it is responsible for capacity, operational efficiency, delay and safety of the complete network (Tianzi et al. 2013). Intersection operational efficiency is dependent on the existing traffic, road geometry and traffic control conditions (Ranjitkar et al. 2014). Based on control type, intersections can be of various types i.e. Signalized, Stop Control and Roundabouts etc. (Garber and Hoel, 2014). Signalized intersections are much effective in controlling traffic as they isolate traffic movements (Oskarbski et. al 2016). Evaluation of transportation related projects is carried out through Measures of Effectiveness (MOEs) (Garber and Hoel, 2014). Multiple MOEs can be selected such as travel time, delay and queue length etc. For signalized intersections, Delay is a recommended MOE as per Highway Capacity Manual (HCM). (Manjunatha et al. 2013). Estimation of delay at signalized intersections has been studied by various researchers and using various methods. However, due to a number of affecting variables, study of delay at signalized intersections is still being conducted (Darma et al. 2005). Traffic Analysis and Simulation tools are widely used in traffic engineering studies due to their usability, reliability and economy. Due to effectiveness in evaluation, signalized intersections studies have also been carried out using these tools. (Tianzi et al. 2013). Operational efficiency of intersections is carried out to assess the situation of how good the intersections can handle the traffic demand.

The purpose of this research is to evaluate the operational efficiency of signalized intersections on two busy intersections of Lahore. Delay is used as a Measure of Effectiveness and recommendations are proposed to improve traffic flow and reduce congestion. Out of many available traffic analysis tools, SIDRA intersection is selected due to its reliability and usability.

2. METHODOLOGY:

2.1 Description of Study Area:

The study was carried out in the metropolitan city of Lahore (31.5204°N, 74.3587°E). Vehicle population in the city has increased from 1.70 Million in 2007 to 4.92 Million in 2016 (PDS, 2008 and PDS, 2017). Traffic congestion and travel time delays are major problems associated with city traffic. Traffic conditions are heterogenous and a larger share of vehicle fleet consists of private vehicles (90%) (PDS, 2017).
Two signalized intersections in urban centre of Lahore are selected for the study. Both the intersections are located on Queens Road, Lahore. Location of intersections is shown in figure 1.

Figure 1: Location of Intersections on Queens Road, Lahore

2.2 Data Collection:

Traffic survey was conducted for a period of 16 hours at both the intersections. For the purpose of traffic analysis, following data was collected:

- Road inventory
- Traffic Volume & Composition
- Directional Distribution
- Signal timings and Phasing data

Manual classified counting technique was used for traffic volume study. Road inventory survey was carried with the help of measuring tape. Signal timing and average speed at intersection approaches were measured with the help of stopwatch. Satellite imagery of both intersections is shown in figure 2:

a) 

b) 

Figure 2: Satellite Image of Intersections, a. Ganga Ram Chowk, Queens Road and b. PSO Chowk, Queens Road

2.3 Traffic Analysis:

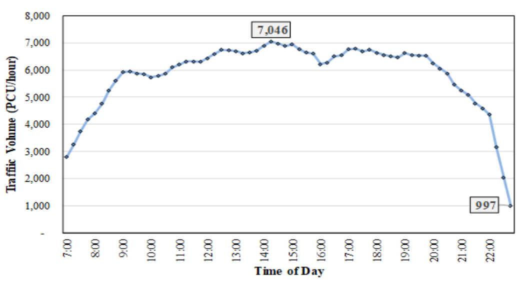
SIDRA Intersection was used to assess operational efficiency of intersections. SIDRA is a traffic modelling tool used for lane by lane analysis of intersections having different control types and lane configuration. Measure of effectiveness (MOEs) including delay, queue length, capacity and emissions are estimated based on inbuilt traffic models and iterative approximation method. (Parakash et al. 2014). SIDRA intersection uses path trace method to measure vehicular delay. Delay estimated in SIDRA is an aggregate of geometric delay, queuing delay, stopped delay, acceleration and deceleration delay (Sisiopiku and Oh, 2001).

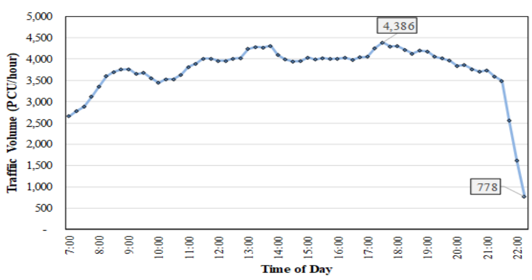
Firstly, Geometry of intersections was outlined based on road inventory data. Afterwards, traffic volume and directional distribution data for each approach was entered into the analysis tool. SIDRA Intersection is unable to differentiate different vehicle types. For that purpose, traffic composition is converted into a single entity termed as Passenger Car Units (PCUs). To estimate traffic for 2023, a growth rate of 3.5% is used. For the purpose of analysis, PCUs for peak hours were used. The peak hours and traffic volume in peak hour for both intersections have been mentioned in Figure 3. Results including Delay at present scenario i.e. 2019 and for future i.e. 2023 was estimated.

3. RESULTS:

3.1: Traffic Volume distribution throughout the Day

16-hour traffic count survey data reveals the fluctuation in traffic flow throughout the day. A time series analysis shown in figure 3 depicts the traffic volume fluctuation in these 16 hours.





a. b.

Figure 3: Time Series Analysis of Traffic Volume, a. Ganga Ram Chowk, Queens Road and b. PSO Chowk, Queens Road

At Sir Ganga Ram Hospital chowk, traffic volume is higher in the middle of the day and traffic in AM peak and PM peak hours is relatively lower. Peak volume was recorded between 2:15 -3:15 pm. At PSO Chowk, traffic volume is also greater in middle of the day, however, maximum peak recorded was between 5pm - 6pm.

3.2 SIDRA Intersection Analysis:

Geometry of the intersections shown in Figure 2 was outlined in SIDRA intersection. PCU values were used for calculation of Delay. Delay analysis for existing scenario 2019 and future scenario 2023 is performed which is presented in figure 4.

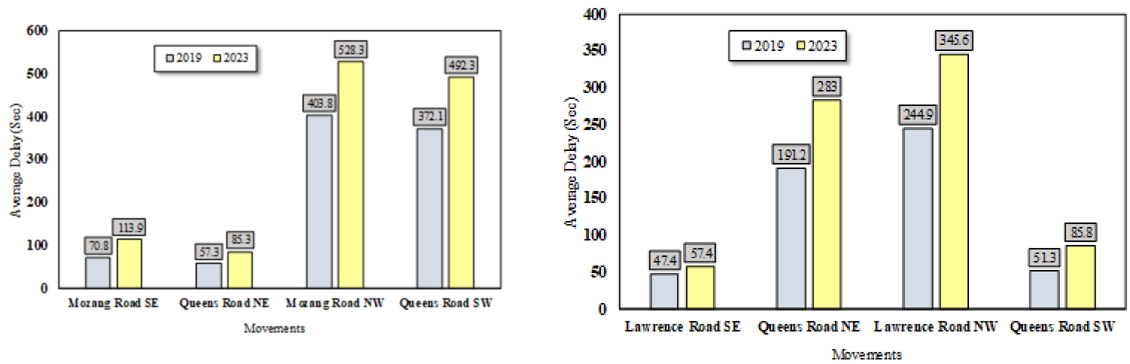


Figure 4: Delay Results in Peak Hour, a. Ganga Ram Chowk, Queens Road and b. PSO Chowk, Queens Road

Results show that delay in existing and future conditions are much higher than acceptable levels. Table 1 displays a comparison of maximum and minimum values of delay on both intersections and acceptable value of delay. It can be seen that even the minimum value is above the acceptable limit of delay. Main reasons of excess delay include absence of signal optimization, abundance of private transport, poor driving behaviour and on street parking near intersections.

Table 1: Comparison of Estimated and Acceptable Delay

Year	Intersection	Approach Delay (sec/veh)		Acceptable Delay (sec/veh) as per Level of Service C (HCM, 2010)
		Max.	Min.	
2019	Ganga Ram Chowk	408.8	57.3	≤ 35
	PSO Chowk	244.9	47.4	
2023	Ganga Ram Chowk	528.3	85.3	
	PSO Chowk	345.6	57.4	

4. CONCLUSIONS AND RECOMMENDATIONS:

Conclusions and Recommendations for the case study are as follows:

- Delay results show that intersections are unable to cater traffic demand in peak hour for both intersections.
- Signalized Junctions i.e. Ganga Ram Chowk and PSO Chowk may be optimized for smooth flow of vehicles.
- Traffic composition in study area is heterogenous and consists mainly of private vehicles. If private transport is replaced by some percentage of public transport, traffic volume can be reduced up to certain level and delays could be reduced.
- On street parking should be prohibited near junctions to incorporate the peak hour traffic.
- The study could be further extended by simulating mitigation measures and applying more comprehensive tools like PTV Vissim and Paramics Discovery.

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Controlling Shrinkage Cracks Propagation in Rigid Pavements Using Banana Fibre Reinforced Concrete

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Abstract

Cracking in Rigid Pavements is a common issue. Out of different types of cracking, drying shrinkage causes visible cracking in Plain Cement Concrete (PCC) as it shrinks and the developed stresses exceeds its tensile strength capacity. Movement of heavy traffic cause propagation and widening of cracks. This ultimately reduce the serviceability and durability of Pavements. The overall aim of the research program is to improve the performance and durability of rigid pavements exposed to heavy traffic loading. The specific goal is to use Banana Fibre in Reinforced Concrete to improve the post cracking behavior of concrete in rigid pavements and limit the propagation and widening of shrinkage cracks. Cylinders of diameter 100 mm and height 200 mm are prepared using mix design ratio of 1:2:4 having water cement ratio 0.6 and fibre content 0.5% by mass of concrete. Specimens are kept in water for 28 days and then tested using ASTM standard C496/C496M-17. The splitting tensile strength of BFRC specimen turned out to be 44% less than that of PCC. The fibres caused a strong bridging effect that shows fibres have a good tensile and bond strength to improve the post cracking behavior of concrete, also the percentage of fibre content has a direct relation with the splitting tensile strength. The advantage of Bridging effect can be utilized to increase the splitting tensile strength of BFRC over PCC by optimizing the percentage of fibre content.

Keywords: Banana Fibre Reinforced Concrete, Rigid Pavements, Shrinkage Cracking, Bridging Effect, Tensile Strength.

1. INTRODUCTION:

Rigid pavements are constructed in areas which are exposed to movements of heavy traffic. The sole purpose of a rigid pavement is to provide efficient, smooth and comfortable ride and long-term durability against dynamic loading. Rigid pavements are expensive in terms of initial cost as compared to flexible pavements but have minimal cost of maintenance. Due to high flexural strength of concrete, vehicular load is homogenously transferred to underlying layers, reducing stress concentration at subgrade level (Chang and Chai 1995). Elastic modulus and shear modulus of rigid pavement is much greater than flexible pavement. Degradation of concrete starts with the early age micro cracking (Guo and Weng 2019). The durability of rigid pavement is controlled by various mechanical properties. These mechanical properties are compromised with the development of cracks. Various types of cracking that a pavement encounter during its life are early age micro cracking, longitudinal cracking, thermal cracking, shrinkage cracking, etc. Concrete made of Portland cement has a brittle behaviour and is weak in tension. Due to this brittle behaviour, concrete has low resistance to temperature and volumetric stresses with low strain capacity in tension and low toughness thus resulting in development of cracks.

When concrete expands or shrinks, stresses are developed which produce cracks on the surface of rigid pavement. Surface of the pavement is exposed to atmosphere and the layer of concrete underlying is not, concrete on surface dries and shrinks at a different rate as compared to underlying layer. The underlying layer of concrete acts as a restraint to shrinkage, resulting in cracking of the surface layer. The governing property of concrete that is responsible for shrinkage cracking is its split tensile strength. During drying shrinkage, stresses are developed in concrete and when these stresses exceed the tensile strength capacity of concrete, shrinkage cracks appear on the surface. After cracking starts, the deterioration rate of concrete pavement increases as the traffic loading boosts crack propagation and expansion. Once Shrinkage cracks appear, the pavement is highly exposed to adverse climatic effects including water rolling effect during raining seasons. Water gets absorbed in these cracks and damages the underlying layers by causing partial settlement as well as thermal cracking due to temperature variation. In such scenarios post construction cost of maintenance increases for remedial measures. Concrete is weak in encountering tensile stresses so it is fortunate that the issue of shrinkage cracks propagation can only be resolved by improving the post cracking behaviour using other additives like reinforcement, fibres, etc.

To enhance the mechanical properties, especially to control the issues of cracks propagation, possible solutions can be admixtures, reinforcement, fibres, etc. Nowadays researchers are widely using fibres as additives to enhance the mechanical properties of concrete. Instead of synthetic fibres like glass fibre, carbon fibre and plastic fibres, plant fibres come under the category of renewable materials that possess a potential to create environmentally friendly products (Kumar et al. 2016). Mostafa and Uddin (2015) investigated the mechanical properties of banana fibre in compressed earth blocks. Fibres create a bridging effect during the development of cracks in concrete and improve its post cracking behaviour (Prasannan et al. 2018). Banana Fibres are lignocellulose material having up to 56% cellulose content that plays a key role in determining its mechanical properties. Banana fibres with a diameter of 80 – 250 mm have a tensile strength ranging from 54 MPa to 754 MPa and density 1350 Kg/m^3 (Ali 2012). Banana fibres are lightweight, less extensible with considerable heat and fire resistance (Sakthivel et al. 2019). Banana fibres, if used as a reinforcing material with different percentages in concrete can play a vital role in increasing the tensile strength of concrete that will reduce the cracks propagation caused after tensile failure.

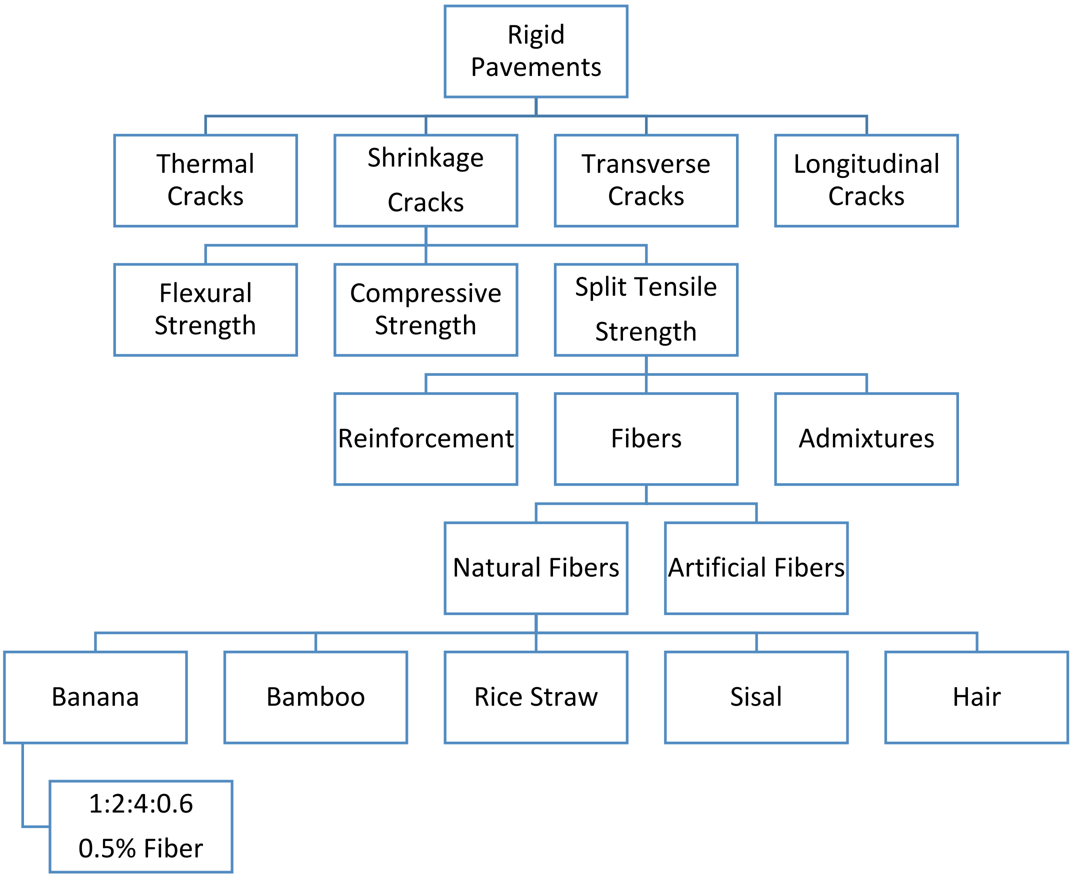

Figure 1: Flow chart representing pavement issues, governing properties and applied solution

Figure 2: Shrinkage cracking in rigid pavement

Prasannan et al. (2018) investigated the mechanical properties of concrete using 1% and 1.5% banana fibre. To the best of author's knowledge, no work has been done to control shrinkage cracks propagation in rigid pavements using banana fibre. Thus, current research is aimed to study the split tensile behaviour of BFRC using 0.5% banana fibre in concrete for its implementation in rigid pavements to achieve better post cracking properties.

2. EXPERIMENTAL PROCEDURE:

2.1 Raw Materials:

Ordinary Portland Cement, locally available fine and coarse aggregates, water and banana fibres are used for the preparation of PCC and BFRC. The length of each banana fibre is 50 mm.

2.2 Mix Design and Casting Procedures:

For the preparation of BFRC, the mix design ratio used is 1:2:4 (Cement: Sand: Aggregate) with 0.6 water cement ratio. Banana fibres are added 0.5% by mass of concrete. Firstly, one third of the coarse aggregates and fibres are added in the drum mixer with three quarters of water and then mixer is rotated for two minutes. Then two third fine aggregates are added and mixer is rotated for another two minutes duration. Then rest of the materials are added and mixer is rotated for three minutes. Slump test is performed to investigate the workability of BFRC as per specifications of ASTM C143 / C143M-15a and then specimens are prepared.

For the preparation of PCC, the mix design ratio used is 1:2:4 (Cement: Sand: Aggregate) with 0.6 water cement ratio. All materials along with water are poured in the drum mixer and then rotated for a duration of four minutes. Same standard test is performed to check the workability of PCC.

2.3 Specimens:

Cylinders for both PCC and BFRC, having 100 mm diameter and 200 mm height are prepared using standard procedure of filling in three layers and temping each layer with 25 blows with temping rod of 16 mm diameter. The unit density of PCC and BFRC is determined as per ASTM standard C138 / C138M – 16. The density of PCC and BFRC specimens came out to be 2426 kg/m^3 and 2196 kg/m^3 respectively.

2.4 Splitting Tensile Strength Test:

To determine splitting-tensile strength of PCC and BFRC cylinders, test is performed according to ASTM standard C496 / C496M-17. Compression testing machine is used to test all cylinders for studying splitting tensile behaviour and determining splitting-tensile strength. The splitting-tensile load-time curves are also obtained for all specimens.

3. RESULTS AND ANALYSIS:

3.1 Splitting Tensile Strength Behaviour:

Splitting tensile test is performed for both PCC and BFRC specimens. Figure 3a shows the strength time curves for both PCC and BFRC. The cracking pattern from the initiation of first crack to the application of maximum load is observed visually. Figure 3b shows the specimens of PCC and BFRC at first crack, cracks at maximum load and cracks at ultimate load. In PCC and BFRC specimens, first crack is observed at 100% and 99.5% of the maximum load respectively. Splitting effect is visualized after maximum load in PCC specimen after which the specimen broke in two pieces. In case of BFRC, a strong bridging effect was observed and the concrete was held by the fibres before splitting due to strong bond between fibres and cement matrix. The cracks widened slowly even after the application of ultimate load. After testing, the specimens were intentionally broken to study the post testing fibre condition.

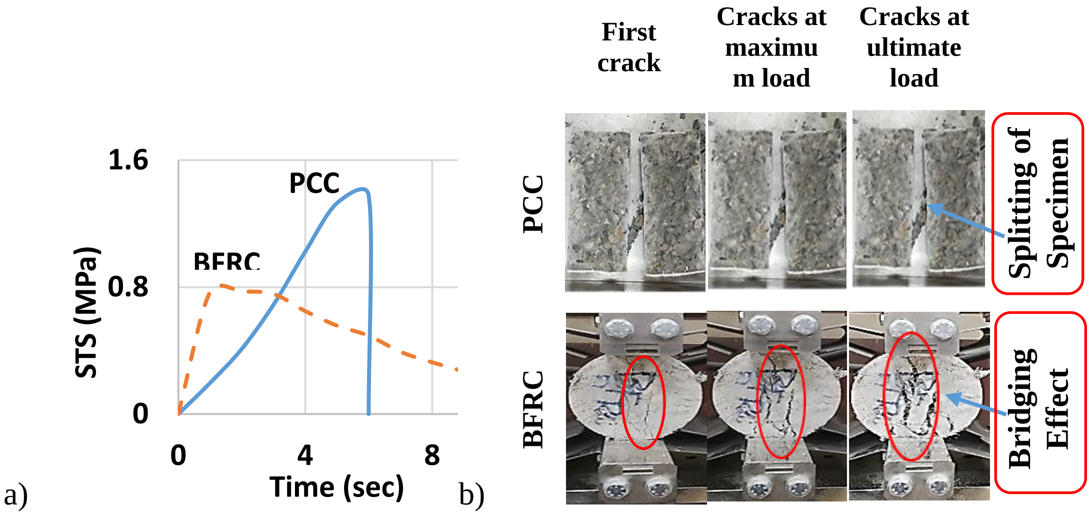

Figure 3: Splitting tensile behavior of PCC and BFRC specimens, a) Strength-time curve, b) Cracking pattern

3.2 Splitting Tensile Strength Parameters:

The maximum load and corresponding splitting tensile strength of PCC and BFRC specimens were obtained and shown in Table 1. The first crack in PCC specimen appeared after 8 seconds at the maximum load of 45 kN. While in case of BFRC, first crack occurred at very first second at the load of 24.30 kN. After first crack the specimen took considerable load due to the fibres holding the concrete. The splitting tensile strength of BFRC reduced up to 44% less than that of PCC. This is due to the balling effect, low percentage of banana fibres, inadequate mixing and improper compaction of concrete. The splitting tensile strength of fibre reinforced concrete increases with the increase in banana fibre percentage.

Table 1: Splitting tensile strength parameters

Specimens	Max. Load (kN)	STS (MPa)
(1)	(2)	(3)
PCC	45	1.38
BFRC (0.5%)	24.40	0.77

4. KEY ASPECTS FOR USING BFRC IN RIGID PAVEMENTS:

The specimens are broken intentionally to visualize the fibre behaviour. Broken specimen is shown in Figure 4. After analysing the broken specimen visually, both fibre breakage and pull out behaviour are noticeable but fibre breakage is observed to be more than fibre pull out. Fibres pull out occurs when tensile strength of fibres is greater than bond strength between fibres and cement matrix. The bridging behaviour is beneficial for controlling the propagation of shrinkage cracks in rigid pavements. Fibres are able to prevent widening of cracks which can reduce the deterioration of concrete in case of rigid pavements. This can increase the lifespan of pavement exposed to heavy traffic.

Figure 4: Fibres breakage and pull-out behavior

The failure of BFRC is observed to be ductile than the brittle behaviour of PCC. The bond between concrete and fibre prevented the spalling of concrete. As it is a pilot study, a drum mixer is used to prepare concrete. The improper mixing, strangling of fibres and poor compaction led to the balling effect in concrete. Also, less fibre percentage i.e. 0.5%, decreased the splitting tensile strength of concrete. Other way round, the strong bridging effect in concrete indicates the presence of strong bond between concrete matrix and fibre.

The observation from the testing shows that the performance of fibre in bearing the tensile stresses has enhanced the tensile property of overall concrete matrix. This property can delay the propagation and widening of cracks in rigid pavements exposed to heavy traffic and adverse climatic conditions which will minimize the deterioration of rigid pavement and increase its durability.

5. CONCLUSIONS AND RECOMMENDATIONS:

The study investigates the post cracking behaviour of BFRC in rigid pavements using mix design ratio 1:2:4 and banana fibre 0.5% by mass of concrete. The splitting tensile strength of BFRC decreased as compared to PCC due to less fibre percentage and balling effect.

Following are the conclusions drawn from the conducted research:

- Brittle failure of concrete changed into relatively ductile failure due to the presence of banana fibre.
- The banana fibre created bridging effect in the concrete improving the post cracking behavior of concrete.

By optimizing the fibre content, the splitting tensile strength of BFRC can be increased. Increased strength and improved post cracking behaviour of BFRC can limit the propagation and widening of cracks thus delaying the deterioration of rigid pavements.

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Evaluation of the Rheological Characteristics of Asphalt Modified with Nano Material

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Abstract

Nano technology is using in all over the world and it has a great effect on characteristics of asphalt. A Nano technology is one of the active research areas in a number of disciplines like Civil Engineering and Construction Materials. A modifier Nano silica (NS) is used by different researchers to start focus on the modification of Pavement materials. The objectives of this study were to enhance the role of Nano technology in Pavement Engineering applications on the basis of its rheological parameters. This paper focus on how to improve the conventional and rheological characteristics of modified binder using Nano Silica (NS). Incorporating of Nano silica (1%, 3%, and 5% by weight of binder) in binder had improved the physical properties of bitumen. Frequency sweep test was performed by using Dynamic Shear Rheometer (DSR) to evaluate complex modulus, phase angle and rutting resistance characteristics of binder. The result showed that by adding 3% NS, penetration and ductility had decreased by 18.29% and 30.1% respectively while the softening point had increased 7.53%. The test result of the DSR showed that rutting resistance had improved 35% by using 3% NS. The optimum percentage of 3% NS was recorded on the basis of rutting resistance values.

Keywords: Nano Silica (NS), Dynamic Shear Rheometer, Rheological and Rutting Resistance.

1. INTRODUCTION:

Asphalt concrete mixture consists of mainly two major components; aggregates and binder(Read and White oak, 2003). Out of total weight of asphaltic mixture, 95% by weight of asphaltic mixture represents the aggregate components and remaining 5% shows the binder component. The binder proportion is small but it has a greater effect on characteristics of obtained mixture (Ahmed, 2007). Many researchers are focusing to enhance the performance of binder which ultimately improve the quality and characteristics of asphalt concrete mixture. Different modifiers were used by them such as clay, rubbers, polymers etc. to enhance mixture characteristics(Chen, Liao and Lin, 2003; Ahmed, 2007). Currently, Nano technology is using in all over the world and it has a great effect on characteristics of asphalt. In Civil engineering it has been used in 1986 in publication of the book Engines of Creations. It has been used in different fields like medicine, water treatment as well as in Transportation Engineering. Researchers start focus on using Nano Silica in different pavement layers (A. Ahmed and T. Mahmood, 2015; Issa, 2016). Rut is a longitudinal grove in roads that is caused by the repeated loads of vehicles mainly at high temperature (Yusoff et al., 2014). Nano silica is mixed with bitumen at a temperature of 160°C in proportion of 1%, 3% and 5% by weight with homogenizer at velocity of 2000rpm (Enieb and Diab, 2017). It has been observed that rutting resistance is improved by incorporating Nano silica in binder. Nano silica act as a filler in mix design due to which it improves the properties of asphalt .The importance of nano silica particles is that it posses oxidizing properties which accelerate oxidizing reaction in the bitumen binder. Advantage of nano silica is that they give us low cost production and enhance performance of asphaltic concrete. As high grade bitumen are used in cold areas, so by modifying these bitumen with nano silica they get stiffer and can also be used in warm areas. As compared to the project estimated cost for the maintenance, the cost used for modifying bitumen is lesser so we can built more durable pavements which will resists the major distresses like rutting,(Mokhtar F. Ibrahim Hassan D. Hassanin (2018)

2. OBJECTIVES:

The main objectives of the research are given below:

1. To improve the physical properties of modified Nano Silica binder.
2. To improve the rheological characteristics of Modified Nano Silica binder.
3. To determine the optimum content of Modified Nano Silica binder on the basis of rutting resistance value.

3. MATERIALS:

3.1 BITUMEN:

The bitumen used in this research is of Penetration grade 80/100 procured from National Refinery Limited (NRL) Karachi, Pakistan.

3.2 NANO SILICA:

Nano silica (SiO_2) is also known as Silicon dioxide. It is in the form of powder. The properties are given in the Table1.

Table 6: Properties of Nano Silica

Properties	
Chemical formula	SiO_2
Molar mass	60.08 gmol^{-1}
Melting point	>1600 °C
Boiling point	2230 °C
Form	Nano powder
Surface Area	spec. surface area 175-225 m^2/g
Diameter	25-30nm

4. METHODOLOGY:

The test methodology of the research is given in Figure 1:

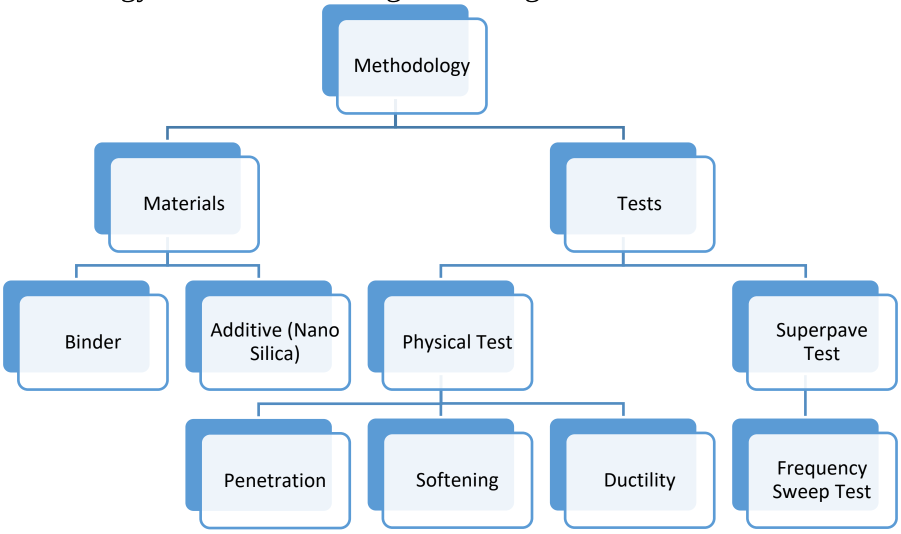

Figure 1: Methodology

5. TESTING PROGRAM

This section we briefly explains the tests performed for the evaluation of rheological properties of bitumen. Experiment is categorized into two section (i.e) conventional tests and advance tests. The conventional tests determine the physical properties while the advance test evaluates the rheological properties.

Figure 2: Penetrometer

Figure 3: Ring and Ball Apparatus

Figure 4: Ductilometer

Figure 5: DSR

5.1 Penetration Test:

Penetration test was performed in accordance with ASTM standard (D5/D5M-13, 2013) as shown in Figure 2. The main purpose of this test is to determine the penetration grade of binder.

5.2 Softening Test:

The main purpose of this test was to determine the consistency of binder in accordance with ASTM standard (ASTM D36/ D36M-14e1, 2014) as shown in Figure 3.

5.3 Ductility Test:

The ductility test was performed to determine the tensile nature of binder in accordance with the (D113-17, 2017) as shown in figure 4.

5.4 Frequency Sweep Test:

Frequency Sweep Test was performed on Dynamic Shear Rheometer (DSR) machine as shown in figure 5 to determine the viscoelastic behavior and rutting resistance of binder in accordance with the (AASHTO T 315-10, 2010). The range of frequencies used for this test was 10 to 0.1 Hz and temperature range was 22°C to 82°C.Strain limit for base binder was kept 12%.

6. TESTING RESULTS:

6.1 Conventional Binder Testing:

The conventional physical properties test results are given in Table 2:

Table 2 Conventional Testing Results for shear mixed Binder

Test	Base Binder	1% NS	3% NS	5% NS
Penetration (0.1mm)	**82**	**74**	**67**	**63**
Softening Point (◦C)	**46.5**	**48**	**50**	**52**
Ductility (cm)	**103**	**83**	**72**	**58**

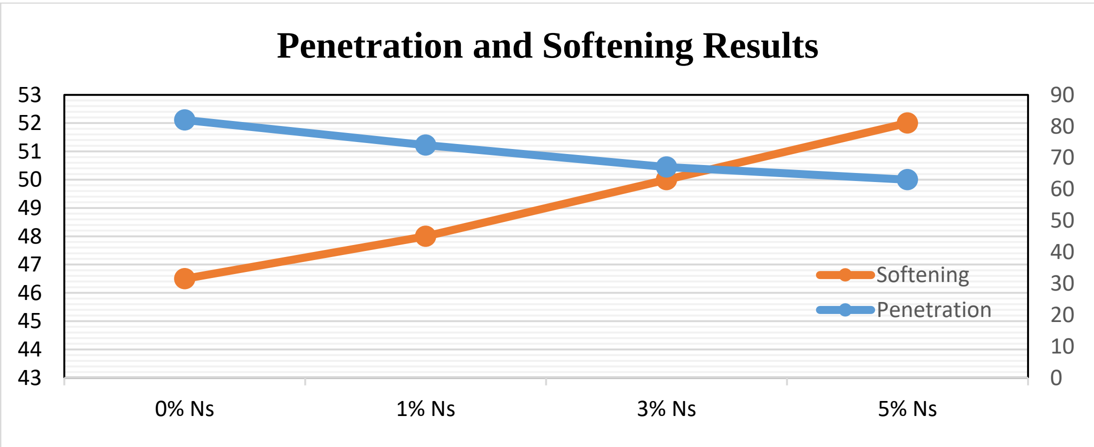

Figure 6: Penetration and Softening Results

Figure 6 illustrated that penetration of the bitumen had decreased by increasing the percentage of Nano Silica. The penetration values of 1% NS and 3% NS showed that it had become 9.76% and 18.29% stiffer respectively than neat bitumen. The overall 23% reduction in penetration value was recorded as result in adding 5% NS. Figure 6 results illustrated that softening point of the bitumen increased by adding NS. The incorporation of NS 1%, 3% by wt. of binder had increased the softening point 3.225% and 7.53% in comparison with neat binder. Similarly, by adding 3% of NS had increased the 4.16% as compared with 1% of NS. Thus results concluded that incorporating 5% NS in base binder had increased the overall softening point 10.58% in comparison with neat binder.

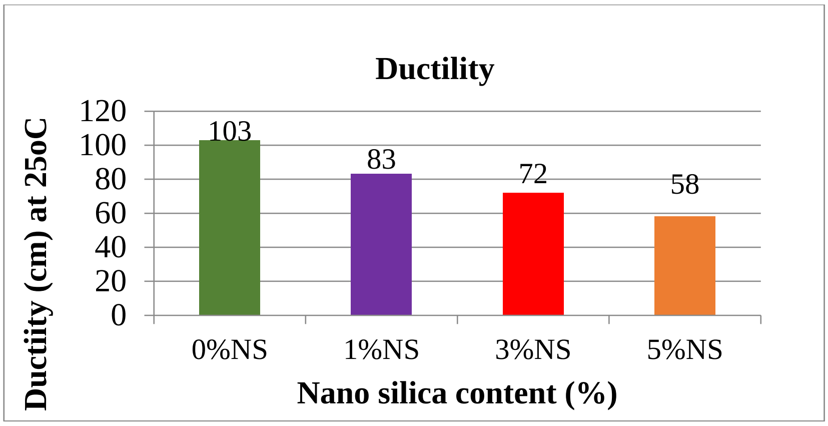

Figure 7: Ductility Test Results

Graphical representation of Figure 7 illustrated that 1%, 3 and 5% NS had significantly decreased the ductility values 19.42%, 30.1% and 44% respectively. There is the significant reduction in the ductility and is reduced by 44% by adding 5% Nano silica in bitumen.

6.2 Frequency sweep Test:

Master curves drawn in Figure 8 shows a relationship between reduced frequency and G*.Master curves were drawn at 58 °C by using sigmoidal parameters. Master curves showed that by adding 1% NS had decreased the complex modulus values while 3% NS had increased the complex modulus (G*) values at lower frequencies while decreased the G* at higher frequencies. It's confirmed that the performance of 3% NS had not only significantly improved the performance of binder at higher temperature but also improved its low temperature characteristics. Similarly, modification by 5% NS showed that by adding the quantity of NS would decrease the high and low temperature performance of pavement.

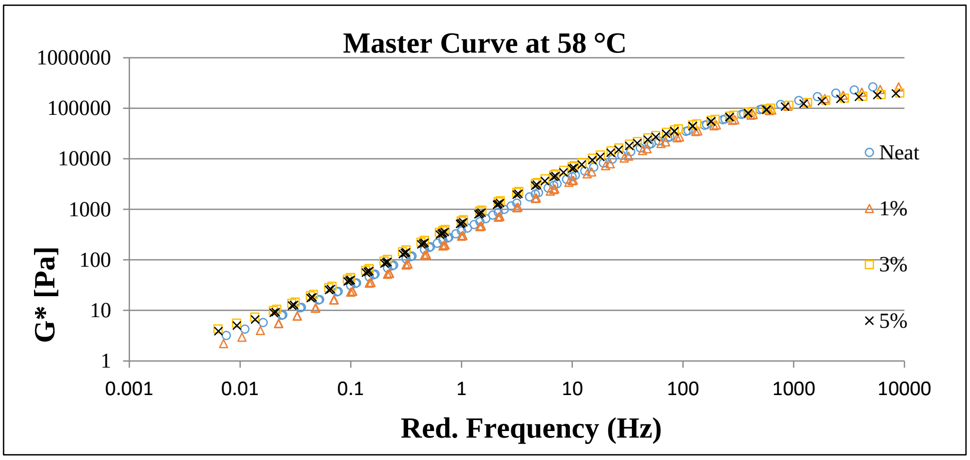

Figure 8: Master Curves Drawn at 58 °C

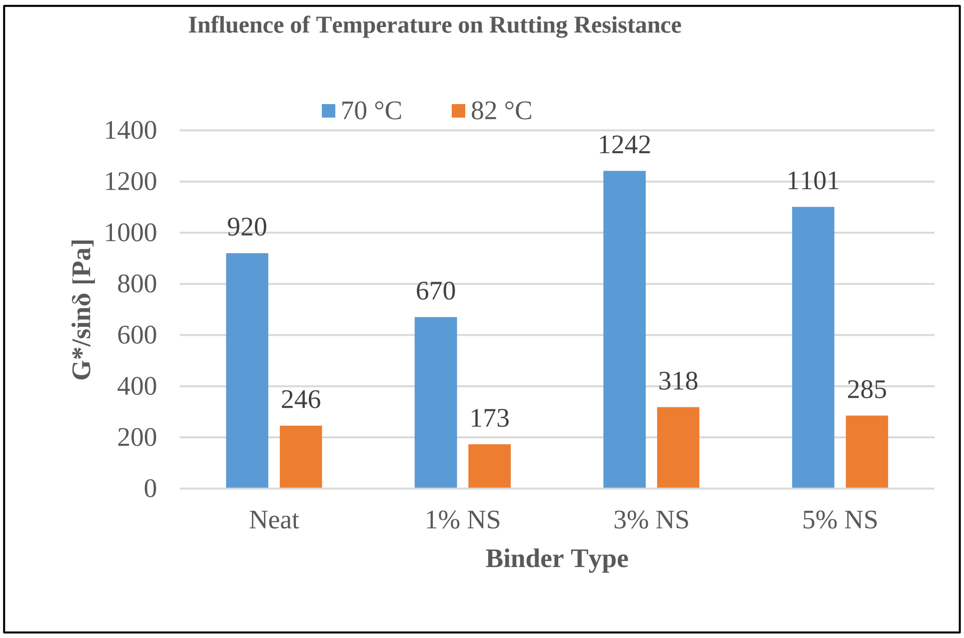

Figure 9: Influence of Temperature on Rutting Resistance

Figure 9 illustrated the influence of temperature on Rutting Resistance G*/sin δ (RR) values. RR values of all the four binders were compared on 10 Hz frequency at temperature of 70 °C and 82°C. RR values at higher temperature 70°C confirmed that 3% NS and 5% NS had passed the Superpave Rut Resistance Criteria (G*/sin δ ≥ 1.00 kPa) (Institute, 2001)at 70°C. Thus it concluded that rutting resistance was improved by 35% and 19.67% at 3% and 5% Nano silica. Thus optimum percentage recommended to use modifier is 3% NS. The results of literature review shows that the most suitable NS content is 3% which decreases the flow values and increases the marshal stability,(Ahmad M. Sawan.,2017).

7. CONCLUSIONS:

- Ductility and penetration of Bitumen was reduced while softening point was increased by incorporating Nano silica in bitumen as shown by the results of conducting conventional tests of bitumen.
- The optimum content of modified NS bitumen noted was 3%.
- The Rutting resistance was improved to be 35% as compared with Neat bitumen.

8. RECOMMENDATIONS:

The recommendations are given below:

- Rutting depth on modified NS asphaltic mix may be evaluated in laboratory by using Hamburg Wheel Tracking Test.
- Ageing tests may be performed on modified NS binders to determine the high temperature performance grade of binder.

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Analysing the Public transport passenger satisfaction in Abbottabad

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Abstract

Passenger satisfaction is considered as one of important factor to shift the commuters from the private transport to public transport. Recent estimations show that the mode share of public transport in metropolitan cities in Pakistan ranges between 16% to 20% and the situation is more critical in other cities. This paper provides the subjective evaluation of the public transport system by means of satisfaction in the Abbottabad city, Pakistan. Research designed a questionnaire and field survey was conducted to evaluate the responses of public transport passengers. Nineteen service attributes regarding public transport service attributes were investigated in the survey. The results indicated that passenger's satisfaction from the overall quality of public transport system and nineteen explanatory variables. The findings of this paper draw the attention of government authorities and service provider towards the performance indicators where the satisfaction lag and that need to be improved to shift the modal ship towards public transportation.

Keywords: Public transport, Passenger Satisfaction, Travel Behaviour, Regional Survey.

1. INTRODUCTION:

Transportation is considered as one of the most significant elements of urban sustainability. Efficient transportation networks have direct impact on the public behaviour and travel patterns, adjoining environment, business activities and national economic growth ((Saif et al., 2018), van Lierop et al., 2018, Wilcox et al., n.d.). However, with rapid urbanization and increasing traffic, cities have become more congested, denser and followed by the incessantly increasing traffic problems (Hoor-Ul-Ain, 2019). A recent study by Errampalli estimated the causes of green-house effect and found that 23% of the greenhouse gases are produced by transport industry and about three fourth of which is generated by vehicles emissions causing half million people causalities (Errampalli et al., 2018).

Public transport plays an important role in the overall success of a city's transportation system. It enhances the mobility and provide access to employment, community resources, education, health and recreational opportunities to public ("The Importance of transit," n.d.). Public transportation also helps to reduce road congestion and travel times, air pollution, and energy and oil consumption (Chapman, 2007).

In undeveloped countries of the world, commuter prefer to use private vehicles instead of public transport due to inadequate public transport networks and due to lack of satisfaction from the service attributes (Irtema, Ismail, Borhan, Das, & Alshetwi, 2018,

F. Rahman, 2015, Nguyen, 2019). Pakistan is one of those countries without a well-organized, properly scheduled public transportation system. It is a well-known fact that Public transport service is moderate in Pakistan. There is a space to take effective measure to enhance the service quality as the study revealed that almost 70% population uses the public transport for intracity travel (Khurshid & Naeem, 2012). Therefore, it is important to ensure well-organized, properly scheduled public transportation system to accommodate the growing population.

In terms of public transport, customer satisfaction is defined as the general level to which the expectation of traveller is meet fully with perceived quality and it is the result of single or collective experiences (Tyrinopoulos & Antoniou, 2008). Enhancing service quality of public transport is very important to retain customer satisfaction (F. Rahman, 2015). A study by Smith has shown that quality of service has a profound influence on customer satisfaction or customer loyalty. Customer satisfaction in transportation is a tool used in a decision, addressing the goals, making justification of resources (Smith, 2010). It is the apparent assessment of product or service (Leem and Yoon, 2004). An empirical analysis revealed a positive relationship between customer satisfaction and service quality in public transport sector of Pakistan (Khurshid & Naeem, 2012).

This research aims to find the gap among the ideal and current scenario of public transport passenger satisfaction by recording the responses from public transport users in city of Abbottabad, Pakistan. Research also aims to find the attributes or explanatory variables that are imparting externalities gap among planned and perceived efficiency of transit system.

2. EXPERIMENTAL PROCEDURES:

Public transport plays a key role in development of city. People use dissimilar modes of transport according to their movability needs. Passengers choice of selecting a specific public transport depends on different factors such as comfort, vehicle travel time, safety etc. Numerous study have found the important service attributes of passenger satisfaction i.e. Waiting time ,cleanliness, seats availability, services at bus stop, seats for females, Driver behaviour and skills, fares(Khurshid et al., 2012), seat availability, travel stability, wait time, Driver attitude, ease of boarding and alighting, Travel time and reliability, The condition of stations or stops (Wong et al., 2017), Platform infrastructure, cleanliness, safety, security, catering and drinking water facility, washroom, toilets and other passenger amenities, stairs/escalators (Ghosh et al., 2017), Accessibility, information, comfort, security, environmental impact, availability, time, customer care (Trompet et al., 2013), Availability of parking space, Traffic congestion, Travel distance and time, fare, accessibility, services frequency, crowding (Tyrinopoulos and Antoniou, 2013). In Abbottabad city major mode of public transport is Suzuki, Carry dabba/carrydhaba, Wagon and others. Suitable Service attributes were selected for measuring passenger satisfaction of Abbottabad city is regarding to fares, driver behaviour, driver driving skills, walking time to access the P.T, Service frequency of vehicles, cleanliness of vehicles, service frequency, enough seats are available, ease of getting on/off from the vehicles, Roads are uncongested and clear, safety in vehicles during evening/night, waiting time, seats are comfortable, vehicles never breakdown on road.

This research developed questionnaire on five-point Likert Scale. Sample size was calculated by using Solvin formula ($n=N/ (1+Ne^2)$) and by taking confidence level of

95%, Z-score of 1.96 and marginal error of 3.3% for 1.3 million population of Abbottabad. Research recorded 710 responses from commuters.

3. DATA RESULTS:

The results of the research unveil the subjective evaluation of transport system in Abbottabad. The research collected the demographical as well as the satisfaction for nineteen explanatory variables to find the important attributes of transit system on likert scale. Figure 1 and Figure 2 shows the demographic statistics of the recorded data. Figure 1 illustrates that the 49.8% of the respondents belongs to age groups of 18-30 years, age group <18 years have 25.49 % participation in 710 respondents. Figure 2 explain the division of respondents based on their profession where the recorded proportion of students was the major proportion in survey and comprises of about 54.37% and least is Unemployed which is 4.23%. As most of businessman use mostly their own vehicle and their percentage in graph is less which is 8.87%.

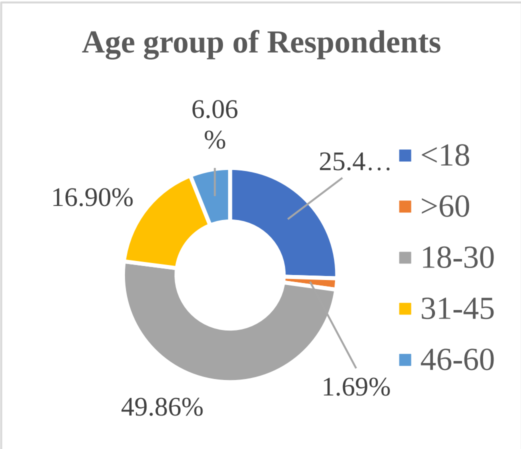

Figure 1: Age group of survey respondents

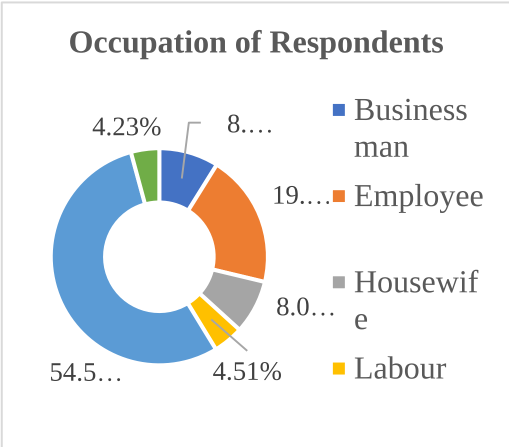

Figure 2: Occupation of survey respondents

Table 1 reveals the detailed demographic profile of the collected survey responses

Table 1: Demographic profile of respondents

Personal characteristics	Number of respondents	Personal characteristics	Number of respondents
Gender		**Public transport usage**	
Male	399	Suzuki Van	378
Female	309	Carry Dabba	150
Others	2	Wagon	57
		Other	125
Age group		**Factor important for journey**	
< 18	181	Cost	113
18-30	354	Time	454
31-45	120	Accessibility	57
46-60	43	Reliability	66
>60	12		
Education		**Time to access nearby public transport**	

Primary	44	< 5 min	205
Matric	232	5-10 min	372
Graduation	227	>10 min	132
Master	165		
Illiterate	42		
Occupation		**Public transport used in a week**	
Student	387	<5 times	299
Employee	141	5-10 times	254
Labor	32	>20 times	157
Businessman	63	**Frequent point of destination**	
Housewife	57	Work	182
Unemployed	30	Educational institute	307
		Shopping	89
		Other	131

Figure 3 is illustrating the frequently used public transportation mode in Abbottabad city. As pink bus service has started its operation after the survey data collection, therefore bus service was not among available mode of intracity public transportation in Abbottabad city. Evident from graph that Suzuki van is major public transport mode in Abbottabad which is 53.24% and second major mode is Carry dabba which is 21.13%. It was interesting to find that Careem service is dominant demand responsive public transportation as respondents mentioned for their preference in comments. Nevertheless, Careem in others option as the demand responsive public transport was not the scope of the study. It is interesting to compare the results of this study with a similar study in Amman where author revealed the public transport satisfaction for bus, mini buses and jitneys against nine service attributes and identified similar deficiencies (Imam, 2014).

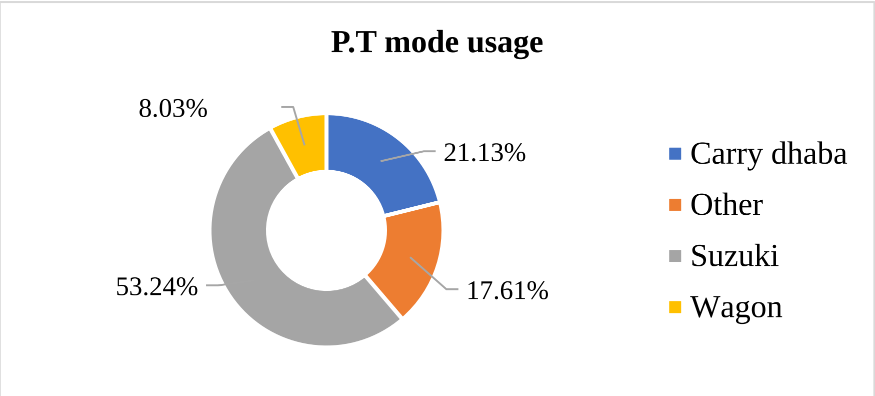

Figure 3: The frequently used public transportation mode in Abbottabad city

3.1 Respondents opinions towards the sevice attributes:

Views about the service attibutes of public tansport of Abbottabd city is presebted in table

Sr.#	How much you are satisfied with following	Very Dissatisfied (%)	Dissatisfied (%)	Neutral (%)	Satisfied (%)	Very Satisfied (%)
1	Fares/ticket	8.73	23.52	21.41	40.56	5.7
2	Walking time	5.7	23.52	21.97	47.89	7.61
3	Service frequency	5.21	22.68	22.48	37.89	7.75
4	Waiting time	5.63	24.23	24.37	38.87	6.90
5	Vehicles travel time	12.11	32.82	23.46	27.46	3.94
6	Ease boarding	13.94	31.41	20.85	29.86	3.94
7	Seats comfortability	19.01	32.39	21.41	22.25	4.93
8	Leg space	24.51	37.32	17.61	16.62	3.94
9	Seat availability	15.35	30.70	23.10	24.08	6.76
10	Vehicles cleanliness	17.75	27.32	25.49	21.97	7.46
11	Vehicle breakdown	9.30	20.42	33.42	29.86	6.90
12	Fatigue	14.08	35.07	24.79	21.55	4.51
13	Driving skills	14.79	28.59	24.79	25.21	6.62
14	Diver behaviour	16.48	26.06	26.34	24.08	7.04
15	Dropoff efficiency	7.04	16.76	20.42	42.82	12.96
16	Security at night	12.11	19.30	23.94	35.07	9.58
17	Route availability	6.76	20.70	24.79	35.92	11.83
18	Roads congestion	31.41	34.79	18.45	11.13	4.23
19	Overall quality of public transport	8.45 %	33.56 %	31.97%	6.90%	2.11%

Table 2: Respondents opinions towards the sevice attributes

It is interesting to see that more than 73% of the survey respondents are not satisfied with the overall satisfaction of public transport. Moreover, the extreme dissatisfaction from the service attribute can be seen where more than 84% of the respondents are not satisfied with the continuous flow of public transport service because of the road congestion. Likewise, similar trend can be seen for the service attribute of public transport causing fatigue, poor cleanliness, poor leg space available and travel time. This study provides an ample opportunity for the transport operators to focus on the lagging service attributes of the transit system to attract more passengers from private transportation. Considering the aforementioned, it should also be highlighted that not just the performance of available public transport infrastructure, but the passenger preferences should be considered while planning the public transport facilities.

4. CONCLUSIONS:

Systematic flow of public transport is a vital factor for development of any sustainable city. This research reveals the philosophy of externalities in transport system and discloses one of two important perspective of accessing the efficiency. Though conversional planning practices focuses on the infrastructural, supply & demand, cost effectiveness in terms of fair box ratio, accessibilty and relevant traits of urban transport systems but the second perspective in terms of perceived quality is usually ignored. Manifestation of the results give us a clear portrayal of usefulness of perceived satisfaction by the users of transport system.

Furthermore, from the aforementioned results, it can be concluded that that there is serious need of improvement in public transport in Abbottabad. As the Suzuki vans are the major public transport mode and major cause of traffic congestion, traffic law enforcement and the traffic management require a serious attention to make Abbottabad a liveable community and a sustainable city.

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Geotechnical Engineering

Paper ID: 701

Effect of Addition of Granular Soils on Physical Properties of Clayey Soil

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Abstract

The experimental work is performed to improve the geotechnical properties of available clayey soil by adding admixtures or granular soil i.e. Sand up to some suitable proportion. The suitable proportion to form the optimum mix in which the granular soil can be added in the clay soil was decided after performing the proctor compaction test. These optimum mixes obtained by the experimental investigation were carried further and checked for index properties of clayey soil. Materials has been used to improve the geotechnical properties of soil, in this regard experimental work was performed and it is studied that the change in properties of clayey soil by adding granular soil sand up to suitable selected proportion. After this we were able to check the change in geotechnical properties of clayey soil. To understand the effect of granular soil on clayey soil different samples were made. To compared properties of these samples different tests were performed on these soil samples. To Check and understand the effect of the addition of granular soil on the engineering properties of a clayey soil, almost forty groups of different soil specimens were prepared and tested at different percentages of granular Soil i.e. 0%, 5%, 10%, 15%, 20%, 25% by weight of the parent soil. In this experimental work the compaction properties of clayey soil with the addition of different percentage of granular soil i.e sand from standard proctor test of sample S_A, it is determined that the value of maximum dry density is increased from 18.30 KN/m^3 to 19.55KN/m^3 (6.45% increased) and sample S_B, it is determined that the value of maximum dry density is increased from 18.00 KN/m^3 to 19.30KN/m^3.From Modified proctor test the two soil sample S_A & S_B, it is observed that the value of maximum dry density is increased from 19.40 KN/m^3 to 20.30KN/m^3 and 19.05 KN/m^3 to 19.90KN/m^3 (4.44% and 4.27% increased).

Keywords: SPT: Standard Procter Test, S_A: Sample A, S_B: Sample B, CS: clayey soil.

1. INTRODUCTION:

In our country Pakistan clayey soil covers a big part of the land. Due to lack of land resources in big cities we started to improve the soil properties by adding some other materials as admixtures to change its properties according to our geotechnical requirements. Many of the civil engineering structures are constructed on weak or soft soil which leads us to develop some soil improvement techniques such as soil stabilization technique. (Coduto, D. P.*et al*. 2014)

For in improvement and increased in shear strength of clayey soil we used granular soils sand for to check the shear strength of clayey soil. After this we are able to checked the change in properties of soil. We compared properties of these samples by performed the different geo technical standard tests to find different values and by plotting graphs to find the required information. (Das, B. M.*et al*. 2013)

It is highly risky that the construction of buildings and others structures on weak or soft soil because such soil is mostly chances of highly settlements due to its poor shear strength and high compressibility. So that the changes in certain desired geotechnical properties of soil specially the most important for buildings and others structures bulk density, load carrying capacity, shear strength and permeability properties of soil can be improved by the use of available different soil improvement techniques such as the use of soil stabilization techniques. (Bowles, J. E.*et al*. 2012)

2. EXPERIMENTAL PROCEDURES:

2.1 Test Standards

In this section we discussed those materials which are being used in the research and their properties related to scope. These tests are carried out according to ASTM code. In methodology section we use approach which was adopted for the goal. Soil as a material that occurs naturally in wide range over the earth and its physical properties of different from one site to other are different from one part of site to the other. So, for civil engineering point of view soil is an un-consolidated agglomerate material without organic matter found at or near the surface of earth crust, with witch and upon which civil engineers build their structures.

2.2 Materials

The materials used in this research were Clayey soil and granular soil which was collected from Ravi river near Lahore Pakistan.

2.3 Methodology

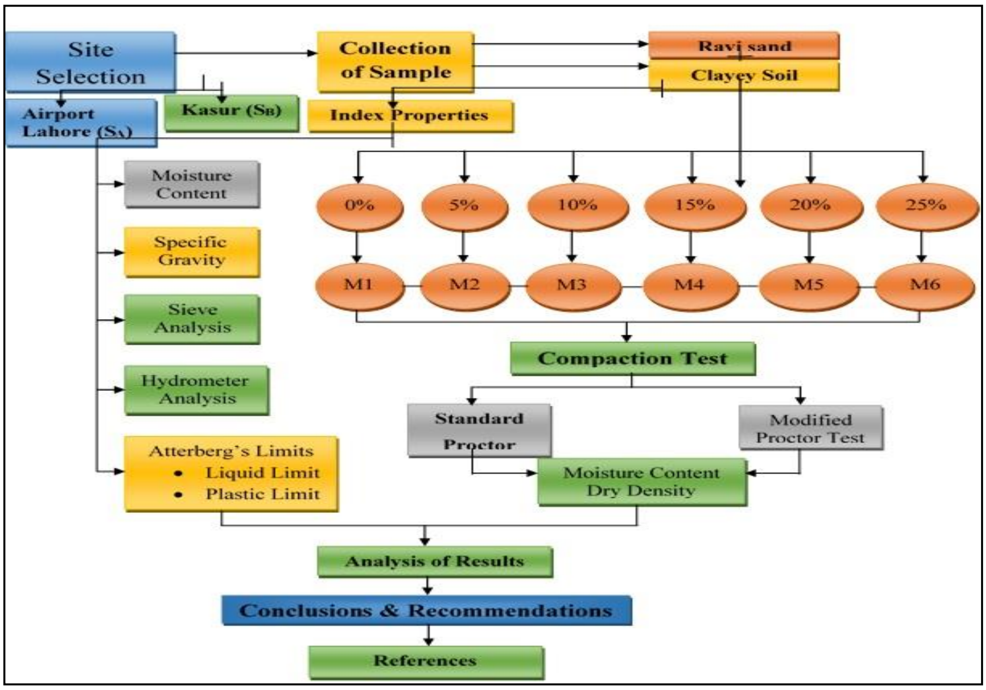

Figure 1: Flow Chart of Research Methodology

4. RESULTS AND DISCUSSIONS:

After taking the sample from the international Airport Lahore and city Kasur the first test we performed was determination of moisture content. Two sample of different site were taken to determine the moisture content. The test was performed after 24 hours of extracting the sample. The moisture content was determined as 13.87% and 6.61% for sample S_A and S_B respectively.

4.1 Moisture Content

After taking the sample from the international Airport Lahore and city Kasur the first test we performed was determination of moisture content. Two sample of different site were taken to determine the moisture content. The test was performed after 24 hours of extracting the sample.
Average Moisture Content of sample 'S_A' located near to Airport Lahore=13.87%
Average Moisture Content of sample **'S_B'** located near to Kasur =6.61%

4.2 Sieve Analysis

Sieve analysis test was performed on two sample to determine the coefficient of permeability C_c and coefficient of uniformity C_u. As the sample is clay so the value of D_{10}, D_{30} and D_{60} and grain size value were shown on graph. So further classification of soil was performed on hydrometer test.

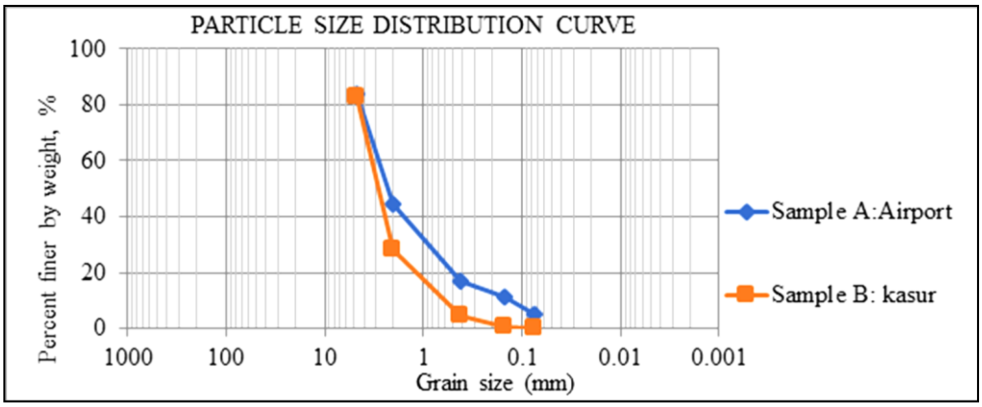

Figure 2: Gradation curve of samples 'S_A, S_B'

Following graph shows the particle size distribution curve of sample 'S_A, S_B'.

Table 1: Coefficient of uniformity and Coefficient of curvature.

Sample	D_{60}	D_{30}	D_{10}	C_u	C_c
A	2.25	0.90	0.15	19.33	1.86
B	3.00	2.00	0.60	3.75	2.96

4.3 Hydrometer Test

ASTM D4221, gives the standard procedure for hydrometer analysis for performing this test on soil sample passing sieve No. 200 for all the soil samples, in order to have an idea about the finer percentage of the soil, like silts and clays. Silt is type of soil particle having plasticity (size ranges from 0.002mm to 0.05mm). Clay is the type of soil having plasticity (size ranges from less than 0.002mm).

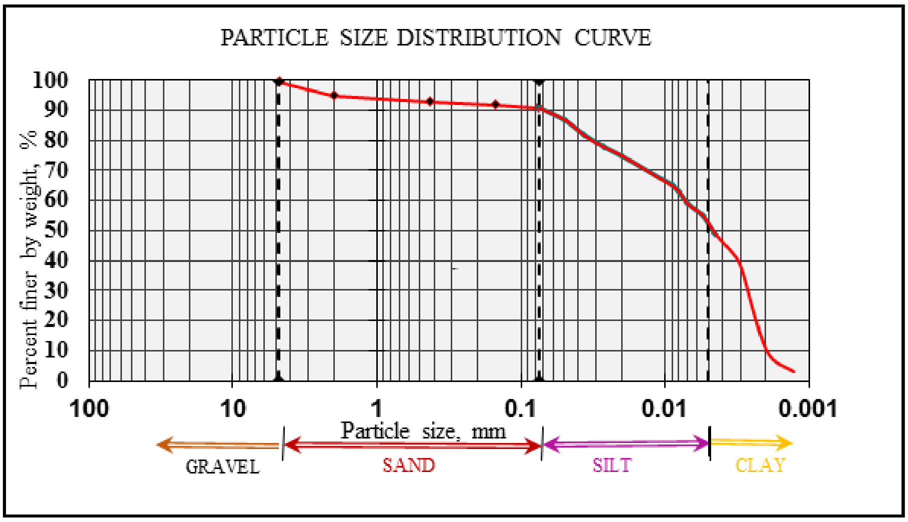

Figure 3: Particle size distribution by sieve analysis

Graph shows the further particle size distribution curve of soil like clay, silt and sand percentages.

Table 2: Sieve and Hydrometer analysis.

Sieve and Hydrometer analysis						
Sample	Coarse Gravel	Fine Gravel	Coarse Sand	Fine Sand	Silt	Clay
A-B	0%	0%	0%	9 %	44 %	47 %

4.4 Atterberg's limits

After the sieve analysis the value of Cc and Cu shows that the soil is greater than 0.75mm so performed Atterberg's limit test for further classification of our soil sample, two tests were performed for the determination of Atterberg's limit.

- Liquid limit
- Plastic limit

4.4.1 Liquid limit

Liquid limit test was performed for further classification of our sample. This test was performed on two samples, in table 4.5.It is shown that the average liquid limit of these two sample is (S_A=23.5%and S_B=22%).this liquid limit is used for classification of soil.

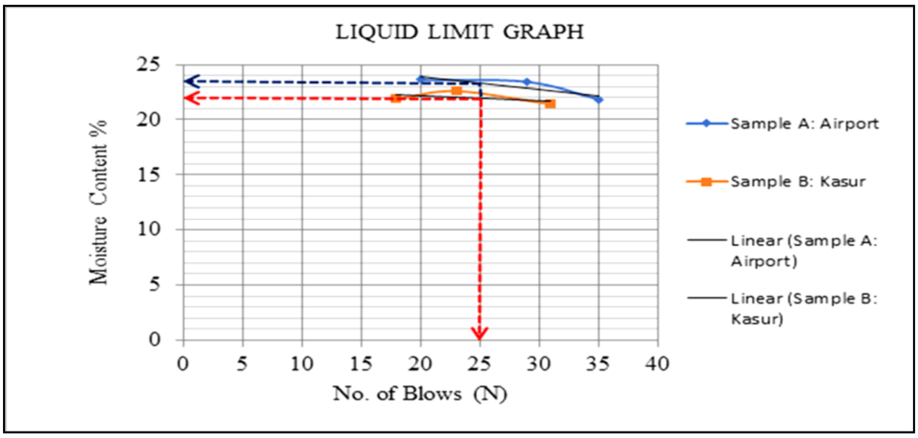

Figure 4: Liquid Limit

Graph shows the liquid limit curve of sample 'S_A, S_B' at the no. of blows 25.

Table 3: liquid limit of sample S_A&S_B.

Sample	Liquid Limit (%)
S_A– Near Airport Lahore	23.5
S_B –Near City Kasur	22

4.5 Fineness Modulus of Sand

Fineness modulus of Ravi sand was determined to study the effect of fineness modulus on density. Fineness modulus of Ravi sand was determined according to ASTM C33 and found 0.9.

4.6 Standard proctor test

The test was performed to measure the maximum dry density and optimum moisture content (OMC) by addition of sand in soil sample.

Table 4: Results of Standard proctor test sample 'S_A' with Ravi Sand.

Sample	Sand added (%)	Density KN/m^3	OMC (%)
1	0	18.30	14.20
2	5	18.50	10.50
3	10	18.70	11.40
4	15	19.10	9.70
5	20	19.25	10.10
6	25	19.55	10.50

Table shows the results of standard proctor test for sample S_A with Ravi sand of different percentages (0% 5%, 10%, 15%, 20% 25%).

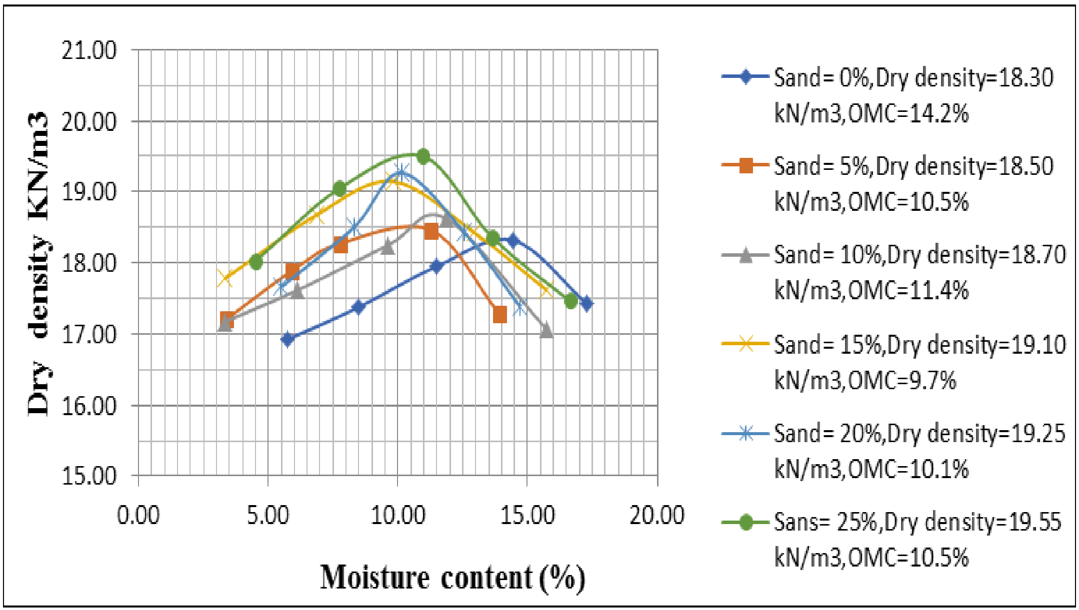

Figure. 5: Standard Proctor test of sample 'SA ' near to Airport Lahore

The test was performed to determine the maximum dry density is 19.55 kN/m^3 and optimum moisture content (OMC) is 14.20 % by addition of sand in soil sample.

4.7 Modified proctor test

The test was performed to determine the maximum dry density and optimum moisture content (OMC) by adding sand in soil sample.
Weight of soil sample =3000gm
Weight of sand added = 0% 5%, 10%, 15%, 20% 25%

Table 5: Results of Modified proctor test sample 'S_A' Airport with Ravi Sand.

Sample	Sand added (%)	Density KN/m^3	OMC (%)
1	0	19.40	11.40
2	5	19.60	9.70
3	10	19.75	9.40
4	15	20.10	9.60
5	20	20.20	10.10
6	25	20.30	9.60

Table shows the results of Modified proctor test for sample S_A with Ravi sand of different percentages (0% 5%, 10%, 15%, 20% 25%).

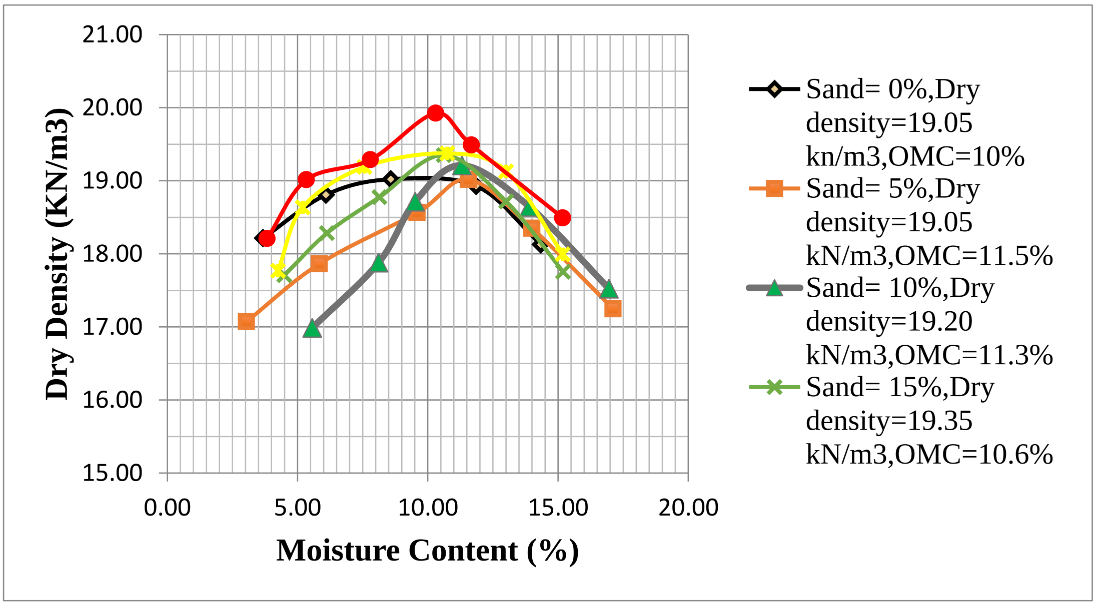

Figure. 6: Modified Proctor test sample ' SA ' located near to Airport Lahore

5. CONCLUSIONS

This research was carried out to find the effect of sand on compaction characteristics of Airport and City Kasur soil sample to correlate different properties of soil such as, maximum dry density and optimum moisture content. This research resulted into following conclusions.

- This soil samples are a fine soil of low plasticity and have 51% clay and 40% silt in it;
- According to USCS soil classification system the soil is CL, ML (clay with low plasticity index);
- For the determinations of compaction characteristics, sand is added 25% with the increment of 5% in each test for standard and modified compaction test;
- From standard proctor test of Airport sample, it is observed that the value of maximum dry density is increased from 18.30 KN/m^3 to 19.55KN/m^3 the sand ranges 0% to 25% (6.45% increased);
- From standard proctor test of City Kasur sample, it is observed that the value of maximum dry density is increased from 18.00 KN/m^3 to 19.30KN/m^3;
- From Modified proctor test the two soil sample, it is observed that the value of maximum dry density is increased from 19.40 KN/m^3 to 20.30KN/m^3 and 19.05 KN/m^3 to 19.90 KN/m^3 (4.44% and 4.27% increased);
- From the result of standard and modified proctor test it is concluded that 25% of sand added in soil is the increase value of the maximum Dry density;
- For the determinations of compaction characteristics, we chose one of the best clayey soils at two different site samples according to maximum Dry density at the optimum moisture content;
- The value of γ of 0% soil was 19.4 KN/m^3 by performing modify proctor test. But after the addition of 25% sand, the density increase to 20.30 KN/m^3.this shows that soil improves its density hence bearing capacity and shear strength of this soil is increased. This shows that this soil can withstand against the settlement; and

- This result helps us to the civil engineer to use the adding material more economical in clayey soil for construction of civil engineering projects.

RECOMMENDATIONS

- The addition of sand and other than material i.e. flies ash, lime, cement and tile waste should be used to study the compaction characteristics of Airport Lahore and city kasur soil;
- More soil sample collected from different area of the site should be tested in same way to verify these results;
- This project covers only a few properties of soil, including particle size distribution, Atterberg's limits, and compaction characteristics of soil by adding sand. Other tests including strength characteristics, plastic index. Liquid limit, compressibility and swell pressure may also be carried out to determine the characteristics of soil by adding sand in it;
- Estimate of Ravi sand should also be considered w.r.t Cost, durability and accessibility for a project; and
- Combination of sand and soil should be studied to determine compaction characteristics of Airport and Kasur soil.

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Paper ID: 702

A Semi-Analytical Framework for Suction Caisson Installation in Sand

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Abstract

Controlling the installation procedure for a caisson foundation requires a preliminary design phase. For suction assisted installation of caisson foundations in sand, such a design phase is important to predict the force required to overcome soil resistance as caisson is pushed into the seabed, and to identify the limits to a safe installation process such as the occurrence of seepage induced piping. The present paper provides a framework where analytical expressions are obtained for the required suction magnitude to overcome soil resistance to caisson penetration, these analytical expressions are derived for a normalized caisson geometry, based on compiled results obtained from finite element analysis of seepage around a caisson wall. The proposed suction predictions for the whole process of caisson installation in sand are validated against field trials reported in the literature.

Key-words: Suction caisson, normalized seepage problem, polynomial regressions, suction profile.

1. INTRODUCTION:

Over the past few decades, suction caisson gained popularity with the rapid development that took place within the oil and gas industry. Economic advantage and easiness of installation and decommissioning added substantial value to this type of foundation, which appears to become a competitive solution for future use as a foundation for offshore wind turbines (Byrne and Houlsby, 2003, Byrne et al., 2002).
Design procedures of caisson installation depend on the type of soil that makes the seabed formation (Houlsby and Byrne, 2005b, Houlsby and Byrne, 2005a). In sand, the driving force pressure differential results into an overall downward force. Such a force acts in conjunction with the seepage taking place around the caisson wall, which reduces frictional resistance. The present paper aims at proposing a standard formulation of the design procedure for the installation of suction caissons in sand. The present formulation uses a standard mathematical description of the installation problem that integrates the solution of the seepage counterpart. In order to achieve this, we propose a normalized framework for the installation process, using a normalization procedure similar to the one adopted in (Harireche et al., 2014, Harireche et al., 2013).

Polynomial regressions are used to obtain standard forms of the normalized pore pressure. The proposed mathematical model will be obliged in accessing the critical condition in suction caissons installation design procedures.

2. NORMALIZED SEEPAGE PROBLEM:

We a suction caisson of radius *R*, height *L*. The depth of caisson penetration into the seabed is denoted as *h*. The seabed profile consists of homogeneous sand. Figure 1 shows a vertical section where only half of the caisson is represented. A cylindrical system with coordinates *r** and *z** is used for the normalized geometry with respect to caisson radius, *R*. A Suction of magnitude $\bar{s}$ is applied, which causes an excess pore pressure, $p^* = p/\bar{s}$.

Seepage equations is: $\nabla^{*2} p^* \equiv \dfrac{\partial^2 p^*}{\partial r^{*2}} + \dfrac{1}{r^*}\dfrac{\partial p^*}{\partial r^*} + \dfrac{\partial^2 p^*}{\partial z^{*2}} = 0$.

Due to impervious caisson wall: $\partial p^* / \partial r^* = 0$ on AD and on z^*-axis due to symmetry.

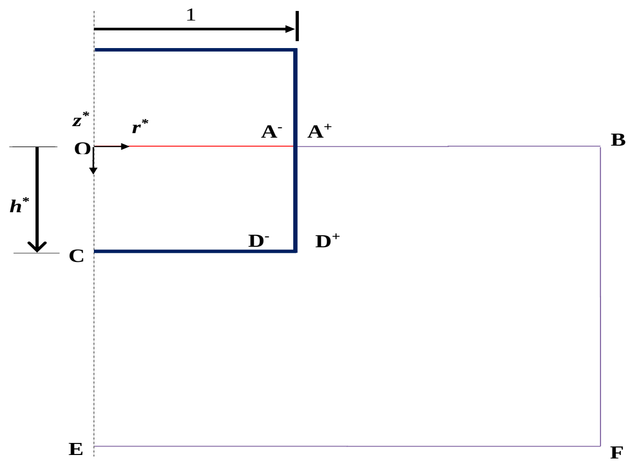

Figure 1: Normalized caisson geometry and surrounding soil

The normalized pressure satisfies the conditions: $p^* = -1$ on OA-, $p^* = 0$ on A+B, BF, and EF. The normalized pressure gradient is denoted by g^*. The normalized seepage problem is solved with finite elements. Preliminary numerical tests have been performed to ensure that boundaries are far enough from the zone affected by seepage so that no disturbance is caused to the predicted results. The mesh, which consists of six-node Lagrange triangular elements in axisymmetric geometry, has been refined around the caisson wall to the extent that ensures convergence. Figures 2 and 3 show the contours of normalized pressure and normalized pressure gradient. Note that in terms of magnitude, pressure gradient is higher on the inner side. This is consistent with the large scale experiments performed by Chen et al. (Chen et al., 2016). A significant drop of inner soil pressure was observed in these experiments and this was induced by seepage. Normalized excess pore pressure, p^* can be represented with the following polynomial regressions:

$$p_o^*(z^*) = \sum_{k=0}^{6} a_k(h^*) \times \left(z^*\right)^k \quad , \quad p_i^*(z^*) = \sum_{k=0}^{6} b_k(h^*) \times \left(z^*\right)^k \qquad (1)$$

Where indices 'o' and 'i' are used to denote pressure values on the outer and inner sides of the caisson wall, respectively. A degree six of the polynomial interpolation ensures a coefficient of determination (R-squared value) larger than 0.999. The coefficients a_k and b_k depend on the normalized penetration depth, h^* and have the expressions:

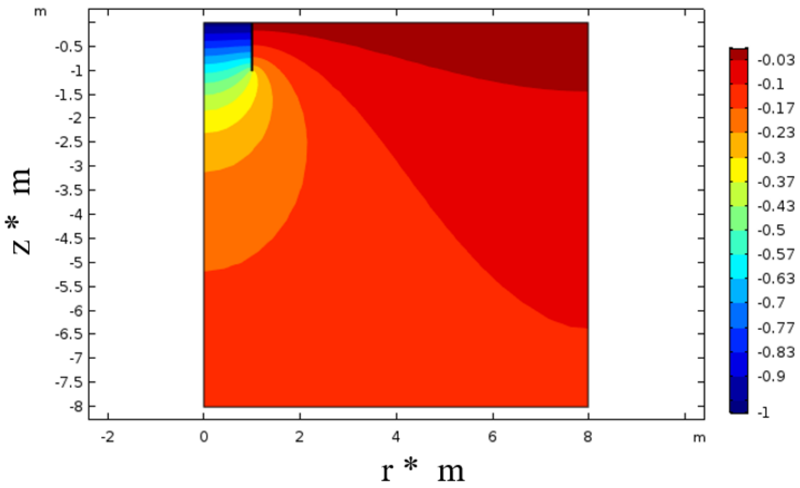

Figure 2: Contours of normalized excess pore pressure

$$a_0(h^*) = \alpha_0 h^* + \beta_0, \quad a_k(h^*) = \alpha_k \left(h^*\right)^{\beta_k}, \quad k = 1,\ldots,6 \qquad (2)$$

$$b_0(h^*) = \gamma_0 h^* + \eta_0, \quad b_k(h^*) = \gamma_k \left(h^*\right)^{\eta_k}, \quad k = 1,\ldots,6 \qquad (3)$$

The constants α_k, β_k, γ_k and η_k, $k = 0,\ldots,6$ are provided in the appendix.

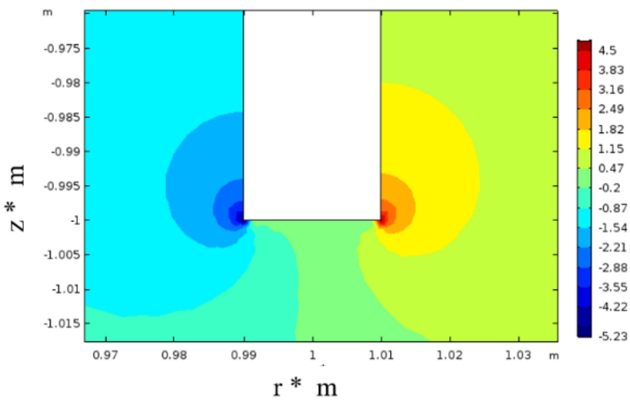

Figure 3: contours of normalized pressure gradient

3. SOIL RESISTANCE TO CAISSON PENETRATION AND SUCTION PROFILE:

At a soil depth z below the mudline, the vertical effective stress is given by

$$\sigma'_{vi}(z) = \gamma' z - \int_0^z g_i(\zeta)\, d\zeta = \gamma' z - \left(p_i(z) + \overline{s}\right) \qquad (4)$$

$$\sigma'_{vo}(z) = \gamma' z - \int_0^z g_o(\zeta) d\zeta = \gamma' z - p_o(z) \tag{5}$$

Where γ' is the submerged unit weight of soil and g_o, g_i denote the z-components of pressure gradients on each side of the caisson wall, respectively. Lateral soil pressure is expressed as:

$$\sigma_{ho}(z) = K\sigma'_{vo}(z) = K\left(\gamma' z - p_o(z)\right) \tag{6}$$

$$\sigma_{hi}(z) = K\sigma'_{vi}(z) = K\left(\gamma' z - \left(p_i(z) + \overline{s}\right)\right) \tag{7}$$

Where K is a soil lateral pressure coefficient. In terms of normalized depth, z^*

$$\sigma_h(z^*) = K\left[2\gamma' R z^* - \overline{s}\left(p_o^*(z^*) + p_i^*(z^*) + 1\right)\right] \tag{8}$$

Note that the term $p_o^*(z^*) + p_i^*(z^*) + 1$ is positive as the sum of normalized pressures on each side of the caisson wall is negative, with a total magnitude less than unity. This shows that the lateral pressure, and hence, the mobilized friction, on the caisson wall, is reduced because of seepage. The lateral frictional force is obtained as the resultant of frictional shear stress acting on both sides of the caisson wall and has the expression

$$F_s = \int_0^h 2\pi R \sigma_h \tan\delta dz \tag{9}$$

Where δ is the angle of friction at the interface caisson-soil. Tip resistance is given by

$$F_t = 2\pi R N_q \int_{R_i}^{R_o} \sigma'_t dr + 2\pi R t\left(\frac{1}{2}\gamma' t N_\gamma\right)$$

Where N_q and N_γ are bearing capacity factors, [3] and σ'_t denotes the vertical effective stress at the caisson tip, defined as:

$$\sigma'_t = \frac{1}{2}\left(\sigma'_{vi}(h) + \sigma'_{vo}(h)\right) = \gamma' h - \frac{1}{2}\left(p_{ih} + p_{oh}\right) \tag{10}$$

Where p_{ih} and p_{oh} are pressure values at caisson tip on the inner and outer sides of the caisson wall, respectively.

Required suction magnitude is given by the ratio of total resisting force over unit caisson cross sectional area and is expressed as:

$$\overline{s} = \frac{F_s + F_t}{\pi R^2} \tag{11}$$

4. RESULTS AND ANALYSIS:

The analytical approach presented in previous sections is validated against field data obtained from trial experiments and reported by (Chen et al., 2016).These field trials took place at Tenby on the south coast of Wales. The seabed formation consists of dense sand with a saturated unit weight of 18.3 kN/m^3, an angle of shearing resistance of 40° and a factor $K\times$ *tan* (δ) estimated at 0.48. The caisson prototype used has a diameter of 2 m, a height of 2 m and a wall thickness of 8 mm. Figures 4 shows the theoretical predictions of installation suction and the results obtained from the field trial. It can be seen from this figure that theoretical predictions are in range. The field test were suspended at 1.4 m depth and hence reflect sudden drop in pressure.

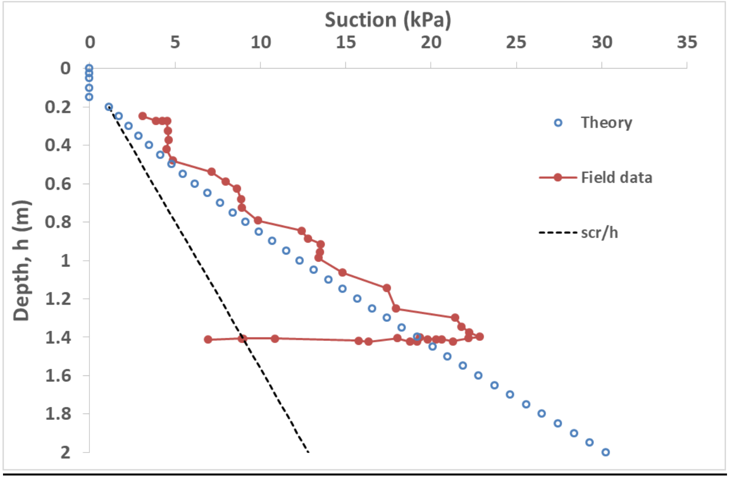

Figure 4: Installation suction. Theoretical prediction and experimental results for field test

The regression model can be applied to layered soil with modifications. However, it cannot be directly applied to bedrock.

5. CONCLUSION:

The proposed unified procedure where seepage was first solved for a normalized caisson geometry, then normalized excess pore pressure was expressed in terms of polynomial regressions. This resulted into an analytical representation of the required suction as a function of normalized penetration depth. The analytical results obtained were compared with the field trails and found to be in agreement. The model can accurately predict the pressure variation against caisson depth and hence the critical conditions for piping.

ACKNOWLEDGEMENTS:

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APPENDIX:

Coefficients α_k , β_k γ_k **and** η_k **in the expressions** $a_k(h^*)$ **and** $b_k(h^*)$, $k = 0,...,6$.

$\alpha_0 = -0.0001$, $\beta_0 = 0.0017$, $\gamma_0 = 0.0001$, $\eta_0 = 0.9983$

$\alpha_1 = 0.1214$, $\beta_1 = -1.4920$, $\gamma_1 = 0.4003$, $\eta_1 = -0.6507$

$\alpha_2 = 0.8737$, $\beta_2 = -2.0812$, $\gamma_2 = 0.9029$, $\eta_2 = -2.0802$

$\alpha_3 = 3.9790$, $\beta_3 = -3.070$, $\gamma_3 = 4.1311$, $\eta_3 = -3.0714$

$\alpha_4 = 8.6348$, $\beta_4 = -4.0763$, $\gamma_4 = 8.9302$, $\eta_4 = -4.0753$

$\alpha_5 = 8.6737$, $\beta_5 = -5.0760$, $\gamma_5 = 8.9749$, $\eta_5 = -5.0763$

$\alpha_6 = 3.3330$, $\beta_6 = -6.0760$, $\gamma_6 = 3.4473$, ,

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Effect of Sugar-Cane Bagasse Ash on Engineering Properties of Low Plastic DGK Soils

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Abstract

Soft cohesive soils that contain the significant percentage of montmorillonite, illite and mica in their mineralogical composition may undergo volume changes upon interaction with water. Pavements and building foundations constructed on such soils may fail due to change in volume with variation in the seasonal moisture content. If encountered, treatment is essential to improve the shearing strength and enhance the load carrying capacity of such foundation/subgrade soils. Numerous studies have been done to improve such soils by adding various materials such as cement, lime, bitumen, rubber and plastic etc., or by chemical, thermal and electrical stabilization. In chemical stabilization, soil stabilization is achieved by chemical reaction of stabilizer (cementitious material) and soil minerals (pozzolanic material). The use of bagasse ash created by sugar cane industries is ideal for chemical stabilization of soft soils as it is economical, environmental friendly and offers a potent solution for weak soil particle bonding. This study has been carried out to examine the stabilization potential of the subgrade soil of D. G. Khan. Bagasse Ash is a by-product of sugar-cane industry, where bagasse is burnt to produce electricity. Bagasse ash contains high silica and alumina contents and is therefore a pozzolanic material, that reacts with calcium to form cementitious calcium silicate and aluminate hydrates. This study shows an increase of almost 30 times in soaked unconfined compressive strength of stabilized soil and a significant increase in CBR values of subgrade soils. One dimensional swell potential of treated soil also found to decrease from 2.5 percent to almost zero.

Keywords: Low Plastic Soil, Sugarcane Bagasse Ash (SCBA), CBR, MDD, OMC, Atterberg's Limits, Minerals

1. INTRODUCTION

Engineering properties of subgrade and embankment soils are always a major consideration from engineering point of view. The current practice in Pakistan is to use in situ soil as a subgrade or embankment material where possible. In general, soft clays are considered unsuitable as foundation material because of their unpredictable and dramatic behavior all over the world significant problems have been faced during the construction of railway tracks, runways, taxiways, highways and embankments because of dramatic change in soil behavior. The most renowned problems associated with clay are swell-shrink and large variation in the properties such as field moisture content, degree of compaction and shear strength with the fluctuating index properties like liquid limit, plastic limit and plasticity index. Change in volume of clays is associated with the change in insitu, moisture content of soil. This change in volume results in reduction in shearing strength of soils eventually results in several pavement distresses. (Mowafy et al., 1990). It is estimated that damage caused due to expansion of clays is more than twice the cumulative damage caused by other natural hazards, i.e., floods, hurricanes, earthquakes and tornados (Jones and Holtz, 1973). Being a citizen of Pakistan, we are concerned with its transportation system. According to Ministry of Industries and Production (2003), transportation sector accounts for 12% of the total GDP. Roads in Pakistan almost carry 92% of passenger and freight traffic. According to NHA, freight growth rate is 3% and passenger's growth rate is 4.5%. Total road network in Pakistan is approximately 260,000km. In which national roads consist of 140726km and farm to market roads consist of 117233km. Only 13000km of roads are managed and taken care by NHA, which comes equal to only 4% of the total road network existing in Pakistan. This 4 % of the total road network caters for almost 80% of the total road traffic in Pakistan. Almost 5000km roads are classified as fair to poor in Pakistan by NHA. As mentioned earlier, enormous growth rate of traffic is causing deterioration and failure of pavement structures. Pavement deterioration is inevitable because the roads are not designed for the traffic load that usually plies on them. To avoid pavement deterioration, high strength construction materials, modern design procedures and versatile construction techniques must be used. The first and foremost thing to keep in mind is the limited resources available for pavement structures. So, there is a stringent need to use the high strength materials and develop the modern building techniques suitable for the concerned conditions

2. LITERATURE REVIEW

2.1 Clayey Soils

Based on mechanical analysis, plastic soils having particle size less than 0.002 mm (2µm) are commonly known as clayey soils. These soils contain minerals like montmorillonite, illite, kaolinite and mica etc. Plasticity and undrained shear strength are the primary characteristics of clayey soils. Clayey soils are mostly made up of minerals, ranging from microscopic to submicroscopic particles derived from weathering of rocks. Plasticity of soil varies from low to very high with moderate to wide range of in-situ water content. Permeability of these soils is very low. While at higher water content, these soils are considerably sticky. (Terzaghi et al 1996). Individual grains of clay can't be seen with naked eye. (Holtz et al 1981). Clayey soils tend to have more swelling potential then other types of soil. Classification of clayey soils based on swell potential and cation exchange capacity is tabulated as:

Table 1: Soil Classification Based on Swell Potential (Seed et al., 1962)

Soil Type	Very High	High	Medium	Low
Swell Potential	> 25	5 – 25	5 – 1.5	< 1.5

Table 2: Soil Classification Based on Cation Exchange Capacity (Yilmaz, I. (2004)

Soil Type	Very High	High	Medium	Low
Swell Potential	> 55	37 – 55	27 – 37	< 27

Table 3: Typical Values of CEC for Various Clay Minerals (Mitchell 1993)

Colloid Type	**CEC (meq/100gm)**
Kaolinite	2-15
Montmorillonite	80-150
Chlorite	10-40
Hydrous Mica (Illite)	10 – 40

Table 4: Rating of Expansive Soil on the basis of Liquid Limit (Day, 2006)

Liquid Limit	Expansion Potential
0-20	Very Low
20-35	Low
35-50	Medium or moderate
50-70	High
70-90	Very High
>90	Extra High

Table 5: Subgrade Classification Based on CBR (TRH4, 1996)

Material Quality	CBR (%)
Good	>15
Moderate	7 – 15
Fair	3 – 7
Poor	< 3

2.2 SOIL STABILIZATION

Soil stabilization is an improvement in the soil properties such as shearing strength and compressibility by performing physical, chemical, biological or a combination of these techniques to meet the engineering requirements. Soil Stabilization results in reducing compressibility of soil, reducing plasticity, increasing bearing capacity and increasing shear strength.

Mechanical stabilization involves techniques like compaction, pre loading, drainage,

etc. Chemical stabilization of soil consists of a process in which different chemical substances are mixed with the soil to improve its engineering properties. The chemicals directly react with soil particles. These reactions are either cementitious or pozzolanic in nature.

2.3 BAGASSE ASH

According to ASTM bagasse ash is classified as pozzolanic material. As per ASTM definition, pozzolans are

"*A siliceous or siliceous and aluminous material, which in itself possesses little or no cementitious value, but will, when in finely divided form and in the presence of moisture, chemically reacts with calcium hydroxide at ordinary temperature to form compounds possessing cementitious properties*." ASTM, C618, (2005). Pozzolanic activity is defined as the measure of Pozzolanic reaction over time in presence of water. The reaction rate is dependent upon particle properties i.e. definite surface area of pozzolan, chemical composition of pozzolan and reaction conditions. Bagasse is an industrial waste produced in sugar industry, that is used as fuel and the ashes produced from combustion of sugarcane bagasse are known as sugarcane bagasse ash (SCBA) which contain high amounts of unburnt matter, oxides of silica and aluminum are most important components of these ashes (Díaz-Pinzon, L., & Ordonez, L. M. ,2002). Sugarcane bagasse ash does have excellent pozzolanic characteristics and is widely used as the pozzolanic material.

2.3.1 Potential Uses of Bagasse Ash

The pozzolanic nature of SCBA and its availability, makes it an attractive material for utilization in engineering applications. Major engineering applications of bagasse ash includes soil stabilization in conjunction with lime or cement, partial replacement of cement in concrete mixes and manufacturing of low-cost mud blocks for building construction.

2.3.2 Effect of Bagasse Ash on Engineering Characteristics of Clayey Soils

Gandhi, K. S, (2012) successfully used bagasse ash to reduce plasticity index of expansive clays. Gandhi.*et.al*., reported that addition of 10% bagasse ash results in decrease in liquid limit from 72% to 52%, PI from 42% to 27% and shrinkage limit reduced from 21% to 15%. Ashish et al, (2015) used bagasse ash to stabilize locally available medium plastic clay and reported that for addition of 10% bagasse ash, liquid limit of soil reduced from 35% to 26% and PI reduced from 13% to mere 9%.
Chhacchia & Mittal, (2015) utilized bagasse ash for the stabilization of clayey soils. They used up to 28 percent of bagasse ash in soil. They reported an increase in OMC from 22.42% to 27.9% and a reduction in MDD from 1.82 g/cm^3 to 1.34 g/cm^3. Ashish et al, (2015) investigated the effect of sugarcane bagasse ash on the engineering properties of locally available medium plastic clay. With addition of 10% bagasse ash, an increase in OMC from 15.3% to 18% and a decrease in MDD from 1.793 g/cm^3 to 1.692 g/cm^3 was observed. Bagasse ash can produce a significant improvement in CBR and swell properties of soil. Ahmed et al, (2015) reported that addition of bagasse ash none, 1%, 3%, 5%, 7% and 9% to the soil samples caused an increase in CBR value at the rate of 6.47%, 8.63%, 10.97%, 12.05%, 13.5%, 13.85% respectively and at the addition of 11% bagasse ash, CBR value decreased to 13.28%. So, 9% was selected as optimum percentage of bagasse ash. Chhacchia & Mittal, (2015) observed that untreated medium plastic clay had a CBR of 2.1%. It increased to 9.8% with the addition of 24% SCBA. But further addition of bagasse ash up to 28% reduced the CBR value to 6.7%. So, they selected 24% bagasse ash as optimum percentage for the soil under study. Gandhi, K. S, (2012) reported a reduction of free swell index from 150% to 80% with the addition of 10% bagasse ash.

3 MATERIALS AND METHODOLOGY

Soil was collected from a town named Shadan Lond of Dera Ghazi Khan Division, Punjab, Pakistan. Location of soil sample's soource is shown in Figure 1.

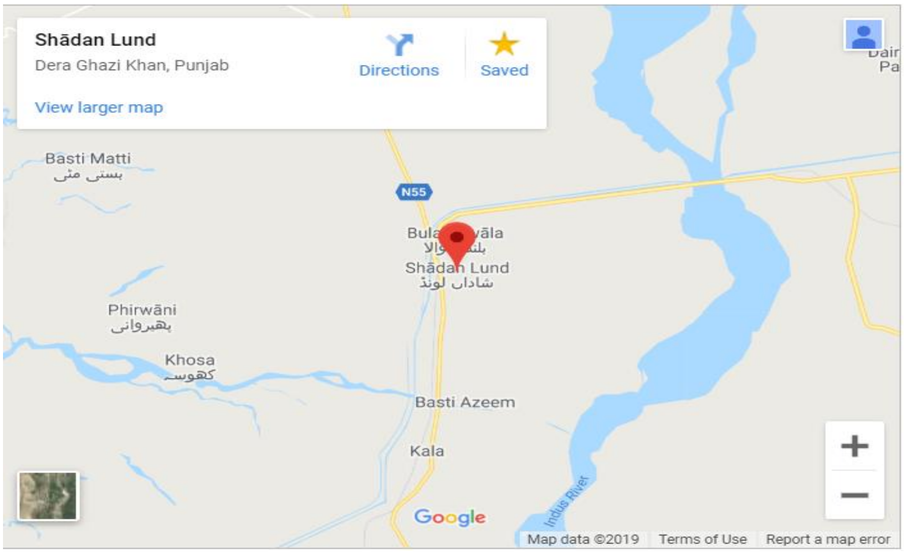

Figure1: Location of Site (From where soil sample was collected

While bagasse ash was collected from Sheikhoo Sugar Mills Limited Sanawan, (figure 2). Muzaffargarh, Punjab, Pakistan. Location of this site is shown in Figure 2. Independent laboratory testing was carried out at geotechnical laboratory, CED, KFUEIT, RYK, Pakistan. Laboratory testing was carried out in two phases as listed below

Phase: 1- Characterization of Untreated Soil
Phase: 2- Characterization of soil treated with Sugarcane Bagasse Ash

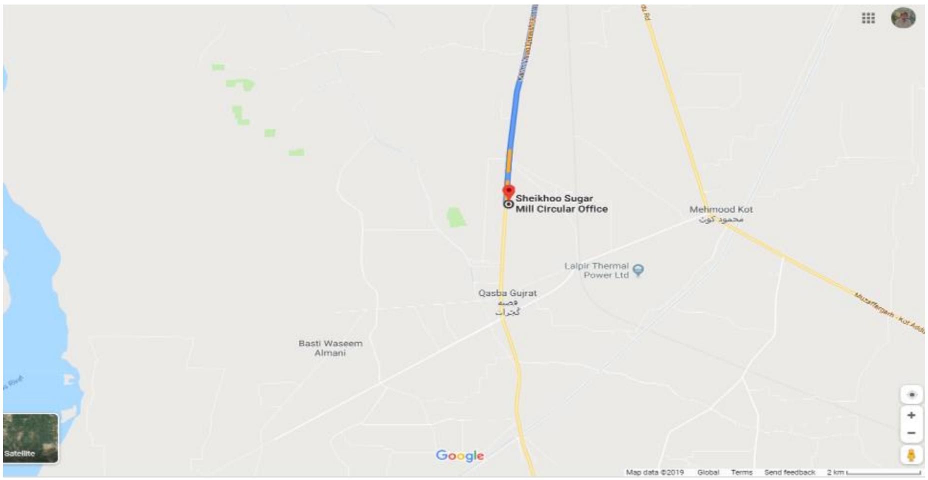

Figure 2: Location of Sheikhoo Sugar Mills Limited (From where SCBA was collected)

Phase 1: Characterization of untreated soil
The first phase of this study consisted of characterizing the untreated soil. In this phase, following properties of untreated soil were examined.

- LL, PL and PI
- MDD and OMC
- UCS
- CBR
- Swell potential

Phase 2: Characterization of soil treated with SCBA

Second phase of this study was aimed to analyze the impact of SCBA on engineering properties of soil.

- LL, PL and PI
- MDD and OMC
- UCS
- CBR.
- Swell Potential

After the analysis of results, optimum dosage/content of SCBA was finalized.

3.1 Soil Sample Preparation

ASTM Standards were adopted to prepare the soil samples for each test. Mixing was carried out by weight. Soil sample was kept in oven for 24 hours to eradicate the field moisture. Dried soil was used for each sample preparation. ASTM standards adopted in this study are as followed

Laboratory tests were performed according to the following ASTM standards,

Sieve Analysis	ASTM D6913-17
Hydrometer Analysis	ASTM D7928-17
Atterberg's limits of soil	ASTM D4318-17
Specific gravity of the soil	ASTM D854-14
Modified Proctor Test	ASTM D1557-12
California Bearing Ratio (CBR)	ASTM D1883-16
Unconfined Compressive Strength (UCS)	ASTM D2166M-16.
Cation Exchange Capacity (CEC)	$e^{(2.63 + 0.002\,LL)}$ Yilmaz (2004)

4 RESULTS AND DISCUSSIONS

4.1 Characterization of Untreated Soil

All the laboratory tests of untreated soil were conducted according to the ASTM standards. Table 6 represents the summary of results of untreated soil characterization.

Table 6 Summary of Untreated Soil Characterization

Soil Property	Values	
Soil Type (USCS)	CL	
% Passing Through Sieve #200	91.4	
LL (Liquid Limit)	49	
PI (Plasticity Index)	25	
Soil pH	8.02	
Maximum Dry Density (MDD) g/cm^3	1.93	
Optimum Moisture Content (OMC) %	13.73	
Specific Gravity (G_s)	2.65	
Clay Minerals (Using XRD)	Predominant Montmorillonite, Mica, illite	
Clay Content %	79	
UCS, (Unconfined Compressive Strength) psi	27	
CBR %	Un-Soaked	3.69
	Soaked	2.14
One Dimensional Swell Potential, %		4.19

4.2 Effects of Bagasse Ash on Atterberg's Limits of Soil

Atterberg's limits of soil reduced with the increase in Bagasse ash content. Table 7 demonstrates the effect of sugar-cane bagasse ash on index properties.

Table 7: Effect of SCBA on Atterberg Limits of Soil

Sr. No.	Soil Sample	DGK Soil	
		Liquid Limit (%)	Plasticity Index
01	Soil Only	49	24
02	Soil + 5% BA	47.3	27.6
03	Soil + 7% BA	42.8	24.3
04	Soil + 9% BA	41.3	22.8
05	Soil + 11% BA	38.7	18.9

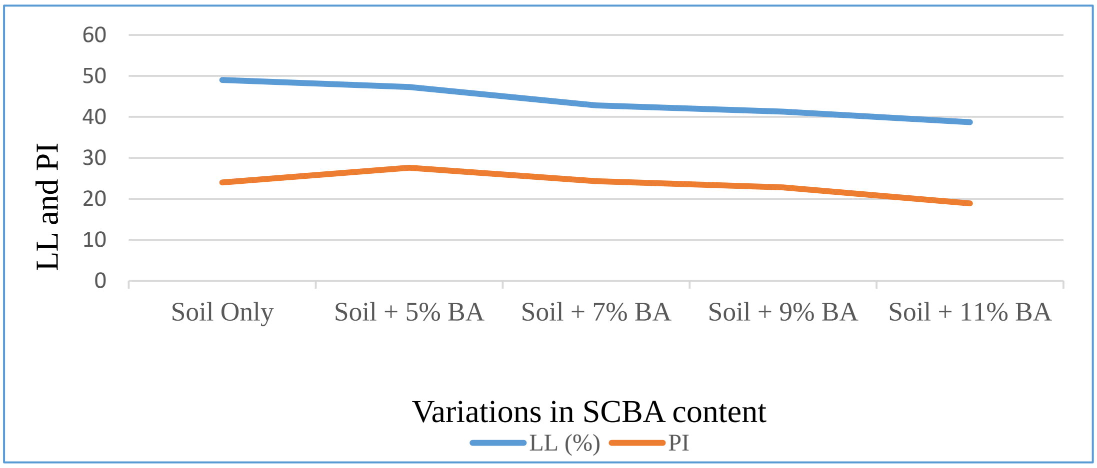

Figure 3: Effect of varying SCBA content on LL and PI of Soil

4.3 Effects of Sugar-Cane Bagasse Ash on OMC and MDD of Soil

Modified proctor test was conducted to analyze the impact of SCBA on OMC and MDD of soil. Addition of Bagasse ash with soil showed considerable change in OMC and MDD of soil. Bagasse ash being a conventional soil stabilizer tend to increase the MC and reduce the density of soil. On the contrary, non-conventional soil stabilizer tend to increase the density by reducing the moisture content. Casagrande apparatus was used to analyze the index properties of soil. Table 8 depicts the real changes in OMC and MDD of DGK soil.

Table 8: Variations in MDD and OMC of Soil

Sr. No	Description	OMC (%)	MDD (g/cm^3)
01	Soil Only	13.73	1.93
02	Soil+ 5% SCBA	15.1	1.81
03	Soil+ 7% SCBA	16.7	1.75
04	Soil+ 9% SCBA	18.6	1.71
05	Soil+ 11% SCBA	17.2	1.79

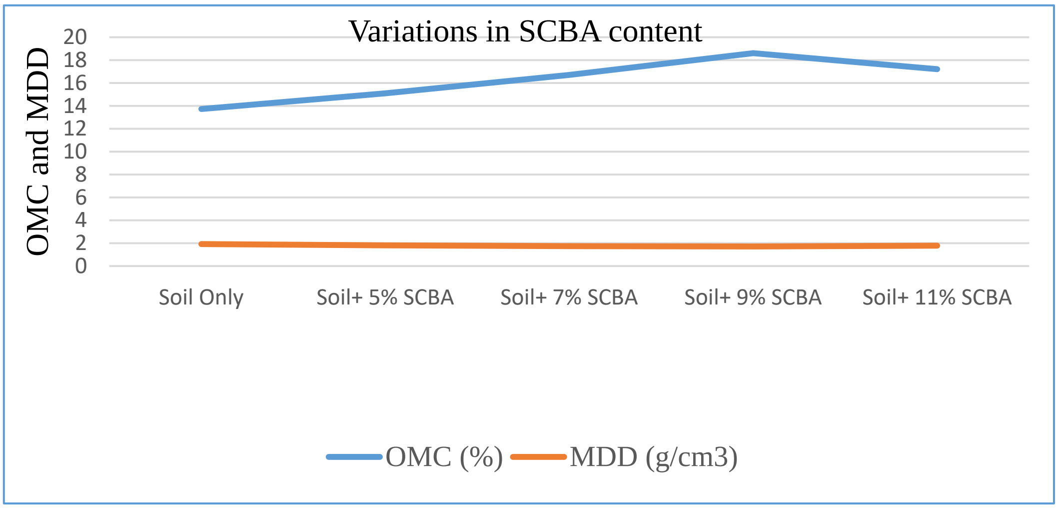

Figure 4: Effect of varying SCBA content on OC and MDD of soil

4.4 Effect of Sugar-Cane Bagasse Ash on CBR of Soil

Samples of CBR were prepared using the MDD and OMC of soil. Un-soaked CBR was conducted to check the behavior of treated soil under dry conditions. While soaked CBR was conducted to assess the CBR under worst conditions i.e. soil exposed to fully moist conditions. For soaked CBR, samples were kept in moist environment for 96 hours. Table 9 shows the impact of bagasse ash on CBR of soil.

Table 9: Variations in CBR of Treated Soil

Sr. No	Sample Description	CBR	
		Un-soaked (%)	**Soaked (%)**
01	Soil Only	3.69	2.14
02	Soil+5 % BA	7.18	2.91
03	Soil+7 % BA	7.93	3.38
04	Soil+9 % BA	9.26	5.18
05	Soil+11 % BA	8.87	4.09

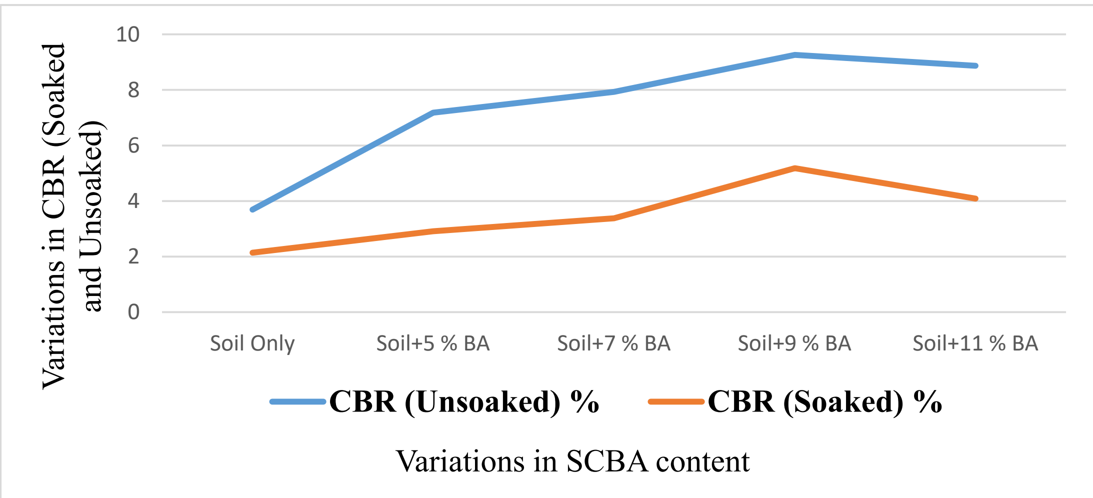

Figure 5: Effect of varying SCBA content on CBR (Un-soaked and Soaked) of soil

5. CONCLUSIONS AND RECOMMENDATIONS

Following conclusions are drawn on the basis of independent laboratory testing conducted on DGK soil (classified as CL).

- Addition of bagasse ash caused the significant reduction in LL and PI of treated soil
- Maximum Dry Density (MDD) of treated soil decreased with the increase in bagasse ash content. After the addition of 9% bagasse ash, MDD started increasing. Which means 9% of bagasse ash content is the optimum quantity which can be used to treat the soil of this class.
- Optimum Moisture Content (OMC) increased with increase in SCBA content. After 9% of bagasse ash, it started decreasing. Change in OMC and MDD can be attributed to the pozzolanic nature of bagasse ash.
- Soil treated with bagasse ash showed considerable increase in CBR under soaked conditions, while under un-soaked conditions improvement in CBR of treated soil was more significant.
- Swell potential of soil decreased to 0.83% from 4.19%, which lies in the limits described by International Building Code (2006).
- On the basis of conclusions, it is recommended that bagasse ash provides an efficient solution for the low plastic soil.

6. RECOMMENDATIONS

- Effect of bagasse ash on engineering characteristics of collapsible soil can be studied
- Shear strength and modulus of resilience of high plastic soil treated with bagasse ash should be extensively examined
- In this study, one dimensional swell potential was taken into account, overall free swell of soil should be examined
- Effect of bagasse ash imported from different places should be studied to standardize its use as a technically potential soil stabilizer
- Bagasse ash can be used in engineering projects as a potential soil stabilizer, as this is abundantly available in Pakistan
- Comparative analysis on impacts of bagasse ash and any other non-conventional (biological) soil stabilizer should be studied to further endorse its usage as a competent soil additive.

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Physical and Strength Characteristics of Fly Ash Stabilized Soft Soil

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Abstract

The engineering properties of soft soils can be improved by mixing with suitable agents and one of them is fly ash. The sole object of this research is to check the effect of varying dosage of fly ash class C on physical and strength characteristics of soft soil. To check the result of fly ash contents on soil properties, a varying dosage from 0-10% was mixed with virgin soil. Soil mineral identification, specific gravity, consistency limits, compaction characteristics, California bearing ratio (CBR) and unconfined compressive strength (UCS) tests were performed on treated as well as untreated samples according to the related ASTM procedures. In addition, UCS tests were performed over an extended period of 0, 3, 7, 14 and 28 days to check the impact of curing period on strength development. The test results showed that California bearing ratio (CBR) and unconfined compressive strength (UCS) increased while plasticity reduced with the increment in fly ash contents.

Keywords: California bearing ratio, fly ash, plasticity, soft soils, unconfined compressive strength

1. INTRODUCTION

In many regions of the world, the common civil engineering problem is the construction of roads and railway over soft soils as such soils exhibit high compressibility and low strength. Subgrade soil with California bearing ratio (CBR) less than 8 are considered as soft soils and not suitable for road construction (Ozdemir, 2016). The common practice to build a railway or highway on soft soils is to replace them with good quality borrow materials. The uneconomical nature of this cut and fill technique forces engineers to upgrade the engineering properties of soft soils using different additives. For the enhancement of physical and engineering properties, one way to strengthen these weak soils is to compact them at their maximum dry density to reduce the pores. Secondly, different treatments can be used in earth-structures to achieve the required properties of geomaterials such as shear strength and compressibility, etc. Soil improvement using various waste products has a double advantage, i.e., the weak ground is stabilized, and hazardous waste from industries is recycled (Show, Tay, Goh, & Ash, 2003). Moreover, the dumping of weak soil and hazardous industrial waste is not required (Sridharan, 2013). There are different types of stabilizers which can be employed to modify the engineering characteristics of geomaterials. The choice of stabilizer based on the nature of chemical reaction with the soil and water. Most commonly used stabilizers include lime, cement, fly ash, and marble dust (Mahvash, López-querol, & Bahadori-jahromi, 2017). Result of mixing fly ash on Atterberg's limits of soft clay was examined by (Jafer, Atherton, Sadique, Ruddock, & Lo, 2018) and concluded that adding fly ash to soft soil causes an increase in liquid limit while the reduction in plasticity index. (Edil et al., 2006) checked the impact of fly ash contents on CBR values of fine-grained soil and concluded that adding 10% fly ash increased the CBR values from 8% to 17%. (Chang, Lund, Page, & Warneke, 1977) studied the physical characteristics of fly ash modified soils and concluded that bulk density reduced with the intensification in fly ash contents. When mixed in the soil, fly ash help to start the process of flocculation of the fine grain particles. Further, it leads to the pozzolanic reactions and cation exchange for the formation of cementitious compounds that improve mechanical characteristics of soil (Prabakar, Dendorkar, & Morchhale, 2004). This process results in the improvement of mechanical performance and workability of soil after compaction (Brooks, 2009). The properties of fly ash treatment have numerous technical guides available in many countries around the world. These guides and recommendations made by various researchers encourage us to back the utilization of this waste product for soil stabilization. The current study aims to assess the outcome of fly ash stabilization on Atterberg's limits, compaction and strength characteristics of soft soil present in Pakistan. Fly ash is a hazardous waste product resulting from coal power plants available easily in Pakistan and cheaper than cement and lime. It is better to utilize fly ash in soil improvement instead of dumping in a fertile land which may pollute the ecosystem.

2. EXPERIMENTAL PROGRAM

The sample of soft soil discussed in this research was retrieved from a roadway site located near district Sheikhupura, Pakistan. An undisturbed soil sample along with a disturbed sample was collected for laboratory testing. Both samples were retrieved after digging a pit to an average depth of 2ft below the ground surface. To keep the in situ moisture contact the undisturbed block sample was sealed with wax while disturbed soil sample was preserved in airtight plastic bags. In-place density and field moisture content were measured from the undisturbed soil sample while the disturbed sample was used for further testing. Various percentages of fly ash, ranging from 0-10% were mixed with the collected soil sample with increments of 2% as specified by most of the researchers. Table 1 illustrates the measured physical characteristics of virgin soil sample used in this research. The list of tests performed on fly ash, virgin soil and amalgamated soil samples is as follows:

a) Water Content Determination (ASTM D-4643)
b) Mineralogical Composition by Petrographic Test (ASTM C-295)
c) Gradation Analysis (ASTM D-422)
d) Atterberg's Limits Test (ASTM D-4318)
e) Modified Proctor Compaction Test (ASTM D-1557)
f) California Bearing Ratio (ASTM D-1887)
g) Unconfined Compression Test (ASTM D-2166)

Table 1: Physical characteristics of virgin soil sample

Test	**Properties**	**Result**
Sieve Analysis	Gravel (%)	1.9
	Sand (%)	32.2
	Silt & Clay (%)	65.9
Atterberg's Limits	Liquid Limit (%)	31.6
	Plastic Limit (%)	18.2

	Plasticity Index (%)	13.4
Specific Gravity	G_s	2.76
Compaction Test	Optimum Moisture Content (%)	9.2
	Max Dry Density (kN/m^3)	19.3
Strength Tests	Unconfined Compressive Strength (kPa)	317
	California Bearing Ratio (%)	3.0
Soil Classification	USCS group symbol	CL

Following overall observations can be drawn based on the tests conducted on the parent soil as well as stabilized soil samples:

- The field water content of the soil sample was calculated based on the average of six samples taken from block sample according to ASTM procedures mentioned in (ASTM, 2014) which resulted to be 7%.
- Determination of in-place density was carried out by taking an average of four core cuter samples taken from block sample which came out to be 17.9 kN/m^3.
- The sieve analysis conducted on soil sample depicts that it has a high percentage of fine particles. The gradation results of both soil sample and fly ash are presented in Figure 1.

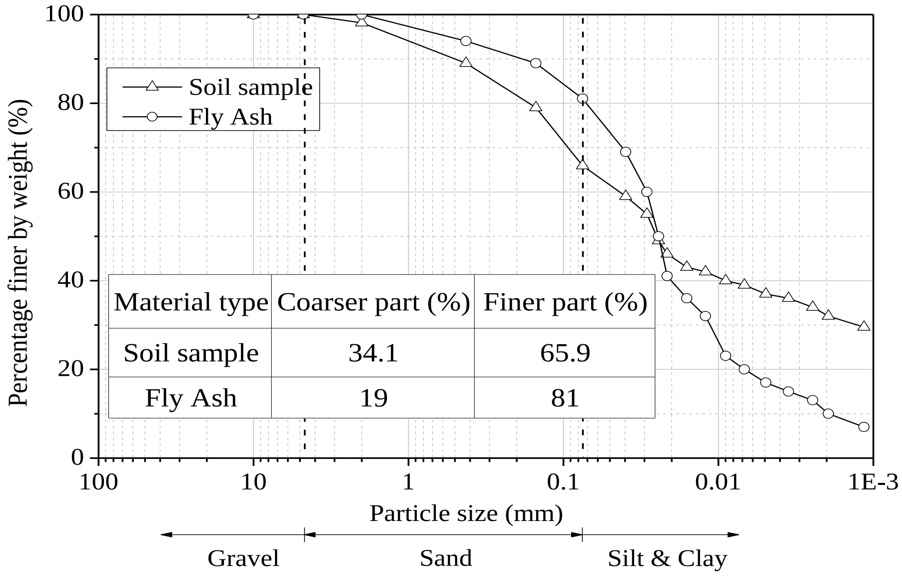

Figure 1. The particle size distribution of soil sample and fly ash

- The mineralogical composition of soil sample and fly ash was determined by petrographic analysis for mineralogical composition following the procedure mentioned in (ASTM C-295). Comprehensive results of the mineralogical composition measured by petrographic analysis are presented in Figure 2.
- Consistency limit state tests were performed on the material finer than sieve # 40 (ASTM International, 2010) and the results indicated that virgin soil sample has a liquid limit (LL) and plastic index (PI) of 31.6% and 13.4% respectively. The same practice was continued for blended samples by mixing fly ash to the soils at the rate of 2% by weight.
- Specific gravity was measured according to ASTM D-854 and based on the average of three trials; it was found that virgin soil has specific gravity 2.76.

- Modified Proctor compaction test was performed according to the guidelines mentioned in (ASTM, 2009) with the help of mechanical mixer and the modified dry unit weight ($\gamma_{d\,max}$) of soil was 19.3 kN/m^3 with optimum moisture content 9.2%.
- Soaked CBR test was performed on virgin as well as fly ash-soil mixes. After compaction of soil mixture, 10 lbs surcharge plates were placed on them. After soaking for 96 hours the prepared samples were tested at 1.3 mm/min penetration rate.
- Cylindrical samples with height/diameter ratio as two (height 76.2 mm and diameter 38.1mm) were tested for unconfined compressive strength (UCS) test in a strain-controlled machine and the final value of UCS was reported based on the average of two samples.

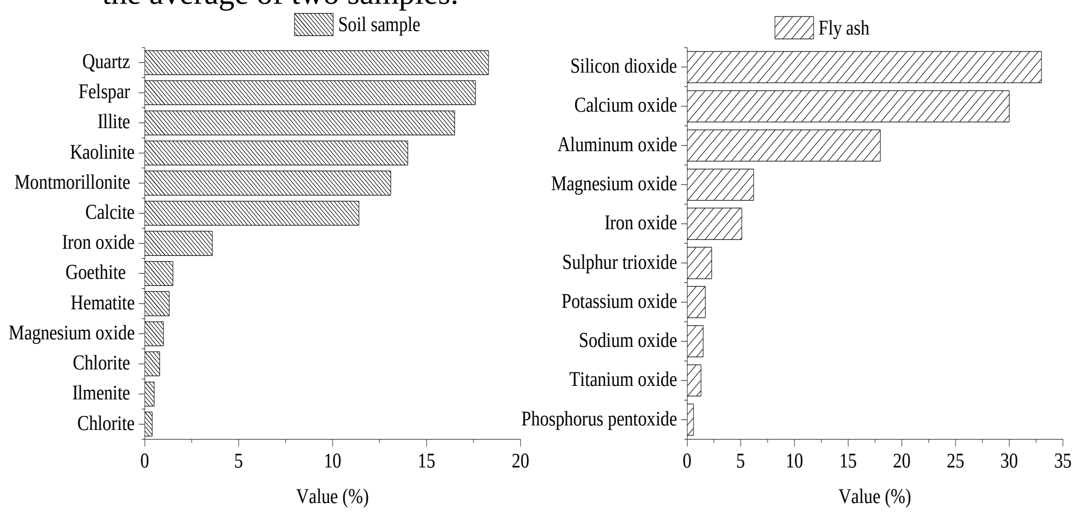

Figure 2. Petrographic Analysis of soil sample and fly ash

3. TEST RESULTS AND DISCUSSION

Detailed outcomes of the tests conducted during this research work on amalgamated soil samples and the result of fly ash on various physical and strength characteristics of the soft soil sample are deliberated below.

3.1 Atterberg's Limits Tests

The outcomes of the consistency limit state tests conducted on the stabilized samples by using fly ash are expressed in Figure 3. From the graph, it is evident that addition of 10% fly ash increased the liquid limit from 31.6% to 35% while the addition of the same amount of additive reduced the plasticity index from 13.4% to 10%.

3.2 Compaction Test

To examine the result of fly ash contents on compaction characteristics ($\gamma_{d\,max}$ and OMC) of the soft soil sample, modified Proctor test was conducted on all treated soil samples and the results are shown in Figure 4. From the graphs, there is a slight decrease in $\gamma_{d\,max}$ by mixing fly ash with the soil while the OMC goes on increasing with the increase of admixture contents.

3.3 California Bearing Ratio (CBR) Tests

Soaked CBR tests were conducted on soil-fly ash composite samples and the results are shown in Figure 5. With the addition of 10% fly ash, CBR value increased from

3% to 8.6% which is well ahead of the threshold value for subgrade material according to the National Highway Authority (NHA) of Pakistan.

3.4 Unconfined Compressive Strength (UCS) Tests

To examine the effect of fly ash contents and curing period on the UCS of the soft soil the samples were made by mixing soil with various fractions of admixture and properly cured to check the mode of strength development. From the test results given in Figure 5, due to the pozzolanic reaction occurring in soil-fly ash mixture, by adding 10 % fly ash to the soil, UCS strength value after 28 days of curing, increased from 317 kPa to 1205 kPa.

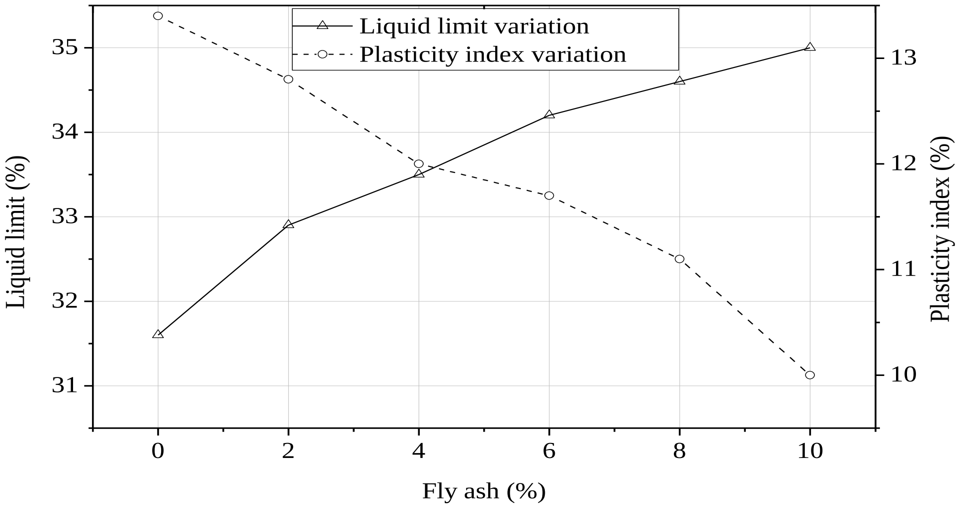

Figure 3. Effect of fly ash on consistency limits of soft soil

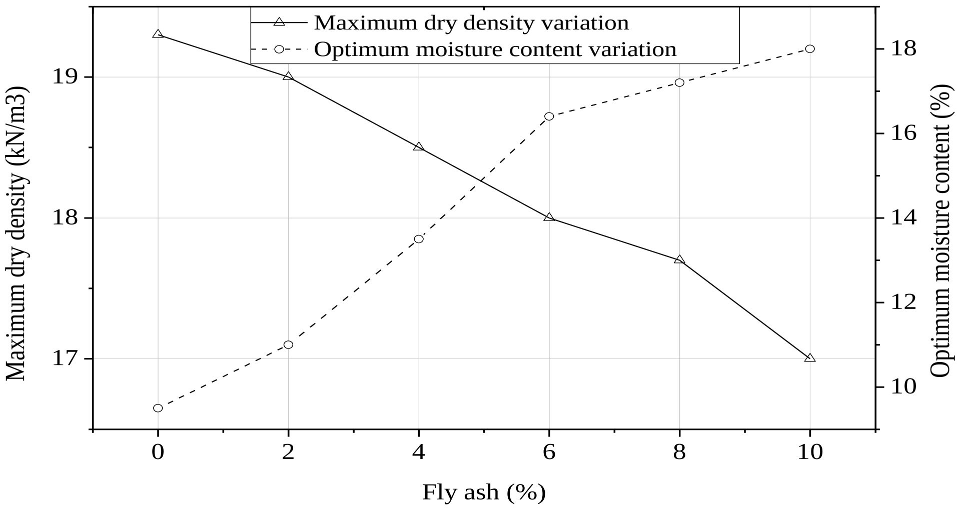

Figure 4. Effect of fly ash on compaction characteristics of soft soil

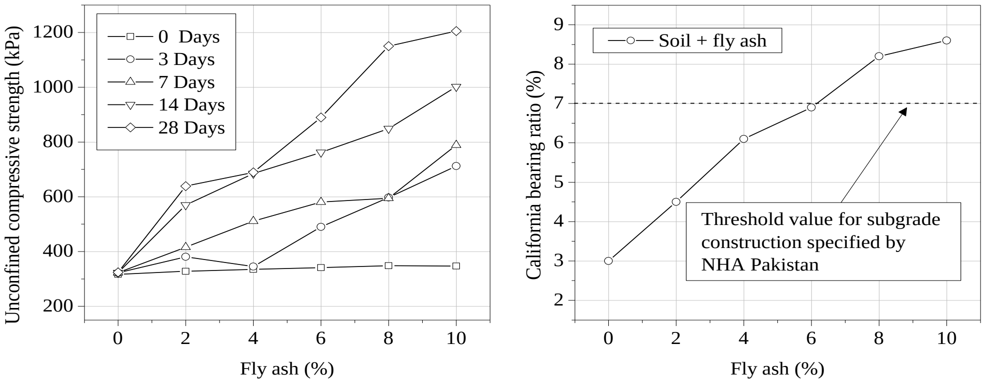

Figure 5. Effect of fly ash on unconfined compressive strength and California bearing ratio of soft soil

4. CONCLUSIONS

From the above study the following conclusions have been made:

- Adding fly ash to the virgin soil reduced the plasticity of the soil.
- The $\gamma_{d\ max}$ of soil slightly reduced with the addition of fly ash contents.
- Mixing of 10% fly ash to soil increased the UCS by almost four folds.
- It is imagined that soil-fly ash mixtures have a mechanism of strength expansion which consists of an immediate exchange reaction of cations and a long term pozzolanic reaction.
- Due to high values of CBR and UCS, it is very beneficial to use of soil-fly ash mixture as a subgrade material to reduce the deformations in these layers.
- However, using this waste product as a soil stabilizer may alter the other properties of soil. Further, it may need an appropriate method to mix the admixture with soil to achieve the desired results. It is better to initiate the use of fly ash from small scale projects.

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Experimental Determination of Capacity of Pile Group and Pile Raft Foundation

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Abstract

Pile raft foundation is at type of foundation that can be used to support heavier structures over a soil of moderate to low bearing capacity through its complex soil-structure interaction. Researchers have carried out numerical studies on the analysis and design of pile raft foundation and pile group and it is proved that pile group and pile raft foundation have difference in load carrying capacities. R. Katzenbach et. al (2005) said that the raft in pile raft foundation increases the stress between the soil and piles and hence contributes to the load capacity of piles in comparison to pile group when piles are free standing. This paper basically provides an experimental verification of their behavior. This paper is aimed to calculate the settlement behaviors of pile group and pile raft foundation experimentally. For this small-scale model is prepared for experimentation, where pile is made up of copper and raft is of aluminum. Experimental model is placed in a soil box and vertical loading is applied through a hydraulic straining frame at a very slow rate. It is found that load carrying capacity of the pile raft is much greater than that of the pile group and vertical settlement is also reduced substantially in case of pile raft foundation.

Keywords: pile group and pile raft foundation, small scale model, settlement

1. INTRODUCTION

Foundation is an element of the structure that transfer the load of superstructure deep in to the ground to that layer which has the required strength. Main purpose of foundation is to bear the load of super structure, to stabilize it against natural forces (earthquake, wind etc.) and to protect it from ground moisture. Shallow foundation is provided when the soil underneath is stiff and has the required bearing capacity to transfer the load in shallow depth while deep foundation is provided where the soil is soft and/or there is a high-rise structure which is more expose to wind and earthquake loads. Pile raft and pile group foundation are types of deep foundation which transfer load by bearing and shearing. In pile raft foundation, raft has a direct contact with soil while pile group foundation has no such a contact between raft and soil underneath. In pile raft foundation load is transferred through the complex soil structure interaction which occurs among piles, raft and soil while pile group is not having such a complex load transfer mechanism. In pile raft foundation, raft control the differential settlement and piles reduce the settlement and stresses in the soil. In pile group foundation as raft has no contact with soil below it and hence doesn't transfer any load while in case of pile raft foundation raft must be in direct contact with the soil underneath and hence load is transferred through piles as well as raft. The load transfer mechanism of pile raft foundation is complex and has no exact solution except the finite element analysis. However some researchers have proposed simplified methods of analysis which are presented below.

a) Poulos and Davis method

This describes the load-displacement behavior of piled raft foundation as a tri-linear curve shown in fig.1. The curve has three regions, in the first region both piles and raft resist the load, in the second region pile capacity is fully mobilized thereby exerting extra demand on raft, while in the third region pile plus raft ultimate capacity is utilized.

b) Randolph method

This method incorporated stiffness of piles, raft and also the interaction between the piles and raft to give a formula for combined stiffness of piles and raft (K_{pr}) as shown below.

$$K_{pr} = \frac{K_p+(1-2\alpha_{rp})K_r}{1-\alpha_{rp}^2({}^{Kr}/_{Kp})} \qquad \alpha_{rp} \cong 1 - \frac{\ln({}^{r_c}/_{r_o})}{\zeta}$$

Where K_p and K_r are the respective stiffness's of the piles and the raft. α_{rp} is the interaction between the piles and the raft.

The load distribution between the piles and raft given by this method as follows:

$$\frac{P_r}{P_r + P_p} = \frac{(1-\alpha_{rp})K_r}{K_p + (1-2\alpha_{rp})K_r}$$

The method also provided a relation to determine the vertical displacement of piled raft foundation (S) as follows:

$S = \frac{P}{K_{pr}}$ where P is applied load on the system.

c) Poulos-Davis-Randolph method

This method has combined the Poulos-Davis and Randolph in such a way that later is used for determining the load distribution among the piles and raft and former is employed to see the load settlement behavior of system. The load P_l at which pile capacity is fully mobilized is:

$P_l = \frac{P_{up}}{1-X}$ where P_{up} is the ultimate load capacity of pile group and X is load ratio of P_r to P_p.

In case the load applied is only up to P_l, the settlement of the system is: $S = \frac{P}{K_{pr}}$

For load beyond P_l, $S = \frac{P}{K_{pr}} + \frac{P-P_l}{K_r}$

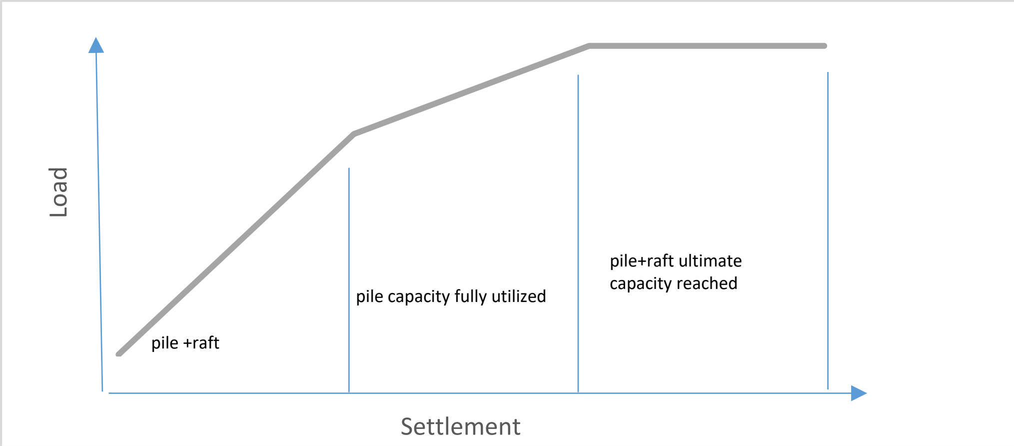

Figure 1: Idealized load settlement behavior (Poulos)

2. EXPERIMENTAL BACKGROUND OF PILE RAFT AND PILE GROUP

Various researchers have conducted experiments to assess the pile group and pile raft foundations. Katzenbach, R., & Moormann, C. (1998) also monitored the experimental model of piled raft foundation and instrumented the model. The testing included the comparison of load settlement behavior of a single pile, single pile with raft and 25 piles with raft as a piled raft foundation while studying the influence of pile-pile and pile raft interaction. The experimental testing showed that only a small part of the load is transferred to soil by the raft, while the load distribution among the piles is such that pile load decreases from the corner piles to the edge piles and reduces significantly to the inner piles. Similarly, M. H. Baziar et. Al (2009) also conducted experimental testing on a small scale 1g physical model that consisted of circular raft and four piles, along with numerical simulation in FLAC-3D and PDR method to study the bearing settlement behavior of piled raft foundation on medium dense sand. The results of these methods adopted were compared and it showed that PDR method works well as long as load applied is in linear range. Beren Yilmaz (2010) conducted laboratory testing on small scale pile raft models with different number of piles. The experimental model included brass nails, aluminum piles, soil box and a soil. Loading jack, loading hangers, displacement dial gauges and data logger were the equipment. A total of 16 soil boxes were prepared and testing was conducted on a single raft, pile raft of 16 piles and raft with 49 piles. Along with this analytical and a numerical method was also employed. It has been concluded that an optimum number of piles are required to control the settlements and above those number of piles, settlement doesn’t decrease and these loads is shared to the soil by both piles and raft.

Instrumentation is a term for measuring instruments which are used to measure, record and indicate physical quantities e.g. load, stress, strain, pressure, distance, settlement etc. In civil engineering practices stress and strain are commonly measured physical quantities in super structure through instrumentation while settlement measurement is preferred in sub structure. The most common instruments used by civil engineers are strain gauges, settlement transducer, load cells, pressure cells and many more. The type of instrument used depends on the type of quantity to be measured. Data from these instruments are recorded by multichannel data logger with which different instruments are connected through its channel and the data logger is connected to a computer programmed to store the data. Specific values of measuring factor are inserted in computer to exactly measure the physical quantity through instrument. In this paper experimental study of pile group and pile raft foundation is carried out in which settlement is important physical quantity to be measured, settlement transducer of 50 mm capacity and load cell are used to be measured the settlement produced in the model and the applied load respectively. UCAM 70 datalogger is used to record the data from the LVDTs and load cell.

3. METHODOLOGY

This research basically presents the comparison of the piled raft foundation and pile group foundation on the basis of their load settlement behavior through experimental testing program. The experimental testing was conducted in Materials Testing laboratory of Civil Engineering department of University of Engineering and Technology, Peshawar. Along with other equipment, lab is equipped with a Hydraulic Straining Frame having a capacity of 200KN.

For this research copper rods (length:2ft, diameter: 0.75inches) are used as pile, aluminum plate (square: 1ft x 1ft, thickness: 0.0196ft) as raft, a soil box (square:3.5ft x 3.5ft, hight:3ft) to carry soil, clayey plus sandy soil, load cell having a capacity of 5tons, linear vertical displacement transducers (LVDTs) of range 50mm and data acquisition system i.e. UCAM 70 data logger having 30 channels is used. Copper is selected as pile because it has sufficient elastic modulus to prevent axil strain in it so that the vertical settlement measured is only the due to injection of pile into soil. Aluminum plate is selected as raft because it bends uniformly when loaded and hence simulates the raft behavior in a good sense.

First an experimental setup is made to represent pile group foundation as shown in fig.2, and then setup for piled raft as shown in fig.3 is also made and the basic difference between the two is that soil is in contact with raft for piled raft foundation and for pile group foundation there is no such a contact. After this setup two LVDTs, one load cells and data logger UCAM 70 is connected to the setup as shown in figures below. Then load is applied to the experimental model and the load settlement data is recorded through UCAM 70. Load is vertically applied at the center of the model through the hydraulic straining frame. The rate of increase of vertical load is very low can be assumed as static load. Vertical load is applied to the model until the vertical settlement is measurable through LVDTs i.e. till 50mm but it is also decided that if excessive bending in raft occurs, loading should be stopped to prevent damage to the material. Therefore, for pile raft foundation loading is stopped at a settlement level of 35-40mm but for pile group foundation load is applied till a settlement of 50mm as there is no such a damage to raft being observed.

Figure 2: Experimental setup for pile group model

Figure 3: Experimental setup for piled raft model

4. RESULTS AND DISCUSSION:

After the application of continuously increasing load on the pile group and pile raft foundation model separately, load and settlement values for both foundation type are obtained and graph between them is plotted. Figure 4 shows the settlement behavior of pile group foundation model in response to the continuously increasing vertical load. It can be seen from the graph that settlement of pile group increases with the increase in load on the model and at small values of load the settlement produce in the pile group model is greater. This is because only piles are transferring load to soil with zero contribution of raft because of no contact between soil and raft. The maximum load sustained by the pile group model is 200lb having settlement of 50mm. This capacity is very low and it is just because of the end bearing resistance of the piles with negligible shaft resistance. Shaft resistance is negligible because copper piles has less friction resistance to the sandy soil as compared to end bearing resistance.
Figure 5 shows the settlement behavior of pile raft foundation model under continuously increasing vertical load. It can be clearly seen that increasing the load increases the settlement of the pile raft foundation model but the settlement in this case is smaller at greater load values as compared to the pile group foundation. The graph of pile raft foundation model has two region, steep slope region corresponds to the load carried by both piles and raft while after the steep slope is a flat slope which shows that the capacity of the piles is fully mobilized and all the load is carried by the raft only. This behavior of pile raft (Fig.5) is in well agreement with Poulos as shown in Fig.1

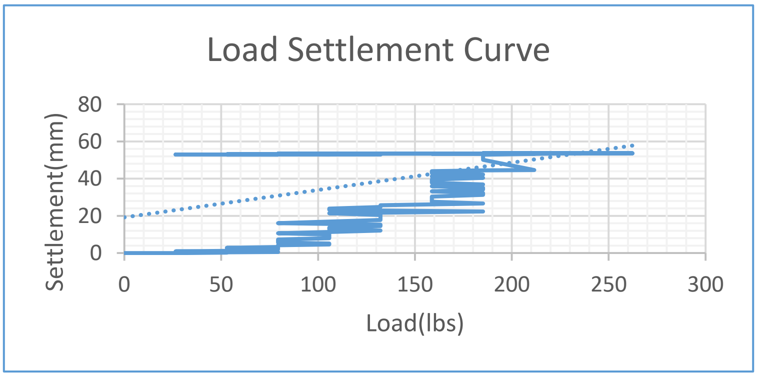

Figure 4: load settlement curve of pile group foundation model

Figure 4 seems to have graph with zigzag pattern but we should focus on the general trend as shown by the trend line of the graph. One of the reasons for that zigzag is at the start of the test soil starts readjusting itself until it gets stable by the soil and pile interaction. In the case of pile raft foundation as shown in figure 5 there is no such a zigzag because the soil adjusts itself quickly during the test because of soil-raft-pile interaction.

5. CONCLUSION:

After comparison of load settlement graph of pile group and pile raft foundation model it is concluded that behavior of both the graphs towards the applied load is same but the load carried by pile raft foundation model is greater than that of load carried by pile group which shows that pile raft foundation has greater load carrying capacity than pile group foundation. The superiority of pile raft foundation is due to raft-soil and pile-raft interaction which doesn't exists in pile group.
It is also concluded that pile raft model is more efficient than pile group model for settlement reduction. Also, as both the foundations have raft but its capacity is only utilized in pile raft foundation, therefore, pile raft foundation is called as an economical alternative to deep foundation. So foundation engineer would prefer pile raft over pile group foundation.

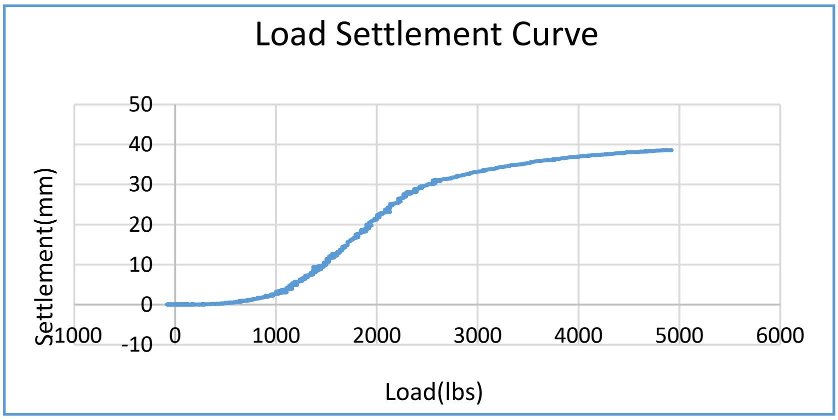

Figure 5: load settlement curve of pile raft foundation model

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Optimum Values for Mixing Ratio and Tire Shred Size of Sand Tire Mix

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Abstract

The increase amount of scrap tire is becoming an enormous concern. The large stockpiles causes an environmental issues, contamination and cause health issues, due to their non-biodegradable nature. So the effective use of scrap tire required a great concern .If these scrap tire is cutted in pieces of appropriate sizes called tire shred, then it can be used for increasing shear strength of sand which is used in mechanically stabilized Earth walls (MSE), earth embankments and for land fill . But using scrap tire for increasing shear strength of sand require a sound knowledge of size of tire shred, mixing ratio of tire shred with sand as well as size of tire shred. The sand gradation also effect the shear strength behavior. This paper present a number of modified proctor compaction test results and gradation test performed on sand of specified specific gravity mixed in different proportion with tire shred of different sizes range from 50mm to 100mm, from which then optimum water content, maximum dry density as well as the size of tire shred is determined. Using that optimum moisture content ,mixing ratio and tire shred size , the increased dry density and hence the increase shear strength can be achieved .If the above optimum characteristics are compromised, the tire mixed with sand will not effectively play its role in increasing the shear strength of sand.

Keywords: tire shred, mixing ratio, optimum compaction ratio, optimum mixing ratio

1. INTRODUCTION:

The shear strength of soil is mainly dependent on two independent parameters that is cohesion “c” and internal frictional angle “θ”. Cohesion is due to binding effect of soil grain and frictional angle is due to friction between soil grains. Shear strength of soil is given by relations. $s = c + \sigma \tan \theta$ Where “s” is the total shear stress, “c” is cohesion, “σ” is normal or overburden stress and “θ” is the angle of internal friction . For pure clear sand cohesion is zero so shear strength also is zero at zero confining pressure and for unclear sand shear strength range between 0.5 to 2 kPa Similarly for highly plastic clay shear strength is about 200 kPa while it is between 10…100 kPa for medium plastic clay .If someone is using only sand in certain projects then its shear strength will be reasonably low .It has been observed that adding elastic materials like small pieces of scrap tires in addition with sand will reasonably increase the shear strength of sand. This pieces of scrap tires are referred as tire shred. Tire shreds basically the pieces of scrap tires cutted into suitable sizes that is ranges from 50mm to 100mm. Every year 13.5 million tons of scrap tires wasted by both developed and developing countries all over the world. if these accumulates in huge amount it becomes environmental unfriendly and may cause diseases in public .The problems due to shredded tires includes fire, polluting the environment and hazardous effect on health as well as effect the agricultural growth of plants .The waste tires can recycled and disposed of without causing any effect on environment . The use of tire shreds as aggregate have most important advantages which includes 1) Light weight having density range from 500 to 1040 kg/m^3. 2) Low Earth Pressure 3) Good Drainage 4) Increase the shear strength when used with sand due to the mechanical properties of rubber. Based on these advantages Tire shreds are used in many application such as landfill covers, slope stability structures, retaining wall backfill, road embankment, road bed support as a backfill in retaining walls.

2. LITERATURE REVIEW

The use of aggregates derived from Scrape tire/waste tire in civil engineering application (ASTM-D6270-08) can reduce the problems resulting from the disposal of tires. According to the Sunthonpagist and Duffy (Ghazavi, M. (2004)), all scrape tire product was investigated the least in terms of production and markets. As the tires are combustible material and can catch fire, there is no harm by using these scrape tires in buried form (Yoon & Kim, K. (2007)), As the tires are light weight aggregate having high strength and low backfill pressure it is used as lightweight material for embankment construction (Bernal, A..Salgado, R., Swan, R. H., & Lovell, C. W. (1997)) It has been observed in previous studies that sand when used with tires get reinforced and can provide greater shear strength than sand itself, having friction angle as large as 65.8 degree being obtained from the mixture of dense sand with 30% tire shreds by volume .Friction angle of sand alone is 34.8 degree (Ahmed, I. (1993)) Humphrey and Manion (Humphrey, D. N., & Manion, W. P. (1992)) evaluated that sand tire mixture can undergo significant compression at low normal stresses. However, most of the compression that occurs is plastic, i.e. compressibility decreases substantially once the tire shred experience load .So, preloading can be done to reduce plastic compression, when the fill has been constructed. Humphrey and Manion (Humphrey, D. N., & Manion, W. P. (1992)) observed that tire shreds and sand-tire mixture can be compacted by using common compaction procedures. It was found that unit weight in the mixture is mainly controlled by the soil content. Whereas, vibratory compaction effort and molding water content appear to have no significant effect. It is reported that the optimum size and sand-tire mix should be determined experimentally, as the shape and size of tire shreds is a main factor of the numerous processing techniques used in production (Edinçliler & Saygılı, A. (2010)). It is reported that by adding 10% of tire shreds by volume in a random arrangement in the dense sand cause a significant increase in shear strength (Edil & Bosscher, P. J. (1992)). From the last few years, recycling of waste tires as construction material (light weight backfill material, drainage layer and thermal insulation) have been considered important to solve the economic and technical problems for a sustainable environment (Bosscher and Eldin, (1992), Tandon, Nazarian, & Picornell, M. (2007). The waste tires, which are shredded into tire chips/granulated rubber, are mixed with sand, which can be used for vibration isolation. Geomaterials derived from Scrap tires are uses in several geotechnical application and important references to related paper can be found in Hazarika and Yasuhara (Hazarika & Yasuhara, K. (Eds.). (2007)). The investigation of the effect of different granulated tire sizes and tire content on shear strength of sand-tire is very limited in terms of details, whereas detail study has been recommended by Prumpotthangkoon and Hyde (Promputthangkoon, , A. F. L. (2007, November)). Foose

studied the feasibility of the application of shredded waste tires to reinforce sand. (Foose & Bosscher, P. J. (1996).
From the above literature review, it can be concluded that those parameters which influence the shear strength and compressibility characteristics directly or indirectly are shape and size of tire shreds, tire shreds content, sand unit weight, mixing ratio, confining pressure and normal stress. However, many studies carried out to find the shear strength of the sand tire mixture by considering one particular size of tire or different size of tire shreds. For this purpose it was concluded that to first perform Proctor compaction tests and obtain the optimum compaction ratio (tire shreds/sand) from which maximum dry density is achieved for a particular moisture content.

3. TESTS FOR OPTIMUM CHARACTERISTICS

Modified compaction test were performed to find the optimum moisture content and maximum dry density. Compaction is the function of moisture content because there is some value of moisture content at we can achieve maximum dry density that valve of moisture content is optimum moisture content(OMC) and the corresponding dry density is maximum dry density. Sand are mixed with tire shred of varying sizes and water content are added with increment of 2% in each case. Numbers of compaction tests are conducted and in case we get dry density and moisture content. Graphs are constructed between dry densities and moisture content. Maximum dry density and optimum moisture content are obtained. This gives the compaction test ratio of maximum dry density which are used for direct shear test.

3.1 Apparatus, Materiel and Sample Preparation

Before performing the tests, first we need the constituent such as sand and tires. For sand, Sieve analysis test is conducted and sand of specific gravity 2.67 is taken. For tire shred the scrap tire are cutted into pieces of various sizes that is (50mm, 75mm and 100mm) shown in figure 1. Modified proctor tests are performed for this purpose in which we have mold and hammer. Mold of weight 10.87kg and volume 0.075ft^3 are used and hammer of weight 10lb are used. The height of rise of hammer is 1.5ft and number of layers of compaction is 5. Number of blows for each layer is 56. So, first of all the required amount of sand and tire shreds are mixed and then the calculated amount of water is added to the mix. And then added to the mold from time to time in 5 layers and compact it in each case.
After that modified proctor tests are performed for each size of tire shreds (50mm, 75mm and 100mm) mixed with different proportion to sand with increment of 2% water by weight. The different mixing ratio are tire shreds/sand (0/100, 20/80, 30/70, 40/60).

Figure 1. Tire shred Sample

Figure 2. Sand Rubber Mix in Compaction Mold

Optimum moisture content, dry density, mixing ratio, and tire shred

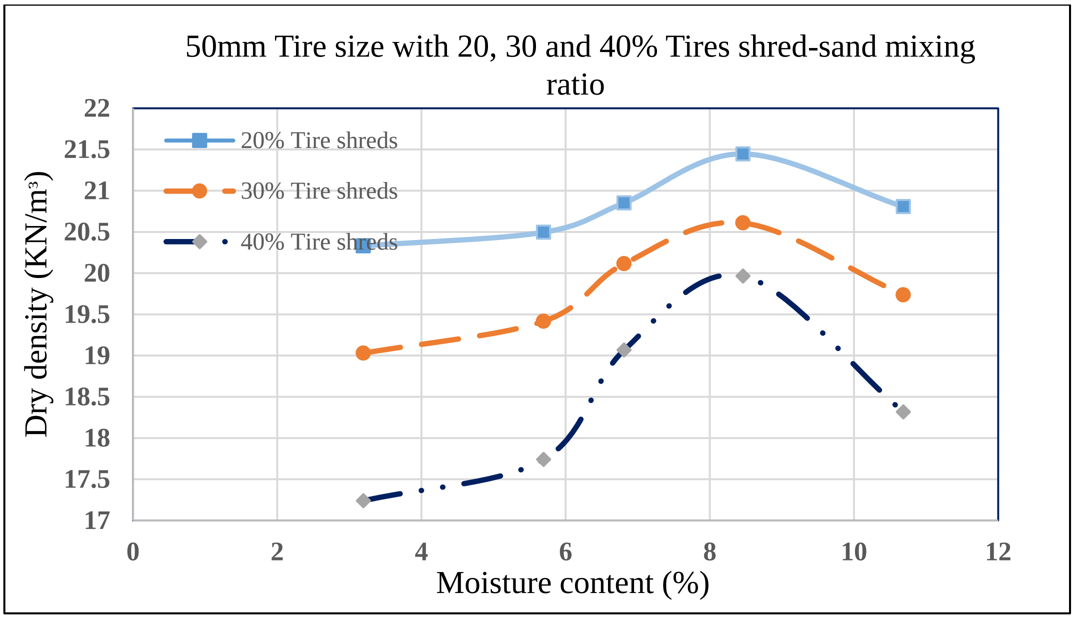

Figure 3. Combine graph for dry density of 50mm tire shred with 20%, 30%, and 40% of tire

The maximum dry density for different mixing ratio of 20%, 30%, 40%, for each size of 50mm, 75mm and 100mm is determined from their respective graph .For 50mm tire shred size, the maximum dry density for different mixing ratio is calculated as above, the graph in figure 3 shows that maximum dry density of 21.446 (KN/m^3) is achieved at moisture content of 8.46(%).
For 75mm tire shred. Data and graph shows that maximum dry density of 19.75 KN/m^{3}is achieved at moisture content of 8.46%.

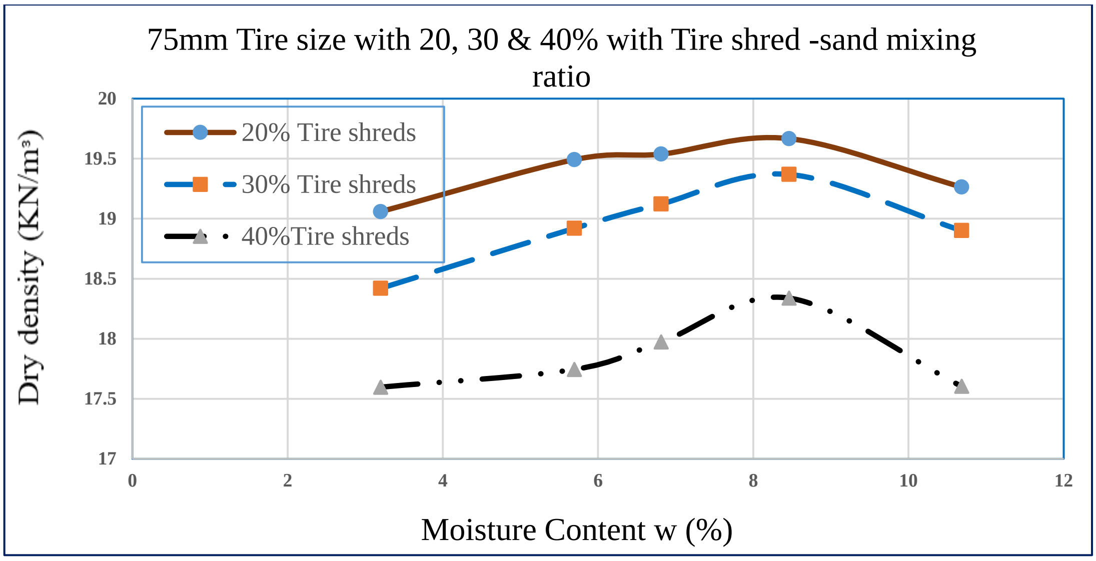

Figure 4. Combine graph for dry density of 75mm tire shred with 20%, 30%, and 40% of tire

For 100mm tire shred the maximum dry density for different mixing ratio is calculated as below. The graph in figure 4 shows that maximum dry density of 17.811 (KN/m^3) is achieved at moisture content of 8.46(%).

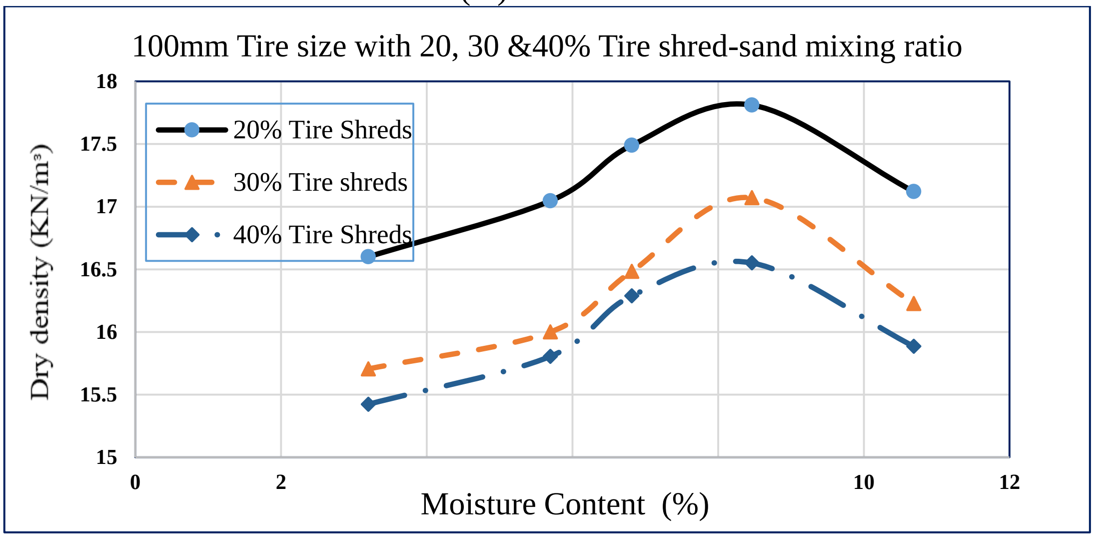

Figure 5. Combine graph for dry density of 100mm tire shred with 20, 30, and 40% of tire

If the combine graph for different sizes of tire shred is plotted for the mixing ratio of 20/80 percent by weight at different moisture content then it can be deduced that for 50mm the dry density will be maximum as shown by the graph below.

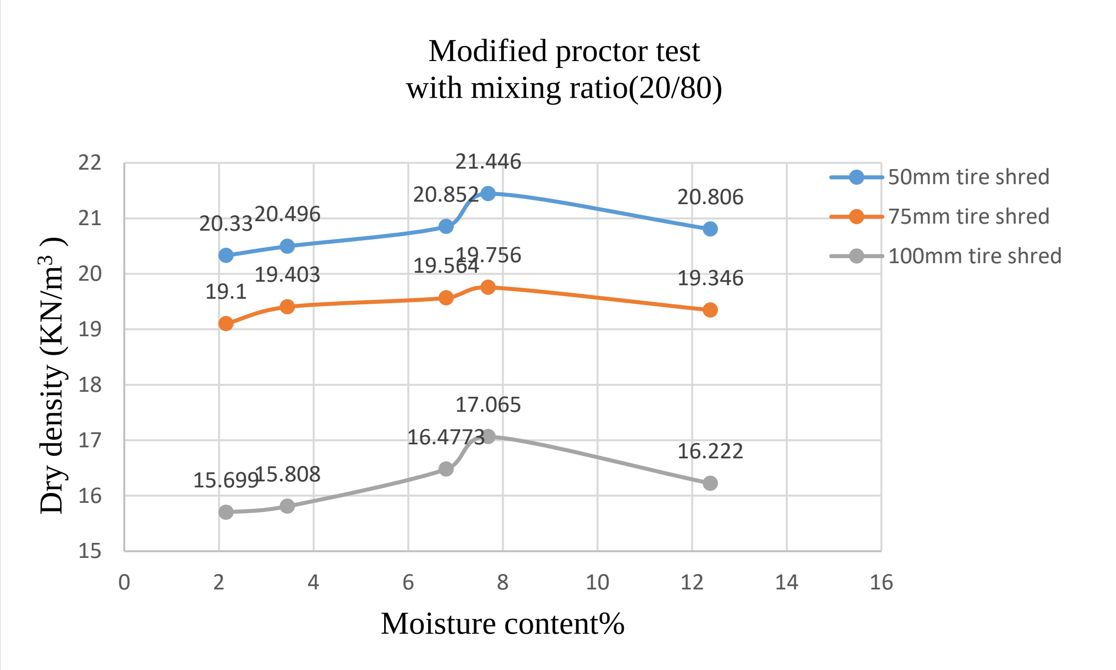

Figure 6. Final graph showing the maximum dry density for 50mm, 75mm, and 100mm tire shred using 20% of tire shred and 80%of sand

4. DISCUSSIONS ON RESULTS

From the above graphs it is determined that For 50mm tire shred maximum dry density 21.446 KN/m^3 is achieved at moisture content of 8.46% with mixing 20% of tire shred and 80% of sand by weight and for 75mm shred maximum dry density 19.75 KN/m^3 is achieved at moisture content of 8.46% with mixing 20% of tire shred and 80% of sand by weight .similarly for 100mm tire shred maximum dry density 17.811 KN/ m^3 is achieved at moisture content of 8.46% with mixing 20% of tire shred and 80% of sand by weight. At optimum mixing ratio 20%by weight of sand segregation is negligible.

5. CONCLUSION

For sand tire mix to have maximum dry density they must be mixed at a certain optimum mixing ratio and optimum moisture content containing particular size of tire shreds. From the above experimental data, compaction tests it is clear that by keeping mixing ration 20/80 that is 20% of tire shred and 80% of sand, moisture content of 8.46% and tire shred size of 50mm we get maximum dry density of 21.446KN/m^3 which is quite greater then that of 75mm and 100mm tire shred for the same mixing ratio of 20/80 which is 19.75KN/m^3 and 17.811KN/m^3 respectively.

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Experimental Study of Comparison of Settlement Behavior of Pile Raft Foundation with Batter and Vertical Piles

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Abstract

Lateral forces on the high-rise buildings and infrastructures causes them to topple down. A simple deep foundation of vertical piles is not enough to full fill the design requirement for lateral forces on these structures. Pile raft foundation is found to be efficient in high rise buildings by reducing the settlement produce in the foundation and increasing its load carrying capacity but in some cases the pile raft foundation with vertical piles is unable to sustain the structure for the lateral load. Therefore, batter piles are used in pile raft foundation which has greater load carrying capacity in both vertical and horizontal direction. Batter piles with different angle with vertical are used to resist lateral load accordingly. This paper presents the experimental study of the comparison of the settlement and load carrying capacity of the pile raft foundation with vertical and batter piles, this study is divided into two parts, first part is experimental study of pile group foundation model having vertical and batter pile in which raft is not active to take load and in second part the pile raft foundation model having vertical and batter piles is studied in which the raft is active to take load. Vertical load is applied on all these foundation type separately and load settlement curves are plotted. It is concluded that pile raft foundation having batter piles has greater load carrying capacity and less settlement than that of pile raft foundation with vertical piles.

Keywords: Pile Raft foundation, Pile group foundation, settlement, batter piles.

1. BACKGROUND AND HISTORY:

Foundation is fundamental part of structure which carry and transfer load from super structure to bearing ground and is located at certain depth from ground surface. Structures ranges from simple residential building to sky scraper have foundation of its required type. On the bases of type (vertical, lateral, Earthquake, wind) and direction of loads on super structure and bearing ground properties foundation type for the structure changes. Shallow foundation is preferred for vertical load and stiff soil while deep foundation is preferred for lateral load and soft soil. Land availability issue in the world causes the shifting of construction industry to areas where soil is not suitable for shallow foundation, deep foundation is preferred in such soil type. Pile foundation is given to structure in soft soil which transfer load both by bearing and shearing. In some situation pile foundation alone does not fulfill the requirement of foundation for a structure, in this case pile and raft are used together to get the required sub structure. In Piled raft foundation load is transferred to soil both by raft and pile and interaction occurs among piles and between pile and raft. In case of earthquake, wind or any lateral load when it exceeds its limit raked piles are used in place of vertical piles or combination of both vertical and raked piles is used.

Piles and raft are used in two different arrangements to carry load, pile group and piled raft foundation. In pile group foundation raft act as pile cap and is not connected to bearing ground. All the load is carried by the piles only, this type of arrangement has small load carrying capacity while in case of piled raft foundation raft lie on the bearing ground. Load is carried by both raft and piles and has greater load carrying capacity than pile group foundation. Different analytical, numerical and experimental studies have been performed on the behavior of pile group and piled raft foundation under vertical and lateral loads and concluded various results which explain the behavior of pile group and piled raft under different loads and external conditions.

Anh-Tuan VU et al (2017) studied experimentally 3-pile and 6-pile batter pile foundation model both for pile group and piled raft arrangement under vertical and lateral load and concluded that introducing batter piles to the pile raft foundation have positive impact on its behavior by increasing its stiffness in both horizontal and vertical direction and reduced settlement caused by both vertical and horizontal loads. Pastsakorn Kitiyodom and Tatsunori Matsumoto (2002) developed numerical analysis method to study the deformation and load distribution in pile raft foundation having batter pile, also parametric studies were performed and concluded that batter pile helps in improving the deformation behavior of pile raft foundation. M. Hajialilue-Bonabb et al. studied the effect of inclination angle of batter piles and concluded that with increase in the inclination angle reduce rigid length of piles and hence causes more settlement during lateral forces. Mahmoud Ghazav et al. (2014) has also studied the performance of batter piles in offshore structures and concluded that the lateral loads efficiency of pile groups increases by introducing batter piles. Nan DENG et al. (2007) has studied attraction of seismic forces in terms of axial force by batter piles in pile group and found that during seismic waves, more axial forces are generated in batter piles rather than vertical piles and that’s due to kinematic interaction between pile group and soil. Bharathi et al. (2019) stated that batter piles have less displacement as compared to vertical piles and their reduction percentage remains up to 25%-50% probably. Z. Li et al. (2016) studied that, in past, Poor analytical, numerical approach and lack of knowledge about batter piles made them poor resistant to earthquakes. Poulos (2006) studied the effects of ground motion on raked piles by considering six-pile group and concluded that with the raking a pile in the absence of ground motion can reduce settlement, lateral deflection and cap rotation but in the presence of ground motion the result is affected adversely as compared to the group having only vertical piles. The ground motion will result in increase in pile load on raked piles such situation may face in case of bridge abutments.

This manuscript presents the experimental study of the settlement behavior and load carrying capacity of pile group and piled raft foundation having vertical and raked (batter) piles subjected to vertical load. Increasing vertical load is applied on the pile raft model placed in sandy soil which is instrumented with transducer and load cell to measure the settlement of the raft and applied load respectively. The load settlement curve of both pile group and piled raft foundation are compared to know their behavior.

2. METHODOLOGY:

2.1 Model preparation and Experimental Setup:

Aluminum plate of dimension 1' x 1' x 0.0196' is used as raft because of its rigidity, high stiffness and transferring equal load to the piles when load is applied at the center of the raft. Four holes at offset of 2 inches from sides of raft are driven and copper piles of diameter 0.75" are attached to the raft through two pins. Piles angle with vertical can be changed from 0^{o} to 40^{o} with 10^{o} degree increment. The raked piles used for this paper are inclined at 10^{o} with vertical. The connection of piles and raft is rigid allowing zero moment when load is applied. Piles can be detached from the raft in case of experimental settlement study of only raft foundation. The model is placed in a clay sandy soil of known properties which is enclosed in a cubic box of size 3' x 3' x 3.5' which gives fixed boundary condition to the sandy soil. In case of pile group model, the raft is about the soil surface while in case of piled raft model the raft lies on the surface of the soil. Figure 1 shows the small-scale model of pile group foundation placed in soil.

Figure 1: Small scale Pile Group Model

2.2 Instrumentation:

To find settlement of raft and piles, two Settlement Transducer are used each having settlement calculation capacity more than 50 mm. The transducers are installed at two opposite sides of the raft which gives different settlement values. Average of these two settlement values are taken for plotting load settlement curve. load cell is used to find the applied load on the model having capacity to measure up to 200kN of load. Data from transducers and load cell are recorded using UCAM 70 data logger. This data logger has 30 channels by which it can be connected to 30 different instruments at the same time and their data can be recorded. Figure 2 shows the instrumentation of transducer and load cell to the pile raft model.

Figure 2: Instrumentation of Pile Group Model

2.3 Load Assembly:

Loading strain frame having applying load capacity of 200kN is used to apply vertical load on the raft. Strain frame consists of a frame which support a hydraulic jack for load application. The position of frame and hydraulic jack can be changed horizontally and vertically respectively. Load cell sandwiched between loading jack and raft measure the load transferred to the raft. A dial gauge is set in the loading strain frame machine which shows the total load applied by the hydraulic jack. Figure 3 shows the loading strain frame set above the model for load application.

Figure 3: Loading strain frame

3. RESULTS AND DISCUSSION:

After experimental setup and instrumentation, continues vertical load is applied on 4 different type of models, two pile group models having vertical and batter piles and two pile raft model having vertical and batter pile inclined at 10^{o} with vertical. The settlements and load are recorded using UCAM 70 a 30 channel Data logger and transferred to a computer programmed for plotting the load settlement curve.

3.1 Pile Group foundation Model with Batter Piles:

Figure 4 shows the settlement behavior of pile group foundation model with batter piles at 10^0. It can be seen from the graph that by increasing the load on the model the settlement of the pile group increases. Also, at small value of load settlement produced in the pile group is greater. The irregularity and zigzag shape of the graph is due to the arrangement of the soil particles when load is applied to the model. Moreover, the negative sign of load shows that the load is applied in the downward direction on the model while the negative sign of the settlement is due to the arrangement of the Transducers. The transducers were fixed on the upper surface of the raft and its gauge is fixed at 60mm settlement. When load is applied on the raft, settlement gauge of transducer starts reducing from 60mm that's why the data recorded has negative sign.

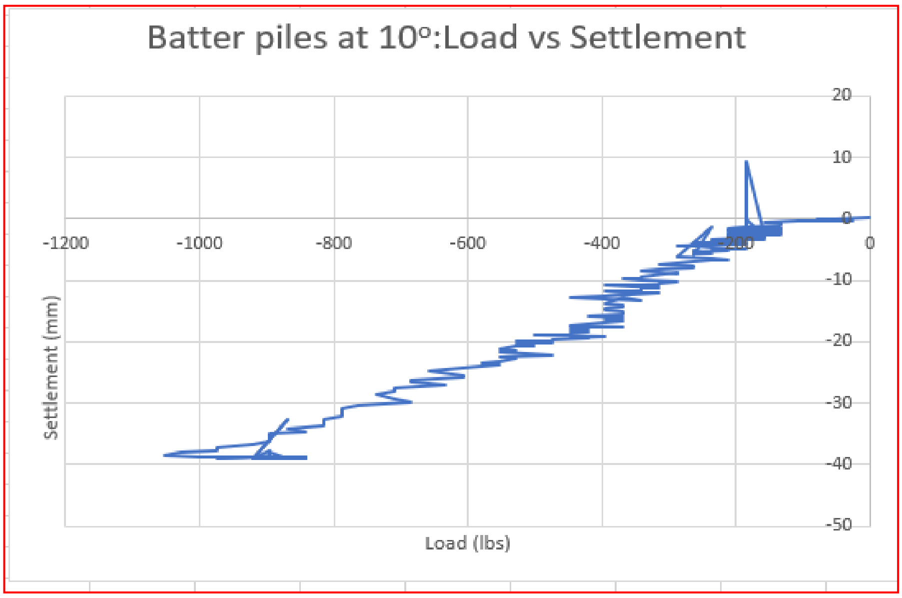

Figure 4: load settlement curve of batter pile group foundation model

3.2 Pile Raft Foundation Model with Batter Piles:

Figure 5 shows the settlement behavior of piled raft model having better pile at 10^0 inclination with vertical under continuously increasing vertical load. settlement values increase with increasing load and at higher load values the values of settlement are smaller as compared to the pile group model. Also, the load carried by the piled raft model is greater than 2000lb with 25mm settlement. The graph shows that at high values of load the capacity of piles to take load is fully mobilized and only Raft is active for taking the load.

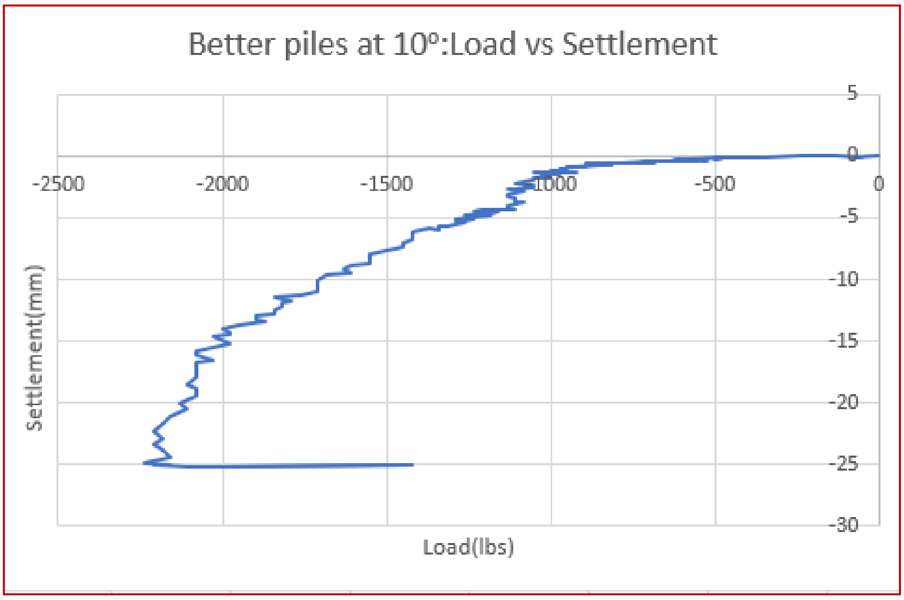

Figure 5: load settlement curve of pile raft foundation model having batter pile

3.3 Pile Group Foundation Model with Vertical Pile:

Figure 6 shows the load verses settlement curve of the pile group foundation model having vertical piles. The trend line of the graph is having constantly increasing slope which shows increase in settlement with vertical load. The total load carried by the model is less than 500 lbs. which is less than that of load carried by pile group foundation with vertical piles. Also, there is more settlement in foundation model than that of the settlement in batter pile group foundation model.

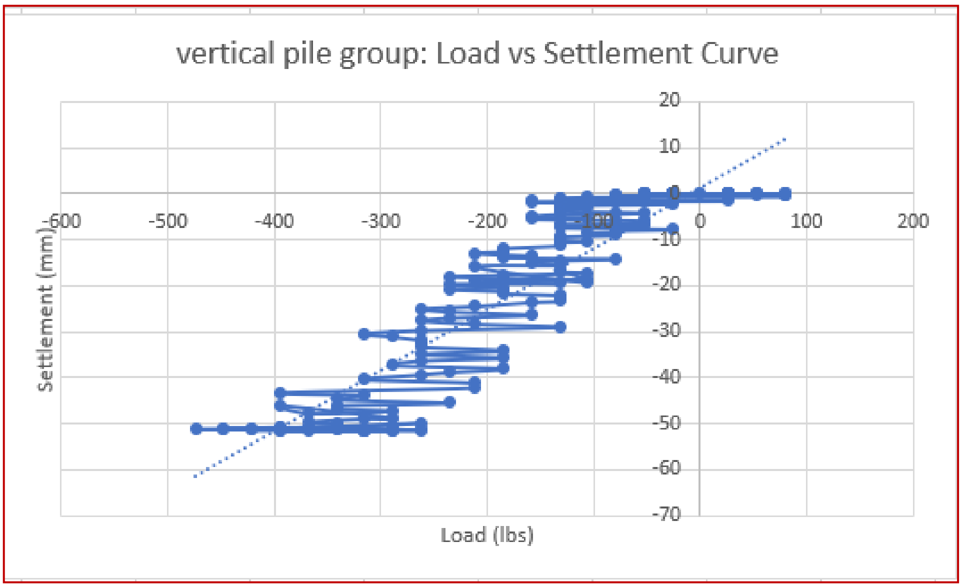

Figure 6: Load Settlement Curve of Vertical Pile group Foundation Model

3.4 Pile Raft Foundation Model with Vertical Pile:

Figure 7 shows the load vs settlement curve of the pile raft foundation having vertical piles and raft lies on the soil surface. The overall graph shows increase in settlement with load with some fluctuation due to the arrangement of the disturbed soil and load cell property i.e. load cell only give load value if the model show reaction to the applied load and in case of the soil particle arrangement the foundation settles without giving reaction to applied load, so the graph obtained is not smooth. The graph shows that the load carried by the model is less than that of the load carried by pile raft with batter piles and also the settlement is more in this case.

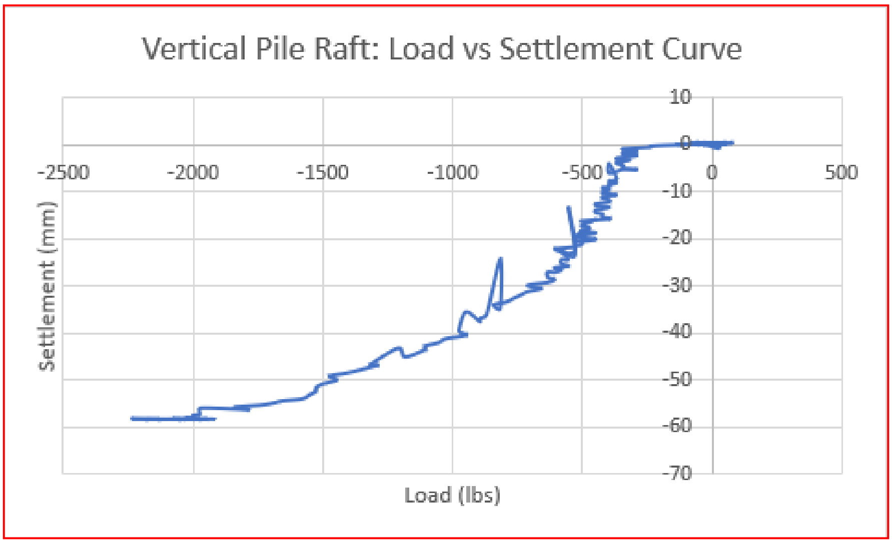

Figure 7: Load Settlement Curve of Vertical Pile Raft Foundation Model

4. RESULT COMPARISON:

Table 1 compares the load carrying capacity and settlement of the pile group and pile raft foundation having batter and vertical piles. In case of pile group foundation model, the load carried by the pile group having batter piles is greater than that of vertical piles group. Also, the settlement produced in batter pile group is less than that of vertical pile group. On the other hand, the pile raft foundation in both vertical and batter pile type carries the same load, but the settlement produce in the batter pile raft foundation is less than that of vertical pile raft foundation.

Table 1: Result Comparison of Pile Raft and pile group Foundation model

Foundation Type	**Pile Group Foundation Model**		**Pile Raft foundation Model**	
	Batter piles with 10^0 inclination	**Vertical Piles**	**Batter piles with 10^0 inclination**	**Vertical Piles**
Load carrying Capacity (lbs.)	921	473	2237	2237
Settlement (mm)	38.9	51.3	25	58.2

5. CONCLUSION AND RECOMMENDATION:

Comparison of pile group and piled raft foundation model having vertical and batter pile at 10^0 inclination shows that pile raft model has greater load carrying capacity than pile group model because in pile group foundation load is carried by only piles while in case of pile raft model both raft and pile are taking the load.

It is also concluded that pile raft foundation can be used in soft soil because off its more contact area with soil and small settlement.

Different values of settlement transducers show that settlement produce in pile raft and pile group is different at different location of the raft which conclude that settlement depends on the condition of soil under the foundation.

It is also concluded that using batter pile instead of vertical pile cause the settlement reduction and increase in load carrying capacity.

It is recommended to study the settlement and load carrying capacity of pile raft foundation when horizontal load is acting on the foundation, Also, the stresses produced in both vertical and batter piles can be studied experimentally by using strain gauges.

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Development and Application of Land Cover Map for Urban Planning and Development Using Geographic Information System and Remote Sensing

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Abstract

The urban land of Abbottabad city is constantly changing due to population growth and migration of people from other regions. It causes major problems of depletion of natural land resources. The main objective of this research work was to study the urban land change in the past 17 years. The other objectives were to quantify landcover classes transformed from one type into other. The methodology include: (i) acquisition of satellite images from Google Earth; (ii) digitization of land covers classes for both years of 2001 and 2018 using the satellite images and (iii) GIS analysis for quantifying the land cover changes. The result indicated 40.02% increase in built-up area, 51.65% increase in forestation, 28.46% increase in open land and 120.93% increase in recreational area in the last seventeen years. There were 40.02% decrease in barren land and 69.33% decrease in agriculture. This study could be utilized for urban planning development and management of urban land of district Abbottabad.

Keywords: Geographic Information System (GIS); Remote Sensing (RS); Urban Land Change; Google Earth Pro; urban sprawl; transformation of classes; landscape study.

1. INTRODUCTION:

Land use suitability assessment is a key factor in any urban and suburban planning and decision-making processes. The assessment is evaluated by a series of criteria involving socio-economic needs. Land use (LU) change is a major issue of concern with regards to change in the global environment. The rapid growth and expansion of urban centers, rapid population growth, scarcity of land, the need for more production, changing technologies are among the many drivers of LU in the world today(Cheruto et al, 2017). The study area was district Abbottabad. This small city acts as a transit point between the northern regions of Pakistan with the rest of the country. Due to this reason, this area attracts a great number of people to settle in it. The other attractions include good weather, high literacy rate, and peace as compared to other cities. The increase in population also gives rise to increase in urban problems and causes urban sprawl. This sprawl results in unplanned settlement, traffic congestion and depletion of natural resources and land resources.

Research work previously conducted using GIS and RS included investigation of the spatial distribution and modeling of existing and future land changes(Akbar et al, 2019); urban green space development using GIS based multi-criteria analysis(Abebe et al, 2017); spatial and temporal land use/land cover assessment is also done using GIS and RS(Foresman et al, 1997); land use change analysis of District Abbottabad using GIS and Land-sat models (Raza et al, 2011); assessment of land use pattern in the district of Abbottabad(Ali et al, 2017).
This research work was conducted to observe the trend of land cover changes occurred in the past 17 years, which could help the planners for urban planning of this city.

1.2. Objectives:

The objectives of this research work are to:
1. Study the urban land changes in the past 17 years;
2. Quantify Urban land area for land use/land cover changes;

1.3. Research Significance:

The purpose of this study was to study the changes occurred in the urban land of the city of Abbottabad. The land use/land cover changes (LULC)in the various classes were observed. These classes were: built up area, forestation, agricultural land, barren land, recreational area and open land. The pattern of LULC was observed and causative factors were identified. It was also studied that how these changes impacted the city over the period of seventeen year. This research could be helpful in urban planning of the city in future.

2. EXPERIMENTAL PROCEDURES:

2.1. Study Area and Data Acquisition:

2.1.1. Study Area:

In the Hazara region of Khyber Pakhtunkhwa, Pakistan, the city Abbottabad is located about 120 kilometers north of Islamabad and 150 kilometers east of Peshawar at an altitude of 1,260 meters(SMEDA). Muzaffarabad lies 77 kilometers to the east of Abbottabad (SMEDA).It is the transit point to the Northern tourist areas. This city is also headquarter of the Hazara division of the KPK province of Pakistan. It has beautiful landscape features along with great climatic conditions and high literacy rate due to which this is preferred and popular city for settlement.
There is increase in population due to which there is intense pressure on available land resources. It caused numerous land cover changes which affected overall scenario of Abbottabad. The location of study area is given in Figure 1.

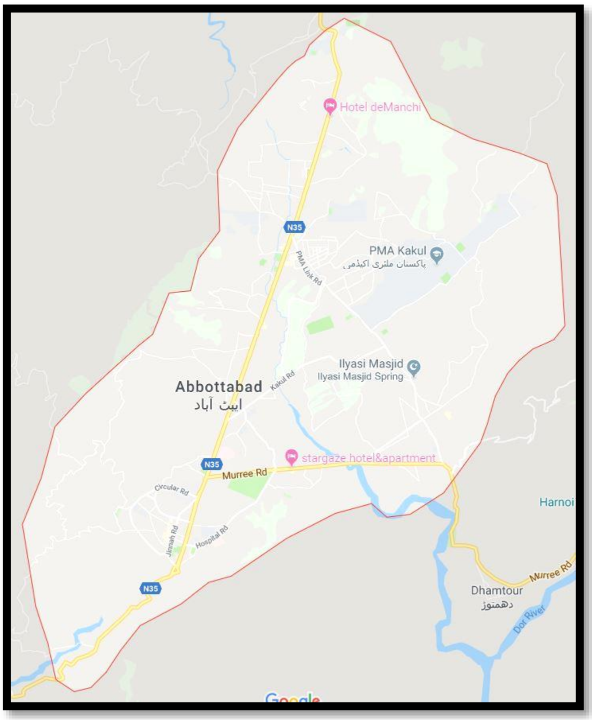

Figure 1. Location of study area

2.1.2. Data and Method:

The images of the study area were obtained using the Google Earth. The acquisition dates for these images are April 23rd, 2018 and March 23rd, 2001. The spatial resolution of these images vary from 0.5m to 2.5m.This resolution is sufficient enough to produce high quality landcover map for urban planning and development of a city.

3. RESULTS AND DISCUSSION:

The polygons for all theclasses of study area were digitized in Google Earth and then these were exported to the Q-GIS software for analysis. The files imported into the Q-GIS were first converted into files that were compatible with this software and then a single map of the study areawas prepared which had all land cover sub-classes.
The land cover sub-classes obtained in Q-GIS were merged together to obtain final classes. After merging classes shape file was produced. This shape file is then converted into a map of 2018 using Q-GIS. Another map of the year 2001 is also made using the same methodology. The areas of all land cover classes were obtained for year 2018. These maps were used for obtaining the areas of different classes for the last 17 years.
The calculation shows that the total area of the Abbottabad city is 24.46 sq. miles. In this area, for the year 2001, the calculations showed that built-up area consisted of 26.11% of the total area of the city; barren land was 19.11%; open land comprised of 16.10% of the area; forestation covered 14.72%; crop area used 22.61% of the land available and the remaining 1.34% was taken up by recreational area. Whereas for the year 2018, the area calculated for the different classes were; built-up areawas36.78%; barren land was10.05%; open land was20.80%; forestation covered22.39%; crop area comprised of 6.99% and the remaining 2.98%was covered by recreational area. These results can also be seen in Figure 2.

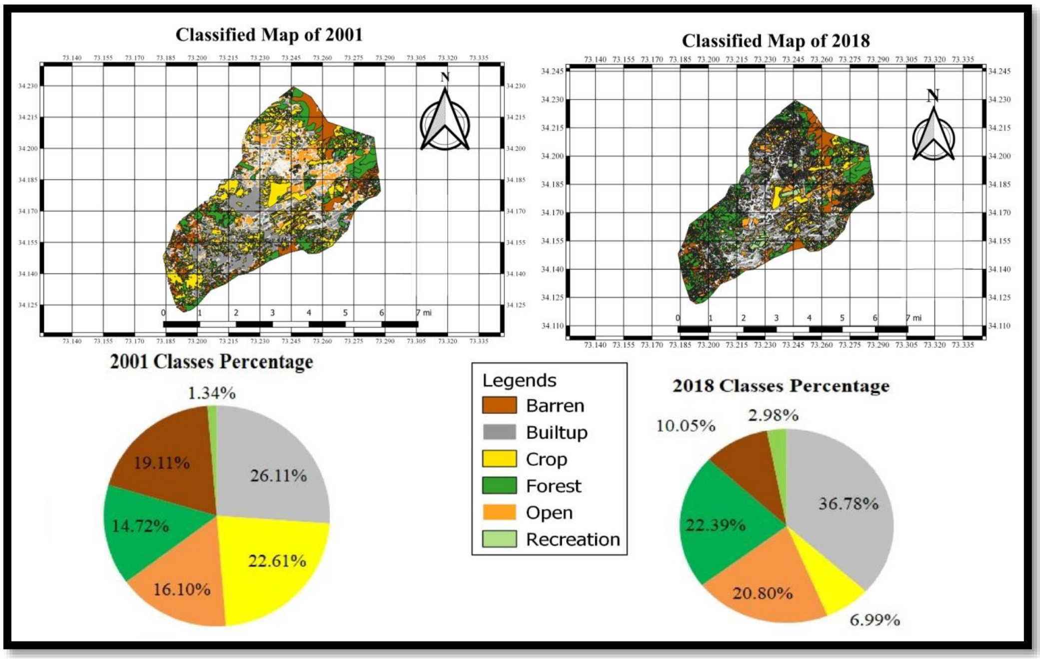

Figure 2. LULC maps with percentages of classes

The increase and decrease in the different classes can be clearly seen from the following table:

Table.1: Difference observed in the different classes over a span of 17 years.

Classes	Percentage Area		Difference in Percentage	
	2001	2018	Increase	Decrease
Barren Land	19.11%	10.05%	-	47.78%
Built-up Area	26.11%	36.78%	40.02%	-
Crop Land	22.61%	6.99%	-	69.33%
Forestation	14.72%	22.39%	51.65%	-
Open Land	16.10%	20.80%	28.46%	-
Recreational Area	1.34%	2.98%	120.93%	-

4. CONCLUSIONS:

GIS and Remote Sensing technology was applied to develop land cover maps for urban planning of Abbottabad city of KPK province. The high-resolution satellite images from Google Earth were processed to produce land cover maps for 2001 and 2018. The classes of each land cover map were: (i) built-up area; (ii) barren land; (iii) open land; (iv) forestation; (v) crop area and (vi) recreational area.

The result indicated 40.02% increases in built-up area, 51.65% increase in forestation, 28.46% increase in open land and 120.93% increase in recreational area in the last seventeen years. There were 40.02% decrease in barren land and 69.33% decrease in agriculture. The results indicated the spread of urban sprawl at faster rate.

The land cover maps could be utilized for effective urban planning and management of the Abbottabad city. GIS and Remote Sensing were found very effective tools for obtaining urban land changes over theperiod of time. Such studies should be accomplished for other cities of Pakistan for town/city planning and development.

This research work can help in improving the state of the city, by controlling the urban sprawl and eliminating the further congestion of the built-up area. If the data used in this research work is used in preparing a master plan, then it can greatly help in the future development of this city. It is recommended that such studies should be carried out for other cities so that their condition can also be improved.

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