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PREFACE

We wish to express our deep sense of gratitude and sincere thanks to Prof. Sunil Kumar, honorable Vice Chancellor, RGPV Bhopal for his able guidance, encouragement and suggestions in organizing this conference. It has been a privilege for us to work under his able guidance. We are grateful to Prof. R. S. Rajput, Registrar, RGPV Bhopal for his valuable support, guidance from time to time. Specially thankful to Prof. S.C. Choube, Technical Education Quality Improvement Programme (TEQIP-III) Coordinator RGPV, Bhopal for providing financial support for this international conference through TEQIP –III and for his guidance and valuable suggestions.

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A LITERATURE EVALUATION OF SMART SENSING TECHNOLOGY IN STRUCTURAL HEALTH ASSESSMENT

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ABSTRACT
There is an important in development exercises in the field of structural designing in the ongoing years. Significant structures like building, bridge, and dam are exposed to serious loading. It also varies with time. In order to monitor the deformation in structure with load, it is required to make periodic observation. Without monitoring the structure can undergo significant deformation which are difficult to observe at the beginning. The deformation is required to be monitored at the beginning of construction, i.e., at pre-construction phase and after the construction, i.e., at post-construction phase. There are a few techniques to determine any deformation in structures. These can be classified into two groups, global and neighborhood. Global technique deals with determination of crack, displacement and deflection in a global scale (on a whole), on the other hand the neighborhood technique tries to determine the deformation of an area w.r.t its neighbor. In this paper, research works using global and neighboring techniques are tried to be compared to find their effectiveness. Global technique can't determine early damage. The neighborhood technique, being confined in character, can distinguish damage inside any limited zone. Henceforth, a procedure is required for Structural Health Assessment (SHA), which should enable persistent checking of structure both locally and globally. Further, these techniques should be applied simultaneously to enable comprehensive analysis in a cost effective way. It is found that, modern sensors these days are capable of monitoring the structures globally and locally giving better results.

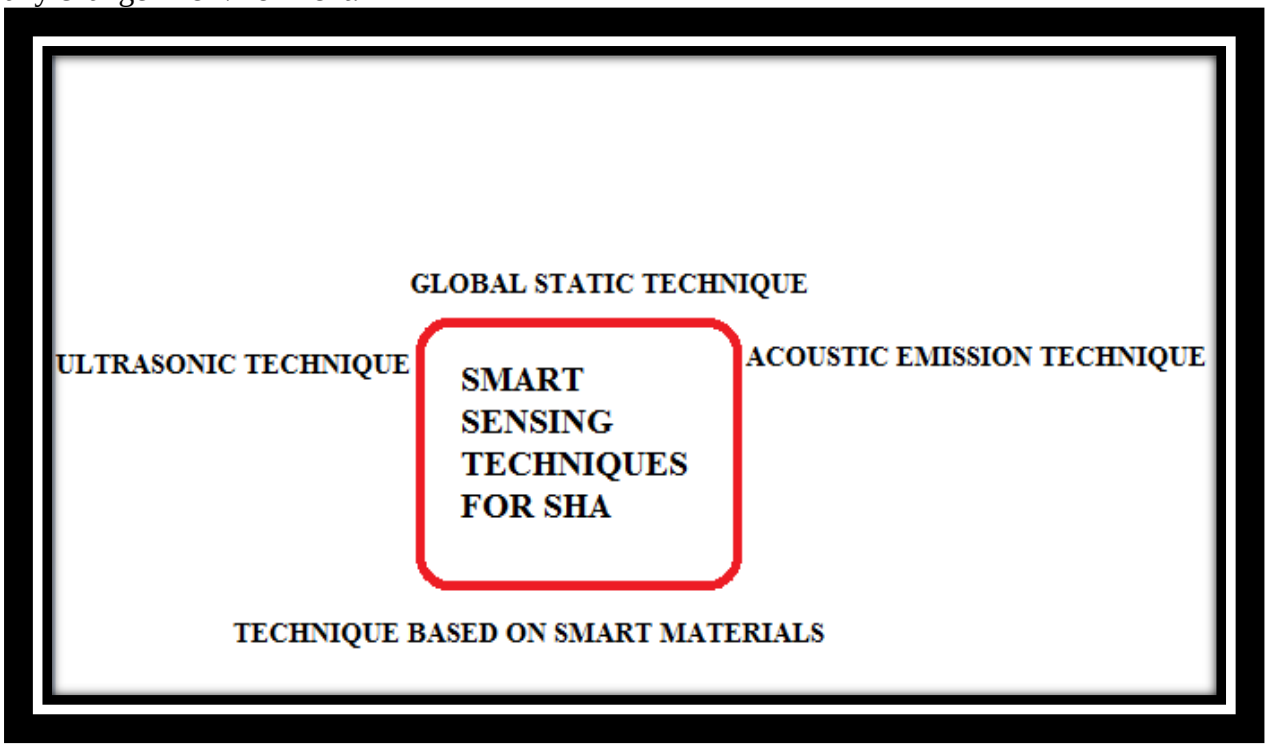
KEYWORDS: Structural Health Assessment (SHA), Global technique, Neighboring techniques, Modern Sensors.

INTRODUCTION
Structural condition assessment is, still, not always direct; such is the case in Health Monitoringof buildings, bridges, and towers. The structural condition is oftentimes required in terms of structural characteristics, i.e., mass, damping, stiffness matrices, damage existence, and/or applied load to the system. These structures are large and consist of various members, which makes such structural condition assessment tough andexpensively. One technique in SHA to improve this difficulty is based on vibration measurement. Though structural characteristics and applied load are tough to assess directly, dynamic behavior, which is a purpose of the structural characteristics and applied load, can be measured. The structural characteristics and applied load information creep around in the dynamic behavior. Structural dependability is expected to be estimated by inverse analyses of the dynamic behavior.
From just limited sensors is insufficient to precisely assess the structural condition. The dynamic performance of these structures is very complex both in spatial and time scale.
Moreover, damage/deterioration is naturally a local phenomenon. Therefore, to comprehend such dynamic behavior, the motion of structures needs to be monitored by compactly located sensors at a sampling frequency sufficiently high to capture salient dynamic characteristics. A successful health assessment shape requires the recognition and combination of a few parts.

Identification of assessment and execution metric is the primary part which is critical information need and should dictate the innovation in question. These realities underline the significance of a robotized health observing structure, which can predict an early damage included collapse, however in addition make the evaluation of auxiliary wellbeing, as and when wanted, at a short notification.

SHA TECHNIQUES

A number of studies have been reported which endeavor to replace the visual inspection by some sort of automated method, which enable more reliable and earlier assessment of the health of the structure. The idea of smart structures was thought to be an alternative to the visual inspection methods. Because of their Inherent 'smartness', the smart materials (such as piezoelectric material, shape memory Alloys, fibre optic materials) exhibit high sensitivity to any change in environment.



FIGTURE: Various Recent Sensing Technology used in The SHA

- Global static technique
- Ultrasonic technique
- Acoustic Emission technique
- Technique based on Smart material

GLOBAL STATIC TECHNIQUE

The Global static technique incorporates applying static forces and dynamic forces on the structure and evaluating the including displacement and deflection. It isn't important to choose the entire arrangement of forces and displacement. In any case, a few load cases might be essential to procure satisfactory information. The resulting conditions are estimated recursively to appear at a lot of part constitutive properties or the structure parameters. Any change in parameters from the designhealthy state gives the sign of the presence of damage. The technique has a number of limitations, especially in the way of practical implementation. Measurement of displacements on a large real-life structure is not an easy task. As a first step,

one needs to establish a frame of reference which requires the construction of a secondary structure on an independent foundation. In addition, employing a number of load cases and computational approach can be very time consuming**.** The application of large loads to cause measurable deflections & strain may necessitate huge machinery and power input. As such, these methods are too tedious and expensive to enable a timely and cost effective assessment of the health of real life structures.

ULTRASONIC TECHNIQUE

The ultrasonic techniques are based on elastic wave propagation and reflection within the material for nondestructive strength and characterization. A piezo-electric probe is employed to transmit high frequency waves into the material. These waves reflect back on encountering any crack, whose location is estimated from the time difference between the applied and reflected waves. These techniques show advanced damage understanding as compared to the global techniques. However, they share few limitations, such as:

- They typically service large transducers and render the structure unapproachable for service throughout the span of the test.
- The measurement data is collected in time domain that requires difficult Processing.
- Since ultrasonic waves cannot be induced at right angles to the surface, they can't detect transverse surface cracks.
- They do not lend themselves to autonomous use since experienced technicians are required to interpret the data.
- Technique is very costly compare to other technique.

ACOUSTIC EMISSION TECHNIQUE

In acoustic emission technique, elastic waves generated by plastic deformations, moving dislocations and disbands are utilized for analysis and detection of structural defects. It requires stress or chemical activity to generate elastic waves, thereby facilitating continuous surveillance. However, the main problem to damage identification is posed by existence of multiple travel paths from the source to the sensors. Also, pollution by electrical interfering and mechanical ambient noise reduces the quality of the emission signals.

TECHNIQUES BASED ON SMART MATERIALS

Smart materials are those materials which have the ability to change their physical properties such as the shape, stiffness, viscosity, etc. in a specific manner under specificstimulus input. Smart materials are one of the components of smart structures. The piezoelectric materials and optical fibers are examples of smart material. Smart structures have ability to sense change in their environment, optimally adjust themselves, and take appropriate action. In the 20th century, metal oxide-based piezoelectric ceramics and other manmade materials enabled engineers to employ the direct and converse piezoelectric effects in several new applications. These materials are generally physically strong and chemically inert, and they are relatively inexpensive to manufacture. The composition, shape and dimensions of a piezoelectric ceramic element can be tailored to meet the requirement of a specific purpose. Ceramics manufactured from formulation of lead zirconate/ lead titanate exhibit greater sensitivity and higher operating temperatures, relative to the ceramics of other compositions. "PZT" material (lead zirconate titanate), is currently the most widely used piezoelectric ceramics.

PROBLEMS IN APPLICATION OF PIEZOELECTRIC MATERIAL FOR SHA

Piezoelectric ceramic elements are used to monitor the health of structures via the Electromechanical Impedance (EMI) technique since mid-nineties. Damage detection is based on assessment of signature with standard signature. Ideally, the change in signature

should be due todescent of health of structure only, not from any other reason. However, it is possible that signature of PZT can change due to other reasons such as:

- **Aging:** PZT patches erode gradually, in a logarithmic relationship with time after polarization. Exact rates of aging depend on the arrangement of the ceramic element and the built-up process used to prepare it. It may possibly induce that change in the benchmark signature.
- **Electrical Limitations:** PZT patches depolarize if it is exposed to a strong electric field of polarity opposite that of the polarizing field. The field of 200-500 V / mm or greater usually have a major depolarizing effect. The degree of depolarization depends on the evaluation of material, the revelation time, the temperature, and other factors. Aconstant current will have a depolarizing effect during each half cycle during which polarity is reverse that of the polarizing field.
- **Mechanical Limitations:** Application of mechanical stress on piezo electric material beyond certain limit can destroy the alignment of dipole. This limit depends upon the type of grade and the brand of the material used to prepare the PZT patches.

CONCLUSION

In this paper a literature evaluationresearch review on Structural health assessment was presented. The evaluation was carried out latest approaches. This paper surveys a collection of basic literatures works that exhibit the ongoing advancement of latest detecting innovations including SHA technique towards their application explicit to SHM. This literature evaluation summary likewise empowers the specialists working around there not exclusively to effectively recognize the area of the exploration work completed previously yet in addition to rapidly discover the pertinent for their very own investigation work.

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EFFECT ON MECHANICAL PROPERTIES OF GEOPOLYMER CONCRETE BY USING CALCIUM NITRATE AS A CURING COMPOUND

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ABSTRACT

Throughout recent decades, the Geopolymer concrete has been explored as an alternative to renewable building materials, enabling the use of manufacturing by-products to minimize CO_2 emissions. Geopolymer concrete made by reacting sodium silicate solution and sodium hydroxide solution with waste materials such as fly ash or ground granulated blast furnace slag (GGBS). Past studies on Geopolymer concrete have shown that it can be appropriate with a workable slump and equal intensity to ordinary Portland cement concrete for structural applications. But however Geopolymer Concrete requires higher temperature for curing. Normal room temperature or water curing has lower mechanical strength. So in this study, Calcium Nitrate ($Ca(NO_3)_2$) was used as curing compound to increased internal heat as well as to increase the mechanical strengths. And it was compared with other curing methods and result shows that use of Calcium Nitrate as a curing compound increased strength of concrete up to 12.71%. However it was still lesser (6.13%) than the strength achieved by oven curing.

KEYWORDS: Geopolymer Concrete, Alkaline Solution, Calcium Nitrate, Strength, Curing

INTRODUCTION

Construction is one of the world's fast growing sectors. According to current world reports, around 260,00,00,000 tons of cement are required annually (M. I. Abdul Arumairaj and P. D., 2012). In another 10 years, this amount will be raised by 25%. Since Lime is the primary source material for ordinary Portland cement, after 25-50 years a severe calcareous shortage may occur. Around one ton of carbon dioxide will be released to the atmosphere as a significant threat to the environment through manufacture of one ton of cement. In addition to the above, an enormous amount of energy is also required for the production of cement. Therefore, discovering an alternate binder is most relevant (M. I. Abdul Aleem and P. D. Arumairaj, 2012).

Professor B. Vijaya Rangan (2008), Curtin University, Australia, stated that, “the polymerization process involves a substantially fast chemical reaction under alkaline conditions on silicon-aluminum minerals that results in a three-dimensional polymeric chain and ring structure” (Hardjito et al. 2004). The final geopolymer composition depends largely on the Si: Al ratio (Si: Al), whereas the components that are most often used for transport infrastructure are usually Si: Al between 2 and 3.5 (Rangan, B. V., 2008 & N. Lloyd and V. Rangan. 2009). The reaction of Fly Ash with an Alkaline solution containing Sodium Hydroxide and Sodium Silicate in their mass ratio, results in a material with three dimensional polymeric chain and ring structure consisting of Si-O-Al-O bonds (Nguyen Van Chanh, Bui Dang Trung, Dang Van Tuan, 2007).

In contrast to the usual Portland cement concrete, the compressive strength of geopolymer concrete is extremely high. Geopolymer concrete has also shown very high early strength.

Geopolymer concrete has a compressive strength approximately 1.5 times greater than that of compressive concrete with ordinary Portland concrete for the same mixture.

MATERIALS AND MIX PROPORTION

FLY ASH

Fly ash used in this study was unprocessed fly ash from torrent power plant (locally available). The whole quantity of fly ash was obtained from one batch and chemical composition of it as per Table 1.

Table 1 Chemical composition of Fly Ash

Chemical composition weight	%	Chemical composition weight	%
SiO_2	53.79	Na_2O	0.37
$Al2O_3$	32.97	K_2O	1.76
$Fe2O_3$	5.51	TiO_2	2.10
CaO	1.84	SO_3	0.46
MgO	0.92	P_2O_5	0.15

ALKALINE SOLUTIONS

The alkaline liquid used was a combination of sodium silicate solution and sodium hydroxide solution. The sodium silicate solution (NaOH = 15.38%, Na_2SiO_3 = 46.15%, and water = 38.46% by mass) was purchased from a local supplier in bulk. The sodium hydroxide (NaOH) in flakes or pellets form having 97% - 98% purity was also purchased from a local supplier in bulk quantity. The NaOH solids were dissolved in water to make the solution. To make alkaline solution first of all to prepare 12 M solution 400 grams of NaOH are dissolved in 1000 gm of water than 1200 grams of Na_2SiO_3 are added in dissolved solution. Then this solution kept as it for 24 hours at normal room temperature.

COARSE AGGREGATES & FINE AGGREGATE

Locally available crushed stones of 10 mm and 20 mm aggregates were used as coarse aggregates. Local river sand used as fine aggregate in the concrete mixtures.

CALCIUM NITRATE

The inorganic compound with the formula Ca(NO3)2 is calcium nitrate, sometimes called Norgessalpeter. Calcium nitrate is used for concrete admixtures that accelerate heat of hydration process. The use with concrete and mortar is based on two effects. The calcium ion speeds up the production and accumulation of calcium hydroxide. This mechanism is also used in cold temperatures and some mixed plasticisers (Justines, H., 2010 and Al-Amoudi, et. al., 2003).

MIX PROPORTION

Table 2 Mix Proportion

Material	MIX M1 (kg/m^3)	MIX M2 (kg/m^3)
Na2SiO3 solution	240.54	240.54
NaOH solution	95.96	95.96
Fly Ash	535	535
Fine Aggregate	450	450
Coarse aggregate	908	908
Calcium Nitrate	0	10.7

EXPERIMENTAL PROGRAMS

Geopolymer Concrete Cubes having grade M30 were made by fly Ash as a 100% replacement of cement, Alkaline Solution, fine aggregate, coarse aggregate and Calcium Nitrate. Dimensions of moulds were 150 mm X 150mm X 150 mm. Demoulding of cubes were done after 24 hours of casting then M1 mix cubes were cured by different methods natural (Ambient Temperature), water and Oven cured; and M2 mix cubes were placed in normal room temperature Mix proportion of Geopolymer concrete as per table – 2. And After it Compression test carried out at 3rd, 7th and 28th day.

RESULTS

Compression tests were carried out on the cubes after 3, 7 and 28 days after casting and result of it as per Table 3.

Table 3 Compression test result

	Compressive Strength (N/mm^2) (average of 3 cubes)		
Curing Type	3rd Day	7th Day	28th Day
Ambient Temperature (M1)	23.67	28.67	36.42
Water (M1)	21.91	29.05	38.45
Oven curing @ 70^0 C (M1)	29.62	35.53	43.57
Curing Compound (M2)	24.63	30.67	41.05

CONCLUSIONS

As per the results of compressive strength test results, strength of Geopolymer concrete depends upon the curing compound. The maximum compressive strength of 43.57 N/mm^2 achieved in oven cured Geopolymer concrete at 70^0 C and least strength was achieved in water cured Geopolymer concrete. Also as per the compressive strength result, oven cured and curing compound cured Geopolymer concrete gains early age strength near to target mean strength required.

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SCALED MODEL TESTS ON THE INFLUENCE OF GEOGRID APERTURE SIZE ON THE DEFORMATION OF GEOGRID REINFORCED RETAINING WALL

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ABSTRACT

Geogrid Reinforced soil retaining walls are internally stabilized systems wherein the fill reinforced with multiple layers of reinforcement behaves as a coherent composite mass and resists earth pressures from the retained fill and other externally imposed loads."Mohammad Saleh Ahmadi(2017),in their work emphasis that –It was found that aperture dimension is influence factor in the pull out resistance (POR) of geogrids and should be selected properly based on the Particle Size Distribution of the Soil (PSD)."Domestic and foreign scholars have done some valuable researches over this, but the reinforcement aperture size effects on the retaining wall failure were still not fully resolved currently in academic sector. Thus in this work two types of geogrids with different aperture size and tensile strength but of similar material are compared by varying the three basic parameters i.e. length, spacing and layer of geogrid, which effect the displacement and failure of the backfill mass of the retaining wall system. Taking the value of these parameters below and within the range, the failure of the structure is analyzed by Load-Displacement graphs using scaled model tests along with photometric images. The result shows that lateral displacement of the backfill mass increased with lesser aperture size as compared to larger aperture size geogrid of higher tensile strength

KEYWORDS: Reinforced retaining wall, geogrid, scaled model tests, lateral displacement, aperture size, spacing, layer, length

INTRODUCTION

Reinforced earth retaining wall is a type of retaining wall which can bear the lateral earth pressure made up of backfill soil, reinforcement and facing system."Zornberg J.G, (2001) in their work emphasis that- Traditional soil reinforcing techniques involve the use of continuous geosynthetics inclusions such as geogrids and geotextile". The involvement of geosynthetics in reinforced soil construction is because of number of factors, including aesthetics, reliability, simple construction techniques, good seismic performance, and the ability to tolerate large deformations without structural distress. Examples include used in reinforced soil design for conventional loading, advances in design for unconventional loading, and in reinforcement materials like, polymeric fiber reinforcements. "R.D.Haltzs (2001) in their work emphasizes that -Geosynthetics as soil reinforcement used for embankments on soft foundations, steep slopes, and for the backfills of retaining walls and abutments". Emphasis is on the materials properties of the geosynthetics required for design and construction. Thus reinforced soil retaining walls (RSW) are widely used in geotechnical engineering practice throughout the world because of its durability and strength. Reinforced soil walls can be constructed using steel strip reinforcement, geogrids, geosynthetics, and geo composite. Geosynthetics manufactured from advanced polymer materials became popular as alternative to steel strip reinforcement in reinforced soi1 walls**.**

LITERATURE REVIEW

Jan Derksen & Martin Ziegler [1],Analysis of bearing capacity of reinforced retaining structures..A medium sand is chosen for the subsoil, fill and backfill of the retaining walls. The model geogrid used for the reinforced structures is a regular biaxial product made of PET.. Results confirmed a considerable increase in bearing capacity with enlarged reinforcement length resulting in an improved bearing resistance. N. Yasufuku& H. Ochiai [2], full Scale Testing of Geosynthetic Reinforced Walls,. The confining effect parameter defines the degree of the apparent incremental confining stress to the current normal stress on the expected sliding plain, was in the range of 0 to 0.3 against the supposed dilatancy angles from 0 to 30 degrees. Mohammad Saleh Ahmadi and Parastoo Nikbakht Moghadam [3], Effect of Geogrid Aperture Size and Soil Particle Size on Geogrid-Soil Interaction under Pull-Out Loading Five types of geogrid samples with various aperture dimensions were produced and used for conducting pull-out tests along with four types of soils with different particle size distribution (PSD) and grading were used for this purpose. R.J. Bathurst & D. Walters [4], Full Scale Testing of Geosynthetic Reinforced Walls Influence of reinforcement spacing ,wall toe restrained facing stiffness on reinforcement on reinforcement on loads/strains/deformations and overall level of safety. The toe of the wall in these experiments carried a significant portion of the horizontal earth forces acting on the hard facing column. Richard. Bathurst [5], Instrumentation of Geogrid-Reinforced Soil Walls, The horizontal displacement response of extensometers mounted on grid layers of an incremental wall test. The comprehensive monitoring of model walls and the quality of the data has allowed the author to identify important mechanisms in the behavior of these complex systems during construction, under working load conditions, and at failure.

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METHODOLOGY ADOPTED

In this study, geo grids of two different aperture sizes 1.8mm X 1.8mm (Type A) and 6mm X 6mm (Type B) on basis of various parameters which influence the stability of backfill soil of a geogrid reinforced retaining wall system comprise of cohesion less sand, as this is the type of soil commonly used for Mechanically Stabilized Earth (MSE) retaining wall system, but for which standard stabilizing procedures have found to be inadequate (Derek V. Morric,1999).

Experimental Work-

1. Soil-

For the model tests cohesion less, dry and clean standard sand is used as the foundation material. The particle size of sand used for the test was passing through600 micron sieve.

2. Geogrid

The glass fiber geogrid is chosen as research material because the strength of glass fiber geogrid is lower than other model grids, it meets the need of scaled model test. Geogrids used are-TGS 2525 and TGS 3030

3. Steel Box

The test setup consists of a strong mild steel (MS) box having 720 mm in length, 220mm in breadth and 600mm in height (internally). Four handles are provided on top side for holding purpose and front face is of toughen glass

4. Universal Testing Machine

Universal testing machine of 100 ton is used for the scaled model experimental testing. Using UTM machine compression load is applied on the modeled wall using a steel square footing

5. Footing Size

Upper plate- 140 mm– rectangular size**,** Height – 100mm**,** Lower plate 20mm square size**,** Material – steel

6. Wall Facing

The facing of reinforced wall is done with the gravel gabion wall along with wrap around facing. Gravel gabion wall is made at 90 degree to the surface. The spacing of grids is maintained using the gravel gabion of two different sizes. For spacing 5cm- gabion height 5cm, spacing 7.5 cm – gabion height 7.5cm

NUMERICAL STUDY

Specific Gravity - (G = 2.62)

Table-1 Specific Gravity by Pycnometer Method

Sr. No.	DESCRIPTION	OBSERVATION		
1	Sample No.	1	2	3
2	Mass Of Pycnometer (W1)	488	488	488
3	Mass Of Pycnometer With Dry Soil (W2)	788	888	988
4	Mass Of Pycnometer With Soil And Water (W3)	1440	1496	1512
5	Mass Of Pycnometer Full Of Water (W4)	1260	1250	1258
6	Specific Gravity (G)	2.57	2.63	2.68

Vibration Table Test

i) Loose State

Table-2 Loose State

No. Of Observation	Weight Of Mould (gm)	Wt. Of Mould+ soil (gm)	Wt. Of Soil (gm)	Density (gm/cc)	Avg. Density
1	5920	9620	3700	1.331	
2	5920	9640	3720	1.338	1.331
3	5920	9620	3700	1.331	

ii) Dense State

Table-3 Dense State

No. Of Observation	Weight Of Mould (gm)	Wt. Of Mould+ soil (gm)	Wt. Of Soil (gm)	Density (gm/cc)	Avg. Density
1	5920	10250	4330	1.558	1.558

Direct Shear Test

Table-4 Summary of Test Result

Soil Property	Loose Sand	Dense Sand
1. Unit weight (Kg/m^3)	15	17
2. Ø (°)	39	45

Table-5 Normal Stress On Sand

	1st Sample	2nd Sample	3rd Sample	4th Sample
Normal Stress (KN/m^2)	27	54	82	112



(a) (b)

Figure 1: a) Gabion wall with Type A geogrid ;b) Scale Model testing with complete setting

RESULTS

The GRS wall is allowed to deform under incremental load from 0 to 3 kN.

On Basis of Number of Layers

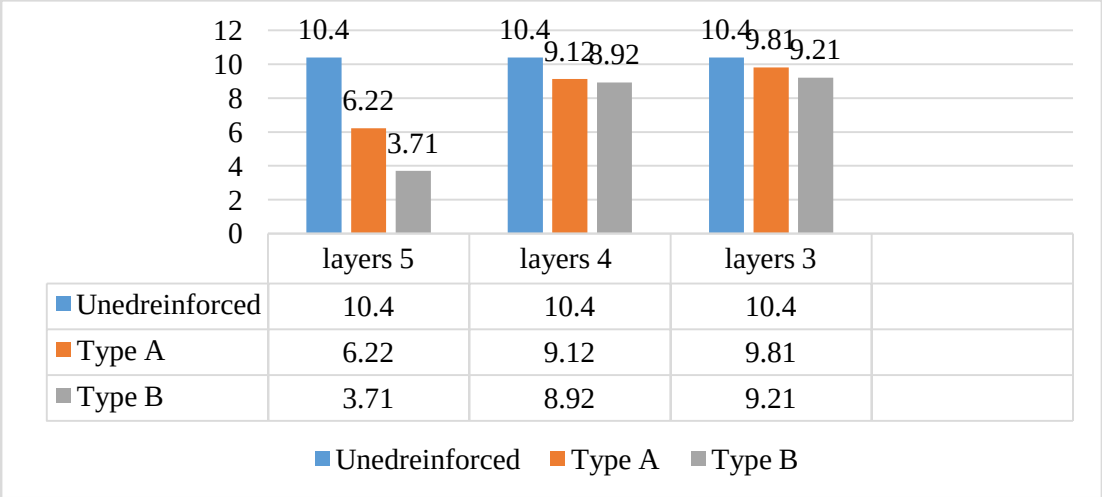


Figure 2 Comparisons of Load- Strain graphs for Number of Layers 3,4 and 5

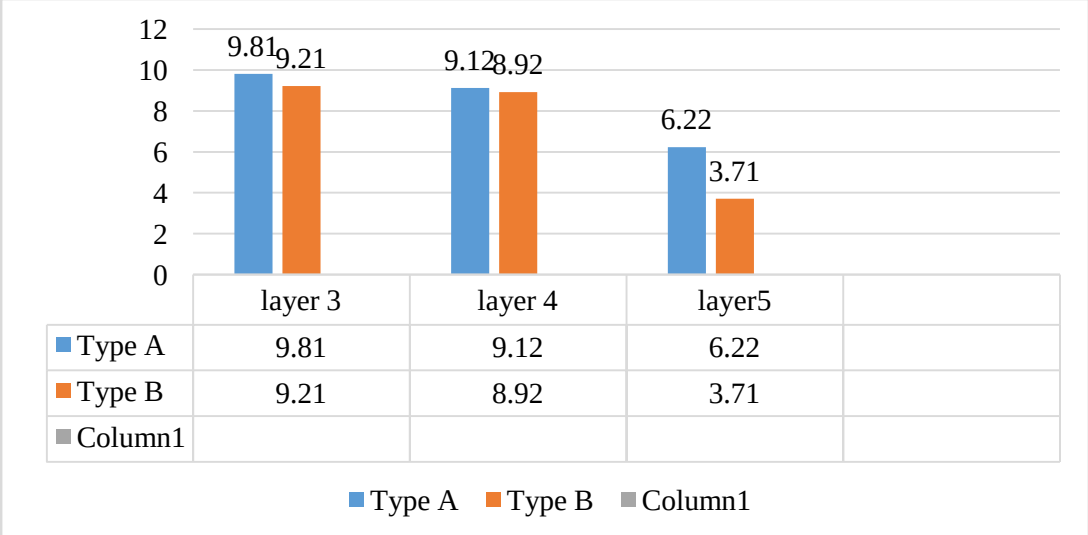


Figure 3 Comparison of Load- Strain graphs for Strain graphs for Spacing 7.5cm

On Basis Of Spacing

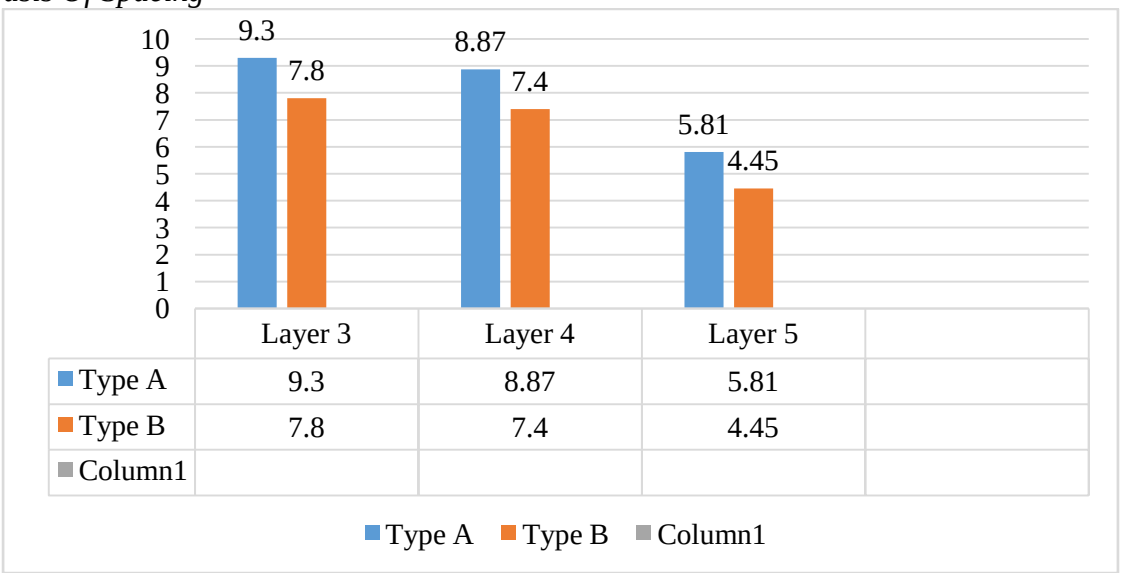


Figure 4 Comparison of Load- Strain graphs for Spacing 5cm

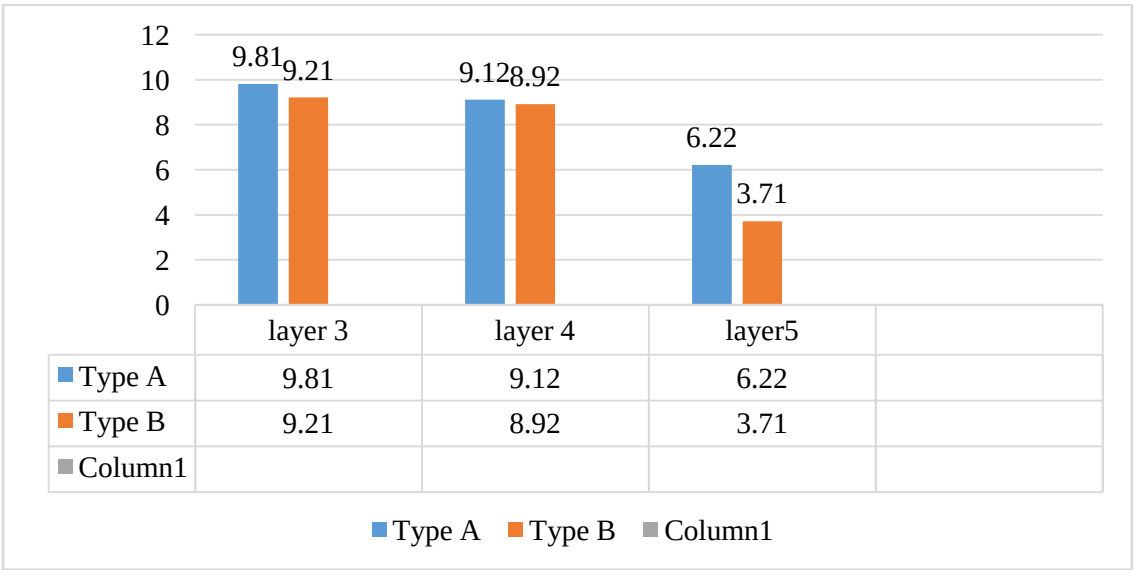


Figure 5 Comparison of Load- Strain graphs for length 35cm

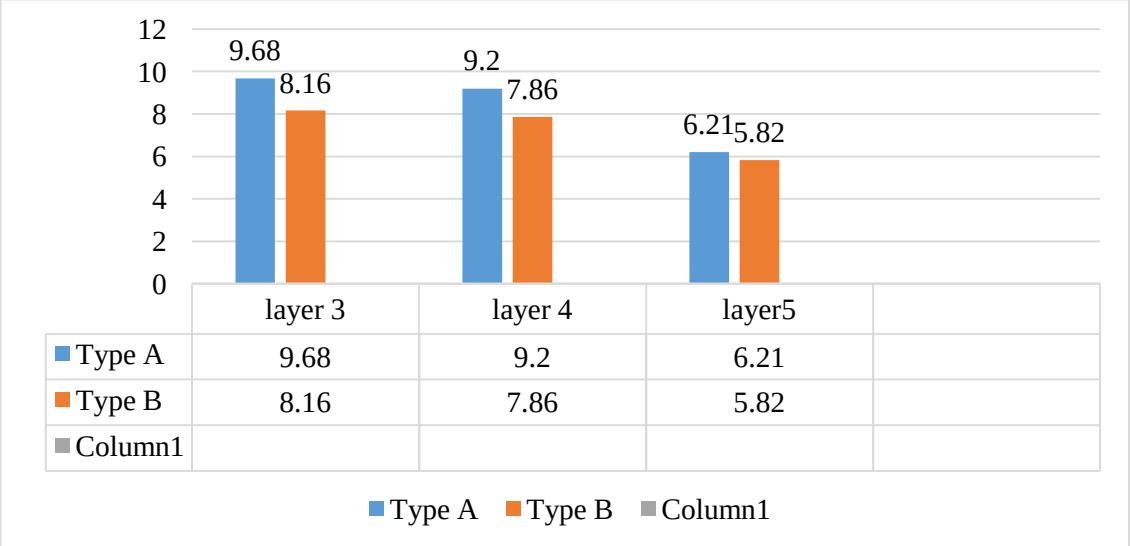


Figure 6 Comparison of Load- strain graph for length 20 cm

CONCLUSIONS

The following are conclusions drawn from the present work are given as follows:

1. TGS 3030 gives the better results in terms of lesser deformation with the same amount of loading.
2. As the number of layers increases from three to five the bonding between the soil the geogrid increases which reduces the displacement.
3. Spacing between the geogrids plays an important role in soil mass strengthening which increases with decrease in spacing.
4. The length of geogrids if increases it affects the economic value but as much soil mass is covered gives resistance against the pull out of geogrid.

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STUDY OF GLASS FIBER REINFOCED POLYMER REBARS- A REVIEW

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ABSTRACT
In reinforced concrete structures, steel and concrete interfacial strength plays an important role. The separation of steel-concrete is one of the principal causes for deterioration or damage of the civil infrastructure. The design of structural members is based on the fundamental assumption that exist the effective bond liking concrete and steel when the structural member is loaded. The behaviour of reinforced concrete element depends on the steel-concrete bond, and the strength capacity of these elements is directly related with the bond. Bond strength between rebar and concrete is essential for reliable performance of reinforced concrete structures. For increasing bond strength between steel and concrete some admixtures have been used with reinforced concrete. Glass fiber reinforced concrete exhibits excellent bonding to steel reinforcement and to old concrete. The glass fiber efficiency is controlled by the resistance of the fibers to pullout, which in turn depends on the bond strength at the fiber interface. The prediction of the interfacial shear strength between the steel bar surface and concrete surface of steel rod bonded into concrete. The present work investigates different properties like compressive strength, tensile strength and bond strength of glass fiber reinforced concrete.

KEYWORDS: F.R.P., GFRP, Tensile strength, Rebars

INTRODUCTION
A composite material combines two or more materials (e.g., fiber reinforcements and binder or matrix resins). These combinations are designed to obtain a material that achieves targeted performance objectives and properties. As an example, fiber-reinforced polymer (FRP) composites are made of thermosetting or thermoplastic resins and glass and/or carbon fibers. The fiber network provides the load-bearing component of the composite while the resin contributes towards transferring loads to the fiber network and maintains fiber orientation. The resin controls the manufacturing process and processing variables. Resins also protect the fabrics from environmental factors such as humidity, high temperature, and chemical attack. There has been significant research on the development of FRP composite materials and their innovative applications. The main aim of this paper is to study various properties like tensile Strength, compressive strength, flexural strength, ductility and creep effect of FRP and to list out the merits and demerits of FRP rebars over steel rebars.

LITERATURE REVIEW

Nkurunziza and Benmokrane(2005) conducted a study to evaluate the creep behavior and change in tensile property of GFRP. They studied its behavior for 417 days (10,000 hr) in different environments and under ambient temperature. Under combinations of different sustained load levels. In their study, 20 GFRP of 9.5 mm dia (circular section) in four series, surrounded by alkaline solution (PH-12.8) or deionized water subjected to 25 and 38% guaranteed tensile strength. A comparitive study in reference with ACI 440. Alkaline solution caused higher drop in strength. Residual strength was too drop to 88 to 68% of the

guaranteed strength with alkaline solution, and for de-ionized water, is almost unchanged. Modulus of elasticity of the bars was very stable. No single case of creep rupture was observed in that study over 417 days of sustained loading at the high level of stress used.

Adam and Said(2015) conducted an analytical experimental flexural behavior study on GFRP bars reinforced concrete beam. Ten beams (120mm width x300mm dep x 2800mm long) were casted and tested upto failure under 4-point bending. Three different amounts of GFRP reinforcement and three grades of concrete (25,45, and 70 mpa) compressive strength were used for that study. Elastic modulus-30,0000 mpa. In Non Linear finite element analysis for studying the flexure behavior of concrete beams by correlating the load deflection relationships with the analystical and experimental ones, cracks were first observed at the tension zone, and under increasing load, cracks propagated in a vertical direction and further new cracks appeared through the shear span. The predicted N.L.F.E. nalysis cracking loads were shown to be in a good agreement with the experimental loads.

Deitz and Harik(2012) studied Forty-five glass fiber reinforced polymer GFRP rebars were tested in compression to determine their ultimate strength and Young's modulus. The rebars or C-bars, produced by Marshall Industries Composites, Inc., had an outside diameter of 15 mm #15 rebar, and unbraced lengths varying from 50 to 380 mm. A compression test method was developed to conduct the experiments. Results were 1. For nonslender rebars (*Lu*,110 mm), the ultimate compression strength ($f_{f\,u,c}$) is equal to approximately 50% of the ultimate tensile strength ($f_{f\,u,t}$); and
Based on limited testing, Young's modulus in compression ($E_{f\,,c}$) can be considered approximately equal to Young's modulus in tension ($E_{f\,,t}$).

Miàs., Torres. et al.(2017) studied analysis and advanced Materials for Structural Design (AMADE), Polytechnic School, University of Girona, Campus Montilivi s/n, 17071 Girona, Spain
Due to the mechanical properties of fibre reinforced polymer (FRP) bars, deflections often drive the design of FRP reinforced concrete (RC) flexural members. This paper presents the results and discussion of an experimental programme concerning eight glass FRP RC beams tested at service load, and subsequently subjected to sustained loading for 250 days. Two reinforcement ratios and two levels of sustained load were considered. The experimental results revealed an effect of the loading– unloading processes and the reinforcement ratio on short and time-dependent deflections. No significant influence of the sustained load level was observed.

OBJECTIVES

Various objectives of the present work are as follows:

1. To study the various physical and mechanical properties of FRP.
2. Then find out various types of FRP.Study of different characteristics and applications of FRP in various Civil Infrastructure constructions.
3. Analyse the cost-benefit Comparison between G.F.R.P. Bar and iron bar
4. Then find out experimentally tests results of different properties of F.R.P.

TYPES OF FRP

1. **Glass Fibers** -Glass filaments normally manufactured by a procedure named direct dissolve, wherein strands with a measurement of 3 to 25 microns have shaped by fast and nonstop drawing from a glass liquid. Glass strands are the most economical, and subsequently the most normally utilized, filaments in auxiliary designing applications. There are a few distinctive evaluations available; however, the most

known have E-glass and the more costly, yet more grounded, R-glass. Glass filaments have portrayed by their high quality, moderate modulus of versatility and thickness, and by their low warm conductivity. Glass strands are regularly decided for auxiliary applications that are not weight basic (glass FRPs are heavier than carbon or aramid) and that can endure the bigger diversions coming about because of the nearly low flexible modulus of the glass filaments. Glass strands have frequently utilized as a part of the assembling of FRP strengthening bars, pultruded FRP basic segments, FRP wraps for seismic redesign, and fiber wound FRP tubes.

2. **Carbon Fibers**- Carbon strands have created by a procedure named organised pyrolysis, wherein one of three potential forerunner strands have subjected to an unpredictable arrangement of warmth medicines (adjustment, carbonization, graphitization, and surface dealing) to deliver carbon fibers with distances across in the scope of 5 to 8 μ. The subsequent filaments can have properties that shift broadly, thus a few classes of carbon strands are accessible, separated in view of their flexible moduli.
3. **Aramid Fibers**- Aramid filaments have made from an engineered compound named fragrant polyamide in a procedure named exclusion and turning. Two firmness evaluations are promptly accessible: 60 GPa and 120 GPa. Aramid filaments have described by high quality, moderate versatile modulus, and low thickness. What's more, FRPs made from aramid filaments have low 8 ompressive and shear qualities because of the exceptional anisotropic properties of the strands. Aramid filaments are additionally defenceless to debasement from presentation to bright radiation and/or dampness.

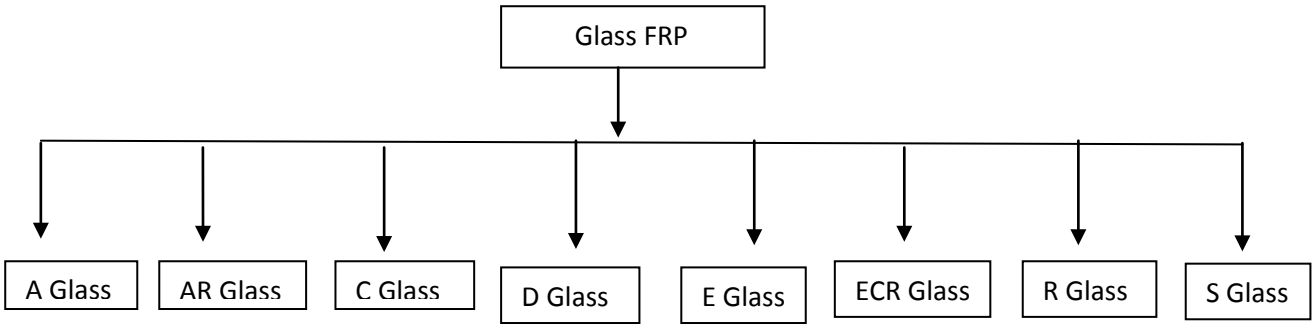


Fig 1: Flow chart showing different categories of Glass F.R.P.

Table 1 below shows the typical mechanical properties of different glass fibre

Table 1: Glass Filament Typical Mechanical properties-

Fiber type	Density, (g/cm3)	Tensile Strength, MPa	Modulus, GPa	Percent Elongation
A-glass (Alkali glass)	2.44	3300	72	4.8
AR-glass (Alkali Resistant glass)	2.7	1700	72	2.3
C-glass (Corrosive resistant glass)	2.56	3300	69	4.8
D-glass (Low dielectric constant glass)	2.11	2500	55	4.5

E-glass (electrically resistive glass)	2.54	3400	72	4.7
ECR-glass (E-glass with higher acid corrosion resistance)	2.72	3400	80	4.3
R-glass (Reinforcement)	2.52	4400	86	5.1
S-glass (also S-2 glass) (brand name)	2.53	4600	89	5.2

CHARACTERSTICS OF GFRP

1. An American Concrete Institute design guide for fiber -reinforced polymer (FRP) - reinforced concrete suggests that ,in order to avoid creep-rupture failure, service load sustained stress should not exceed 0.20 times the design tensile strength of glass-fiber-reinforced-polymer(GFRP) reinforcement. Where ff_u is equal to the guarantee tensile strength multiplied by an environmental reduction factor. The 0.2 value was put in place prior to 2001, was conservatively chosen due to both limited test data and a lack of standards for both materials and test methods. The analysis indicates that a 114-year service life and a 1.67 safety factor as is used by ACI design guide for FRP-reinforced concrete, a sustained stress limit of $0.30ff_u$ is conservative value to avoid creep rupture of GFRP bars that confirm to ASTM standards.
2. Ductility-Ductility is a material property that characterises the ability of the material to undergo substantial deformation and is important in limiting the deformations of the structure such as deflections and rotation in beams as cardinal objectives of design is to ensure that the structure will not fail abruptly in the event of extreme circumstances. Thus, the structural materials are required to possess adequate ductility so that brittle failures are avoided or reduced to barest minimum.
3. Tensile strength-It has been seen that the stress-strain curves followed an almost linear graph up to stress values close to the ultimate tensile strength.
4. Shear behaviour-Most F.R.P. bar composites are relatively weak in interlaminar shear where layers of unreinforced resin lie between layers of fibers. Orientation of the fibers in an off-axis direction across the layers of fiber will increase the shear resistance, depending upon the degrees of offset. For F.R.P. bars, this can be accomplished by winding fibers transverse to the main fibers.
5. Bond behaviour-In F.R.P. bars, it is postulated that bond force is transferred through the resin to the reinforcement fibers, and a bond-shear failure in the resin is also possible. When a bonded deformed bar is subjected to increasing tension, the adhesion between the bar and the surrounding concrete breaks down, and deformations on the surface of the bar cause inclined contact forces between the bar and the surrounding concrete .

APPLICATIONS OF FRP

1. The material characteristics of FRP reinforcement need to be considered when determining whether FRP reinforcement is suitable or necessary in a particular structure.
2. The corrosion resistance of FRP reinforcement is a significant benefit for structures in highly corrosive environments such as seawalls and other marine structures, bridge decks and superstructures exposed to deicing salts, and pavements treated with deicing salts.
3. In structures supporting magnetic resonance imaging (MRI) units or otherequipment sensitive to electromagnetic fields, the nonmagnetic properties of FRP reinforcement are unrivaled.
4. FRP reinforcement has a non-ductile behavior that is partially compensated by its high tensile strength. The use of FRP reinforcement should be limited to structures that will

significantly benefit from other properties such as the non corrosive or non conductive behavior of its materials. Due to lack of experience in its use, FRP reinforcement is not recommended for moment frames or zones where moment redistribution is required.

5. FRP reinforcement should neither be used as reinforcement in columns nor other in compression members, nor as compression reinforcement in flexural members. It is acceptable for FRP tension reinforcement to experience compression due to moment reversals or changes in load pattern. The compressive strength of the FRP reinforcement should not, however, be neglected.
6. Typical FRP reinforcement products are grids, bars, fabrics, and ropes. The bars have various types of cross sectional shapes (square, round, solid, and hollow) and deformation systems (exterior wound fibers, sand coatings, and separately formed deformations).

RESULTS:

The following results were drawn from the test conduted:

GFRP has light weight and it is cheaper than the conventional steel bars. The Table-2 shows the weight and cost benefit comparison.

Table-2: A weight and cost benefit Comparison between G.F.R.P. Bar and iron bar

G.F.R.P.					STEEL				
Dia in mm	Weight of rod per 12m length	Weight of rod per metre length	Cost of rod per 12m length	Cost of rod per metre length	Dia in mm	Weight of rod per 12m length	Weight of rod per metre length	Cost of rod per 12m length	Cost of rod per metre length
4.5	0.31	0.026	66.00	5.50	6		0.22		
6	0.66	0.055	120.00	10.00	8	4.65	0.39	186.00	15.50
7	0.84	0.070	147.00	12.25	10	7.44	0.62	288.00	24.00
8	1.1	0.092	188.00	15.67	12	10.56	0.88	418.00	34.83
12	2.45	0.204	388.00	32.33	16	18.96	1.58	750.00	62.50
16	4.41	0.370	770.00	64.17	20	29.628	2.469	1240.00	103.33

It is found that GFRP rebars have high tensile strength, less modulus of elasticity,less elongation and are non conductive in nature.

Table-3: The main indicators of steel and fiber glass reinforcement

Material	Steel	FRB- Polymer bound glass fiber
Tensile strength (MPA)	390	1250
Modulus of Elasticity (Gpa)	200,000	50,000
Elongation (%)	25	2.2
Coefficient of Thermal Expansion (10-6/^{o}c)	13-15	9-12
Density (ton/m^{3})	7.8	1.9
Corrosion resistance to aggressive environment	Appearance of rust products	Stainless material preserving properties even in alkaline concrete environment
Thermal Conductivity	Conducts temperature	Does not conduct temperature
Electrical Conductivity	Electrically conductive material	Dielectric
Ecological Compatibility	Electrically friendly material	Does not emit harmful and toxic substances during storage and

		use
Durability	Terms are prescribed in special building codes	Not less than 80 years

It has been observed that according to the tensile strength the size of GFRP bars are found to be lesser than metal rebars. Table-4 shows the size substitution of metal rebar by FRP rebar.

Table-4: Table of substitution – by Tensile strength

Metal Rebar	Fiber glass Rebar (FRP)
Ø 6	Ø 4
Ø 8	Ø 6
Ø 10	Ø 7
Ø 12	Ø 8
Ø 14	Ø 10
Ø 16	Ø12
Ø 18	Ø 14
Ø 22	Ø 16
Ø 25	Ø 18
Ø 28	Ø 20
Ø 32	Ø 22

CONCLUSIONS

From the above test results it can be concluded that in comparison with steel;

1. FRP have higher resistance to corrosion,
2. FRP have higher tensile capacity, and
3. FRP have lower weight.
4. FRP are also non-conductive for electricity and non-magnetic.

Thus, for structures built in or close to seawater or at similar corrosive environment, where electromagnetic neutrality and/or electric insulation are required, or exposed to deicing salts, FRP may become invaluable.

Rapid installation: FRPs can be fast implemented due to modular, pre-fabricated, and light weight units that eliminate forming and curing efforts necessary for conventional materials such as concrete decks or elaborate welding and riveting needed in heavy steel construction.

Unfortunately, FRP are not free from hindrances that need to be resolved before they can be prescribed as a reinforcing material for concrete structures.

5. The major obstacles are the high initial cost,
6. low modulus of elasticity,
7. lack of ductility and absence of design guidelines.

FRP reinforcement should not be relied on to resist compression.

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QUALITY ASSESSMENT OF SURFACE WATER BODIES OF CITY OF BHOPAL AFFECTED BY CONSTRUCTION AND DEMOLITION PROJECTS

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ABSTRACT

The purpose of research is to suggest the study of quality assessment of surface water bodies of city of Bhopal due to affected by construction and demolition projects. Absolutely pure water in never found in nature and large number of impurities present in water due to construction and demolition projects of city. Community life required water supply and sewage disposal. Without proper disposal, the wastes of community can create intolerable nuisance, spread diseases, and public health hazards and we need to water surface bodies free from odour, tubidity, bacteria, dissolved gases, harmful salts, poisonous metals, radio-active substances, phenolic compounds, and it should be non corrosive. the aim of this research shows that we required water quality assessment of surface water bodies time to time and we have to take necessary steps for water quality

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INTRODUCTION

Water is the most essential parts in our life. Water is needed by man, plants, animals and without water they cannot be survival, water maintain ecological balance in the relationship between living things and environmental impacts in which they live. Community life required water supply and sewage disposal. Without proper disposal, the wastes of community can create intolerable nuisance, spread diseases, and public health hazards .The planning, design and operation of water and waste water require a high degree of skill and judgment. The complete planning designing and construction of water supply schemes become the works of civil engineers. Therefore, for every civil engineer it is necessary to have the full knowledge of it.Quality assessment of surface water bodies dependent on study of suspended, dissolved and colloidal impurities present in surface water. Therefore, we remove impurities only into certain extent so that it may not be harmful to the public health.

Activities which generate C & D waste in cities / towns are mainly from: (according central pollution control board March-2017)

1. Demolition of existing, old dilapidated structures;
2. Renovation of existing buildings (residential or commercial);
3. Construction of new buildings (residential or commercial or hotel etc.);
4. Excavation/ reconstruction of asphalt/ concrete roads;
5. Construction of new fly over bridges/ under bridges/ sub-ways etc.; and
6. Renovation/ Installation of new water/ telephone/ internet/ sewer pipe lines etc.
7. Present collection and disposal system.



Figure 1.

LITERATURE REVIEW

S.No.	Author's	Title	Publication	Remarks
1.	Harish nema ,prof.charan singh thakur	Process monitoring and performance evaluation of 10 MLD Sewage treatment plant at kotra Bhopal india.	IRJET ,2018	Waste water, sewage treatment plant kotra, Bhopal mp
2.	Nidhi gupta, pankaj Pandey, jakir huassin	Effect of physicochemical and biological parameters on the quality of river water of narmada Madhya pradesh india.	National water research center , ELSEVIER, 2017	Water quality indices, water quality standard, Narmada Madhya Pradesh india
3.	Derara chalchisa, moa megersa, abebe beyene	Assessment of the Quality of drinking water in storage tnaks and its implication on the safety of urban water supply in developing countries.	Environmental system research, SPRINGER, 2017	Assessment of Drinking water, safe water
4.	Shikha saxena, nisha singh, anjali bajpai	Physic-chemical assessment of water quality in and around Jabalpur city of Madhya Pradesh india	IJCSR, 2016	Assesment of water quality of surface water narmada river Jabalpur india

5.	Jaladhi vavliya, bhavsar dhruv, utkarsha kavadi, mahroof mohammad	Online performance assessment system for urban water supply and sanitation services in india.	ELSEVIER, 2016	Performance assessment, capacity building
6.	Mir mohammad ali, mohammad lokman ali, md. Saiful islam, md. Zillur rahman	Preliminary assessment of heavy metals in water and Sediment of karnaphuli river, Bangladesh	Environmental nanotechnology, monitoring & management, ELSEVIER, 2016	Assessment of heavy metals & water sediment coastal river bangaladesh
7.	Jyoti jotwani, bharti jain, suman malik	Assessment of potability of different water sources at Bhopal city.	International journal of environmental sciences, 2014	Assessment of water supply from different sources in Bhopal city
8.	Ram kumar kushwah, avinash bajpai, suman malik	Characteristics of waste water in sewage treatment plant of Bhopal india.	Journal of chemical and pharmaceutical research, 2011	Assessment of sewage treatment plant of bhopal
9.	Akhilesh jinwal, savita dixit	An assessment of fluoride concentrations in different districts of Madhya Pradesh india.	International journal chemistry science, 2009	Assessment of ground water, flouride
10.	Y. zhao, X. H. Xia, Z. F. yang, F. wang	Assessment of water quality in baiyangdian lake using multivariate satatistical techniques.	ELSEVIER, 2011	Assessment of water quality, physical chemical parameters

PROPOSED METHODOLOGY

Life cycle assessment and itspurpose-

Life cycle assessment (LCA) is a tool to evaluate theenvironmental effects of a product or process throughout its entire life cycle. An LCA entails examining theproduct from the extraction of raw materials for the manufacturing process, through the production and use of the item, to its final disposal, and thus encompassing the entire product system. A schematic representation of a product life cycle is given-

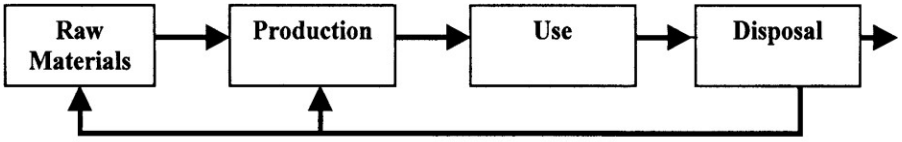


Image II

NEW PROJECTS IN BHOPAL CITY-

1. Smart city projects Bhopal
2. Bhopal metro projects
3. Commercial complex project

4. Residential projects
5. New fly over /under bridges and sub-ways etc
6. Installation of new water/ telephone/ internet/ sewer pipe lines etc.

CONCLUSIONS

The purpose of research is to suggest the study of suspended, dissolved and colloidal impurities present in surface water bodies and we remove to their impurities and it should be free from poisonous metals like iron, manganese, lead, arsenic, radium, stronsium, harmful salts, etc. Review of all the papers and result shows that we required water quality assessment time to time and the aim of this research is to we need to quality assessment of surface water bodies of city affected by construction and demolition projects.

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EFFECT OF USING MARBLE POWDER AS A PARTIAL REPLACEMENT OF CEMENT ON THE STRENGTH OF CONCRETE

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ABSTRACT
With increased urbanization in the past decade, a huge demand for Concrete as a building material has surfaced in recent times. This has led to an increase in the demand for related materials like cement and aggregates. However, this demand has led to an increase in the overall budget of any construction project and thus urged civil engineers to find alternate cheaper and easily available materials as complete or partial substitutes for building materials. Marble as a building material especially in palaces and monuments has been in use for ages. However, the use is limited as stone bricks in walls or arches or as lining slabs in walls, roofs or floors, leaving its wastage at the quarry or at the sizing industry generally unattended for use in the building industry itself as filler or plasticizer in mortar or concrete. This huge unattended mass of marble waste consisting of very fine particles is today one of the environmental problems around the world. One of the logical means for the reduction of the waste marble masses calls for utilizing them in the building industry itself. Thus this research work was carried out to assess the effect on the strength of concrete when this waste marble powder was used as a partial replacement of cement. The concrete grade of M20 was examined in this research work and it was observed that up to 20% of the quantity replacement an increase in the strength was noted but further increase in marble powder quantity led to a reduction in the strength. Thus it was concluded that such type of partial replacement of cement with marble powder in M20 concrete shall be recommended up to a limit of 20%.

KEYWORDS:budget, alternate, marble powder, strength, replacement

INTRODUCTION
Concrete is a composite material composed of coarse granular material (the aggregate or filler) embedded in a hard matrix of material (the cement or binder) that fills the space among the aggregate particles and glues them together. Concrete is widely used for making architectural structures, foundations, brick/block walls, pavements, bridges/overpasses, highways, runways, parking structures, dams, pools/reservoirs, pipes, footings for gates, fences and poles and even boats.

In modern times, researchers have experimented with the addition of other materials to create concrete with improved properties, such as higher strength or electrical conductivity. For a very long time, strength was the only criteria on which the concrete designs were based. However, in due course of time the concept of durability has gained major importance in the design of concrete.

"Cement", commonly Portland cement, and other cementitious materials such as fly ash and slag cement, serve as a binder or a Ligare for the aggregate. Water is then mixed with this dry composite, which produces a semi-liquid that workers can shape (typically by pouring it into a form). The concrete solidifies and hardens to rock-hard strength through a chemical process called hydration. The water reacts with the cement, which bonds the other components together, creating a robust stone-like material.

"Chemical admixtures" are added to achieve varied properties. These ingredients may speed or slow down the rate at which the concrete hardens, and impart many other useful properties. "Mineral admixtures" are becoming more popular in recent decades. The use of recycled materials as concrete ingredients has been gaining popularity because of increasingly stringent environmental legislation, and the discovery that such materials often have complimentary and valuable properties.

Marble is a non-foliated metamorphic rock composed of recrystallized carbonate minerals, most commonly calcite or dolomite. Marble as a building material especially in palaces and monuments has been in use for ages. However the use is limited as stone bricks in wall or arches or as lining slabs in walls, roofs or floors, leaving its wastage at quarry or at the sizing industry generally unattended for use in the building industry itself as filler or plasticizer in mortar or concrete. The result is that the mass which is 40% of total marble quarried has reached as high as millions of tons. This huge unattended mass of marble waste consisting of very fine particles is today one of the environmental problems around the world.

One of the logical means for reduction of the waste marble masses calls for utilizing them in building industry itself. Some attempts have been made to find and assess the possibilities of using waste marble powder in mortars and concretes and results about strength and workability were compared with control samples of conventional cement-sand mortar/concrete.

In this study, marble powder was used as a partial replacement of cement in concrete. For each variation in quantity of replacement of cement, strength of the concrete cubes were assessed. The procedure that follows has been described below.

LITERATURE REVIEW

Rai et al. (2011) studied the influence of marble powder/ granules in concrete mix and concluded that increased waste marble powder (WMP) or waste marble granule (WMG) resulted in increased workability and compressive strengths of the mortar.

A Ergün (2011) conducted a study to study the effect of using diatomite and waste marble powder as partial replacement of cement on the mechanical properties of concrete. His study revealed that the concrete specimens containing 10% diatomite, 5% WPM and 5% WPM +10% diatomite replacement by weight for cement had the best compressive and flexural strength and the replacement of cement with diatomite and WMP separately and together using a super plasticizing admixture could be utilized to improve the mechanical properties of the conventional concrete mixtures.

Shirule et al. (2012) also used marble dust powder as a partial replacement of cement and tested the cylinders as well as cubes for strength. They concluded from their study that the optimum percentage for replacement of marble powder with cement and it is almost 10% cement for both cubes and cylinders.

METHODOLOGY ADOPTED

To carry out the proposed study cubes of concrete (1:1.5:3) with varying partial replacement of cement with the marble powder were cast and tested at three different intervals of 3,7 and 21 days. Their results were compared with those of standard (1:1.5:3) mortar and concrete cubes.

The Marble Powder used in the experiment was purchased from the Siyaganj Area of Indore. Before initiating the test properties of materials were determined according to respective IS codes. The properties are shown in Table 1.

Standard cubes of size 15cm x 15 cm x 15 cm were used to cast the concrete cubes.

Table 1: Properties of Raw for concrete

Material	Fineness/Fineness Modulus	Specific Gravity
Cement	0.225 m^2/g	3.21
Fine Aggregate	2.36	2.71
Coarse Aggregate	2.96	2.85
Marble Powder	1.5 m^2/g	2.67
Grade of Cement Used - OPC 43		
Consistency of cement is 30%. Initial and final setting time of cement is 50 minutes and 520 minutes respectively		

Table 2 below gives the detailed proportioning of the concrete mix.

Table 2: Concrete Mix (1:1.5:3) Proportion

	Type I	Type II	Type III	Type IV	Type V
% replacement of cement with marble powder	0	10	20	30	40
Cement	4050	3645	3268	2854	2450
Fine Aggregate	6075	6075	6075	6075	6075
Coarse Aggregate	12150	12150	12150	12150	12150
Marble Powder	0	405	816.8	1225	1634
Water	2025	1822	1634	1427	1225
Water/cement ratio	0.5	0.5	0.5	0.5	0.5

Three cubes of each Type were cast in standard cube moulds. The concrete was compacted using a vibrator. The moulds were opened after 24 hours and the cubes were soaked in the curing tank for curing. The standard procedure followed was :

(i) Clean the mounds and apply oil
(ii) Fill the concrete in the molds in layers approximately 5cm thick
(iii) Compact each layer with a vibrator.
(iv) Level the top surface and smoothen it with a trowel.

The test specimens were stored in moist air for 24 hours and after this period the specimens were marked and removed from the molds and kept submerged in clear fresh water until taken out prior to test.

These cubes were subjected to Universal Testing Machine (UTM) and were checked for strength. The observations of each composition after the testing were noted and calculations were performed as below:

Size of the cube =15cm x15cm x 15cm.

Area of the specimen (calculated from the mean size of the specimen)=225cm^2

Compressive stress the cube can bear = load at which the cube fails in the UTM / area of the specimen.Similar calculations were done for 3,7 and 28 days.

RESULTS

The results of strengths of each cube sample, upon subjecting them to load failures in UTM were noted and summarized in Table 3 below.

Table 3: Values of strength for each replacement sample

Percent replacement of concrete	Strength of the concrete in N/mm^2		
	3 days	7 days	28 days
0%	11.7	14.5	20.1
10%	12.5	15.9	20.5
20%	14.7	18.7	24.0
30%	11.4	14.5	18.6
40%	11.6	14.8	19.0

To better analyse the results they have been represented in the graphical form below in Figure 1.

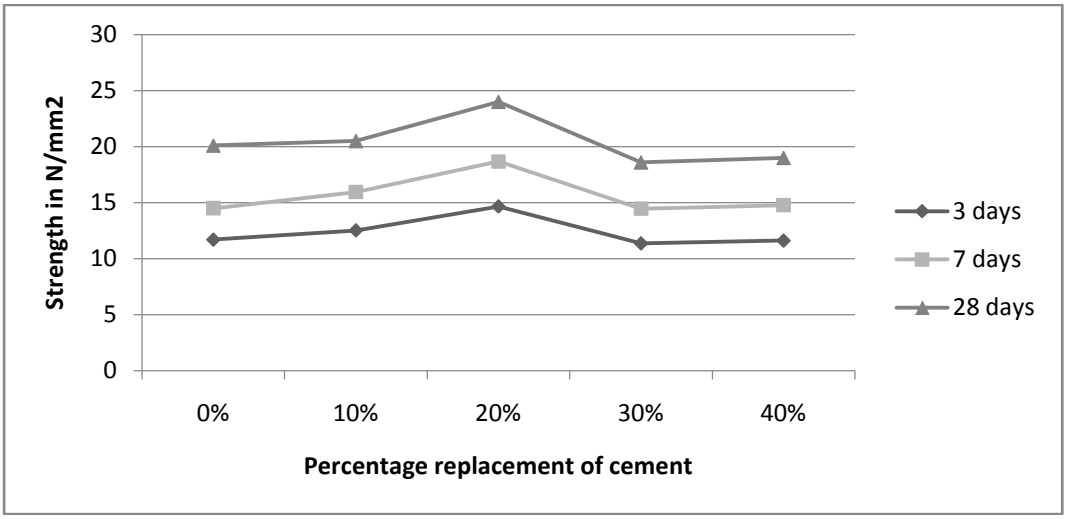


Figure 1: Variation of strength with variation in replacement of cement with marble powder

CONCLUSIONS

The following are conclusions drawn from the present work are given as follows:

1. The feasibility of replacing cement with marble powder upto 10%, 20%, 30% and 40% was demonstrated in M20 grade of concrete.
2. Experiments with M20 grade of concrete suggest that 20% replacement of cement with marble powder resulted in the best concrete strength. Also 10% replacement gave satisfactory results. However 30% and 40% replacement resulted in poor strength of concrete.
3. This may be an effective way of disposing off the waste marble powder generated from the marble industry.
4. This may be perceived as an economic cost-cutting measure in big projects where huge quantities of cement are involved. Around 20% of this cement quantity maybe replaced by marble powder thereby reducing the cost of cement.

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COMPARITIVE ASSESSMENT OF PROPERTIES OF PLASTIC COATED AGGREGATES WITH FRESH AGGREGATES

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ABSTRACT
Disposal of plastic is a roaring challenge globally. There is massive generation of single use it cannot be disposed off naturally. Hence plastic has found its disposal ultimately in various industries including the construction industry. The paper includes assessment of the aggregate properties with plastic coated aggregates. The aggregates procured from market were heated and then coated with a layer of LDPE single use plastic. Certain common tests namely Los-Angeles Abrasion test, Impact Value test, Crushing Strength Test and others were performed on these coated aggregates. The results of these tests were compared with those of fresh aggregates. It was found that plastic coated aggregates yielded either better or similar results than those of fresh aggregates, for instance plastic coating was found to increase the aggregate impact value. Thus this inference may be used as a lucrative option for disposal of single use plastic. Also this process shall generate an additional job for rag pickers and thus enhance employability.

KEYWORDS: LDPE, plastic coated aggregates, Crushing Strength Test

INTRODUCTION
Plastic waste is a severe threat to the environment. Plastic is user friendly but not eco friendly because it is not bio-degradable. India generates close to 26000 tonnes of plastic in a day making it the 15th biggest plastic polluter globally.
The plastic used was the waste plastic bottles, LDPE/HDPE bags, wrappers, collected from the nearby houses and apartment and from the dump yards. On the basis of physical properties, plastic is classified as thermo plastic and thermo set. Thermoplastic can be remolded and thermo set cannot be remodeled by application of heat. Thermoplastic constitute 80% of total post consumer plastic waste generated while thermo set plastics constitute 20% of total post consumer plastic waste generated.
Plastic waste when used in coating aggregates, were found to improve the performance of aggregate, improving in abrasion, slip resistance, crushing, durability and fatigue life. This plastic coated aggregate can be used in different civil engineering works after testing in laboratory.

Waste Plastic And Its Soureces

Waste plastic	Origin
low density polyethylene (LDPE)	Carry bags, sacks, milk pouches, Cosmetic detergent bottles
High density polyethylene (HDPE	Carry bags, bottle cups, house hold articles etc.
Polyethylene teryphthalate (PET)	Drinking water bottles etc
Polypropylene(PP)	Bottle caps and closures, Wrappers of detergent, microwave Trays etc.
Polystyrene(PS)	Yoghurt pots, clear egg packs, Bottle caps, protective packing etc.
Poly vinyl chloride(PVC)	Credit cards, toys, pipe and gutters

LITERATURE REVIEW

V.S. Punith (2001) found some encouraging results in his study to improve the performance of bituminous mixes of road pavements. He heated waste plastics (polythene carry bags, etc.) at around 130°C and softened them. Further thermo gravimetric analysis on this plastic showed that the reisno gas was evolved in the temperature range of 130-180°C. Softened plastics have a binding property. Hence, it was concluded that they may be used as a binder for road construction.

Sabinaetal (2001)studied the comparative performance of properties of bituminous mixes containing plastic/polymer (PP) (8% and 15% by wt of bitumen) with conventional bituminous concrete mix (prepared with 60/70 penetration grade bitumen). Improvement in properties like Marshall Stability, retained stability, indirect tensile strength and rutting was observed in plastic modified bituminous concrete mixes.

Mohd.Imtiyaz (2002) concluded that the mix prepared with modifiers showed higher resistance to permanent deformation at higher temperature.

Dr. R. Vasudevan (2007) stated that the polymer bitumen blend is a better binder as compared to plain bitumen. Blend increased softening point and decreased penetration value with a suitable ductility. When it was used for road construction it was found to withstand higher temperature and load. The coating of plastics reduced the porosity, absorption of moisture and improved soundness. The polymer coated aggregate bitumen mix formed better material for flexible pavement construction as the mix showed higher Marshall Stability value and suitable Marshall Coefficient. Hence the use of waste plastics for flexible pavement was concluded as one of the best methods for easy disposal of waste plastics.

METHODOLOGY ADOPTED

1. In the present study,plastic waste made out of PP(Poly propylene), PE(Poly ethylene), PS(Poly styrene), cut into a size between 2.36mm and 4.75mm was used in shredding machine (PVC waste should be eliminated)
2. Collected plastic waste sorted as required thickness.
3. The aggregate was heated to 1650C.
4. Shredded plastic was added to hot aggregate. The plastic got softened and coated over the surface of the aggregate.
5. At constant temperature mixture was stirred manually for about 20-30min.
6. Mixtures of different percentages of plastic were prepared & subjected to different tests namely: :-
   a) Crushing test
   b) Los Angeles abrasion test
   c) Impact test

RESULT

Different testing was done with different plastic percentage as per IS specification.

a) Crushing test: - Aggregate crushing value test on aggregate gives a relative measure of the resistance of an aggregate crushing under gradually applied load. (As per IS specification). The values have been reported in Table 1 and represented graphically in Figure 1 for different plastic percentages.

Table 1: Crushing values for different plastic percentages

Test	PLASTIC CONTENT						
	0%	5%	6%	7%	8%	9%	10%
Crushing Value	18.28	13.5	7.95	8.7	8.1	9.23	11.07

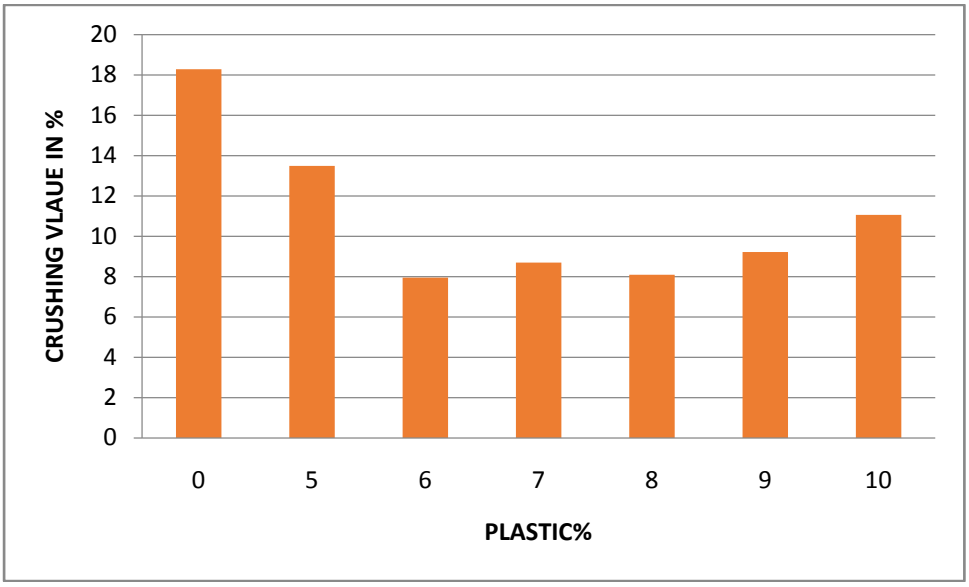


Figure 1: Crushing values at different plastic contents

b) Los angeles abrasion test: - Abrasion test is the measure of aggregate toughness and abrasion resistance such as crushing, degradation and disintegration. (As per IS specification). The values have been reported in Table 2 and represented graphically in Figure 2 for different plastic percentages.

Table 2: Abrasion values for different plastic percentages

Test	PLASTIC CONTENT						
	0%	5%	6%	7%	8%	9%	10%
LOS ANGELES ABRASION Value	11.2	11.26	8.43	8.92	8.23	9.87	10.52

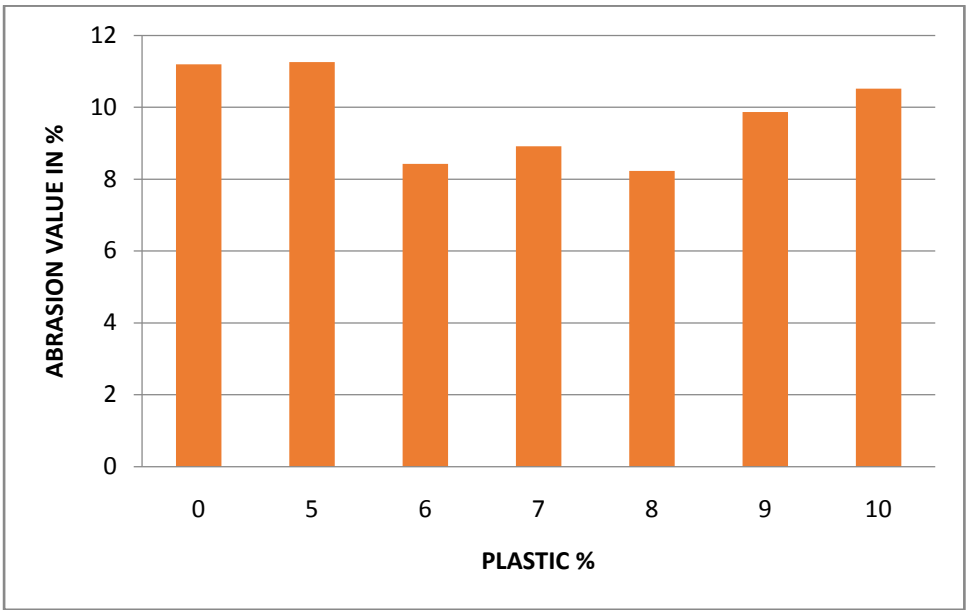


Figure 2: Abrasion values at different plastic contents

c) Impact value test: - The aggregate impact value is a measure of resistance to sudden impact or shock. . (As per IS specification). The values have been reported in Table 3 and represented graphically in Figure 3 for different plastic percentages.

Table 3: Impact values for different plastic percentages

Test	PLASTIC CONTENT						
	0%	5%	6%	7%	8%	9%	10%
Impact Test	18.28	13.5	7.95	8.7	8.1	9.23	11.07

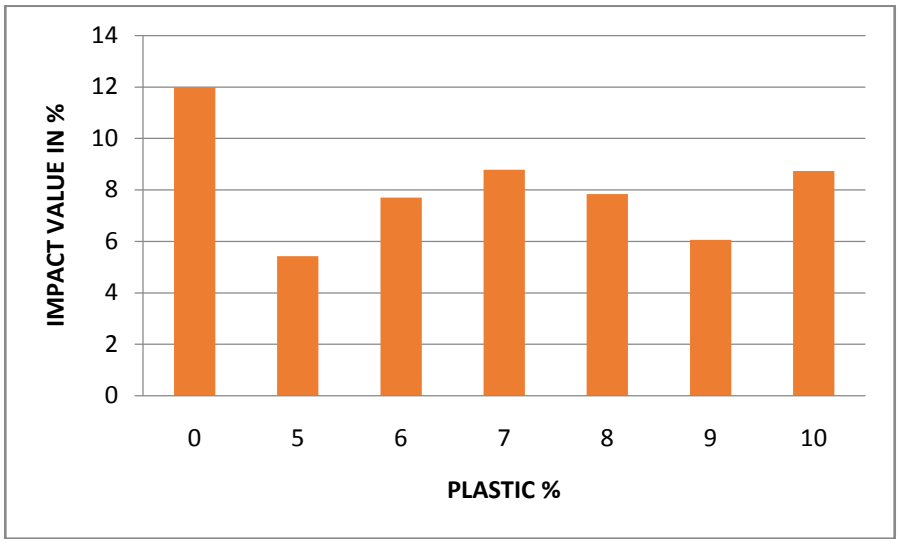


Figure 3: Impact values at different plastic contents

CONCLUSIONS

The following are conclusions drawn from the present work:

Different percentages of plastic coated aggregates were used to find different properties of aggregate. In present study only three properties i.e. Impact test, Crushing test and Los Angeles abrasion test was performed on plastic coated aggregate and fresh aggregate and comparative study has been done.

- It is found that on adding 8% of plastic in aggregate for coating, the crushing value, impact value and abrasion values gave most satisfactory results.
- This may be perceived as an effective alternative to dispose off single use plastic which is in actual a ecological hazard, due to its non-biodegradable nature.
- This may also create additional employment for rag pickers and also indirectly contribute in maintaining cleanliness of the country (solid waste management).

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STUDY ON EFFECT OF BRACING IN MULTI STOREY BUILDINGS

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ABSTRACT
Seismic retrofitting is a technique used to modify existing structure and to strengthen the structure so that it can withstand seismic variations without any failure. About 60-70% buildings of our country are vulnerable to seismic activity of various intensities. Most of the structure constructed in the past either do not have sufficient lateral stiffness or they are often non-ductile in nature, thereby making them more prone to seismic variations and need adequate seismic strengthening.Buildings which are situated in earthquake prone areas need to be retrofitted so that they can resist ground motions effectively. In this study, a (G+2) RC frame is analysed using Staad Pro V8i and then the frame is retrofitted by means of different types of steel bracing system (Inverted-V, X, V and K). The frames are subjected to various seismic load combinations such as (1.5 (Dead load + Live load), 1.2 (Dead load + Live load + Earthquake load in X direction), 1.2 (Dead load + Live load + Earthquake load in Z direction), 1.5 (Dead load + Earthquake load in X direction). The frames are analysed as per IS 456 and IS 1893. The results with and without retrofit frame are compared and it is inferred that steel bracing system considerably reduces the lateral displacements when the structure is subjected to seismic loads.It is also evident that X bracing system considerably minimizes the lateral displacement to a great extent.

INTRODUCTION
Seismic retrofitting is the modification of existing structures to make them more resistant to seismic activity, ground motion, or soil failure due to earthquake. The purpose of seismic retrofitting is to enhance structural capacities (strength, ductility, stiffness, stability and integrity), so that performance of the building can be raised to a desired level. Seismic retrofitting is mainly done to meet seismic safety requirements. Many buildings do not meet the seismic design requirement because they are not designed for seismic loads or they are designed as per old IS code.

Public safety, economic considerations, material survivability is achieved by seismic retrofitting.The earthquake loads acting on the structures are termed as lateral loads because they are acting in horizontal direction. The building behaves like a vertical cantilever during ground motions and acceleration generated by the seismic wave get transmitted through the vibrating structure to the masses at different floor levels, thereby generating horizontal seismic forces.

Many techniques have been proposed by the researchers which can help in resisting the earthquake loads. Some researchers found out that soft storey needs some serious retrofitting measures. Three methods were identified to retrofit open ground storey are brick masonry, steel bracing and use of RC structural wall in ground storey. A comparison has been done and the author proposes structural wall method to be best suited for open ground storey as it gives maximum strength and ductility(Waghmare, 2011). Steel plating method is used to retrofit beams and concrete jacketing to retrofit columns (Munshi Md Rasel, 2015). (Amritha Ranganathan, 2015)has made comparison among concrete jacketing, steel jacketing, FRP wrapping to reach up to a conclusion that which method is more suitable to retrofit the local

members. It is found that the structures which were constructed previously using codes and seismic zone that no longer exists in present times. This arises the fact that these structures need some retrofitting measures to withhold the current obligations.

(J.O.Afloyan, 2016)Studied the reinforced concrete building frame with or without retrofit and analysed the frame using Staad Pro software. The results are compared with relevant codes of practice. The structures that are structurally deficient having soft storeys or improper design were investigated by performing dynamic tests. Super reinforced with flexibility method has been developed to prevent the load carrying capacity of structure under excessive lateral deformation, comparison of collapse behaviour with or without retrofit to check the effectiveness of SRF (Yousok Kim, 2012). (Dr. Suchita Hirde, 2013)Studied the performance of two industrial building which needs retrofitting. Two cases were observed, one needs retrofitting because of faulty design and other needs retrofitting because of operational purposes. (Krishnaraj R Chavan, 2014)Studied that the steel bracing system is applied to reinforced concrete frame. He suggested different types of bracing system such as Diagonal bracing, V-bracing, inverted V bracing and X bracing. The author then compares the results obtained from Staad Pro software in terms of lateral displacement, storey drifts, axial force and base shear.
Addition of shear walls, steel bracing system, reduction of irregularities, concrete jacketing, steel jacketing, FRP wrapping, base isolation devices are some of the methods used to retrofit the structures either globally or locally.
In this study, different types of steel bracing system have been used (V, inverted-V, X and K bracing) to retrofit the structural frame which is assumed to be in seismic zone IV. Different types of load combinations are studied to check the response of the structure in every possible manner. Bracing system can be added to the exterior side of the structure so it does not interfere with functional use of the building.The models are analysed according to IS Code 1893:2002 using STAAD PRO V8i software. The results of with or without retrofit are compared to evaluate which bracing system resists the seismic forces more efficiently.

METHODOLOGY

The RC structure modelled in this study is a two storey frame (G+2). The frame is analysed using STAAD PRO V8i software. It is considered to be an ordinary moment resisting frame. It is 2-3m bay structure having total height of 9m. The structure is located in seismic zone IV. The live load considered on terrace is 1.50kN/m^2, floor level is considered to be 2.50kN/m^2 and Floor finish is taken as 1.0kN/m^2. The cross-section of beams and columns are 300mm X 230mm. Slab dimensions are 3m x 3m. The member which is used to retrofit the frame is of cross section 230mm x 230mm. Parapet wall is also considered on terrace. The thickness of brick wall is assumed to be 230 mm thick. Unit weight of concrete is taken as 25kN/m^3. M25 grade of concrete and Fe 415 grade of steel is command for study. All the calculations are done according to IS 456 and IS 1893.
The design seismic load considerations are according to seismic zone IV. Soil considered being medium soil site with 5% damping (S_a/g=2.50) as per IS 1893:2002 (part 1), clause 6.4.5.Fundamental time period of 0.389 seconds as per 1893:2002 (part 1), clause 7.6.1. Importance factor of 1.5 is taken for public buildings 1893:2002 (part 1), table 6, clause 6.4.2, zone factor of 0.24 according to IS 1893:2002 (part 1), table 2, clause 6.4.2, and response reduction factor of 3 taken up for ordinary moment resisting frame as per IS 1893:2002 (part 1), table 7, clause 6.4.2.
Calculation of seismic forces according to IS 1893:2002-

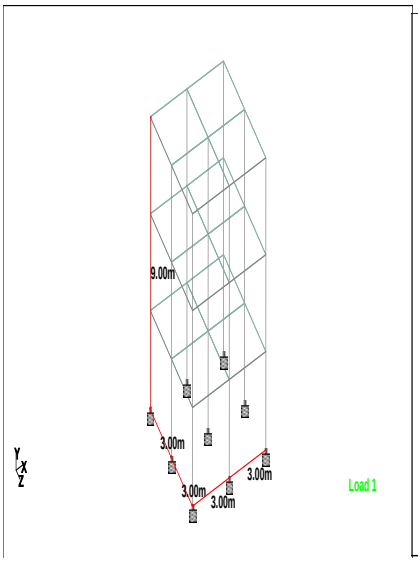
A_h= Z/2 x I/R x S_a/g,

Where,

A_h= Design horizontal seismic coefficient
Z= Zone factor for the Maximum Considered Earthquake (MCE) and service life of a structure in a zone.
I= Importance factor is given according to the use of structure considering its economic value, functionality, prone to failure etc.
R= Response reduction factor, depending on the perceived seismic damage performance characterized by ductile or brittle deformation.
S_a/g= Average response acceleration coefficient

$V_B = A_h$ x W;
Where,
V_B = Design base shear,
W = Total seismic weight
The structure is subjected to various load combinations so that we can obtain the maximum possible displacement of the structure.

- 1.5 (Dead load + Live load)
- 1.2(Dead load + Live load + Earthquake load in X direction)
- 1.2(Dead load + Live load + Earthquake load in Z direction)
- 1.5 (Dead load + Earthquake load in X direction)



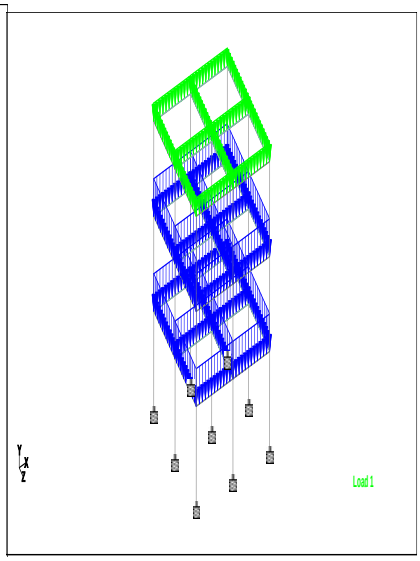
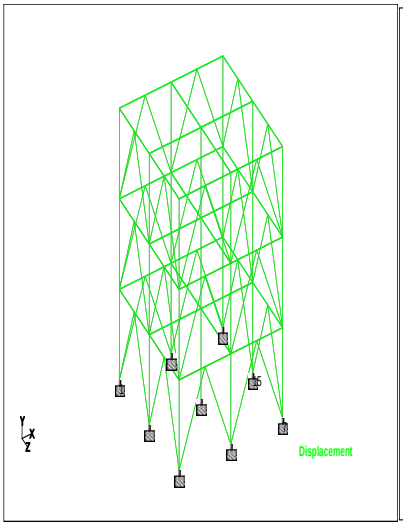


Fig. 1 Line diagram of plane frameFig .2Loading pattern in plane frame



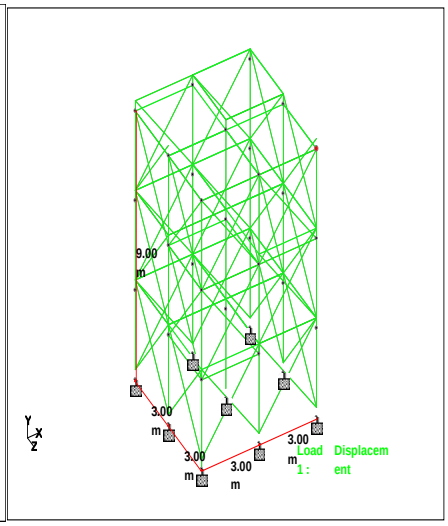
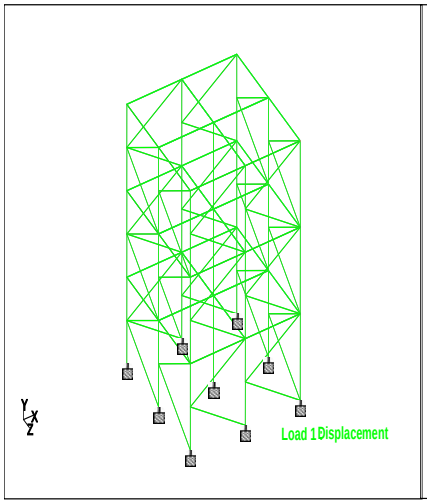


Fig. 3 Inverted-V type bracingFig. 4 X type bracing



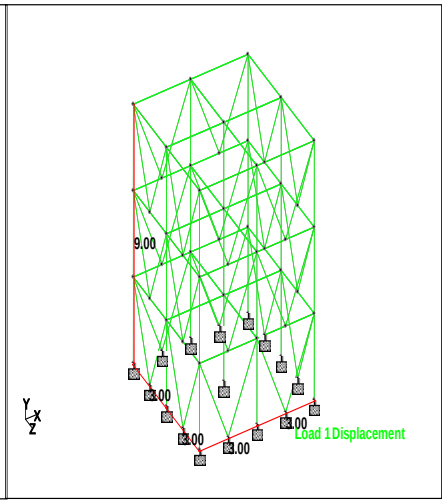


Fig. 5 K type bracingFig. 6 V type bracing

RESULTS AND DISCUSSIONS

The graph is plotted between displacements (mm) Vs Number of storeys with different load combinations to evaluate the results of plane frame (non-retrofitted frame) and the frames which are retrofitted by means of bracing system.

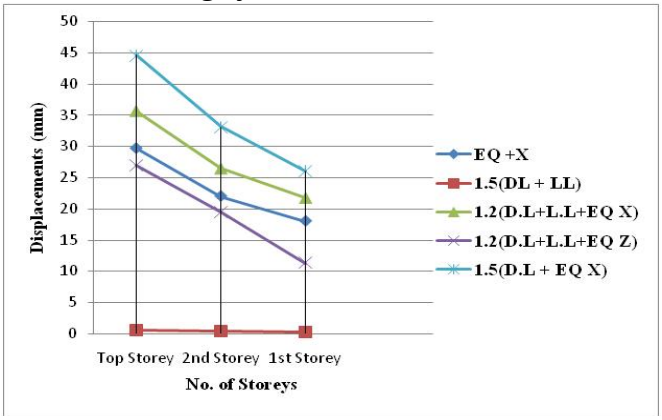


Fig. 7 Displacements (mm) in plane frame

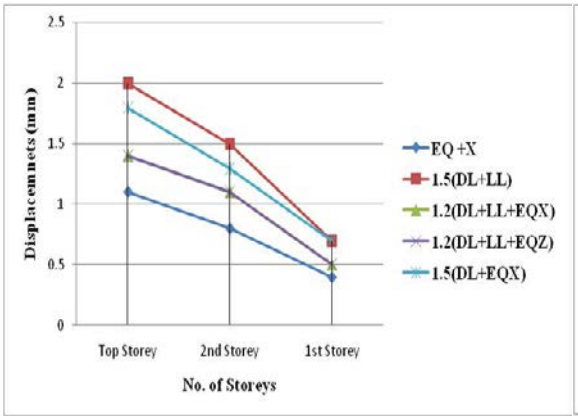


Fig. 8 Lateral displacements in Inverted-V braced frame

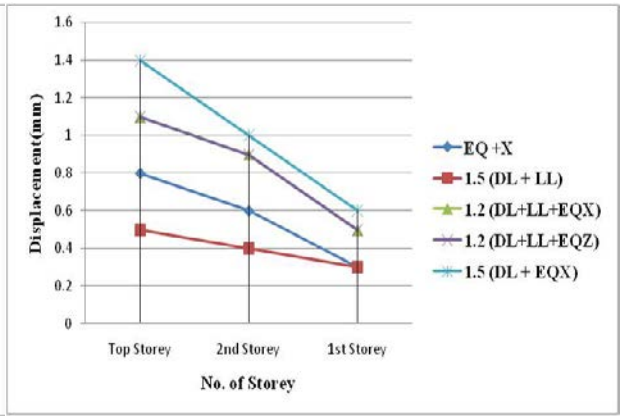


Fig. 9 Lateral displacement in X braced frame

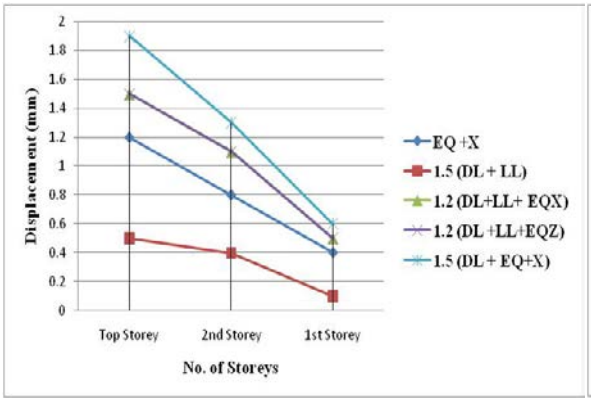


Fig. 10 Lateral Displacement of V Braced Frame

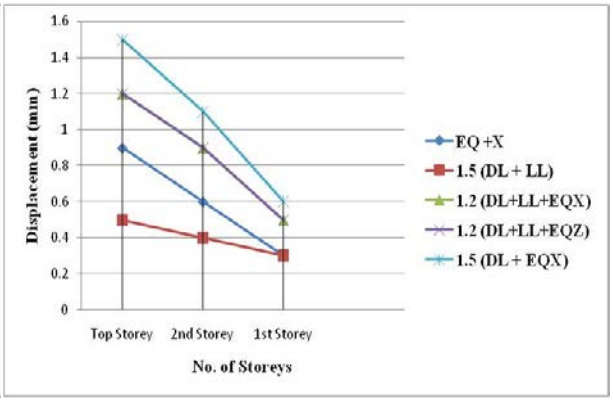


Fig.11 Lateral Displacement of K braced frame

Table 1. Lateral Displacement in Plane Frame

S.No.	L/C	Top Storey (mm)	2nd Storey (mm)	1st Storey (mm)
1	EQ +X	29.7	22.0	18.0
2	1.5 (DL +LL)	0.5	0.4	0.3
3	1.2(DL+LL+EQX)	35.7	26.5	22.4
4	1.2(DL+LL+EQZ)	27.0	19.5	11.8
5	1.5(DL + EQX)	44.6	33.1	26.7

Table 2. Lateral Displacement in Inverted-V Braced Frame

S.No.	L/C	Top Storey (mm) Node 36	2nd Storey (mm) Node 33	1st Storey (mm) Node 30
1	EQ +X	1.1	0.8	0.4
2	1.5(DL + LL)	2.0	1.5	0.7
3	1.2(D.L+L.L+EQX)	1.4	1.1	0.5
4	1.2(D.L+L.L+EQZ)	1.4	1.1	0.5
5	1.5(D.L + EQ X)	1.8	1.3	0.7

Table 3. Lateral Displacement in X Braced Frame

S.No.	L/C	Top Storey (mm) Node 36	2nd Storey (mm) Node 33	1st Storey (mm) Node 30
1	EQ +X	0.8	0.6	0.3
2	1.5 (DL + LL)	0.5	0.4	0.3
3	1.2 (DL+LL+EQ+X)	1.1	0.9	0.5
4	1.2 (DL+LL+EQ+Z)	1.1	0.9	0.5
5	1.5(DL+ EQ+X)	1.4	1.0	0.6

Table 4. Lateral Displacement in V Braced Frame

S.No.	L/C	Top Storey (mm) Node 36	2nd Storey (mm) Node 33	1st Storey (mm) Node 30
1	EQ +X	1.2	0.8	0.4
2	1.5 (DL + LL)	0.5	0.4	0.1
3	1.2 (DL+LL+ EQX)	1.5	1.1	0.5
4	1.2 (DL +LL+EQZ)	1.5	1.1	0.5
5	1.5 (DL + EQ+X)	1.9	1.3	0.6

Table 5. Lateral Displacements in K Braced Frame

S.No.	L/C	Top Storey (mm) Node 36	2nd Storey (mm) Node 33	1st Storey (mm) Node 30
1	EQ +X	0.9	0.6	0.3
2	1.5 (DL + LL)	0.5	0.4	0.3
3	1.2 (DL+LL+EQX)	1.2	0.9	0.5
4	1.2 (DL+LL+EQZ)	1.2	0.9	0.5
5	1.5 (DL + EQX)	1.5	1.1	0.6

The lateral displacements of RC frame without retrofit under different load combinations are stated in table 1. It is found that the top storey is subjected to maximum displacement of 44.6 mm at a load combination of 1.5(DL + EQ in X direction). Similarly, in the 2nd storey, the displacement is 33.1 mm and 26.7 mm in ground storey. Table 2 shows the lateral displacements of retrofit frame in which inverted-V bracing system is used. Similarly, in table 3, 4 and 5, the lateral displacements of X, V and K braced frame are shown.After evaluating the results obtained from STAAD PRO V8i software, it is found that the ground storey has been collapsedand caused serious damage to the structure.

After analysing the retrofitted frame, it is observed that the lateral displacement of the structure is substantially reduced. The structure is so rigid that it bears all the load combination successfully without failure. All the beams and columns are intact with no rupture at all. The maximum displacement of top storey is found out to be only 1.9 mm in V braced frame due to combination of loads. The steel bracing system (V, Inverted-V, X and K) which are used to retrofit the structure, it is found that X type of steel bracing significantly reduces the lateral displacements.

CONCLUSIONS

From this study, it is inferred that X bracing system gives the best suited results in terms of lateral stiffness.The lateral displacements of braced frame are well within the limits and are considerably lower as compared to plane frame.The load combinations are according to IS

1893:2002. In plane frame, the ground storey fails when subjected to seismic loads but the braced frame is so rigid that it can withstand all the seismic load combinations.

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SEISMIC ANALYSIS OF A MULTI-STOREY BUILDING WITH AND WITHOUTFLOATING COLUMNS CONSIDERING PLANE AND SLOPING GROUND

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ABSTRACT

This study highlights the importance of presence of floating columns in the analysis of a building. The study deals with G+4 storey normal building and G+4 storey building with floating columns in which location of floating columns is changed from first floor to second floor and further upper floors. A regular floor plan of 16 m × 16 m size is considered for the structure. This study also deals with ground condition i.e. location of building on plane and sloping ground. Staad Pro software is used for modelling and analysis of structural members. Total ten models are studied. The comparison of results of analysis is done in terms of time period, peak storey shear and storey drift considering seismic zone V.

KEYWORDS: Floating Column, Ground Condition, Plane and Sloping Ground.

INTRODUCTION

Buildings with floating columns are a typical feature in the modern multi storey construction practices in the whole world and also in urban India. If irregular features like floating columns are added in buildings, a considerably higher level of engineering efforts is needed in the structural design. Floating columns are sometimes provided to create column free space on some floors of building and sometimes for aesthetic and other functional requirements.

OBJECTIVES OF STUDY

1. To perform response spectrum seismic analysis of a building with and without Floating Columns located on plane and sloping ground.
2. To compare the performance of building on plane and sloping ground with and without floating columns.
3. To study the effect of location of floating columns on seismic performance of buildings resting on plane and sloping ground.

LITERATURE REVIEW

1.Bhavya B S and Jisha P (2016)[1]presented study about analysis of G+ 7 building andtotal twelve models are created . The recommendations such as the effect of Infill, Bracings and Shear wall are introduced in the building in order to improve the seismic performance.The structure is assumed to be situated in earthquake Zone III & V on a medium soil (type II).

2.IshaRohilla (2015)[2] discussed the critical position of floating column in vertically irregular buildings has been discussed for G+5 and G+7 RC buildings for zone II and zone V.

The response of building such as storey drift, storey displacement and storey shear has been used to evaluate the results obtained using ETABS software.

3. S.B. Waykule (2016)[3] presented study about analysis of G+5 Building with and without floating column in highly seismic zone v. Two models are created 1st having floating columns at 1st floor and without floating column building. Equivalent static analysis and time history analysis are carried out on all models.

RESEARCH METHODOLOGY

BUILDING DESCRIPTION

Model consists of G+4 storey RCC building having four bays in each direction with width of bay as 4m. The story height for 1st storey to 4th storey is 3.2m and ground storey height is kept as 3m respectively. The RCC frame consists of beam of sizes 0.23m × 0.4m and column of sizes 0.4m x 0.23m for 1st,2nd,3rd,4th storey and 0.4m × 0.4m for ground storey. Slab thickness is taken as 150mm. The models are analysed on plane as well as sloping ground. The concrete of grade M20 and steel of grade Fe 415 are used.

Table 1 Building Description

S. No.	Name	Description
1	Plan dimension	16 x 16 m
2	Storey height	Ground Floor-3.0m, 1st to 4th Floor-3.2m
3	No. of storeys	G+4
4	No. of bays	4 ×4
5	Soil type	Medium Soil
6	Seismic zone	V

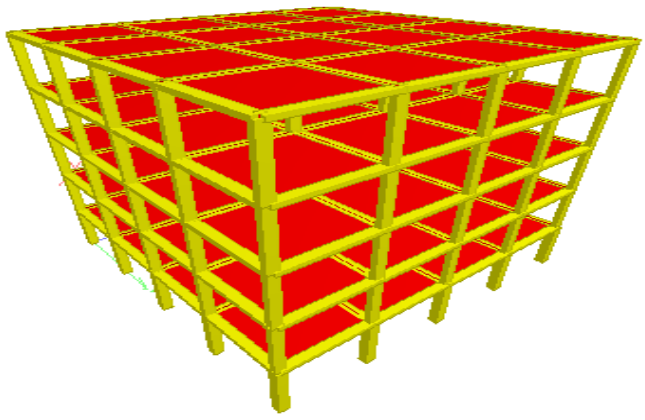
LOADS

Dead load of each floor include floor load, floor finish load and brick infill load as per relevant IS codes and Live load on floors =4 KN/m^2 (as per IS 875 part1 and part2)[4,5].

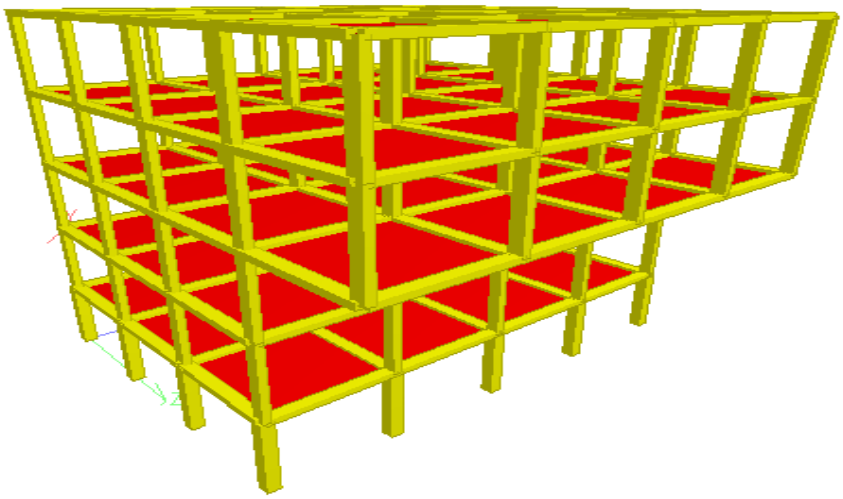
MODELLING AND ANALYSIS

Total 10 models are made with different location of Floating Columns. Five models are considered on plane ground and five other models are considered on sloping ground. The various models are analysed by using linear static analysis

Model 1- Structure without floating column on plane ground
Model 2- Structure without floating column on sloping ground
Model 3- Structure with floating column on 1stfloor on sloping ground
Model 4- Structure with floating column on 2ndfloor on sloping ground
Model 5- Structure with floating column on 3rd floor on sloping ground
Model 6- Structure with floating column on 4th floor on sloping ground
Model 7- Structure with floating column on 1st floor on plane ground
Model 8- Structure with floating column on 2nd floor on plane ground
Model 9- Structure with floating column on 3rd floor on plane ground
Model 10- Structure with floating column on 4th floor on plane ground



Model 1- Structure without floating column on plane ground



Model 9- Structure with floating column on 3rd floor on plane ground

RESULTS AND DISCUSSION

Comparison of seismic response parameters such as time period, base shear and storey drift has been done for various models.

Time Period

The time period of the structure for particular mode shape is the time required to complete the oscillation for corresponding mode shape.

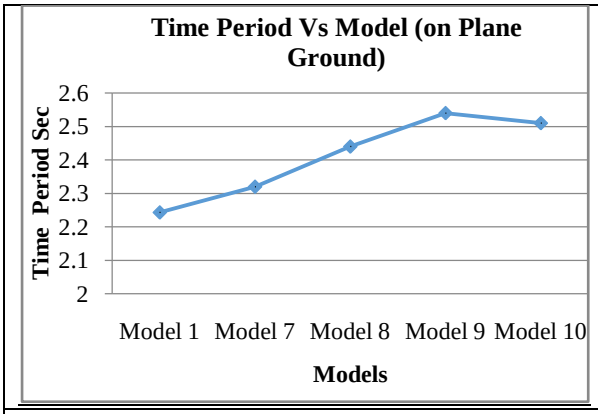


Figure 3 Variation of time period for various Models on plane ground

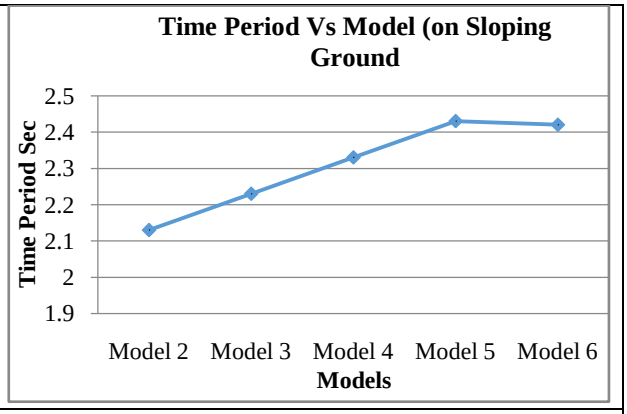


Figure 4 Variation of time period for various Models on sloping ground

After analysing various models, we can summarise the results as-

1. When the building with and without Floating Columns is located on plane ground then minimum magnitude of time period comes in building without Floating Columns (model 1) and maximum value of time period comes in the building with Floating Column on 3rd floor (model 9).
2. When the building is located on slopping ground then minimum magnitude of time period comes in building without Floating Columns (model 2) and maximum value of time period comes in building with Floating Columns on 3rd floor (model5).

Peak Storey Shear

Peak storey shear is the estimation of maximum expected lateral forces at different storeys that will occur due to seismic ground motion.

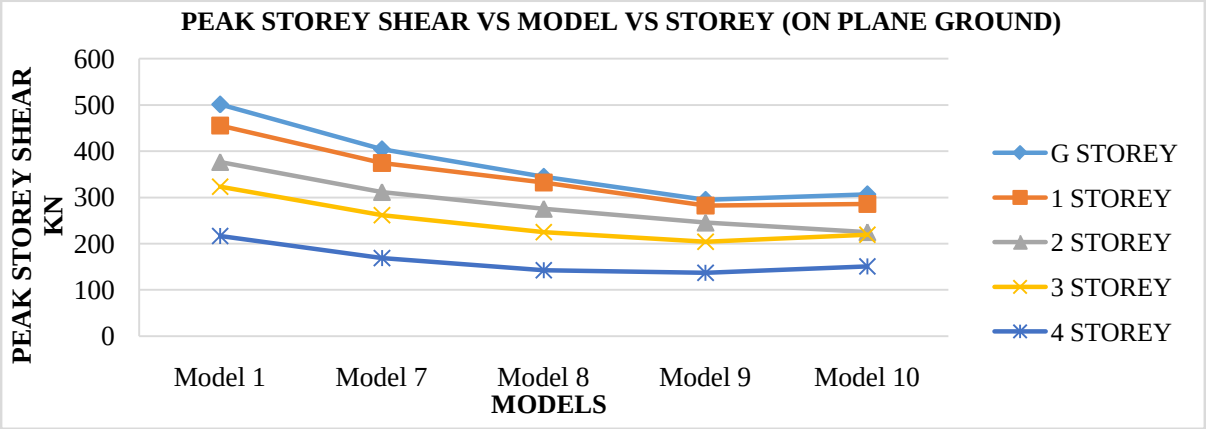


Figure 5 Variation of peak storey shear for various models on plane ground

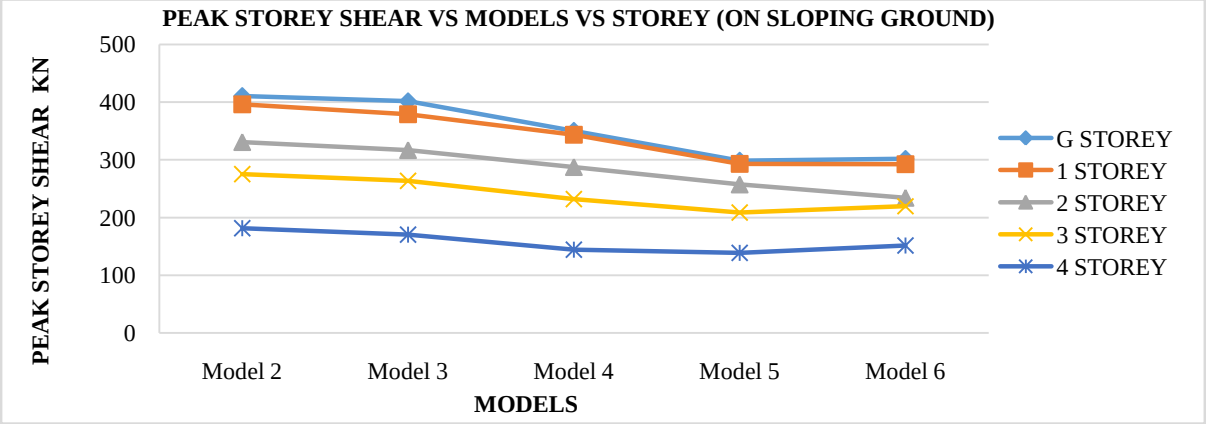


Figure 6 Variation of peak storey shear for various models on sloping ground

After analysing various models, we can summarise the results as-

1. When the building with and without Floating Columns is located on plane ground then maximum magnitude of peak storey shear comes in building without Floating Columns (model 1) and minimum value of peak storey shear comes in building with Floating Column on 3rd floor (model 9).
2. When the building is located on slopping ground then maximum magnitude of peak storey shear comes in building without Floating Columns (model 2) and minimum value of peak storey shear comes in building with Floating Columns on 3rd floor (model5).

Storey drift: -Storey drift is the relative displacement of the adjacent floors and calculated as the difference of deflections of the floors at the top and bottom of the storey under consideration.

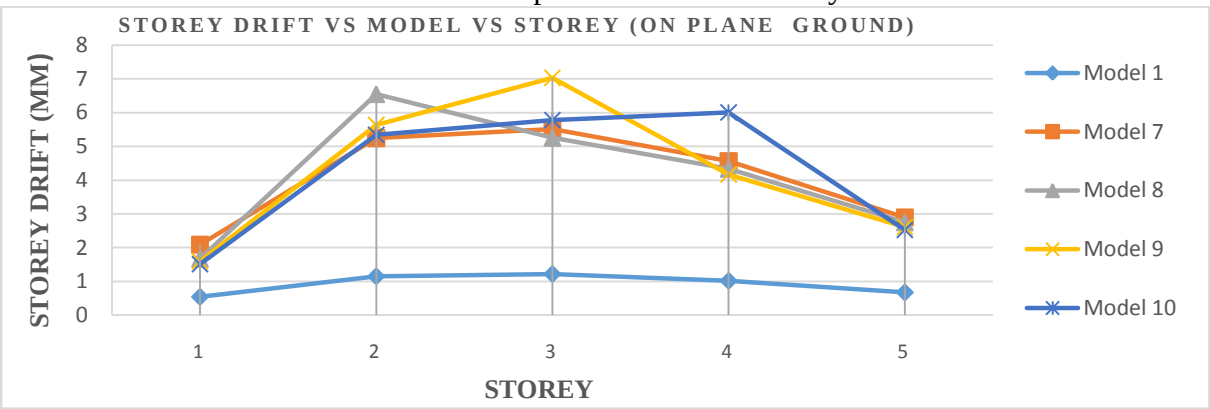


Figure 7 Variation of storey drift for various models on plane ground

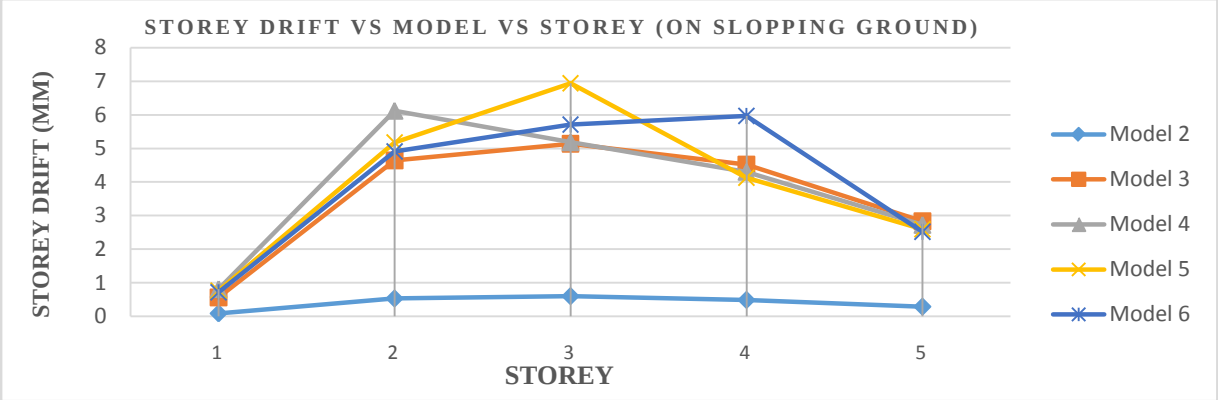


Figure 8 Variation of storey drift for various models on slopping ground

After analysing various models, we can summarise the results as-
1. When building with and without Floating Columns is located on plane ground then minimum value of storey drift comes in building without Floating Columns (model 1) and maximum value of storey drift comes in building with Floating Column on 3rd storey (model 9). Storey drift is maximum on 3rd storey as compared to other storeys in model 9.
2. When the building is located on sloping ground then minimum magnitude of storey drift comes in building without Floating Columns (model 2) and maximum value of storey drift comes in building with Floating Columns on 3rd floor (model5). Storey drift is maximum on 3rd storey as compared to other storeys in model5.

GROUND CONDITION

After analysing various models, we can summarise the results as for building on sloping ground (slope of 10^0 in this study), not much variation is observed in various parameters.

CONCLUSIONS

The following conclusions are drawn from this study -

1. It is observed that building with Floating column has more time period as compared to building without Floating column. It is also observed that when the location of floating column is changed from first floor to second floor and further upper floors the value of time period goes on increasing. Although there is slight decrease in value of time period when the floating column lies in the top storey. This shows less stability and resistance against earthquake of a building with floating columns.
2. It is observed that building with Floating column has less peak storey shear as compared to building without Floating column. It is also observed that when the location of floating column is changed from first floor to second floor and further upper floors the value of peak storey shear of each storey goes on decreasing. Although there is slight increase in value of peak storey shear when the floating column lies in the top storey. Thus, it shows less resistance against earthquake force at base and other storey levels for a building with floating columns.
3. In case of building with Floating column the value of storey drift is more as compared to building without Floating column. It is also observed that when the location of floating column is changed from first floor to second floor and further upper floors the value of storey drift goes on increasing. Although there is slight decrease in value of storey drift when the floating column lies in the top storey. It can also be concluded from the study that value of storey drift is maximum for the storey where floating columns lies i.e. when the floating column lies on second storey the storey drift of second storey is to be maximum but there is one exception in case of first storey.

4. For building on sloping ground (slope of 10^0 in this study), not much variation in various parameters is observed.

Finally, we can conclude that floating columns should be avoided as far as possible in earthquake prone regions and if they are unavoidable, then the structure should be strengthened by providing bigger size of beams and columns. It is not always easy to provide heavy beams and columns everywhere then adopt some other remedial features such as bracings, shear wall etc. along with proper structural design following the conditions of IS 1893-part-1: 2002, IS-456: 2000 and other relevant IS codes.

FUTURE SCOPE

1. Higher storey buildings can be studied after applying irregular structures like floating column.
2. Same study can be done for ground having slope more than 10^0.
3. Steel building can also be studied with floating columns.

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DEVELOPMENT OF LOW ENERGY ECO-FRIENDLY NAAC BLOCKS WITH ITS PHYSICAL PROPERTIES ANALYSIS

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ABSTRACT

Pre-Cast construction blocks is nowadays used in most construction industries, they are used in masonry work, partition, foundation, column, furnaces, road, footpath and flooring. Where the blocks (bricks) are used in masonry work. Bricks or Precast blocks used since 8300 BC and fire clay bricks dominate the construction from 12th century. At present there are several blocks and bricks available in market. In modern era many scientific innovation and technical studies are being performed to develop a low cost, environment friendly, energy efficient, light weight, workable, and fast working pre cast block. This paper compiled such pre cast construction blocks which is known as **Non-Autoclaving aerated concrete block** which is a modified version of AAC blocks by eliminating Autoclaving process to conserve the energy and fuel requirement to attain the high temperature (140-180)˚C and high Pressure (1.3 to 1.4) MPa for 120-180-minute duration. This work uses sun dry method with wet gunny bag curing method to achieve the required strength and glass powder and steel fiber is also used to enhance the properties of the NAAC blocks. Basic material used to develop these are Fly-Ash, OPC, Lime, Gypsum, Aluminium Powder, Steel Fiber, and Glass Powder. To understand the properties of these blocks some test (Bulk Density, Compressive Strength, water Absorption and IRA test) were perform.

KEY WORDS: Waste Management, environmental concern, low energy material, Autoclaving, embodied energy and carbon equivalent.

INTRODUCTION

Precast Blocks is principally used as a building material is the construction of walls and partition but now these precast blocks are also used in Roads, Footpath, Roof, Flooring, Column, Foundation, Furnaces and in Elevations also. The term precast blocks (construction material) associates that, these blocks formed and hardened before they are brought to the working site. These precast blocks used as structural material since 8300 BC. In present time several precast blocks are available in the market for example Abode Blocks, Fire Clay Bricks, Porothem Hollow Bricks, Clay Burnt Bricks, Calcium Silicate Blocks, Concrete Blocks, Fly-Ash Bricks, CLC Blocks and AAC (Aerocon) Blocks. All these building material haves their own advantages and disadvantages based on their physical, chemical properties and environmental impact. In current era where properties like Strength, Durability, Serviceability, Cost and some other factors but their environment impact on land, water and air and its raw material also affects their use and availability in market. The greenhouse gases emission contributed by the construction sector in India is about 22% and there is an ever-increasing demand for building material. As per Indian building construction practices so that requirement of eco friendly and low carbon emission construction materials are required and to understand the environmental impact of material and construction their embodied energy and embodied carbon equivalent analysis is required where

EMBODIED ENERGY-Embodied energy is used to express the energy requirement for the manufacturing of any building, construction work, industrial produced and construction

material. This energy includes all kinds of energy required for processes like construction, processing, production, transportation, maintenance, renovation, and demolition.

EMBODIED CARBON EQUIVALENT- the embodied carbon equivalent is the total amount of carbon di oxide emitted during the above processes which is expressed in kg CO2 per Kg of material

There is no defined methods and tools to find and calculate the but there some basic calculative methods to calculate the embodied energy of any material, product, and building. These methods are following

Process Analysis – Embodied energy analysis by the Process Analysis involves the energy data from the manufacturing factory. The embodied energy is calculated by the using of this the with amount of material used.

Input-Output analysis-In input/output analysis processes the direct and indirect energy inputs in the process of production of building material. Because of that, this method is considered relatively complete.

Hybrid analysis- This method combines the elements of both the input-output analysis and process analysis to achieve a more accurate value of embodied energy.

Table 1 Embodied E. and Embodied. C. E. of material (IFC World Bank Group)

S. No	Material	Embodied Energy (MJ/kg)	Embodied Carbon Emission (kg CO_2/kg)
1	Cement	6.70	0.196
2	Fly ash	9.3×10^{-3}	4×10^{-3}
3	Active Lime	4.5	0.13
4	Aluminium Powder	155	2.1
5	Steel Fiber	20.1	0.482
6	Gypsum	1.80	0.0025
7	Waste Glass Powder	0.012	7.8×10^{-3}
8	Water	9.2×10^{-3}	3.5×10^{-3}

NAAC BLOCKS

Non Autoclaved aerated blocks are just similar to the AAC blocks but the main different between these blocks are in NAAC Blocks the Autoclaving process is eliminated and in place of Autoclaving some other method of cuing which required less energy is used which help to reduce the energy requirement as well as carbon emission done by the AAC blocks. For this research curing is done by wet gummy bag for maintaining the moisture and also at day time blocks are left open in front of sun to provide ambient temperature which help in the crystalline process into the blocks. This method required at least 28 days curing for blocks to achieve the significant strength and blocks were gaining their strength continuously after the 28 days also.

LITERATURE REVIEW

[1] A.Y. Melnky et al. (2013), investigated Non-Autoclaved Aerated Concrete properties made by salt industrial waste, The researcher was used 10% of carbonated waste and sulphate waste.In addition, 0.1 kg of polypropylene fibres was also added with same Carbonate and sulfate-containing waste and then tested for their properties and compares the behaviour of all samples. Results show maximum strength is getting by the mixing of carbonated waste with PP fiber which is 2.8 MPa as compared to sulphate waste samples.

[2] Rana Shabbar et al. (2017) presented an investigation on the influence of the aluminium powder at different percentages and tested for the strength, modulus of elasticity, density, and

porosity. Researcher mixed the Al powder in the variation of 0.25% start with 0% to 1% of Al powder content and tested for their physical properties. The test results show the lowest compressive strength was achieved with 1% of Al powder. Results shows that with the increment of al powder density of sample reduce and porosity increase.

[3] Abdul Ghaffar et, al (2014), researcher investigation of the behavior of steel fiber in structural concrete. For the investigation, the compressive and flexural test on eleven mixed batches of concrete containing 0% to 5% "Hooked" steel fiber with an interval of 0.5% by weight of cement.The maximum percentages increase in compressive strength and flexural strength achieved were 6.15% and 15.86 at 3% and 4% fiber content respectively.

[4] Shweta Patil et, al (2014), investigate the properties of steel fiber reinforced concrete like compressive strength and flexural tensile strength. Author used 4 different aspect ratio 40, 50, 60 and 70 and steel fiber content 0.5% to 2.5% with 0.5% interval. After analyzed the results it was obtain that binding wire in concrete significantly increases the flexural strength, at constant percentage 1.5% of steel fiber content with increasing aspect ratio of fiber from 40 to 70.

[5] Bharat Nagar et al. (2016), investigate the behaviour of waste glass powder as replaced with fine aggregate in various percentages such as 5%, to 50% in concrete. it was observed that maximum compressive strength is achieved at 20% glass powder content.Test results show that for the flexural and split tensile strength 25% glass powder as a sand replacement gives optimum results with 6% and 5.5% increments into the strength respectively.

[6] S. Lalitha et al. (2017) presented an investigation on waste glass powder used in concrete in the content of 5, 10 and 15%. To compare the experimented blocks, plain concrete of grade M25 is also prepared which 28 days compressive strength is 27 N/mm2, and split strengthis 2.47 N/mm2, and samples with glass powder in 5%, 10%, and 15% is made and their compressive strength increase by 51.7 %, 37%,and20.7% individually. Similarly split tensile strength increasedby 3.23%, 7.28%, and3.64%.

METHODOLOGY

NAAC BLOCKS – Non-Autoclaved Aerated Blocks precast construction blocks which are light weight, excellent thermal and sound insulating eco- friendly construction material which can be used as wall material in frame type structure. These are modification of AAC Aerocon blocks in which by the eliminating of autoclaving process blocks were made. For the manufacturing the NAAC blocks raw material used is similar to the AAC blocks but by the studies of previous research paper it was observed that strength achieved by blocks facing problem that why for this research work waste glass powder is used to enhance the properties and steel fiber also used which as a reinforced material both material helps to achieving the sufficient strength. By the eliminating of autoclaving process huge amount of energy requirement is reduce by which low cost, low embodied energy and low embodied carbon equivalent NAAC blocks was made. Material used for the manufacturing the NAAC blocks are mentioned below

FLY ASH - The fly ash collected from the local industry is class C fly ash which contains nearly 70% of Silica Oxide with Aluminium Oxide and Iron Oxide.

ACTIVE LIME - For research work,high-quality Quick lime is used which contending more than 75% of CaO+ MgO.

OPC- OPC of grade 43 was used for the manufacturing the NAAC (Non-Autoclaved Aerated) blocks samples.

GYPSUM-Gypsum is used for this research work to control the setting time and allow the past 2 to 4 hours for complete the chemical reaction to generate the sufficient amount of hydrogen gas.

ALUMINIUM POWDER–Al powder is used as air entrained which react with calcium and produce hydrogen gas.

$2Al + 3Ca(OH)_2 + H_2O$ — $3CaO.AL_2O_3 + 3H_2$

Aluminum powder + Hydrated lime — Tricalcium aluminates + Hydrogen

The high-quality Aluminum powder used for this research work which has the following properties comes into the 10g pack with poly cover packing.

STEEL FIBER - For this research micro crimped steel fiber from the Kasturi Company is used having round shaped with 10 mm long, 50 aspect ratio and 485 Mpa minimum Tensile Strength.

WASTE GLASS POWDER - Waste Glass Powder is a great pozzolanic material which is generated by cutting and polishing of glass. For this research work glass powder collected from glass cutting and polishing factory.

MIXED PROPORTIONING

In my research work, all raw material mixed in predefine quantity according to the standard AAC blocks manufacturing company with the addition of glass powder and steel fibers in a different percentage variation. Glass Powder use in 10, 20 and 30 % of fly ash and steel fiber used 1,2 and 3% part of cement.

Table 2 Raw Material Percentage Specification for NAAC Block

Material	**Fly Ash**	**Lime**	**OPC**	**Gypsum**	**Al Powder**
Quantity (%)	50%	30%	15%	4.9%	0.1 %

Table 3 Percentage Variation of G. P. and S. F. in Different Sample

Sample	**S1**	**S2**	**S3**	**S4**	**S5**	**S6**	**S7**	**S8**	**S9**
Glass Powder	10%	20%	30%	10%	20%	30%	10%	20%	30%
Steel Fiber	1%	2%	3%	1%	2%	3%	1%	2%	3%

For the manufacturing the NAAC blocks with above material all raw material mention above is weight batch and then mixed with water and AL powder is mixed at last and the mixed again for 1 minute to make homogenous mixture then it places into the mould. The paste is filed 2/3 of the mould and 1/3 mould is left for the expanding which is done due to generation of hydrogen gas. This gas trap into the paste and made spongy structure and after 24 hours sample is demoulded and place exposed into the sun for ambient temperature and for maintaining moisture wet gunny bags also used. 28 days proper curing is required for these blocks to achieve strength but these blocks gain strength but after 28 days blocks are ready to used.

TESTS FOR MECHANICAL PROPERTIES

UNIT WEIGHT AND BULK DENSITY (IS-6441 PART 1)

IS 6441 part 1 is adopted first in 1972 and then last reaffirmed is done is 2001. This IS code is used to determining the Unit weight and bulk density of autoclaved cellular concrete product, determined by the calculating ratio between the weight of the specimen and volume of the specimen.

DETERMINATION OF COMPRESSIVE STRENGTH (IS-6441 PART 5)

IS-6441 Part 5 covered the method for determining the compressive strength of Autoclaved cellular concrete. Test specimen can be used for determining the compressive strength is 15 cm edge length cube or other cuboid sample with minimum 7.5mm large edge. The specimens shall be loaded at the rate of 0.5 Kgf/cm^2 to 2 Kgf/cm^2 in such a way that failure occurs within 30 seconds.

WATER ABSORPTION TEST (IS- 2185)

Water Absorption test is the measurement of the amount of water brick or blocks absorbs when placed in water (fully immersed) for 24 hours. For the common practice for the concrete block hollow (Open and close Cavity) IS-2185 (Clauses 8.5 & 9.1).
The amount of water absorbed (%) was calculated by

$\frac{A - B}{B} \times 100$ = _ _ _ _ _**Eq- 2** Where; A = Wet mass of the units&B = Dry mass of a unit

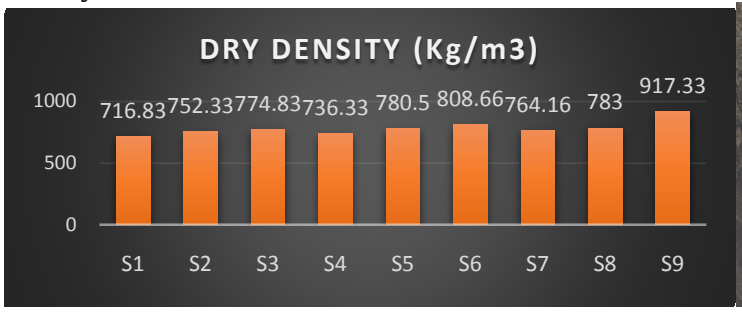
EMBODIED ENERGY EQUIVALENT& EMBODIED CARBON EMISSION EQUIVALENT CALCULATION

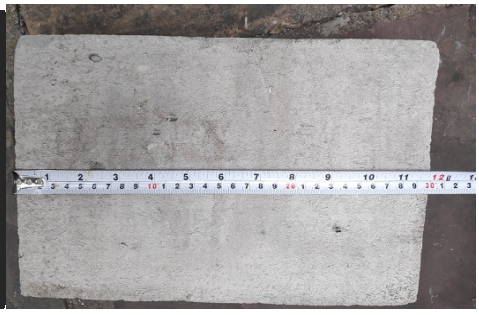
Embodied energy = Quantity of the material × Embodied energy coefficient
And
Carbon Emission Equivalent = Quantity of the material × Embodied Carbon Equivalent

RESULTS AND DISCUSSION

UNIT WEIGHT AND BULK DENSITY (IS-6441 PART 1)

For the research purpose, 9 different sampleswere made as per different design composition.All blocks were cast into 300mm×200mm×100mm and measured for the density.



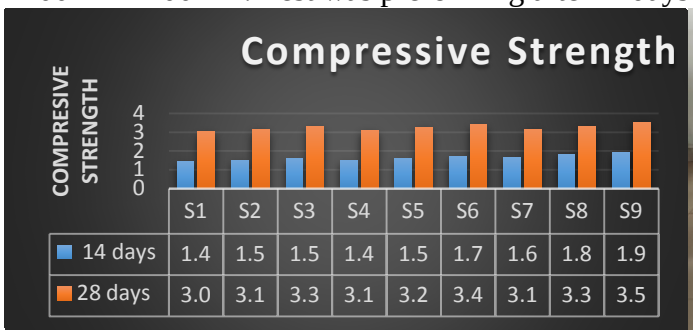


Graph No.1 Bulk Density of Test Samples Image No. 1 Measurement of Test Sample

Results show that with the increasing amount of steel fiber and waste glass powder density and specific gravity also increase.

COMPRESSIVE STRENGTH TEST RESULTS OF NAAC BLOCKS (IS-6441 PART 5)

This test is preforming as per IS-6441 Part 5 specification, For the compressive strength test 9 samples were prepared and for each sample have 3 blocks which are in size of 200mm ×100mm ×100mm. Test was preforming after14 days and 28 days curing.



	S1	S2	S3	S4	S5	S6	S7	S8	S9
14 days	1.4	1.5	1.5	1.4	1.5	1.7	1.6	1.8	1.9
28 days	3.0	3.1	3.3	3.1	3.2	3.4	3.1	3.3	3.5



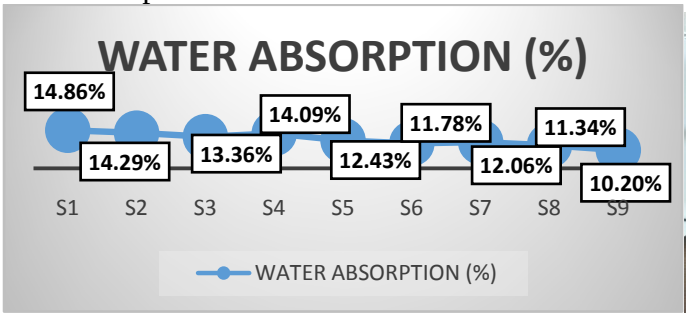
Graph No. 2 Compressive Strengthof SamplesImage NO.2 Compressive Strength. Testing

The minimum strength achieved by the sample S1 1.416 MPa for 14 days and 3.016 MPa for 28 days and the maximum strength achieved by S9 is 1.933MPa and 3.53 MPa

respectively.Results show glass powder and steel fiber helps to improved strength of the blocks.

WATER ABSORPTION AND DRY DENSITY TEST (IS- 3495 (PART-2))

This test is conducted on the specification as per IS- 3495(Part-2). For this research work, 9 different samples of NAAC blocks are used with 300mm × 200mm × 100mm size and the results are presented and discussed below



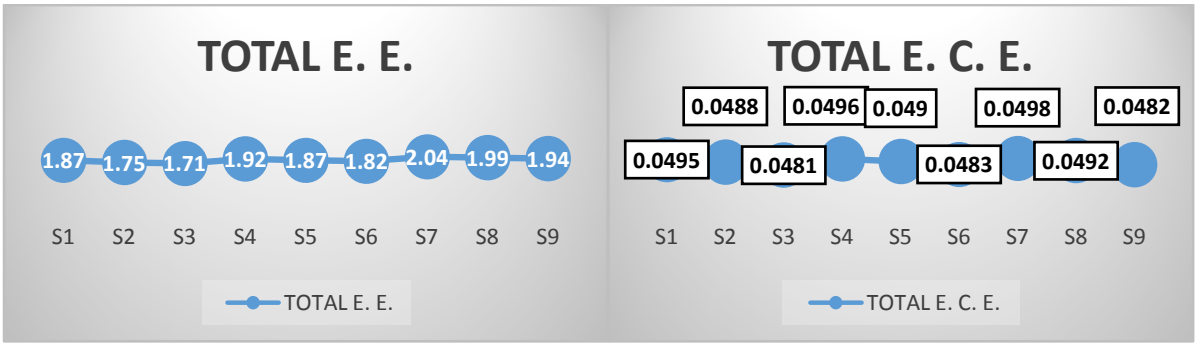


Graph No.3WaterAbsorptionTestResults ofSamples ImageNo. 3. Oven Drying of Blocks

The test results show that with the increasing the waste glass powder content in NAAC Blocks the water absorption of the blocks reduces. The water absorption of sample S1 is 14.88% is the highest value in all sample and sample S9 have the lowest value 10.21% of water absorption.

EMBODIED ENERGY OF NAAC BLOCKS

Embodied energy is one of the main factors to understand the material characteristics related to their impact on the environment. For this research work Embodied energy is calculated according to the Hybrid Input-output analysis. Direct embodied energy is calculated by the material embodied energy and the indirect embodied energy involves transportation, operational human and power consumption energy. Embodied carbon Equivalent is calculated similar to the Embodied Energy calculation their results shows in table below.



Graph No. 4 Total Embodied Energy and Embodied Carbon Equivalent of NAAC Samples

CONCLUSIONS

This research work is concentrated on the AAC autoclaved Aerated Concrete Blocks. These blocks are known as low weight, eco-friendly green building material but when it was properly observed the total embodied energy equivalent of AAC blocks was 3.5-3.6 and embodied Carbon emission was 0.076-0.102 which is higher than the conventional clay bricks.So that in this research work Aerated blocks were manufactured without Autoclaving.After performing the tested it was observed that the use of steel fiber and waste glass powder improved the physical properties of NAAC blocks. As results shows that by the

increasing the amount of steel fiber and waste glass powder the density of blocks increase by 28 % compressive strength increase by 17% and the IRA value reduce by 45% and water absorption reduce by 19%. In term of environment concern the embodied energy equivalent of blocks lies in the range of 1.71 to 1.94 which is much lower than the AAC blocks and their percentage variation is 19%. Similarly, the Embodied Carbon Equivalent lie in the range of 0.048 to 0.050 and their percentage variation is 3.6%. Results shows that these blocks are eco-friendlier and more fulfilled the all physical requirements of construction blocks.

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BEHAVIOR OF RC BUILDINGS WITH SPECIALLY SHAPED COLUMNS UNDER SEISMIC LOADS – AN OVERVIEW

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ABSTRACT

With the new preferred concept by architects of avoiding column's offsets in structure, the interior of structures are prepared. Structural engineers are also required to design the structure from architect point of views. For that, many structural engineers are using specially shaped columns as they are much better in placement and helpful in achieving objectives along with strength, stiffness & behavior of structures.In all structures, columns are most important component as they are acting as path for load transferring to foundation. Also, deflection of structures, stiffness of buildings, drift capacity of buildings, moment of inertia of structures, overturning moment etc. are directly or indirectly dependent on columns position, columns cross-section, and sizes. Any alteration in such parameters of columns may change the behavior of structures and its usefulness in any high rise structures subjected to static as well as dynamic loading.

Many papers studied, includes the comparative study of effects of rectangular columns & specially shaped columns, effects of building configuration & Bracing and shear wall system on behavior of symmetric plan buildings with different height subjected to seismic loads. This paper presents a review of previous work done on symmetric RC buildings consist specially shaped columns, bracings & shear walls system subjected to seismic loads and the study of effects of specially shaped columns, bracings & shear wall system on various parameters influenced by seismic loads.

KEYWORDS: Specially Shaped Columns, Seismic Loads, RC Buildings

INTRODUCTION

There are a lot of construction of high rise structures with symmetrical and asymmetrical shape and irregularities in plan. These high rise structures are subjected to various loads (such as static and dynamic) along with their combination. The loads when acting on the structure along with the combination of lateral loads sometime adversely affect the structure, their behavior and performance.

In all structures, columns are most important component as they are acting as path for load transferring to foundation. Also, deflection of structures, stiffness of buildings, drift capacity of buildings, moment of inertia of structures, overturning moment etc. are directly or indirectly dependent on columns position, columns cross-section, size etc.

The specially shaped columns are most suitable for increasing seismic resistance & stiffness of structures. The most prefer shapes are L-shaped, Plus shaped & T-shaped cross-section columns.



Fig.1 (a)



Fig.1 (b)



Fig.1 (c)

Any alteration in such parameters of columns may change the behavior of structures and its usefulness in any high rise structures subjected to static as well as dynamic loading.

LITERATURE REVIEW

Comparative study of SMRF structures with different height & analysis methods

The analysis of structures under seismic loads are most important when they are constructed in zone III, IV & V as per Indian Standard Code. A lot of work is carried out to study the behavior of structures of different heights.Sallal Ali Kadhim (2018)simply analyzed the ten storey RC building under effect of wind and seismic forces. The work carried out in ETABS software[1].Ismail Najif (2008)performed analysis (using Staad Pro V8i software) of 5 storey & 10 storey regular structure of both MRF & SMRF with an objective of study of structural behavior of special moment resisting frame & which is the best structural system between them. The effect of response modification factor on structural system are also studied [2].

The availability of various analysis methods helps to understand the actual performance of structures subjected to seismic loads. The effects on seismic influenced parameters can also be predict to know lateral forces resistance requirement of structures.Nair and Akshara (2017)done study of seismic analysis method of structures. They carried out the detailed study of all four types of analysis method that are linear static analysis (equivalent static analysis), linear dynamic analysis, nonlinear static analysis (pushover analysis) & nonlinear dynamic analysis [3]. Similarly, VelipGuruvendra (2018)performed seismic analysis over RCC building which is G+6 structure having 20m x 25m plan area and building is considered as office building in Zone V (Bhuj, magnitude of 7) using time history method in Staad Pro V8i software[4].

Likewise, Kumar H. (2019) deals with the study to compare seismic performance of RCC and Composite building. For study G+7 RCC building and Composite frame building are considered for analysis in zone V and having 16.5m x 16.5m plan area. The equivalent static method and response spectrum method are used for the analysis. Whole work is done in SAP2000 software [5].

Study the effects of Specially Shaped Columns on performance of SMRF structures with different height & analysis methods.

The columns play an important role in seismic behavior of structures, as they are path of loads transfer to base. The use of specially shaped columns helps in increasing the lateral resistance of structures. In construction of seismic resisting structures, specially shaped columns are mostly prefer as they provide resistance and strength in both directions. Yang et al. (2008) designed and analyzed some frame structure with rectangular columns and specially shaped columns as per Chinese code and technical specification. They carried out comparative studies and the outputs by nonlinear dynamic analysis of proposed structures [6]. Rahaman et.al. (2018), Gumble and Pagade (2015), Abhilash and Gowda (2016)

&Pajgade and Agrawal (2016) deals with the study and assessing the response of specially shaped columns structure subjected to seismic loads consider in various seismic Zone using different analysis software. The comparative study are also carried out in order to determine the difference in behavior of structures with conventional columns & specially shaped columns [7] [8] [9] [10]. Tang and Wu (2014) carried out the study of seismic performance of irregular RC frame in elevation (Refer Fig.2) with specially shaped columns. They performed nonlinear seismic response analysis in SAP2000 software by inputting ground motion in two direction. They conduct response spectrum analysis and elasto dynamic time history analysis for structure for checking whether chosen seismic wave satisfy requirement of criteria. Also they examined seismic performance of the structure under rare seismic action and given a primary evaluation on the structures [11].

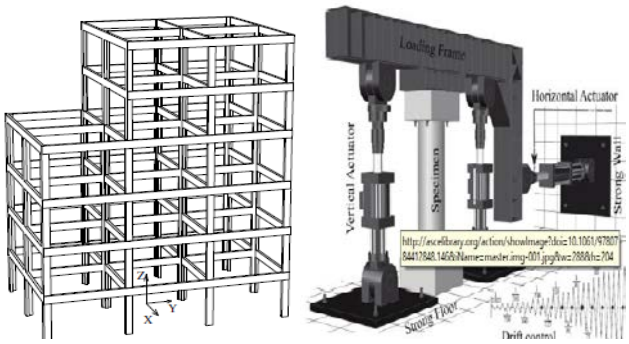


Fig.2Irregular RC frame in elevationFig.3 Test Setup

The experiment work were also done related to specially shaped columns such as Li & Pham (2013) presents an experimental investigation carried out on ten reinforced concrete columns with L shaped section, which were subjected to simulated seismic loading using loading apparatus (Refer Fig.3). This paper deals with the study to investigate the suitability of using L shaped RC column in regions of moderate to high seismic risk. They takes ten half scaled RC columns which were tested under the combination of a constant axial load and double curvature bending simulated seismic load. The Effects of Specially Shaped Columns on Asymmetric RC Building with Shear wall Subjected to Seismic Loading result so obtained were examined in terms of cracking pattern, hysteretic response, initial stiffness, shear strength & drift capacity. This experiment provides information for both structure design and assessments[12].

Likewise, Konapure&Muddiddi (2018)carried out analysis for the most influential structural parameter by studying the seismic performance of multistory building. They evaluated dynamic property such as time period of multistory building analytically & experimentally. They used ETABS software for the analysis. The G+5 building is modeled and analyzed. Similar steel building scale model was also prepared to carryout test on shake table. After whole analysis work and studies, various parameters were compared such as maximum bending moment, maximum shear force, longitudinal reinforcement, shear reinforcement of beams & columns [13].

Study the effects of building configurations & alteration of seismically effective parameters on performance of SMRF structures with different height & analysis methods

The building configuration are one of most important aspect which play an important role in the seismic performance of the structures. Related to such concept, Alashker et al. (2015)

deals with the study of building configuration effects on seismic performance of RC building have same area and same height. They analyzed four different plan Effects of Specially Shaped Columns on Asymmetric RC Building with Shear wall Subjected to Seismic Loading model having same area of 400m^2 (i.e. 20m×20m, 25m×16m, 28.5m×14m, 40m×10m) with height of 15.2m each. Nonlinear pushover analysis has been used to evaluate the seismic performance. As a result, they compare base shear & top displacement & performance point & yielding pattern of the structures [14].

Some study were also carried out in reference of seismically effective parameters such as fundamental time period such as Bhuskade&Sagane (2017) performed analysis of building to study the variations of natural time period according to various parameters of buildings. They analyzed ten building in which five storey bare frame building with plinth beam at ground floor level is considered as benchmark building or chosen as a basis. With the alteration of parameters in different type of buildings, the various effects are considered due to stiffness, mass, height and column orientation to study natural period. To study the effects, the whole work is done in ETABS Software. In this case study the result they studied are taken from book "Earthquake Behavior of Building" of C.V.R. Murthy [15].

Likewise, Joseph &Joshma (2017) studied the influence of different types of infill on fundamental time period of RC structure. The different infill used for analysis work were solid concrete blocks, hollow open cavity concrete block, aerated concrete blocks, common burnt clay brick & performed foam cellular concrete block infill masonry [16].

Study of Seismic Base Isolation and Energy Dissipation Concept.

In seismic base isolation, the concept of decoupling of super structure and sub structure is used to reduce transfer of seismic energy to super structure. The entire super structure acts as single unit and hence the inter storey drift becomes negligible. Adithya G.S. et al. (2016)carried out the study to give the overview of the nonconventional system which includes the concept of seismic base isolation, types advantages and limitations of base isolation system, the working principles, advantages and limitations of different passive energy dissipation devices and the basic working principles of active, hybrid & semi active control systems. This paper concluded that nonconventional techniques are more reliable and effective for earthquake resistance design techniques as compared to conventional methods[17].

Similarly, in energy dissipation system, a major portion of input seismic energy is dissipated by using shear walls, bracing system and other mechanical devices. Mehta and Rana (2017)studied the behavior of the RCC building with different position of shear wall. They used four different model of RCC building one without shear wall and other with different position of shear wall. The Staad Pro V8i software is used for whole analysis. The parameter like storey drift, lateral displacement, and base shear are studied and suitable location of shear wall are determined among these models [18].

Likewise, B.S. Naveen Kumar et al. (2015)carried out the seismic performance evaluation of RC building consisting low, medium & high rise building frame located in seismic zone V. They adopted steel bracing system in low, medium & high rise building to find out possible earthquake mitigation system [19]. Similarly, Ahmed &Kalurkar (2017)deals with the studies of the effect of different type of bracing system on the storey stiffness, on the fundamental time periods and fundamental frequency. They modeled eleven models with different bracing

system (models such as bare frame, one concentric X bracing system, three frames with diagonal bracings & six frames with eccentric bracings). The modelling and analysis work is done in ETABS software. After the analysis, they compared various outputs such as fundamental time period, stiffness, maximum displacement & base shear.

CONCLUSIONS

1. This study includes the review of past studies to understand the performance of symmetrical structures subjected to seismic loadswith different height & analysis methods.
2. It also includes the study on effects of specially shaped columns on RC building with different heights subjected to seismic loads.
3. It includeseffects of bracing & shear wall system on symmetric building subjected to seismic loads, the effects of building configurations & alteration of seismically effective parameters on performance of structures with different height & analysis methods.
4. This study concludes that performance of structures with specially shaped columns are much better than conventional building with rectangular columns. Many seismically effective parameter such as storey displacement, storey drift, and fundamentalstime period are less for specially shaped columns building with more stiffness and lateral load resistance capacity.

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ANALYSIS OF INTZE WATER TANK FOR DIFFERENT ANGLES OF CONICAL DOME

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ABSRACT
This paper concerns with the wind analysis of RCC Intze type elevated water tank whichis subjected to live load ,dead load and wind load as per IS codes. Angle of Conical dome plays an important role in the analysis of the tank. This paper deals with the study of water tank in which the angle of conical dome vary as 45^0, $51^0$and 56^0 degrees for same capacity of water tank i.e. $1000m^3$.

Keywords: Intze tank, Angle of conical dome

INTRODUCTION
Water tanks are the structures used for storage of water for domestic and industrial purposes. The tanks may be classified as overhead, on ground and underground water tanks, depending on the location of the tank. Depending on their shapes the tanks are named as Rectangular, Circular and Intze types in various shapes. In case of large diameter elevated circular tanks, thicker floor slabs are required resulting in uneconomical domes. In such cases, Intze type tank with conical dome and bottom spherical dome provides an economical solution. Since the Intze type tank has an optimal load balancing shape, it is widely preferred. This type of container and supporting structure has been extensively used in India since last few decades. Analysis and design of the tank plays an important role in making structure safe, economical and durable.

OBJECTIVES OF STUDY
1. To perform the analysis of Intze tank considering wind load.
2. Analysis of Intze tank on empty and full condition with different angles of conical dome for samecapacity of the tank.

LITERATURE REVIEW
1. Deepa Dubey (2017)[1] - This work aims to study the performance of the Overhead water tank ofIntze type under the action of basic wind loads in different terrain category. Designed Wind forces for zone I is about 19 to 21% less than the that of zone II, about 35 to 37% less than that of zone III, about 45 to 47 % less than that of zone IV, about 55 to 57% less than that of zone V, about 71 -73% less than that of zone VI. .

2. Shubham Gautam (2017)[4] - This paper concerns with the study of deviation in bottom domeangle in which the angle of bottom dome selected is 35^0 and 50degrees for same capacity of water tank i.e$1000m^3$. Intze tank with bottom dome angle 35 degrees is more appropriate as compared to 50 degrees.it is because the value of base shear, storey displacement and max. bending moment at base is more with 50 degree bottom dome angle as compared to 35 degrees . Also these values are more in case of water tank with full condition as compared to empty or half full condition.

3. **V.L. Swarupa And R. Ramakrishna (2017)[5]**- The purpose of the work is to understand behavior ofthe water tank subjected to wind with straight and inclined leg. There is a decrease of displacement for Inclined Leg Water Tank by 11.1% when compared to Vertical Leg Water tank for same wind load. While no change was observed for displacement in y direction(along height), a decrease of about 29%displacement was observed in z direction (along lee-ward face) for inclined leg water tank compared to vertical leg water tank subjected to same wind load.

RESEARCH METHODOLOGY

In the present study of analysis of an Intze tank, model generation of the tank with different angles of conical dome is done under tank empty and tank full conditions. Analysis of different models is performed for wind load. Based on comparison of results of various models some conclusions are drawn. The details of tank are given in Table 1.

Table 1. Tank Details

S.N.	Particulars	Size/value	Size/value	Size/value
1	Angles of Conical dome (degrees)	45^0	51^0	56^0
2	Capacity of tank (m^3)	1000	1000	1000
3	Unit weight of concrete (kN/ m^3)	25	25	25
4	Grade of concrete (Mpa)	M30	M30	M30
5	Grade of steel (Mpa)	Fe415	Fe415	Fe415
6	Thickness of top dome (mm)	100	100	100
7	Rise of top dome (m)	2	2	2
8	Diameter of tank (m)	14	15	16
9	Height of cylindrical wall (m)	5.5	5	4
10	Thickness of cylindrical top (mm)	200	200	200
11	Thickness of cylindrical bottom (mm)	300	300	300
12	Section of Top ring beam (m)	0.4x 0.4	0.4x 0.4	0.4x 0.4
13	Rise of conical dome (m)	2	2	2
14	Thickness of conical dome (mm)	500	500	500
15	Rise of bottom dome (m)	1.8	1.8	1.8
16	Thickness of bottom dome (mm)	250	250	250
17	Number of columns	8	8	8
18	Section of bottom ring beam (m)	1.2x0.75	1.2x 0.75	1.2x 0.75
19	Total height of staging (m)	18	18	18
20	Diameter of columns (m)	0.75	0.75	0.75
21	Section of bracing (m)	0.5 x 0.5	0.5 x 0.5	0.5 x 0.5
22	Section of bottom circular beam (m)	1.2 x 0.75	1.2 x 0.75	1.2 x 0.75

LOADS

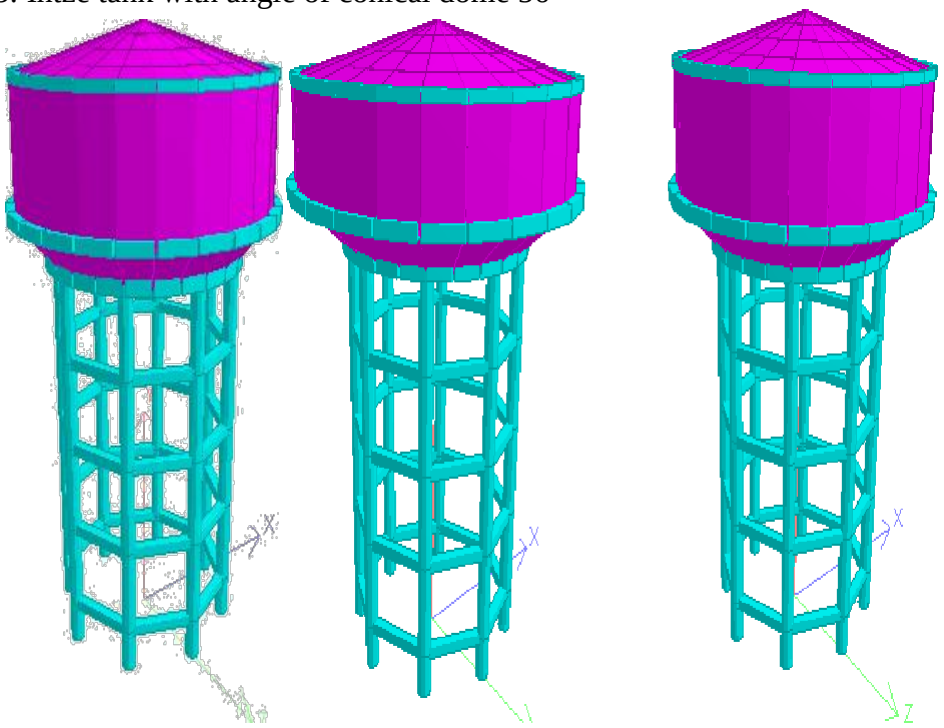
Dead load includes weight of structure and water and Live load 1.5 KN/m^2 as per IS-875 part-2[2] and wind load as per IS-875 part-3[3].

MODELLING ANDANALYSIS

The Tank is modelled using software Staad Pro V8i (SS6). The following models of Tank are considered

Model 1: Intze tank with angle of conical dome 45°

Model 2: Intze tank with angle of conical dome51°
Model 3: Intze tank with angle of conical dome 56°



(A) **(B)** **(C)**

Figure 1. Models of Intze tank with angle of conical dome 45^0 , 51^0& 56^0

RESULTS AND DISCUSSION

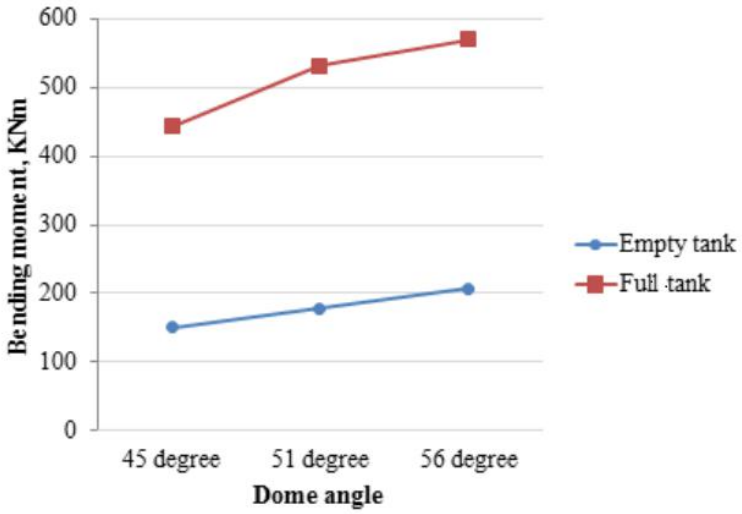
Parameters such as Bending moment, node displacement and hoop tension are found through analysis of various models of the tank.

Bending moment

Bending moment is the reaction induced in a structural element when an external force or moment is applied to the element causing the element to bend.

Table 2 B.M. for different angles of conical dome

Angle of ConicalDome	Bending moment(kNm)	
	Emptytank	Fulltank
45 degree	166.1	554.61
51 degree	185.96	633.44
56 degree	205.78	643.81



After analyzing various models, we can summarize the results as-

1. As the angle of conical dome is increased from 45^0 to 56^0 bending moment is also increased in both the conditions i.e. Empty and tank full condition.

2. Comparing empty and full tank condition, maximum value occur in tank full condition, because of water which applies hydrostatic pressure.

Node Displacement

Node displacement is the displacement of nodes from its position due to force.

Table 3. Node displacement for different angles of conical dome

Angle of Conical Dome	Displacement in (mm)	
	Empty tank	Full tank
45 degree	4.606	11.047
51 degree	5.033	11.126
56 degree	5.666	11.432

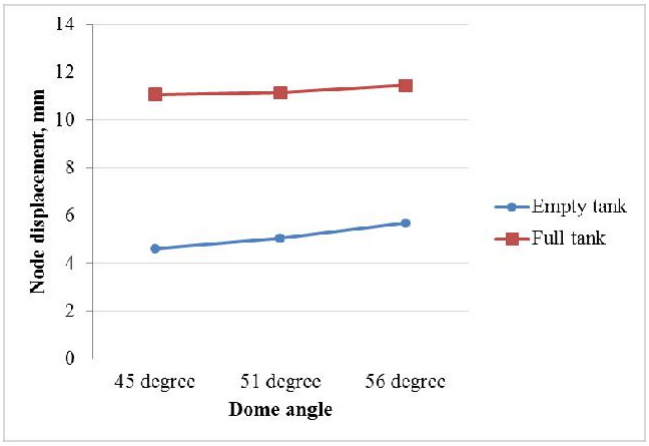


Figure 3. Variation of node displacement withangle of conical dome

After analyzing various models, we can summarize the results as-

1. As the angle of conical dome is increased from 45^0 to 56^0 , node displacement also increases in both the conditions i.e. empty and full tank condition.
2. Comparing empty and tank full condition, maximum values of node displacement occur in tank full condition, because of water which applies hydrostatic pressure.

Hoop Tension

The Hoop stress is the force exerted circumferentially (perpendicular both to the axis and to the radius of the object) in the cylindrical wall.

Table 4. Hoop tension for different angles of conical dome

Angle ofConical dome		Empty tank			Full tank	
	Hooptension(Bottom dome) kN	Hoop tension(Cylindrical wall)kN	Hooptension(Conicaldome) kN	Hooptension(Bottom dome) kN	Hoop tension(Cylindrical wall) kN	Hooptension(Conicaldome) kN
45 degree	87.75	55	207.5	734.25	173	448.5
51 degree	120	82.5	238.5	679.5	218.75	493.21
56 degree	151.5	133.5	260.45	584.5	324.75	563.3

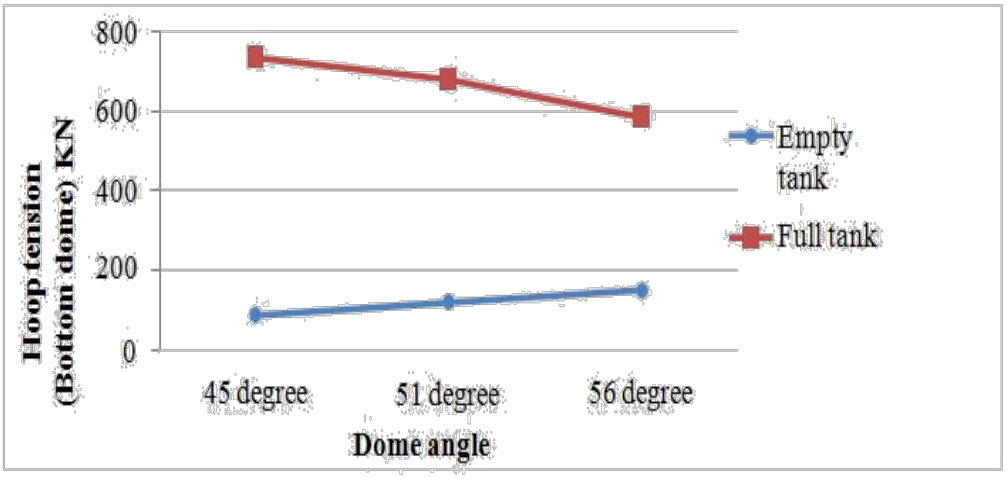


Figure 4. Variation of hoop tension in bottom dome with angle of conical dome

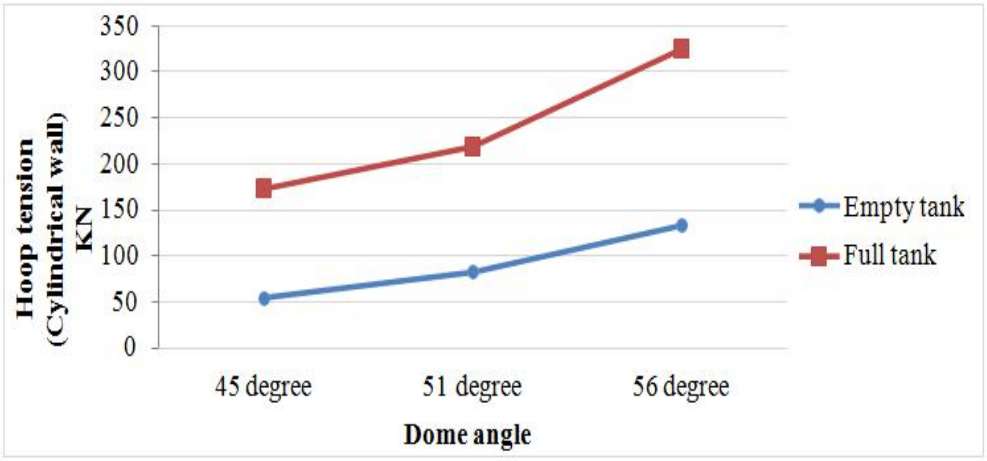


Figure 5. Variation of hoop tension in cylindrical wall with angle of conical dome

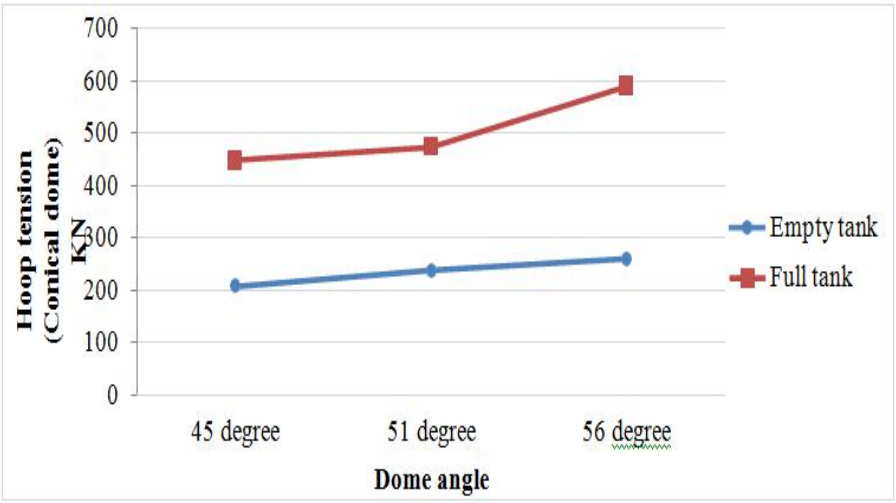


Figure 6. Variation of hoop tension in conical dome with angle of conical dome

After analyzing various models, we can summarize the results as-

1. In empty condition hoop tension in bottom dome increases with the increase in angle of conical dome from 45^0 to 56^0. Although as an exception, hoop tension in bottom dome decreases with increase in the angle of conical dome for tank full condition.
2. Hoop tension in cylindrical wall as well as conical dome under both empty and tank full condition increases with the increase in angle of dome.
3. Values of Hoop tension in bottom dome, cylindrical wall and conical dome are more for all the threeangles 45^0,51^0 and 56^0 under tank full condition compared to empty condition.

CONCLUSIONS

The following conclusions are drawn from this study

1. It has been found that as the angle of conical dome is increased from 45^0 to 56^0, bending moment and node displacement also increase in both conditions i.e. tank full and empty condition. Moreover values of bending moment and node displacement at all dome angles are found more under tank full condition compared to empty tank condition. Hence it is concluded that dome angle has a significant impact on bending moment and node displacement.

2. For both, tank full and empty condition, hoop tension in cylindrical wall and conical dome increases significantly when angle of conical dome is changed from 45 to 56 degrees.
3. In case of tank full condition, hoop tension in bottom dome decreases when angle of conical dome is increased from 45 to 56 degree. while opposite behaviour is observed in empty tank condition
4. As the bending moment and node displacement is found minimum when the angle of conical dome is 45^0, from design point of view lesser reinforcement will be required, making the structure economical .
5. Hoop tension in cylindrical wall and conical dome is found minimum in case of 45^0 angle of conical dome. Lesser the value of hoop tension lesser will be the thickness of section as well as requirement of reinforcement making the structure economical.
6. In case of tank full condition, hoop tension in bottom dome is found maximum when the angle of conical dome is 45 degrees, that will have marginal effect on the economy of the structure

FUTURE SCOPE

1. Same study can be done for conical dome angle less than 45 degrees and more than 56 degrees for the same capacity.
2. Study can also be done by changing the dimension so rise in bottom dome and top dome and diameter of bottom dome.

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RESPONSE STUDY OF RC INFILLED BUILDINGS IN SEISMIC ANALYSIS

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ABSTRACT
The reinforced concrete framed buildings with infill panels are the most commonly built buildings in the developing countries. It is common observation that the behaviour of the building is altered by the presence of the infill walls in the frame, under the lateral forces. Even in case of the design of reinforced concrete buildings, without proper consideration of seismic provisions, the infill element in frame can contribute to potentiate the structure under seismic effects. However it is also experienced in design practice to ignore infill effect to make it conservative, which can impact opposite, for vertical irregularity having discontinuous infill. It supports the need of infill modelling in framed buildings under seismic analysis. Indian Standard IS 1893 gives a multiplication factor in case of not considering the stiffness of infill but assessment of its feasibility is also seems to be required. The paper reviews the contribution and effect of infill modelling in seismic response of the building; also it presents the study of some code recommendations and the need of procedures of robust seismic design for such buildings.

KEYWORDS: Framed buildings; Infill; Lateral forces; Potentiate.

INTRODUCTION
The infill walls are normally considered as a nonstructural element in the concrete frame. In general practice, a period reduction factor for the RC frame without infill walls is considered for the effect of infill walls on the stiffness of frame structures and the constraint of the infill walls on the RC frame is disregarded. The failure pattern of the frame changes due to consideration of the constraint impact of infill walls.

The properties of the frame highly dependent on location of the infill walls and it can play a vital role in increasing stiffness of frame.So, in the design process, the influence of the infill walls must be considered.

The reinforced or un-reinforced masonry walls are also constructed in RC frames in seismic vulnerable areas, but sometimes a general lack of knowledge of behavior of this mixed structural system is observed. All around the world, the infill with masonry in general is used in the buildings.

This paper presents an attempt to study the effects of the infill in the reinforced concrete frame by means of the study of literatures available on this topic.

LITERATURE REVIEW
Amanat and Hoque (2006) performed modal eigenvalue analysis and 3D finite element modelling of regular RC framed buildings to study infill and its effect on fundamental period of vibrations of buildings. Author observed the higher period values by analysis than athe code values, in case of without infill modelled frames, but both the values were found in the close agreement in case of analysis of frame with modelled infill panels. Author also discussed that the variation of the period is not found due to the randomly distributed infills,

if the total infilled panels is kept unchanged. But, the variation in infilled panels may show some change in the period values. Authors suggested some guidelines for determining the period based on modal analysis, sensitivity analysis and the findings of this study after analysis of the considered frames with the help of rational approach.

Bendimerad et al. (1991) summarized the seismic behavior of the infilled RC-MRF buildings and discussed that the time period of RC-MRF buildings increases during earthquake reflecting the non linear behavior. The data base of 18 RC-MRF structures was studied in this work to evaluate the fundamental time period. They also discussed that on time period of building, the effect of detailing is not significant, but, the effect of non-structural elements is considerable to influence the period of framed buildings. In the study of infilled panels it was found the lesser impact of masonry panels on the stiffness of the building. The conventional methods for determining the period was also discussed and the proposed method was presented through this work. The work presented the rational approach of compiling the time period of the studied frames. The work presented the deformational structural response of considered frames and suggested some period considerations in the pseudo-elastic analysis.

Dubey and Kute (2013) experimentally investigated the bare frames, RC frames and partially infilled reinforced frame. The considered frames were tested under collapse condition applying the horizontal loads and the results were compared for different frames.A mathematical model was also proposed for partially infilled and braced frame to evaluate the ultimate load.

Dolsek and Fajfar (2008) analysed a four storey RC infill frame building by N2 method, based on spectrum approach and pushover analysis, to understand the impact of infills on seismic responses and made the comparisons with the results of bare frames. The diagonal struts were employed for the modelling of panels of infill and the study for damage of structure along with study of shear failure of columns were presented in the paper. The N2 method used in this work was implemented as per Eurocode8. The method combines the consideration of Pushover method of MDOF model and response spectrum method of SDOF model. By comparing the capacity and demand, the seismic assessment was done in this work and it was concluded that the infill beneficial for the structure both near limit state of collapse and at the damage limitation.

Eleftheriadou and Karabinis (2012) correlated the calculated periods to several recorded damage in infill building. Important conclusions have been presented on the basis of study of the parameters as height, structural type, etc. showing them as parameters influencing the seismic response. In addition, the qualitative characteristics of the cases of the considered seismic damage were considered to present a damage scale for the measurement in post-earthquake surveys in Greek. The structural and economic damage index was presented to study the seismic damage.

Gates and Forth (1978) considered the non structural elements in form of stiffness of building and its effect on time period and proposed an expression based on the study of ground motion data recorded for 27 RC infill frames during earthquake.

Hashemi and Mosalam (2006) analysed prototype of 5-storey (RC) infilled frame and un-reinforced wall of masonry of its first storey on a shake-table experiment. In the analysis, the demand parameters were studied to establish the relation with prototype structure with the

help of modeled structure. The dynamic properties of the RC prototype structure along with the local and global responses with the impact of the URM infill wall on the behaviour of considered structural was presented by the help of the shake-table tests results. Finally, distribution of forces in various elements of the structure was examined with comparative study of results of experimental and analytical study and reported substantial effects of infill walls on nonlinear behaviour.

Hatzigeorgiou and Kanapitsas (2013) analysed different building configurations of RC infilled structures to create a formula to estimate the period based on the vibration study of considered pre-existing structures of Greece. The external and internal infill walls were taken care in the considered models in the analysis. The effect of SSI on time period was also considered for taking into account the effect of the flexibility of soil of elongating period value. A nonlinear regression analysis was used for obtaining the empirical equations of period of vibration to achieve an effective but simple formula considering the evaluated datasets of considered buildings. The paper presented the comparative analysis of the developed empirical expression comparing it with the similar type of expressions from the different literatures.

Jain et al. (1997) studied the effect of stiffness of reinforced concrete infilled framed buildings for time period evaluation and presented the performance outcome. Authors also discussed the difficulties in the period analysis and presented the reviewed provisions of relevant design codes. 3 RC Frames with infill were analysed using ambient vibration test and 3D dynamic analysis. The obtained results were compared with the experimental results. Author suggested that the all possible sources of stiffness should be incorporated for the computation of the period to assess the seismic forces.

Mostafaei and Kabeyasawa (2004) observed that, in general practice, influence of the infill masonry in the stiffness is generally not acknowledged in the process of design, and the bare frames are considered. The author presented an analysis of the structure of the Bam telephone center to study a realistic approach of modelling of infill masonry walls. 3 different categories of modelling were considered as bare frames, frame and infill masonry, and frame with light infill panel. The analysis was done to assess the roll of masonry infill in terms of structural performance. It was concluded that the large nonlinear deformations of considered building can occur without infill masonry walls during the earthquake. The analytical modelling of the infill walls with openings, further studies were recommended.

Oliveira and Navarro (2010) have attempted to develop relation between construction parameters with the frequency or the period of the infilled building studying the dynamic characteristics of different infilled buildings of Portugal with the help of ambient vibration technique. 197 reinforced concrete buildings were analysed to evaluate natural period and it was reported in the form of a function of storey of building and the height for different typologies and situations. The correlation pattern and the parameters having significant influence are characterized. Comparisons with published formulae show similarity of considered parameters but some differences in approach also. The analysis results of increase in the periods of vibration were also discussed due to alteration of the seismic behavior. The method of forced vibration was studied by

Pan et al. (2014) analyzed 116 residential infilled buildings ranging from 4 to 30 storey using ambient vibration tests. The paper presents the results of evaluation for the effect of the aspect ratio, SSI and the period-height relationships under different site properties. It was

reported that the period values of buildings situated at soft-soil site are about 40% longer than firm soil site and the effect of aspect ratio can be considered as insignificant. The comparison between the measured periods with those estimated using the proposed formulas has been presented.

Trifunac et al. (2001) have analysed a 7 storey RC infilled building in Van Nuys, California performed for the apparent frequency. Using the data of recorded response of 12 earthquakes, the soil-structure interaction model was considered with a foundation description, surrounding soil and building structure, and responses of results of Fourier analysis were recorded. Zero-crossings analysis and windowed Fourier analysis were employed for analysis of instantaneous apparent frequency. It was discussed that the the apparent frequency is not constant, it changes from earthquake to earthquake. It was also suggested for other instrumented buildings to perform similar studies.

CONCLUSIONS

The following points of the conclusions are drawn from the literature study work as:

It was suggested that the infill acts as a diagonal strut (Smith 1966) on investigation of square frames.

The bottom of column and the column beam junction are the locations of the possible plastic hinges in case of partially infilled and braced RC frames (Dubey and Kute 2013), and these frames show less deflection and higher lateral load carrying potential in comparison to bare frames.

The experimental results were found close to the theoreticalvalues (Mali 1981) on investigation of failure and cracking behaviorof infilled frames under uniformly distributed vertical loading with racking loads.

(Benjamin and Williams 1958) found the scale models to be consistent with the prototype and no scale effect was observed on testing of considered prototypes and models of reinforced concrete infill walls.

(Dolšek and Fajfar 2008) highlighted for the ground motion intensity above a certain level that a mechanism of soft story to be formed and collapse will occur in case of low global ductility.

In case of infill frames, the infill has same effect on period of frame as the effect of number of bay on period values (Kose 2009).

In case of partially infilled RC framed buildings, the story drift and deformations in the column is lesser along with the decreased quantity of damage overall frame and column, in comparison to bare frames (Hashmi and Alok 2008).

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EFFECT OF SHEAR WALL IN THE DESIGN STABILITY OF BUILDINGS

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ABSTRACT
The necessity of comprehensive knowledge of the structural behaviour is observed as there is rapid growth in the number of high-rise structures in practice throughout the world. This supports the necessity of precise assessments of overall stiffness and strength of the structure. For resisting the lateral loads induced by dynamic forces commonly wind or earthquake, it is essential to ensure adequate lateral stiffness of building. Apart from the solutions like providing the bracing members in frames, infilling the frame, enhancing rigidity to the joints etc., the shear wall construction is another solution to provide high in-plane stiffness to resist the lateral forces.
The purpose of this paper is to study the attempt made by researchers in the field of analysis related to shear wall structures. The work is in compliance with the study of shear wall structures along with the association of frames, infilled frames etc. The assumptions and extent of the research made have been investigated, presenting the summary of in chronological order.

KEYWORDS: Comprehensive; stiffness; earthquake; shear wall; infilled frames.

INTRODUCTION
In modern time as the rate of construction of building or structural system is increasing day by day along with the advancement of variousanalysis and design technique for structural system and safety. The main function of any structure system is to carry a gravity loads along with the lateral loads which affect the stability of the building. Thefocus of the study in this paper is effect of the shear wall on building stability. Generally, the lateral forces in the building is commonly resisted by 3 systems-

- The rigid jointed moment resisting frame,
- The braced frame
- The shear walls.

Shear wall is the system of structure which is effectively used for resisting the lateral forces and to provide the adequate building stiffness which may occurs due to wind load or earthquake. Generally when RC walls, with very large in-plane stiffness are used, they provide needed resistance to lateral loads, have the ability to dampen vibration, and keep the lateral displacement within permissible limits, which generally acts as the vertical cantilever beams and resist the in-plane stresses like bending moments, and shear caused by lateral loads in the plane of the walls and also carry vertical gravity loads. The main purpose of the shear wall is to transfer the lateral forces which occurred due to wind or seismic forces to the diaphragm located at the base of the structure or to the foundation of the building.
The location of shear wall depends on various structural and architectural aspects of the building but generally shear wall is located near lift or staircase location in the building because of less usability. The different types of shear wall which are used are steel plate shear wall, concrete shear wall, plywood shear wall and RC hollow concrete wall masonry.

In this paper,modelling and experimental approaches has been discussed which are analysed and perform by the various researcher across the globe. This paper also discusses about the various design parameters like lateralstiffness, stressgenerated, storeydrift, effect of openings on shear wall, effect of windborne debris, freeze thawand the overall stability of structure.

LITERATURE SURVEY

Esmaili et al. (2008) studied the structural aspects of 56 storey long tower located inhigh seismicity zone, which is in Tehran. In this paper high storey structural system having shear wall with irregular opening are analysed under both gravity and lateral load and promulgate the problem occurred in behaviour of structural elements such as shear wall, coupling beam etc. In this structural system study, 3 general consideration are taken into account which areconstruction sequence loading, time dependent effects andoverall torsion. In this paper they explored the seismic effectiveness of the of the structural system i.e. building have enough level of stiffness or ductility to satisfy the codal assumptions. Also, the outcomes of axial load on ductility of shear wall and effective contribution of coupled walls via coupling beams. After this general study of ductility level in shear wall of tall building they concluded optimality and conceptuality of building design with some useful information which could be very useful for designers.

Asfa et al. (2010) in present days the FRP (fiber reinforced polymer) is used extensively for the strengthening of the concrete. FRP concrete prevents the brittle failure and also impart suitable ductility to system. Here the study was carried out on the effectiveness of FRP on the structural system with wall openings at different positions by FEM with help of ABAQUS software. Laboratory test on concrete shear wall used to calibrate the software resultsand verification of coupled shear wall and FRP strengthened beam is carried out.Then, investigation for effect of strengthening around the opening of coupled shear wall is carried out by FRP at different angles i.e. horizontal vertical (Cw-90),diagonally (Cw-45) and combinations (Cw-45,90). In this research two cases of wall behaviour were observed and verified, the two cases are CB with and without diagonal barand then latter, the wall is retrofitted andcomparison of it with others wall is carried out. For the verification of efficiencyof retrofitting methods, they concluded that Cw-45 gained more stiffness, increases the ultimate load carrying capacity and reduced the crack formation, while Cw-45,90 having highest ductility. whileCw-45,90 and Cw-90 having capacity in decreasing order respectively. From this study it can be easily concluded that the combination of horizontal, vertical and diagonal FRP layers convince to most efficient strengthening method and improve the structural behaviour.

Wdowicki et al. (2012) in this paper a satisfactory match is found on comparing a 3D shear wall assembly with are having a step to step change in the cross section by using a variant of the CCM(continuous connection method). There are various methods to analyse the shear wall structure which are available are FEM, method of frame analogy and CCM.Out of this method ,in current scenario FEM is one of the most effective method which is widely used for complex structure like shear wall with couple beam and braced systems,but in complex structural analysis there always a scope for development of other methods to reduce the complexity in computational effort and increasing the numerical efficiency in finite element method. Here the author presented a continuous connection method with various differential equation for system of shear wall. For design and analysis of coupled shear wall CCM is regarded as one of the most efficient and simple method. The main aim of this paper is to provide the effective differential equation of coupled shear wall system with non-uniform cross section across the height of the building using the variant of CCM .The results which

are obtained from present study are compared analytically and experiment and they observed a satisfactory results.

Titiksh et al. (2015) he studied the seismic performance of various framing structure like OMRF, SMRF and BSF (braced steel frame) and concluded the best suited framing structure according to Indian code scenario. For this purpose, author design G+4 storey structure in zone 5 according to Indian codes. Parameters which was considered here are used material, maximum node displacement, maximum shear on beamandmoment, maximum stress on plate,storey drift,average deflection at storyand also provides guidelines on structural economy criteria. They concluded that storey drift is least in BSF, stresses in column is less in BSF because of braces, lateral loads are best resisted by BSF.Overall BSF is more effective but it proves extremely costly and SMRCF is more effective than OMRF.
Soni et al. (2016)studied and analysed the behaviour of shear wall and towards the lateral stability of shear wall. They analysed G+10, G+20, G+26 SMRF building using finite element method by using STAAD PRO software.Loads were considered according to IS Standards. The analysis of structure frame model that include shear wall at different location has been done at location 1(interior) and location 2(exterior).Three load cases has been considered i.e. DL ,LL, WL and various parameters like interstory drift, base shear and lateral displacement and von mises stresses are compared . They concluded that overall deflection of structure at location 2 for G+10, G+20, and G+ 26 is more as compared to location 1 and also concluded that the location 1 of shear wall is better than the location 2 because in location 1 stress distribution and deflection is less.

Husain et al.(2016)in this study Husaincompare SW in structural system at a different position which is subjected to seismic loading .A comparative study is based on the 56 models with was analysed using finite element method with the help of SAP2000 V14 computer software. Each model consists of ten storey each all having same properties. The main aim ofthis study is to find out the effective type, appropriate shear walllocation in the building and to provide an efficient conclusion for the building under the lateral loads.In this paper 9 buildings have been considered according to bays number in X direction.Sidewall system (SW), middle-core system (MC), double-core at different symmetric locations , are the three different structural system of shear wall which are studied , also a model with no shear wall system in the building has been considered for comparative study. Study of various parameters like shear force, bending moments, drift etc is made for the optimization of structure. In last, the results observed are that the shear resisting capacity of the structural system is more as compared to that without SW system. For length up to 20 meters sidewalls arrangement has been considered good and for lengthgreaterthan 20m shear wall arrangement with double core is considered as ideal.

Honarparast et al. (2019) studied 20-story coupled shear wall in considered which was in Western seismic Canadian zone. This CSW building is designed with the old code so the main objective in the paper is to highlight the deficiencies by checking the dynamic performance of old designed CSWs and compare it to the CSWs designed by new code and also assess the effectiveness of externally bonded fibre reinforced polymer (EB-CFRP) retrofitting on the seismic response of deficient of CSWs .The CSWs are designed according to modern NBCC 2015 ,Canadian Standard Association (CSA) and old National Building Code of Canada (NBCC) 1941. With the help of RUAUMOKO program nonlinear time history analysis is carried out on old,new and CFRP retrofitted wall. The results concluded from analysis of CSW1941, CSW2015, and CSW1941 retrofitted with CFRP (CSW1941-R) that CFRP retrofitting method was more effective method to increase the seismic

performance of deficient old building also this also decreases the parameters likes storey displacement ,CBs rotation ,ductility demand, interstory drift and also improve the yielding sequence inwalls and CBs.

Saini et al. (2019) studied the damage assessment of wood frame shear wall subjected to lateral wind load and windborne debris impact. This study investigates the structural response of WFSW (wood frame shear wall) subjected to wind load and windborne debris impact and also to examine the factors which are influencing the extent of damage in windstorm. They simultaneously consider the wind force and the impact of windborne debris and carried out the damage analysis. For this detailed finite element models are developed to calculate the structural response of WFSW system, and the validation is done by the experimental study.The performance of WFSW was examined using various measures,such as penetration depth, absorbed energy, and deformed shape. Including the effect of various contributing factors, such as angle of attack, impact location, moisture content and nailspacing. The outcome of this study is then employed to introduce a capacity loss index , the damage accumulated found greatly dependent on the structural details like position of stud and panel,the lateral load capacity of SW may decreaseup to 30%,also the change in moisture content affects the structural damage.

Hung et al. (2020),the experimental study was conducted on failure behaviour of the four squat shear wall(FSSW) with high strength concrete (HSC)and HSS(high strength steel) with UHPC(ultra-high performance concrete) and in adjunct to the experimentation the suitability of existing design code ,tie and soften strut model for assessment of high strength squat wall ,shear strength was evaluated. Varioustypes HSC material is used for construction of FSSWand different grade deformed steel bar is used for reinforcement. The four specimen were HSC/75-HS(SD785) i.e. wall specimen made using high strength concrete containing 75% volume fraction of fibers and reinforced with SD785 steel bars,UHPC/0-HS(SD785) ,UHPC/150-NS(SD420) and UHPC/150-HS(SD785). One day prior to test the mechanical test on material is conducted and afterward cyclic loading tests on wall is conducted via hydraulic actuators. The lateral deflection on wall was tracked by using LVDT (linear variable displacement transducer). Author analysed the various results parameter like cracks number, dissipation of energy capacity,stiffness, strain in reinforcement, deformation and concluded very effectively with detailed numerical explanation in paper.
Rong et al.(2020)performed the experimental study of shear wall subjected to lateral load after freeze thawdamages. In recent times a very few experimental research have been performed on behaviour of shear wall structural system on the frost-damaged reinforced shear wall. Eight RC shear wall specimens were tested on lab by creating artificially freeze-thaw successive cycle to investigate the seismic performance. The major parameter includesthe number of freeze-thaw cycles (FTCs), strength of concrete andaxial load ratio. The seismic behaviour of experiment specimens was calculated in terms of deformation capacity, hysteretic behaviour, damage process, load carrying capacity,capacity deformation, shear deformation capacity, and energy dissipation. The results of study shows that there are many difference in actual and experimental condition ,by increasing the strength of concrete ,by use of air entrainment, onecan improve the frost resistance ,also the main type of damage due to frost-thaw damage is that decrease in the bond strength. They also stats the various limitation like there is need of more research in this field to predict the actual freeze-thaw damagesand for higher strength of concrete.

CONCLUSIONS

The following important conclusion obtained from the study of above literature which are-

Esmaili et al. (2008) recognise the effect of time dependence of the concretei.e.it make concrete element more sensitive to the time dependency which are having different stiffness. Shear wall for both gravity and bracing system can't be used simultaneously on increasing the axial load, moment of inertia of section increases and lesser axial force provide more stability and ductility.

Asfaet al. (2010) found that the diagonal FRP bracing gained more stiffness combination of diagonal and vertical gained more ductility. Diagonal Combination also increases theultimate bearing capacity and reduced crack formation is found. He concluded the combination of all combination leads to the most efficient strengthening method.

Wdowicki et al. (2012) concluded the differentialequation for the solution of assembly of shear wall which is having a step wise changes in the cross section with the help of continuous connection method.

Titiksh (2015) et al after the study they concluded that BSF is most effective framing system but is not economical. SMRCF is more effective than OMRF but less effective than BSF.

Soni et al. (2016) concluded about the location of shear wall in the structure system. they found the interior arrangement of the shear wall is better than that of exterior in terms of less stress generation, less nodal displacements.
Husain et al. (2016) the results observed are that the shear resisting capacity of the structural system is more as compared to that without SW system. Sidewalls arrangement has been considered good and the arrangement for double core shear wall is found ideal for length up to 20 m and more than 20 m respectively.

Honarparast et al. (2019) The results concluded from analysis of CSW2015, CSW1941,and CSW1941 retrofitted with CFRP (CSW1941-R) that CFRP retrofitting method was more conducive method to increase the dynamic performance of deficient old building also this decreases the parameters likes storey displacement ,CBs rotation ,ductility demand, interstory drift and also enhance the yielding sequence in walls and CBs.

Hung et al. (2020) Author analysed the various results parameter like cracks number, dissipation of energy capacity, stiffness, strain in reinforcement, deformationand concluded very effectively with detailed numerical explanation in paper.

Rong et al.(2020) The results of study show that there are many differences in actual and experimental condition. By increasing the strength of concrete, by use of air entrainmentone can enhance the resistance against frost. The main type of damage due to frost-thaw is that decrement in the bond strength.

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STUDY OF FLAT SLABS IN REINFORCED CONCRETE BUILDINGS

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ABSTRACT
Flat slab reinforced concrete building facilitates for several advantages over conventional moment-resisting frames such as faster construction procedure, reduced building height, ease of formwork etc. Combination of the flat slab and post-tensioned slab can provide more effective than the conventional slab, enabling for designing of long-span frames with improvement in aesthetic view, but, from the safety point of view, the comprehensive analysis of structural effectiveness of flat-slab construction is required under earthquake conditions. The assessment of the behaviour of the flat slab structure having its combinations with drop panel, infill walls, shear wall etc. have been discussed in this paper. The paper presents the recent research on flat slab systems in high-rise buildings with nonlinear dynamic analysis to assess the influence of modelling the RC flat slab elements, on the response of the building system under seismic excitation.

KEYWORDS: Moment-resisting frames; Formwork; Flat-slab; Nonlinear.

INTRODUCTION
The flat slab system has slabs supported on columns without beams, with or without drops, with or without columns heads, unlike the conventional slabs system where the beams are used. The transfer of load in a flat slab takes place from slab to columns and then directly to the footing of the system. For the reduction of reinforcement in support region and to provide additional shear strength, the flat slabs are thickened near columns positions. On the other hand in the conventional slabs, the load transferred from slabs to beams and then to columns. It causes the increment of the weight of the structure and makes the formwork costly when compared to flat slab structures. In the conventional slabs, the slab thickness is small and beam depth is large and thus more formwork is required as compared to that of flat slabs. In this slab, the dead load is more as compared to that of the flat slab.

The flat-slab system is popularly used in building constructions due to the many advantages over conventional moment-resisting frames. Since beams are absent in the flat slabs, it has many advantages like lower building height, unobstructed space, good ventilation and lighting, easy provision of services like pipes lines and the wires beneath the slabs, and have shorter construction period and also has functional as well as architectural and economical aspects. The formwork uses is simple. The structural efficiency of flat-slab is high. It is efficient in resisting gravity load. But, the flat-slab effectiveness is reduced by its poor performance under seismic loading conditions. It has large lateral displacements due to beams absence which resulting in lower lateral stiffness. Due to above the deformations causes' damage to members which are non- structural when subjected to moderate intensity seismic excitations. It is more flexible and has more lateral drift when subjected to seismic loading conditions.

The failure of punching occurs because of the transfer of unequal moments and shear forces between columns and slabs is also the problem in flat slab constructions. Under the

seismic action, the unequal moments produces high shear stresses. The flat-slabs has major deduction in stiffness caused due to the cracking which occurs from different loading conditions like; temperature effect, lateral loads, shrinkage effect and the gravity loads. Therefore, in the high seismic regions, the flat-slab can be used for the resistance of vertical load along with shear walls or other structural elements for the resistance of the lateral loads on the structure. In these cases, for the resistance of the gravity loads, the connections of slab column must undergo the transverse deformations of the lateral load-resisting elements without any punching shear failure. The flat slab system with certain modifications can effectively perform with acceptable earthquake risk. Modifications with some structural elements improve the lower bearing capacity of the flat slab and increase stiffness and strength, improves overall earthquake performance of flat-slab.

In this paper, some modelling related approaches are discussed such as stiffness degradation effect, for the seismic analysis and static analysis of flat plate system. This paper also discusses the requirement of lateral load resistance system such as the shear wall, infill walls, edge beams, in the flat slab constructions to improve overall seismic behaviour.

LITERATURE REVIEW

Fraser (1983) proposed rules to find the effective width of the slab in flat plate floors and the effective stiffness of the equivalent beam in normal conventional buildings. These are evaluated from some parametric studies by using finite element techniques, which are used for both laterally and gravity loaded buildings. A building model is analyzed by both methods and the results are evaluated. The formulas are presented for calculating the stiffness of the equivalent beams. It is found that the frame stiffness evaluation becomes simpler, due to the elimination of equivalent columns calculations. It is also observed that the predicted sway deflections of flat plate building evaluated by rules are in close agreement with the actual deflections.

Moehle and Diebold (1985) examined the output of the flat-plate frame under seismic excitation of base motions. They tested 2 storeys, 3-bay flat-plate frame structure experimentally and subjected dead load on the prototype, and then loaded for seismic excitations of low, moderate, and high intensity. The test structure design based on the direct method specified in the ACI Building Code (ACI 318-83) for vertical gravity loads and seismic loads, the static lateral force method is used. The detailing requirement based on ACI 318-83. The lateral load stiffness of the test structure is analyzed and evaluated by equivalent frame models and the effective beamwidth. It is found that the lateral load stiffness decreases with an increase in the level of drift. This reduced stiffness is not incorporated in both models hence the stiffness evaluated using both models are more at working loads. The inertia of slabs is reduced to rationally estimate stiffness. It is also found that at the peak base shear condition, there is the uniform variation of lateral inertial loads over height, and it is contrary to the inverted triangular distribution which is given in design.

Durrani et al. (1994) evaluated theresults of a 9-story flat-slab structure during the Loma Prieta, California, earthquakeof 1989 by using System identification, equivalent static techniques as given in codes and the 3D linear dynamic analysis. For this, the static and dynamic analysis is performed on 4 different models by using GTSTRUDL (GTICES: "GTSTRUDL" 1991) which is a finite element based program. The models include dead load and live load variations, shear wall variation and floor slab variation. The system identification results are used to validate the analytical model. It is found that the analysis results from system identification and the analysis of the dynamics, indicates torsional mode

as lowest mode of vibration, and results for the torsional mode from system identifications and dynamic analysis are same.The response results show that at the building centre, the NS behaviour is dominated because of the presence of shear wall due to initiation of frame action of the shear wall. The base shears evaluated from the dynamic and static analysis are the same. However, the drift values evaluated from the static analysis are lower than both methods.

Luo et al. (1994) give an equivalent frame approach for non-linear seismic analysis for RC flat plate building, based on hysteretic model and concept of effective slab width. The determination of the factor of effective slab width and hysteretic parameters are evaluated from the response of laboratory tests. The justification of the proposed method is done by evaluating the measured and the observed results of 2-bay flat-plate assembly experimented under seismic loading. From the laboratory tests, it is observed that there is a reduction of stiffness takes place with the increment in drift level. The results of the elastic analysis can deviate from the actual results based upon the assumed stiffness reduction factor and effective slab-width. Thus the non-linear equivalent frame method is given in which slab stiffness degradation effect is incorporated in calculations and the method is also based on the equivalent beam-slab concept, which incorporates effects of both stiffness and the moment transfer capacity of connections.

Erberik and Elnashai (2004) evaluated fragility curves of flat slabs masonry infill walls building of medium-rise. In this evaluation of a 5 storey flat slab infill wall model the inelastic dynamic analysis and in-elastic pushover static analysis is performed for the calculation of seismic susceptibility of the flat-slabs models. The pushover analysis performed by use of ZEUS-NL software. The analysis results include eigenvalues, capacity curves, storey drifts, response spectra etc, of Flat slab building, is compared with moment resisting frames along with codal provisions. They observed that the seismic damage for flat-slab and moment-resisting frames structures are almost the same. From the fragility curves, they observed that the flexibility of flat-slab structure is more than the normal conventional frames because of the absence of beams. The curves also reflect the importance of the infill walls that dominates response. As the damaged infill panel not contributed to transverse resistance, the structure reached inter-story drift limit level more speedily than the normal moment-resisting frame. The results indicated that the infill walls addition in the flat slab structure created a major increment in the stiffness of the system. Fragility curves are the graph between the probability of failure and the spectral displacements to access seismic loss.

Kim and Lee (2005) proposed method includes the development of super elements for the study of flat slab system with the help of the matrix condensation technique. The compatibility conditions are induced by the use of fictitious beam in the advancement of super elements development. As it is tough to use equivalent frame method with equivalent beams flat slab structure analysis with openings. So it is necessary to use finite element technique for the correct analysis but it requires more calculations time. They did the static and dynamic analysis using SAFE and MIDAS/SDS software. They compare the analysis results of finite element technique, equivalent frame method and that of the proposed super element method. They observed that the equivalent frame method gives less accurate results than the finite element technique results and it cannot be used in flat slabs with openings. The finite element method is used in openings but takes more computation time. They propose an analytical method using super element gives good results with mesh results and reduces the computation time. The stiffness degradation effect due to cracks is also considered in the proposed method.

Hueste and Bai (2007) evaluated the seismic performance of a 5story Reinforced Concrete flat slab building which is situated in the Central United States and which is designed for the code requirements and provides retrofit techniques. Structural response is computed using in-elastic static pushover analysis and in-elastic dynamic time history analysis with the help of finite element technique using ZEUS-NL structural analysis program. The responses of models are assessed using FEMA356 criteria include Collapse Prevention level, Life Safety level, and Immediate Occupancy level. The responses of dynamic and static analysis are compared. They observed that the Flat slab not fulfilled the CP limits; several retrofit techniques are evaluated including use of the shear wall, use of Reinforced Concrete column jackets, and use of external steel plates to constrain the regions of plastic hinges of the columns. The building is again analysed using ZEUS-NL and seismic evaluation is done with the FEMA 356 criteria. They observed that responses for the in-elastic static pushover analysis and the in-elastic dynamic analysis for the original un-retrofitted structure have fulfilled the prediction of the global structural response. The addition of shear wall provides improved seismic performance of the Flat slab building and all other retrofits techniques improve the overall seismic performance.

Apostolska et.al (2008) evaluated earthquake performance of a reinforced concrete flat slab system with the help of different model includes Frame, Flat-slab strengthened by perimeter beam and RC walls, Purely flat-slab, Flat-slab strengthened by RC walls and Flat-slab strengthened by a perimeter beam. To evaluate the seismic resistance of flat slab system seismic static and dynamic analysis is executed by using the finite elements technique with the help of SAP2000v10.0.9 advanced computer program and response parameters are compared among different models and the effect of designed modifications on the dynamic characteristics of structure, on the bearing capacity and deformability of the flat-slab structure is studied. They observed that the fundamental period and displacements of the flat-slab system is more in comparison to framed systems. The modal vibration of the first mode is characterized by torsion. It is also examined that the flat slabs systems have lower stiffness capacity and a strength capacity in comparison to the framed system. The alteration of the flat slab system with some structural elements improves the low deformability and bearing capacity of the system and provides improved seismic behaviour of flat slab structure during lateral load and seismic excitations.

Han et. al (2009) developed a method for the slab stiffness reduction factor calculations in beams width models for the estimation of moments and lateral drifts for the flat slabs against the dynamic loading. The nonlinear regression is conducted using data collected from test results of reduction factors. The validation of transverse stiffness estimated by the proposed technique is done on the performance of a test conducted on the flat slab connection system. It is observed that the reduction factor of stiffness depends upon the level of applied moments and remain unaffected by the amount of steel until the yielding of slab reinforcements. Lateral load and slab stiffness reduction factors both are different. It is observed that the proposed technique with the stiffness reduction factor correctly validates the estimation of the lateral stiffness of the test model.

Sen and Singh (2016) evaluated the flat slab building performance of various heights which was design for vertical loads using various provisions of code. They analyzed and designed various flat slab models including with and with-out the progression of reinforcement in the bottom of the slab, with and without the use of edge beams along with the variable height. They use equivalent frame approach, nonlinear static (pushover) analysis using SAP 2000 software considering P-Delta effect. They found that all the models

collapsed under the maximum considered earthquake (MCE). The punching shear dominated the major failure. The punching shear capacity of the system is improved by the use of edge beams and the performance of flat slab is improved by providing continuity in the slab bottom reinforcement. They observed that the flat slab system not performing adequately in high earthquake zones and needs primary lateral load resistance system such as shearwall. They proposed a displacement-based method for the estimation of the requirement of size and the number of shear for the lateral resistance of the system to improve overall seismic performance.

Cosgun and Sayin (2018) studied the causes of failure of existing partially collapsed 2-storey reinforced concrete flat slab car parking structure supported on columns under vertical loads. They did their study in 3 stages. Firstly they examine the present state of collapsed structure with the help of data available from plans and drawings of the existing structure. In the second stage, samples are collected from the structure for the conduction of mechanical tests and in the third stage 3D finite element model is prepared by using SAP2000 software for performing the numerical analyses based on the information gathered from the laboratory experiments and field examinations. They found that Flat slab systems are inefficient for structures subjected to heavy vertical loads but these systems efficiently resist light load cases. The main reasons of collapse are the non-fulfilment of the checks for punching in the design, Live loads are underestimated in the design phase are less than the loads specified in regulations and the bond length detail is not applied.

CONCLUSIONS

The following conclusions are highlighted here from the studied literature as:

Fraser (1983) concluded that the frame stiffness evaluation becomes simpler, due to the elimination of equivalent columns calculations.

Moehle and Diebold (1985) found that the lateral load stiffness computed by both effective beam width and equivalent frame models are more than actual. The reduced stiffness, due to the formation of the crack, is not incorporated in these methods. For this, they reduced the inertia of slab to account reduced stiffness.

Luo et al. (1994) also concluded that there is a reduction of stiffness takes place with the increment in drift levels. The linear elastic analysis results can diverge from the observed actual results. Thus they proposed non-linear equivalent frame approach to incorporate slab stiffness degradation effect, which also includes the effect of moment transfer capacity of connections.

Kim and Lee (2005) observed that the equivalent frame method cannot be used for flat slabs with openings and finite element method take more computation time. So they give some modifications in modelling techniques by using super element in the finite element method for faster calculations.

Han et. al (2009)concluded that the reduction factor of stiffness depends upon the level of applied moments and remain unaffected by the amount of steel until the yielding of slab reinforcements.

Durrani et al. (1994) found that the presence of shear wall dominated the response of building due to initiation of frame action of the shear wall. The base shear evaluated from the dynamic and static analysis is the same. However, the drift values evaluated from the static analysis are lower than both methods.

Erberik and Elnashai (2004) found the flexibility of the flat slabs structure is more than normal conventional frames because of the absence of beams. They also observed thatthe infill walls addition in the flat slab structure created a major increment in the stiffness of the system.

Hueste and Bai (2007) concluded that flat slab not fulfilled the Collapse Prevention limits, so several retrofit techniques are evaluated. They found that the addition of shear wall provides improved seismic performance of the Flat slab building and all other retrofits techniques improve the overall seismic performance.

Apostolska et.al (2008)observed that the fundamental period and displacements of the flat-slab system is more in comparison to the framed system and the flat-slab system have lower stiffness capacity and strength capacity. The alteration of the flat slab system with some structural elements improves the low deformability and bearing capacity of the system and provides improved seismic behaviour of flat slab structure during lateral load and seismic excitations.

Sen and Singh (2016) found that the flat slabs are not performing adequately in higher earthquake zones and need lateral load resistance system like shearwall. The failure is due to punching and it is improved by the use of edge beams.

Cosgun and Sayin (2018) concluded the causes of brittle failure of flat slab buildingsand found that these are not efficient for heavy loads but these systems efficiently resist the light load. The main reasons for brittle failure are the non-fulfilment of the checks for punching shear capacity in the design phase, live loads are underestimated in the design phase and the bond length detail is not applied.

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ANALYSIS OF FRAMED STRUCTURE WITH TRIPLE FRICTION PENDULUM BEARING UNDER SEISMIC LOADING

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ABSTRACT
In early practice of civil engineering it was accepted that increase in stiffness will lead the structure to withstand seismic forces. But later in 1974, after invention of base isolation friction pendulum by Zayas, it was observed that base isolation provided to a structure reduces the overall stiffness of structure as well as response generated due to seismic forces. Thus use of friction pendulum bearing as base isolator came to use for seismic resistant designing. The main purpose of base isolation to increase time period of structure, so as to decrease spectral acceleration, base shear, storey drift and other parameter.
Later, it was found that displacement of bearing plates of Single Friction Pendulum Bearing (SFPB) were large as it was oscillating within single pair of plates, which occupies a large area and reduced the effectiveness of bearing. Thus to increase the usability of bearing rather than single pair of plates, multiple pairs were used so as to occupy relatively less space by moving in different mechanism within small space. Those bearing were termed as double friction pendulum bearing and triple friction pendulum bearing.
Triple friction pendulum was highly preferred when it comes to select for base isolation, because of its adoptive behavior under different levels of earthquakes. Its adoptive nature of stiffness and damping is due to five different mechanisms within the bearing.
Paper summaries seismic responses of 11 storey R.C.C. framed structure under base isolation and for reference its responses are compared with similar fixed base structure. The results indicate by addition of triple friction pendulum at base of structure, tremendous decrease in responses like storey drift, storey shear, spectral acceleration, input energy was observed. Also, it leads to increase in fundamental time period of structure.
The performance of Triple friction pendulum was also observed in terms of re-centering capability of the bearing. To check re-centering displacement graph for bearing was examined.

KEYWORD: Triple friction pendulum, base isolation, optimization of triple friction pendulum

INTRODUCTION
Basic principle of base isolation is to decouple the substructure from superstructure introducing flexibility to structure hence reducing the stiffness of overall structure. To minimize the transmission of potentially damaging earthquake ground motions into a structure is achieved by introducing flexibility at the base of structure in the horizontal direction while at the same time damping element is introduced to dissipate energy and hence restrict the amplitude of motion caused by the earthquake.
Purpose of base isolation is as under:

- Reducing stiffness
- Increase fundamental period
  (decrease natural frequency of structure)
- Increase damping

Triple Friction Pendulum Bearing (TFP) is seismic isolator uses the characteristics of a pendulum. It lengthen the natural period of the isolated structure hence develop less earthquake force.
It functions over 3 components:

- Articulated friction sliders
- Spherical concave sliding surface
- Lateral displacement restrainers

It is obvious that the main purpose of base isolation is to increase natural period of structure, so as to effectively decrease the base shear, inter-storey drift. The effective period of vibration of the superstructure is increased due to the reduced lateral stiffness of the isolation system.

LITERATURE REVIEW

1) Apostolos A. Sarlis and Michael C. Constantinou (2013) presented a revised model of the Triple Friction Pendulum bearing. No assumptions are made on the location of the resultant forces at each sliding surface. No constraints on the values of the coefficient of friction are required, provided that all sliding surfaces are in full contact. The inertia effects of each of the moving parts of the bearing are accounted for in the derivation of the equations describing its behavior.
2) Troy A. Morgan et. al. (2008) provided numerical evaluation to characterize a new multi-stage isolation bearing that can progressively adopt different hysteresis property at different time and condition as per requirement. He also showed experimental results to validate those results. The results showed that these multi plate friction pendulum bearing have greater advantage than those 3 independent pendulum mechanism whose stiffness and damping has to be selected as per need.

METHODOLOGY ADOPTED

In this study, R.C.C. structure with Triple Friction Pendulum Bearing (TFP) was analyzed using SAP2000 software and sensitive analysis as a procedure was followed in which independent variables were adjusted so as to obtain an desired performance.

In sensitive analysis, 7 independent design parameter of triple friction pendulum were selected,

1) Effective radius of curvature of sliding surface 1 and 4.
2) Effective radius of curvature of sliding surface 2 and 3.
3) Coefficient of friction on sliding surface 1.
4) Coefficient of friction on sliding surface 2 and 3.
5) Coefficient of friction on sliding surface 4.
6) Displacement capacity of sliding surface 1 and 4.
7) Displacement capacity of sliding surface 2 and 3.

NUMERICAL STUDY

The RCC framed building of 11 storey height and 30mx30m in plan is considered for present study, details of which are shown below:
STRUCTURAL DATA:
The details of 11 storied RCC building is shown below:

Plan size	30x30 m^2
Frame height	44 m
Concrete	M30
Rebar	HYSD500
Beam	600x400 mm

Column	800x800 mm
Slab thickness	200 mm

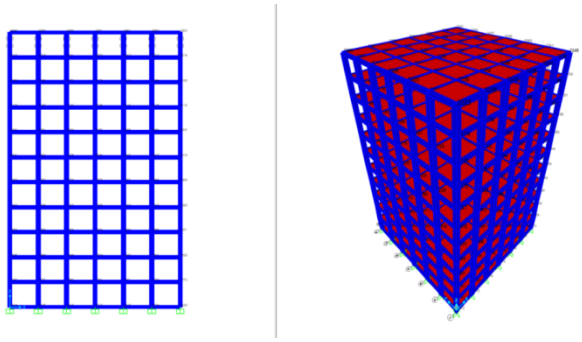


Figure-1 Elevation of Structural model of RCC Building in SAP2000 software

TIME HISTORY DATA

Loma prieta earthquake (0.209g PGA) is taken to observe performance of isolated RCC building. The plot of time vs acceleration is shown below:

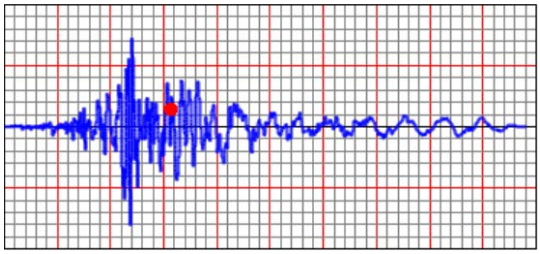


Figure-2 Loma prieta earthquake Time history

BEARING DATA: (refer figure for R and μ)

R_1 & R_2	2.236m & 0.99m
μ_1 & μ_4	0.045
μ_2 & μ_3	0.03

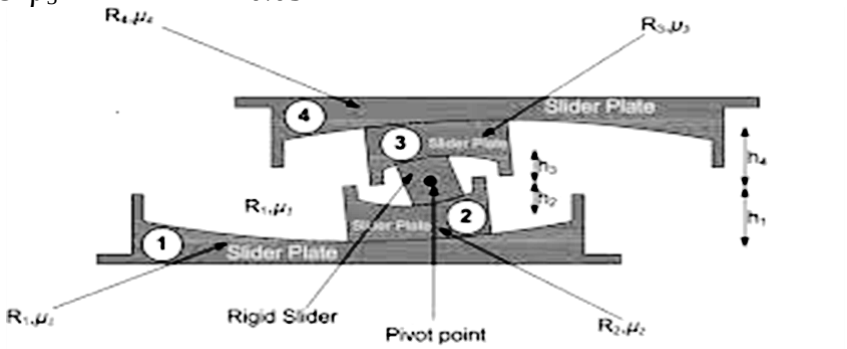


Figure-3 Components of TFP bearing

RESULTS

The study was performed on RCC building with and without TFP base isolator. The earthquake time history Loma prieta was used to perform Model time history analysis. Expected remarkable change observed is summarized below:

a) Fundamental time period

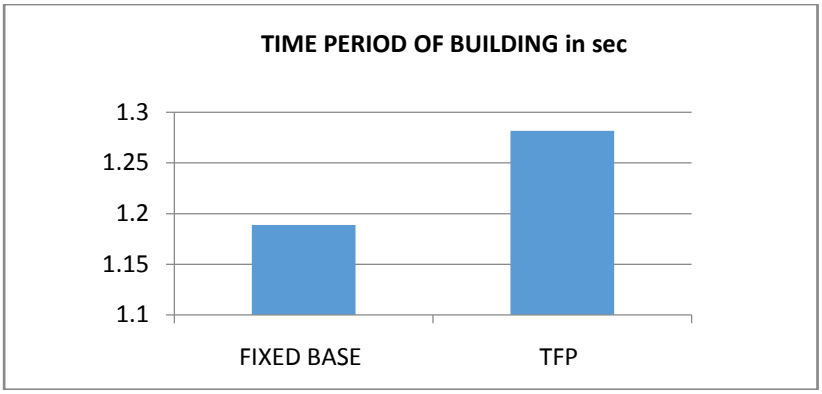


Figure –4 Fundamental time period comparison

c) Base shear

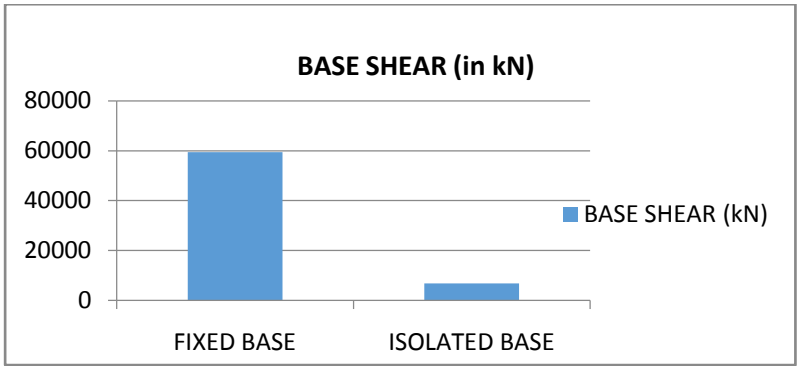


Figure –5 Base shear comparison

In this case of isolated frame decoupling of super structure of the frame reduces the stiffness of system resulting in drastic reduction (90 %) of base shear as shown in Figure above.

d) Storey displacement

The storey displacement observed at different level is plotted below:

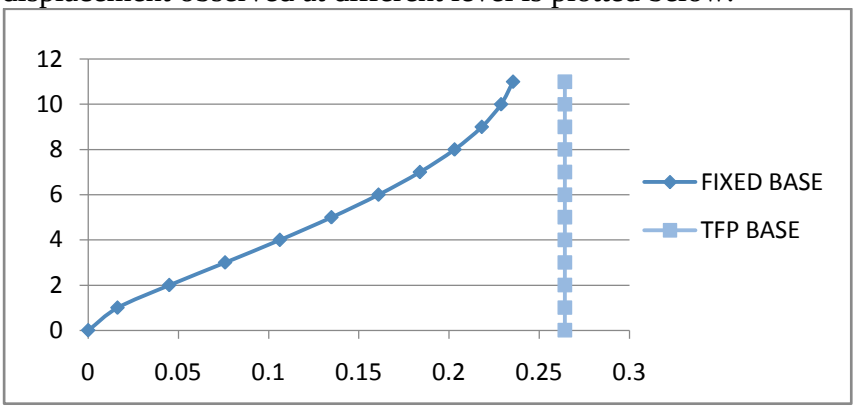


Figure –6 Plot of Maximum Storey displacement in m vs storey height

In case of fixed base frame, though the maximum displacement is less than case of isolated base frame but the point need to be noted is drastic reduction in storey drift of the upper part of the frame. As upper frame above isolator is decoupled from base, it moves as a rigid body leading to reduction in storey drift.

Variation in Maximum storey drift at various floor level observed is shown below:

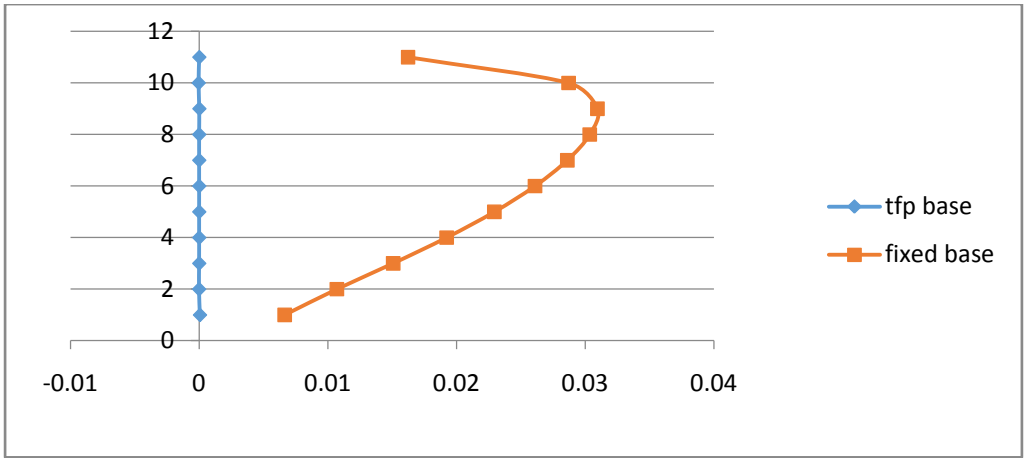
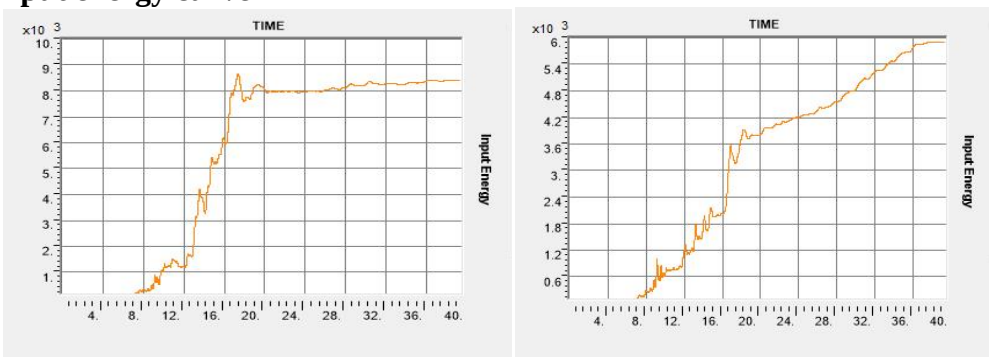


Figure 7–Plot of Maximum Storey drift in m vs storey height

e) Input energy curve



(a) Fixed base frame (b) Isolated frame

Figure-8 Input energy curve for 11-storeied RCC building

Curve shows sudden spikes with higher slope of energy within system in case of fixed base frame, while in isolated base frame the energy within system reduces to a great extent.

RECENTERING OF ISOLATOR

"Re-centering capability" is property of all friction bearing to return to its original position. It is one of the major check in designing triple friction pendulum for its proper working and centering mechanism.

Check for re-centering capability is done by checking displacement graph of bearing under excitation.

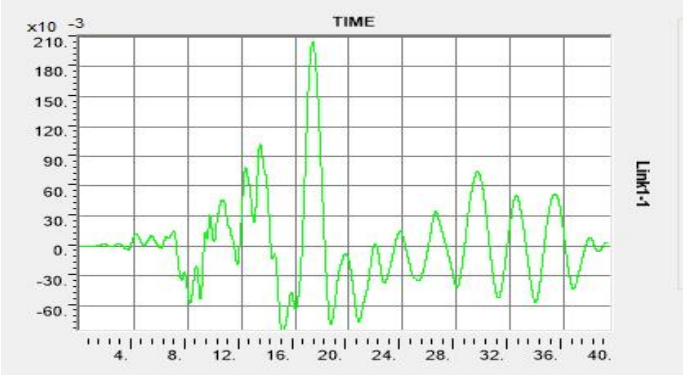


Figure-9 Time vs Displacement of Isolator showing recentering capability

CONCLUSIONS

1) Introducing Triple Friction Pendulum bearing leads to increase in fundamental time period of structure roughly 30% higher than fixed base condition.
2) Base shear was reduced to 90%, due to reduction in stiffness of structure by providing base isolation.
3) Storey drift was reduced to great extend in base isolated structure.

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DIFFERENT TECHNIQUES FOR ON SITE CORROSION MEASUREMENTFOR STEEL STRUCTURES – A REVIEW

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ABSTRACT
It is impractical to disprove that the steel has the superior position over any other construction materials. Steel usually gets decayed when interactswith the environmental forces. Basically corrosion is denominated as the degeneration or the deterioration of metal or steel, when metal interacts with the environment. The existence of very tiny or small amount of electrolyte on the unprotected exposed surface of the metal can cause the motion of electrons from anode to the cathode encouraging corrosion. Corrosion of metal or steel is the main reason that sources the destruction and rapid premature failure of the structure. The corrosion rate of steel relies on the several components of environment. Assessment of corrosion rate becomes the most important factor for the structural designers. The prolongation and restoration of the building structures and bridges demand essential inspection surveyand different techniques for assessing the rate of corrosion. There are severalnon-destructive and electrochemical techniques of measurement of corrosion rate for RCC structures as well as steel structures. In this present study, a detailed review on different methods for assessment of corrosion rate of steel in steel structures and different applications are discussed.

KEYWORDS: Corrosion, steel structures, electrochemical techniques, non-destructive technique.

INTRODUCTION
Steel corrosion is the most devastating occurrence that is disastrous for a structure aspect and this also sources structural failure of a component. Steel, due to interdependence with the environment in the presence of moisture and oxygen, destructs the properties of metal material thereby resulting in corrosion of steel.The process of corrosion is the chemical or the electrochemical response between the metal material and the surrounded environment that leads to the degradation of metal material and the metal properties. Corrosion reduces the cross sectional area of the steel, initiating cracks and pits on the surface of the steel thereby depleting the strength of the metal.Corrosion can be detected by various ways. Among which, most common is the visual inspection system. The existing visual inspection system does not provide adequate and accurate results to define the core structural defects. Hence different electrochemical and the non destructive testing techniques has been found more appreciable in the current world scenario to expound the corrosion rates.

LITERATURE REVIEW
A general extensive research on the literature survey is carried out to specify the research work done by many researchers across the world on the behaviour of the corrosion activity on steel, the experimental methods, the corrosion measurement techniques and the corrosion protection methods. Songand Saraswathy (2007)presented a study on various electrochemical techniques and Non destructive techniques for assessing the corrosion rate in steel. Combination of various NDT measurement techniques were suggested to get best results of the corrosion state of the reinforcement bar in a certain structure. Van Dalen (2008) studied

about the atmospheric stress corrosion cracking and the relative non-destructive method. Further, the general principle of the penetrant testing has been discussed and the penetrant testing technique has been carried out.Rathi (2010) carried out the experiment by open circuit potential method to measure the rate of corrosion for tor steel, mild steel and CRS steel by selecting four concrete blocks under certain exposure conditions. Rao (2012) studied about the magnetic flux leakage technique describing the working principle, limitations, and applications of the NDT test.

Layssi et al. (2015) reported about the difference between the rapid chloride permeability test (RCPT) and the electrical resistivity of concrete. It was concluded that the resistivity test procedures are much easier and faster than the RCPT.Zaki (2015) listed a brief study about various non- destructive testing corrosion assessment methods. Acoustic Emission technique is considered to be the most effective for assessing the corrosion in steel.Hodalet al. (2017) studied different methods of corrosion measurement of steel. The factor affecting the corrosion rate of steel such as the pH of the surrounding nature, pollutant levels, time of wetness were listed in this study. Odrobinak (2018) presented a structural study on bridges in different areas to approximate the loss of corrosion in the steel plates by using Rapid NSS test.

CORROSION MONITORING ASSESSMENT

DyePenetrant Inspection (DPI) / Liquid Penetrant Inspection (LPI)

The method of dye penetration is a simple, economical, non-destructive material examination procedure to notice any surface crack, corrosion pitting etc. Dye penetration inspection is suitably employed to disclose the weaknesses that are interrupted on the surfaces of the metal. LPI technique is based on the principle of a pigmented dye or liquid penetrant is drained in to the cracks or faults on the metal surface by capillary action. Overspill surface dye penetrant is extracted from the faults after a certain dwell time and the developer is put on the surface. Developer is powder type material that behaves as a blotter and removes all the dye from the metal flaws. Liquid penetrant in fluorescent color is most widely used when extreme crack sensitivity is necessary. Very minute crack of 150 nanometer can be identified using DPI. Dye penetrant can be applied to any metallic or non metallic surfaces but it is not applicable for rough and dirty surfaces. Therefore, it is necessary to clean and prepare the surface for dye penetration inspection before testing.

Open Circuit Potential Measurement (OCP)

Open circuit potential is also termed as half cell potential (HCP) measurement and this is the most typical technique used in the timely inspection of the structures. Open circuit potential measurement is mainly based on the principle that involves assessment of corrosion ability of steel with courtesy to a standard reference electrode. Reference electrode used in this technique is copper/ copper sulfate electrode (CSE) and saturated calomel electrode (SCE), silver/ silver chloride electrode. The observation and the assessment of the corrosion in the steel structureare much needed. Apparently there exists to be numerous techniques for the observation, detection and assessment of corrosion in steel structures but there is no consent regarding which technique gives the correct accurate assessment of corrosion levels. OCP is the gauging of voltage measured between the working electrode and the reference electrode (RE). Voltmeter is engaged in the measurement of OCP. Electrometer with high impedance input is used to measure the voltage in the absence of current. The values obtained from OCP test only gives the probability of the corrosion occurred and this test cannot specify the corrosion rate of steel.

SurfacePotential Measurement (SP)

Surface potential measurement is a non-destructive technique for noticing the anodic and the cathodic regions of the metal. This measurement gives us the indirect probability of the

corrosion that will occur. In surface potential measurement technique, two reference electrodes are involved in the testing. One is the fixed reference electrode that is fixed at the symmetric point and the other one is the moveable reference electrode that is moved along the node points of the structure. High impedance voltmeter is used for measuring the potential difference of the moveable reference electrode when placed against the fixed reference electrode. Anodic area shows higher positive potential results where there is high probability of corrosion. And as high is the potential difference between the anode and the cathode regions more is the corrosion possibility of the structure.

Acoustic Emission (AE)

Acoustic emission is a most powerful non-destructive field testing techniques for investigating the habits of metal that are undergoing corrosion, stress corrosion, delamination etc. when the structure is exposed to the sudden change of stresses, this generates the acoustic elastic waves in the structure. Sudden changes in stresses include change in load, pressure, temperature that spread to the surface of metal structure and thus recorded by acoustic emission (AE) sensors.Acoustic emission sensors are the significant objects of the AE testing system. When the structure is aroused by an acoustic stress wave the AE sensor produce the electrical signal. Electromagnetic devices such as piezoelectric device, magnetostrictive devices, and capacitive microphone produce acoustic signals. These devices detect and convert the high frequency electric waves that are generated from the sudden changes in the structure directly to the electrical signal.

Tafel Extrapolation (TE)

Tafel extrapolation is an electrochemical technique used for quantifying the rate of corrosion based on Tafel slopes and magnitude of the corrosion current (i_{corr}). Rate of corrosion in Tafel extrapolation can be studied operating a elementary Tafel slope value substitution to find the current of corrosion.Corrosion current is given by Eq. (1):

$$i = i_{corr} \cdot \left(e^{S_1(E-E_{corr})} - e^{-S_2(E-E_{corr})}\right) \quad \text{Eq. (1)}$$

where, $i$is the corrosion current at any time,$i_{corr}$is the corrosion current, $S_1$is the slope of anodic branch (2.303/β_a), $S_2$is the slope of cathodic branch (2.303/β_c), $E$is corrosion potential at any time, E_{corr} is the corrosion potential, $\beta_a$is anodic Tafel constant, β_c is the cathodic Tafel constant.Further corrosion rate in µm/yr is obtained by Eq. (2)

$$Corrosionrate = \frac{0.129 i_{corr} \text{ .E.W}}{\text{dA}}$$

Eq. (2)

Tafel extrapolation technique can quantify very low rates of corrosion and corrosion rates of structure can be continuously examined by this technique.

Magnetic Flux Leakage (MFL)

This is the NDT techniques used in the monitoring of corrosion that employs electromagnetism principle to detect the flaws or decay in the structural steel. Corrosion or pitting of steel is diagnosed using magnetic flux or transverse field inspection (TFI). This technique has high speed of inspection. Tough magnets are employed to magnetize a metal structure with tough magnetic fields. If any defects are detected on the field distortions will be indicated on the magnetic flux, showing up the existence of pitting, metal loss or corrosion of steel. Metal flux leaks from the steel where ever the cracks or corrosion is detected.

Eddy Current Testing

Eddy current testing is an electromagnetic non destructive testing technique employed that is used to discover and identify the sub surface and surface cracks or corrosion on the metal material. This technique is based on the principle of electromagnetic induction. Eddy current testing device is incorporated with an AC current source and a coil attached to this AC current source. A coil conducting an alternating current is held very near to the metal surface or wrapped around the metal. Voltmeter is employed in this testing to compute the change in the voltage across the coil. Use of Ammeter can also be made in the place of voltmeter for evaluating the change in current. The AC currents produce Eddy current in the structure close to the surface. Maximum rise in AC current develop the magnetic field and as the current falls down to zero, the magnetic field is collapsed.

Visual Inspection

Visual inspection is the most ordinary and familiar NDT technique for monitoring of corrosion. A well up skilled expert investigator can carry out a very functional assessment of the structure visually just by using very simple tools like torch, binoculars, magnificent glass. Visual inspection is executed once in a month, every six months or once every year depending on the significance of the structure. Visual inspection is the first step in any maintenance or the quality check procedure of the structure. This is not suitable for very minute flaws or any major defects in the structure. If the problem is seriously extreme and visual inspection is no more significant foe the assessment of the problem than more complex techniques are involved in the structural inspection.

CONCLUSION

In the present review, different techniques for corrosion measurement are discussed.Assessment of corrosion rate becomes the most important factor for the structural designers. The prolongation and restoration of the building structures and bridges demand essential inspection survey and different techniques for assessing the rate of corrosion.Visual inspection is the basic and first step in assessing the corrosion and cracking of the structures. Based on this report, one can decide the method of corrosion and crack measurement. Dye penetration inspection technique and half cell potential measurement techniques are most used techniques for corrosion and crack width determination. Acoustic emission is also a most powerful non-destructive technique for field measurement of corrosion.

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ANALYSIS OF BEHAVIOR OF REINFORCED CONCRETE BEAM LAMINATED BY FIBER REINFORCED POLYMER SHEET

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ABSTRACT

One of the major causes affecting the long term performance of reinforced concrete structure is corrosion of steel reinforcement. This Research is to find the impact corrosion on RC element as well as developing ways to inhibit rebar corrosion and also strengthening of RC element using Fiber Reinforced Polymer (glass and carbon) sheets. CFRP & GFRP sheets however have property of acting as barrier for chloride ions and carbon dioxide molecules to prevent corrosion of the reinforcement bars and strengthening RC structures.

An Experimental study on corrosion inducement in large scale reinforced concrete beams by impressed current technique and their strengthening using Glass Fibre Reinforced polymer sheets is presented. 5 Beams (1 uncovered + 5 covered with CFRP & GFRP mats having different pattern). Corrosion in beam have been introduced by impressed current technique, where the beams will be exposed to a DC current of 15 V for a period of 150 hours in 5% NaCl solution and degree of corrosion of RC beam have been noted for different pattern of FRP sheets. RC beams, have been subjected to Nondestructive Test (Rebound hammer and Ultrasonic pulse velocity test) and Destructive test (center point load flexure test).

New techniques emerge daily for rehabilitation or strengthening, one of these involves externally bonding of fiber reinforced polymer (FRP) laminates to the structural member. Interest in this technique has been wide-spread and continues to flow because of their high tensile strength to weight ratio, high resistance to fatigue and corrosion, low-cost, negligible clearance loss, high durability and ease of installation. Scores of studies have shown that rehabilitation or strengthening with FRP laminates is successful in restoring or increasing the strength of concrete members.

KEYWORDS: Glass fibre reinforced polymer, Carbon fibre reinforced polymer corrosion, reinforced concrete.

INTRODUCTION

Composite materials made of fibers in a polymeric resin, also known as fiber-reinforced polymers (FRPs), have emerged as an alternative to traditional materials for repair and rehabilitation. For the intents of this document, an FRP system is determined as the fibers and resins used to create the composite laminate, all applicable resins used to stick it to the concrete substrate, and all applied coatings used to protect the component materials. Coatings used exclusively for aesthetic reasons are not considered part of an FRP system. FRP materials are lightweight, noncorrosive, and exhibit high tensile force. These fabrics are readily usable in various varieties, ranging from factory-made laminates to dry fiber sheets that can be wrapped to conform to the geometry of a structure before adding the polymer resin [10, 12].

The magnitude of reinforcement corrosion has a significant effect on flexural strength, deformational behaviour, ductility, bond strength, and mode of failure of the reinforced-concrete structures. As the result,the degree of corrosion is considered as one of the main parameters to predict the useful service life of corroding reinforced-concrete structures. Due

to the adverse effect of environmental factors, increased structural load requirements, etc, reinforced concrete structures degrade day by day. The corrosion of steel reinforcement can lead to the premature failure in the structural members. After corrosion initiation, its progression forms additional corrosion products that have an increase in volume compared to the non corroded metal. Longitudinal corrosion cracks are formed along the corroding reinforcing bar when the tensile stress in the concrete surrounding the reinforcing bar exceed the tensile strength of the concrete. The cracking ultimately results in the spalling and delamination of the concrete. Reinforcement bars in concrete beams possess a passive film around them due to the highly basic medium of concrete. Corrosion occurs when this passive layer encounters a break down due to chloride ions in de-icing salts or due to neutralization of concrete by atmospheric carbon dioxide. Reinforced concrete flexural members such as bridges, slabs, beams that have been damaged due to to the corrosion of steel reinforcement are often rehabilitated with fibre reinforced polymer (FRP) composites. This research investigates passive protection of the steel reinforcing bar in RC flexure members that are treated with surface bonded FRP i.e.Carbon & glass fibre reinforced polymer. This research synopsizes the effect of different factors on the extent of corrosion as well as developing ways to inhibit rebar corrosion using Glass Fibre Reinforced Polymer (FRP) sheets. The GFRP sheets have been wrapped using different patterns and the efficiency of each pattern is investigated.[9,13].The magnitude of reinforcement corrosion has a significant effect on flexural strength, deformational behaviour, ductility, bond strength, and mode of failure of the reinforced-concrete structures. As the result, the degree of corrosion is considered as one of the main parameters to predict the useful service life of corroding reinforced-concrete structures. Due to the adverse effect of environmental factors, increased structural load requirements, etc, reinforced concrete structures degrade day by day. The corrosion of steel reinforcement can lead to the premature failure in the structural members. After corrosion initiation, its progression forms additional corrosion products that have an increase in volume compared to the non corroded metal. Longitudinal corrosion cracks are formed along the corroding reinforcing bar when the tensile stress in the concrete surrounding the reinforcing bar exceed the tensile strength of the concrete. The cracking ultimately results in the spalling and delamination of the concrete. Reinforcement bars in concrete beams possess a passive film around them due to the highly basic medium of concrete. Corrosion occurs when this passive layer encounters a break down due to chloride ions in de-icing salts or due to neutralization of concrete by atmospheric carbon dioxide. Reinforced concrete flexural members such as bridges, slabs, beams that have been damaged due to to the corrosion of steel reinforcement are often rehabilitated with fibre reinforced polymer (FRP) composites. This research investigates passive protection of the steel reinforcing bar in RC flexure members that are treated with surface bonded FRP i.e. glass fibre reinforced polymer. This research synopsizes the effect of different factors on the extent of corrosion as well as developing ways to inhibit rebar corrosion using Glass & carbon Fibre Reinforced Polymer sheets. The GFRP & CFRP sheets have been wrapped using different patterns and the efficiency of each pattern is investigated[9,13].

LITERATURE REVIEW

A. Research Background

Bonacci J et.al. (1998), the study concluded that the simulation of corrosion in large-scale reinforced concrete columns and their repair using carbon fibre reinforced polymer (CFRP) sheets is presented. Seven columns were subjected to an accelerated corrosion regime, wrapped using CFRP sheets, then tested to structural failure and (or) subjected to further post- repair accelerated corrosion, monitoring, and testing. Accelerated corrosion had been achieved by adding sodium chloride to the mixing water, applying a current to the

reinforcement cage, and subjecting the specimens to cyclic wetting and drying. Results showed that the CFRP repair greatly improved the strength of the repaired member and retarded the rate of post-repair corrosion [1].

Dolan, Cet.al. (1999), the study concluded that Strengthening of reinforced and pre-stress concrete (RC and PC) members using externally bonded FRP laminates is today a well-accepted technology that is becoming popular among designers and contractors. Near-surface mounted FRP reinforcement represents an alternative way to improve the flexural and shear performance of concrete structures. In some instance, it is the only suitable technology that can be efficiently applied, for example, when upgrading beam-column joints or for the flexural strengthening of compression members. In this paper, bond related issues, flexural and shear design recommendations, and design examples are discussed and proposed. The paper is an attempt to provide designers with the first comprehensive protocol for the rational implementation of the technology [2].

K.A. Saudki et al. (2000), the study concluded that the viability of carbon fibre reinforced polymer (CFRP) laminates for the strengthening of corrosion damaged reinforced concrete bridge girders is addressed in this paper. Ten reinforced concrete beams (100 × 150 × 1200 mm) with variable chloride levels (0-3%) were constructed. Six beams were strengthened by externally epoxy bonding CFRP laminates to the concrete surface. The tensile reinforcements of three un strengthened and four strengthened specimens were subjected to accelerated corrosion by means of impressed current to 5, 10, and 15% mass loss. Strain gauges were placed on the CFRP laminates to monitor and quantify tensile strains induced by the corrosion process. Following the corrosion phase, the specimens were tested in flexure in a four-point bending regime. Test results revealed that CFRP laminates successfully confined the corrosion cracking, and the total expansion of the laminate exhibited an exponential increase throughout the corrosion process. All the strengthened beams exhibited increased stiffness over the unstrengthened specimens and marked increases in the yield and ultimate strength. The CFRP strengthening scheme has been able to restore the capacity of corrosion damaged concrete beams up to 15% mass loss [3].

C .Lee& J.F. Baonacci (2000), the study concluded that the simulation of corrosion in large-scale reinforced concrete columns and their repair using carbon fibre reinforced polymer (CFRP) sheets is presented. Accelerated corrosion has been achieved by adding sodium chloride to the mixing water, applying a current to the reinforcement cage, and subjecting the specimens to cyclic wetting and drying. Results showed that the CFRP repair greatly improved the strength of the repaired member and retarded the rate of post-repair corrosion. Moreover, subjecting the repaired column to extensive, post-repair corrosion resulted in no loss of strength or stiffness and only a slight reduction in the ductility of the repaired member. The repair of corrosion-damaged columns using CFRP sheets has been shown to have the following benefits: (1) It greatly improved the strength and ductility of the corroded member, increasing the load-carrying capacity of the corroded column by 28%. (2) It impeded post-repair corrosion, resulting in a 50% decrease in the rate of corrosion after wrapping and a similar decrease in the natural corrosion currents. (3) Even when subjected to significant post-repair corrosion, the structural performance of the repair system has been not compromised [4].

Subhash C Goel et. al. (2000) concluded that the performance of the rc research on new materials, elements and systems concluded that research work carried out in both states in developing and application of novel and innovative materials for use in advanced composite structural elements and systems (Research For Innovation - RFI) under the US-Japan Cooperative Earthquake Research Program on Composite and Hybrid Structures. The research work includes Fibre Reinforced Plastics (FRP), and High-Performance Concretes

(HPC). Use of FRP materials has been studied in RC-FRP panels, fixing and toning up of existing RC members by FRP, and use of FRP in electrical installations [5].

C.E. Bakis et.al (2001) study State-of-the-Art Review states that a concise state-of-the-art survey of the fibre-reinforced polymer also known as fibre -reinforced plastic. Composites for construction application in civil engineering is awarded. The report is formed into separate sections on structural shapes, bridge decks, internal reinforcements, externally bonded reinforcements, and standards and codes. Each segment includes a historical review, the current state of the art, and future challenges. This paper attests to the many potential applications of FRP composite materials in construction, although the demand for brevity prevents all topics from being fully addressed. It can be stated that the amount of experience with diverse kinds of FRP construction materials varies in conformity with the perceived near-term economic and safety benefits of the cloths. In the example of externally bonded reinforcements, for example, the immediate cost and safety benefits are clear, and adoption of the material by industry is widespread. In other cases where FRP materials are believed to be primary load-carrying parts of social organizations, field applications still maintain a research flavour while long-term experience with the material accumulates. Standards and codes for FRP materials and their function in the structure are either published or currently being written in Japan, Canada, the United States, and Europe. These official documents are typically similar in format to conventional standards and codes, which should facilitate their acceptance by governing agencies and governing bodies. The most significant mechanical differences between FRP materials and conventional metallic materials are the highest strength, lower stiffness, and linear-elastic behaviour to the failure of the former. Other differences such as the thermal expansion coefficient, moisture absorption, and high temperature and fire resistance need to be seen as good [6].

Tamer A,EI Maaddawy & Khaled A. Saudki (2003**),**study concluded that Accelerated corrosion by means of the impressed current technique is widely used in concrete durability tests. In this study, the influence of varying the impressed current density level between 100 and 500 mA/cm2 on the actual degree of steel reinforcing bar corrosion as well as on the concrete strain behaviour due to expansive corrosion products has been experimentally investigated. Twelve reinforced-concrete prisms (150*250*300) mm were used. The prisms were reinforced by two No. 10 reinforcing bars. Corrosion has been induced by means of impressed current using electric power supplies. To depassify the steel reinforcement, 5% NaCl by weight of cement has been added to the concrete mix. The strain response due to the expansion of corrosion products has been measured at each face of the prisms. At the end of the corrosion phase, all the corroded reinforcing bars were removed, cleaned according to the ASTM G1-90 standard, and weighed to get the actual degree of mass loss. The results showed that, up to 7.27% mass loss, accelerated corrosion using the impressed current technique has been effective in inducing corrosion of the steel reinforcement in concrete. With respect to Faraday's law, the use of different current densities has no effect on the percentage of mass loss. However, increasing the level of current density above 200 mA/cm^2 results in a significant increase in the strain response and crack width due to corrosion of the steel reinforcement [7].

Markus Bergstrom (2006), the study concluded that How do existing loads during repair and strengthening affect the structure? The research carried out and presented in this thesis clearly shows that the repaired structural concrete member is affected by an existing strain field, and this could severely decrease the load carrying capacity. Does the stiffness of the structural member decrease due to deterioration issues? The test shows that the stiffness of the structural member is heavily affected by steel corrosion. The test has shown that strengthening increased the bending stiffness. This conclusion is based on both local and global stiffness measurements. How can the load carrying capacity be determined by a

probabilistic approach? A probabilistic evaluation is a very strong tool in estimating the load carrying capacity of deteriorated, repaired and strengthened concrete structures [8].

Sangeeta Gadve et.al. (2010), the study concluded that Corrosion Protection of fibre reinforced Polymer-Wrapped reinforced concrete. Reinforced concrete (RC) flexure members such as bridges, slabs, and beams that have been damaged due to the corrosion of steel reinforcement are often rehabilitated with fibre-reinforced polymer (FRP) composites. This paper investigates passive as well as active protection of the steel reinforcing bar in RC flexure members that are treated with surface bonded FRP. Passive protection is investigated using glass FRP (GFRP) and carbon FRP (CFRP) sheets that have been adhesively bonded to the beam specimens. Active protection has been studied using electrically conductive carbon fibre wraparounds as an anode while the reinforcing bar is used as a cathode. Accelerated corrosion is inflicted in RC beams by immersing them in saltwater and passing anodic current through the reinforcement. For investigating active protection, the adhesive has been modified to impart electrical conductivity. Specimens were exposed to a highly corrosive environment for a specified time. Flexure strength, mass loss, and half-cell potential are reported as metrics of performance of the samples. The proposed technique has been very effective in retarding the corrosion of steel. This paper investigates the passive as well as active protection of the steel reinforcement in FRP-wrapped flexure members. Passive protection is achieved by creating a barrier layer by refuting the ingress of corrosive elements to the reinforcement by the FRP wraps. Active protection has been successfully achieved using electrically conductive CFRP wrap as an anode. The flexure capacity of the protected samples was found to be much higher than the control samples. The actively protected sample exhibited maximum capacity and the lowest mass loss. The loss of bond between steel and concrete that has been observed in the tension specimens in the previous investigation is overcome by avoiding overprotection by selecting the appropriate protection current density [9].

D. Baggio et al. (2014**),** concluded that the use of fibre reinforced polymer (FRP) sheets to shear strengthen reinforced concrete (RC) beams has been studied and proven repeatedly. However, numerous strengthening configurations and materials can be combined to maximize the increase in strength and repair. This study investigates the effectiveness of using commercially manufactured carbon FRP (CFRP), glass FRP (GFRP) and fibre reinforced cementitious matrix (FRCM) sheets to increase the shear capacity of RC shear critical beams. Nine RC shear deficient slender beams were tested. The test variables included the use of three different FRP (CFRP, GFRP, and FRCM), and the presence and type of FRP anchors (CFRP or GFRP). The experimental results revealed that applying FRP sheets increased the overall shear capacity, full depth u-wrapped FRP sheets perform better over companion partial depth u-wrapped FRP sheets, the use of FRP anchors further improved the shear capacity and ductility of failure, FRP strengthening could change the mode of failure from a shear to flexural failure and FRP debonding has been delayed with the presence of FRP anchors. A comparison of the experimental load capacities with predicted values indicates that they correlate well with predictions using the current Canadian design codes. The result shows that Beams strengthened with partial depth GFRP sheets with and without anchors showed a 52% and 36% increase in shear capacity, respectively, over the control beam and the presence of the FRP anchors has been effective in halting the debonding of the GFRP sheets. Beams strengthened with CFRP sheets with and without anchors showed a 67% and 75% increase in shear capacity over the control beam, respectively, with both beams exhibiting a ductile flexural failure mode [10].

Ji Hua Zhu et al. (2014), The study concluded that An investigation has been performed by using carbon fibre-reinforced polymer (CFRP) as the anode material in the impressed current cathodic protection (ICCP) system of steel reinforced concrete structures. The service life

and performance of CFRP were investigated in simulated ICCP systems with various configurations. Constant current densities were maintained during the tests. No significant degradation in electrical and mechanical properties has been found for CFRP subjected to anodic polarization with the selected applied current densities. The service life of the CFRP-based ICCP system has been discussed based on the practical reinforced concrete structure layout. A comprehensive experimental program has been carried out to study the electrical and mechanical behaviours of a CFRP plate in simulated ICCP systems with varied solutions [11].

Kolluru Hemanth Kumar et. al (2015), study shows that Referable to the adverse effects of the surroundings along the social system, reinforcement corrosion in concrete may cause detrimental effects viz. The decrease of the effective area of support which ultimately equates to strength reduction, internal stress caused by rust products formed during corrosion consequently lead to cracking and spalling of concrete, reduction of bond strength and change in the bond-slip behaviour in between concrete and steel reinforcement. Hence the prevention of corrosion in steel reinforcement of RC structures has been an area of prime concern for researchers in the past few years. The present paper deals with the influence of GFRP on corrosion of steel reinforcement in RC beams. The experiment includes construction of ten (150×150×700 mm) reinforced concrete beams. Eight beams were strengthened by externally epoxy bonding of GFRP on concrete surface in different patterns. The reinforcements of four strengthened beams (one of each pattern) and one un strength end beam has been exposed to accelerated corrosion by the impressed current method. After the corrosion phase, the beams were subjected to three-point bending. The beam has been covered in FRP only at the bottom and the Flexural strength without exposure increased by 82.61% (Table-1) as compared to uncovered beam thus it could be concluded that using FRP on the tensile side is pretty effective for flexural strengthening. Another bottom covered beam has been then exposed to corrosive environment and the theoretical corrosion percent in this case came out to be 17% which has been lower than that in plain beam but its strength as compared to non-exposed bottom covered beam decreased by 32.57% because the FRP debonded due to severe cracks developed just at the interface of covering. Thus, it could be concluded that only tensile face being covered is effective for strengthening but not for corrosion inhibition [12].

Nabeel Shaikh & Sheetal Sahara (2016), study shows “Effect of impressed current on corrosion of Reinforcing bar in Reinforced concrete” International Journal of Advances in Mechanical and Civil Engineering states that Corrosion of steel reinforcing bar is one of the major causes of premature deterioration reducing service life of reinforced concrete (RC) structures. This increases maintenance and repair cost of the RC structures. In RC structures, corrosion of steel in natural condition is a very slow process. Hence generally accelerated corrosion techniques are used in the laboratory to simulate natural corrosion process. An experimental work has been carried out to study the effect of impressed current on corrosion of steel reinforcing bar in concrete. In this experiment, three different voltages 4V, 6V and 10V were used and have been impressed on the reinforced bar in concrete. A cylindrical specimen of 100mm diameter and 200mm height reinforced with bar diameter 16mm and 20mm concentrically were used and studied for the effect of different voltages applied [13].

Rahul Dubey & Saurabh Samander (2016), in their experimental analysis on steel corrosion in RCC structures by Impresses Current Technique, using FRP Mats to inhibit Corrosion concluded that equally the scope of applications for fibre-reinforced polymer (FRP) sheets in civil engineering constantly increases, there is a lot of concern with relevance their performance in essential environments the fireplace behaviour of composite materials is very necessary since advanced physical and chemical processes like the glass transition and decomposition occur once these materials area unit subjected to elevated and high

temperatures, probably resulting in extensive loss of stiffness and strength. This article shows a study on FRP sheets as a corrosion inhibition of concrete [14].
Shahzeb Bardii & Shilpa Patil (2016), the study concluded that Effect of FRP on corrosion on Reinforcing bar in reinforced concrete has been analysed. From the experimental results obtained, it can be concluded that application of FRP wrap is effective in controlling the corrosion. It helps in reducing the reinforcement mass loss and also the deterioration of the concretestructure [15].

B. Research thrust

As discussed in reviewed literature many researchers have discussed about inhibition of corrosion using fibre reinforced polymer mats i.e. Glass, Carbon and other etc. Many researchers have tried wrapping of GFRP and CFRP mats with different patterns on RCC structural elements (beam, column etc.) and structures were subjected to flexure test and calculation of mass loss of steel embedded in concrete. The corrosion in steel embedded in concrete has been achieved using the Impressed current technique. Many types of research have been done to inhibit corrosion by active protection and passive protection both. The ultimate target of each researcher has been to inhibit corrosion, strengthen existing corroded structures, improve flexural strength, compressive strength etc. After reviewing literature, gap has been found that work has not done on some patterns of wrapping of GFRP mats over RCC members and compressive strength has not determined using rebound hammer test and ultrasonic pulse velocity test, specific cube strength, Elastic modulus (static and dynamic both) has not determined using Ultrasonic pulse velocity test.

METHODOLOGY

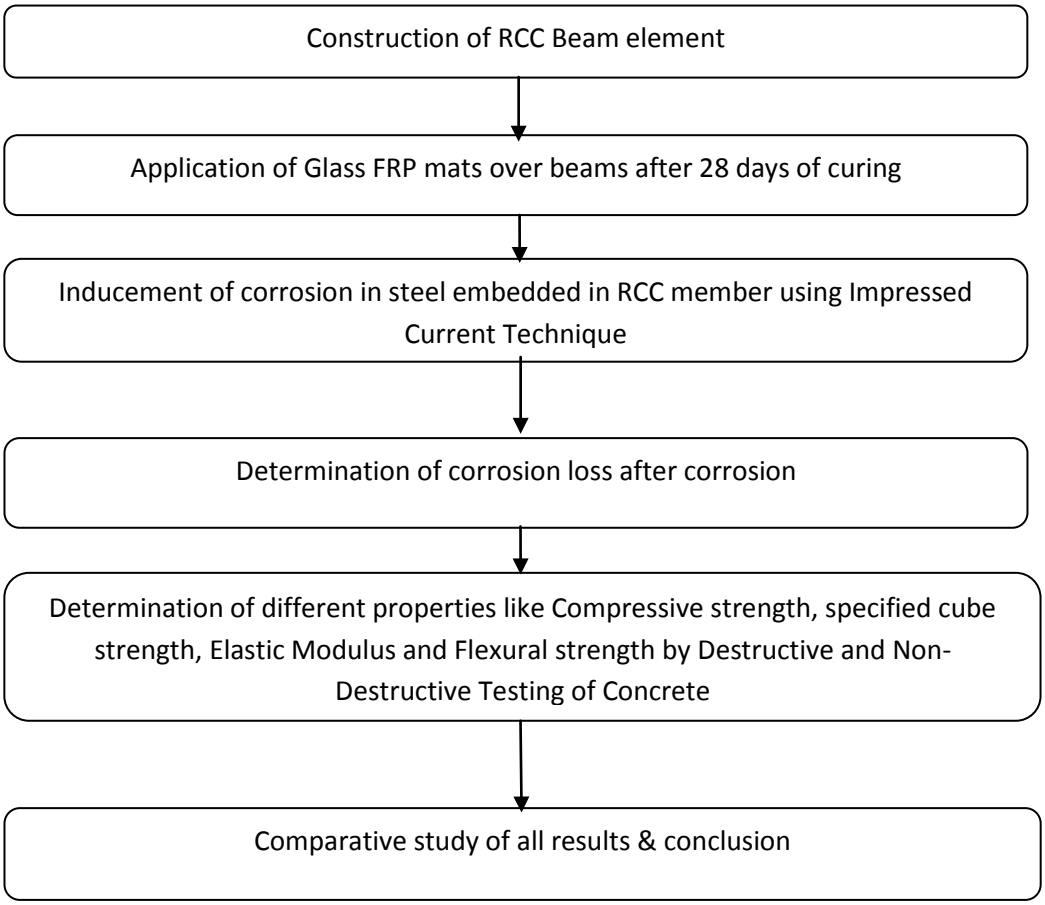


Figure 1: Flow Chart of Methodology

Research have been gone through no. of phases, it includes construction of

1. Construction of RCC Beam
   a. Selection of Material- Pozzolana Portland Cement (PPC) conforming IS 1489 (part – I) have been used in this research along with river sand as fine aggregate and crushed stone as coarse aggregate conforming IS 383, clear water free from suspended particles (conforming IS 456) have been used for this research.
   b. Concrete mix design – concrete mix of M30 concrete have been designed as per recommendation given under IS 10262: 2009.
   c. Structural design - Structural design of the beam is done as per recommendation given under IS 456 : 2000 by limit state method (LSM) and specification of beam are two steel bars each of length 65 cm and diameter 8 mm (Fe 500) at the bottom along with two steel bars each of length 65 cm and diameter 8 mm (Fe 500) at the top, Four, 2-legged stirrups of 8 mm diameter (Fe 500) are placed at equal distance from center to center, clear cover at all the sides was set at 20 mm.
   d. Casting – 72 Reinforced beam of size 15×15×70 cm is casted.
   e. Curing – Curing is done in clean water at room temperature.
2. Covering or wrapping of the beams with FRP sheets 4 mm thick (glass and carbon) in 6 different patterns listed below and applying of FRP sheets contain different steps i.e. grinding of beams, applying primer on surface of beam, cutting FRP, mixing and applying of resin, wrapping or covering of beam with selected pattern.

Table 1: FRP Patterns Applied on Beams

S.No.	FRP Pattern	Pattern Name
1	Uncovered Beam.	A
2	Only side covered with FRP.	B
3	Only bottom covered with FRP.	C
4	Bottom completely covered, & sides covered upto 5 cm with FRP.	D
5	Bottom completely covered, & sides covered upto 10 cm with FRP.	E
6	Bottom and sides completely covered with FRP.	F

3. Corrosion in reinforced bar is introduced by impressed current technique. Impressed current technique have been used to accelerated corrosion in beams. A Plastic water tank meant for filling up water and immersing the beam into it up-to a certain depth, The depth of water chosen was 7.5 cm i.e. half the depth of the beam was immersed into water, The water was then mixed with Sodium Chloride so as to prepare a 5% solution as recommended in the standard testing procedure, DC power supply of capacity 30V, 4A was used as the source of current, The reinforcement bars of the beams protruding outwards were then connected to the positive terminal of the DC power supply, thus making it the anode, A stainless steel hollow rod in L-shape was used as the cathode and thus connected to the negative terminal of the power supply The power supply was maintained at 30V, 20V subsequently during pilot study and it was observed that the current values were exceeding the design limit in these cases, thus a constant voltage of 15V was finalized to be applied on a sample beam and it was observed that significant cracks were developed after a time of nearly 145 hours at this voltage on uncovered beams, Thus, all the beams were decided to be applied for corrosion at 15V for a time of 150 hours thus, generating similar conditions for all the beams, wrapped in varying FRP patters so that the corrosion inhibiting properties of each pattern could be studied, In general, wet and dry cycles need to be given during corrosion, i.e., the beams need to be taken out of the water after 6-7 hours of corrosion so as to replenish oxygen that is necessary for effective corrosion. To avoid this situation, a diffused aerator of the type used in aquariums was used so as to

pump air into the water thus replenishing the oxygen content and minimizing the need for wet-dry cycles, Also, it is generally observed in similar studies that in accelerated corrosion mechanism, the protruding bars detatch just at the surface of the beam due to excess content of oxygen. To avoid this condition, the surface of beam at the location where the bars were protruding, was covered with araldite epoxy.

4. Beams laminated with CFRP & GFRP haven tested by non-destructive test like rebound hammer test & ultra-sonic pulse velocity test to carry out compressive strength, cube strength along with elastic modulus of concrete and flexural strength test have been performed on beams to evaluate flexural strength. All these beam have been tested after 28 of curing and result have been carried out before and after corrosion.

RESULT & DISCUSSION

A. Corrosion

Corrosion in beam is introduced by impressed current technique, total mass of dipped iron was calculated by taking the average mass of 8 mm bars as 0.401 kg/m run and it came out to be 1042 gram for four bars of 650mm long, 15V current is passed through beam for 150 hours and The mass of rust produced per unit surface area of the bar due to applied current over a given time can be determined theoretically using the following expression based on Faraday's law M_{th}= (W*I*T)/F

Where,

M_{th}= theoretical mass of rust per unit surface area of the bar (g/cm^2);

W = equivalent weight of steel which is taken as the ratio of atomic weight of iron to the valency of iron (27.925 g);

I_{app}= applied current density (Amp/cm^2);

T = duration of induced corrosion (sec);

and F = Faraday's constant (96487 Amp-sec).

table 2 & figure 2 shows result of corrosion induced in beam laminated by GFRP and CFRP in 5 patterns along with one uncovered beam.

Table 2: Result of Corrosion in Beams

Pattern	GFRP		CFRP	
	Theoretical Weight loss	Practical Weight loss	Theoretical Weight loss	Practical Weight loss
A	25.56%	22.46%	26.54%	22.46%
B	23.65%	18.67%	24.84%	20.09%
C	19.41%	14.32%	22.37%	16.24%
D	16.25%	10.65%	19.64%	12.15%
E	15.37%	9.77%	18.55%	10.87%
F	11.88%	5.64%	16.32%	9.59%

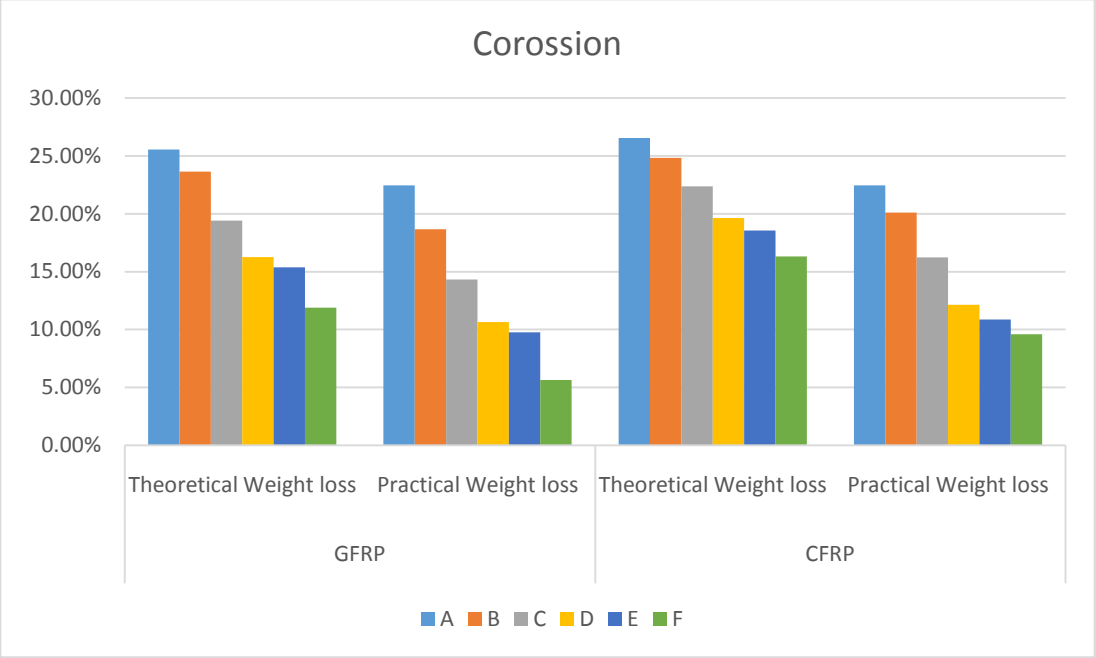


Figure 2: Result of Corrosion in Beams

B. Rebound Hammer Test

Rebound hammer test have been performed on beams to evaluate compressive strength of concrete.

The compressive strength of different beams having different patterns of Glass FRP and Carbon FRP has been calculated using Rebound hammer. Rebound hammer provides rebound no. corresponding to the hardness of the surface of the beam, more the hardness of the beam's surface more the compressive strength of the beam. Beam's have been tested before and after corrosion and rebound no. is noted. At least 9 readings have been taken at each surface of the beam and then the average of these readings has been calculated and corresponding to this average rebound no. compressive strength is calculated using Fig provided with Instrument. Table 3 and figure 3 shows result of compressive strength of concrete before and after corrosion after 28 days of curing by rebound hammer test.

Table 3: Compressive Strength of Beam by rebound hammer test

Pattern	CRPF		GRPF	
	Before Corrosion	After Corrosion	Before Corrosion	After Corrosion
A	34.25	30.47	34.25	30.47
B	37.74	34.12	34.81	31.35
C	38.08	35.64	36.16	33.64
D	38.96	36.33	36.93	34.47
E	39.55	37.91	37.83	35.89
F	40.67	39.11	38.21	35.06

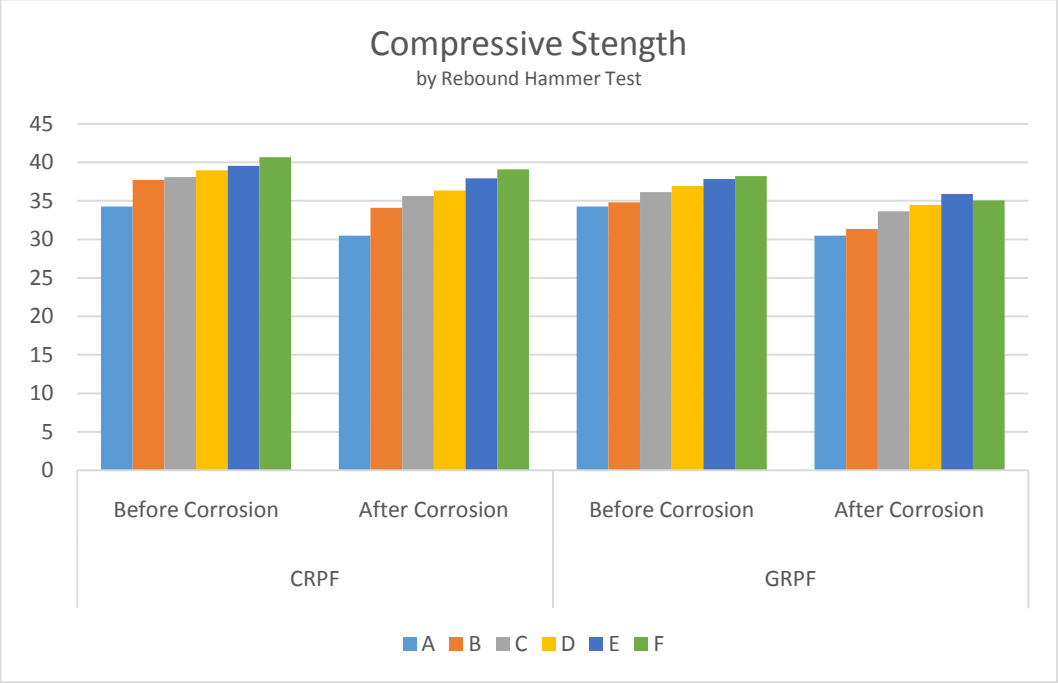


Figure 3: Compressive Strength Test by Rebound Hammer Test

C. Ultra-sonic Pulse Velocity Test

To find out specified cube strength, compressive strength and Elastic modulus it is necessary to find out first ultrasonic pulse velocity. In this study, ultrasonic pulse velocity is measured as distance travelled by pulse divided by the time taken by pulse to reach upto receiver end. Calculation of ultrasonic pulse velocity for each of the beam is measured with the help of ultrasonic pulse velocity equipment. After calculation of velocity specified cube strength of beams, compressive strength and the elastic modulus is calculated using figures. The comparative study has been carried out between results of beams of different patterns. Table 4 and figure 4 shows compressive strength of concrete of concrete laminated with CFRP and GFRP, table 5 and figure 5 shows specific cube strength of concrete, table 6 and figure 6 shows elastic modulus of concrete.

Table 4: Compressive Strength by Ultra-sonic Pulse Velocity Test

Pattern	CRPF		GRPF	
	Before Corrosion (MPa)	After Corrosion (MPa)	Before Corrosion (MPa)	After Corrosion (MPa)
A	35.08	29.64	35.08	29.64
B	35.88	31.42	36.12	30.55
C	36.15	32.56	36.47	31.55
D	36.52	33.87	36.92	32.15
E	36.94	34.21	37.34	32.85
F	37.31	34.79	37.85	33.45

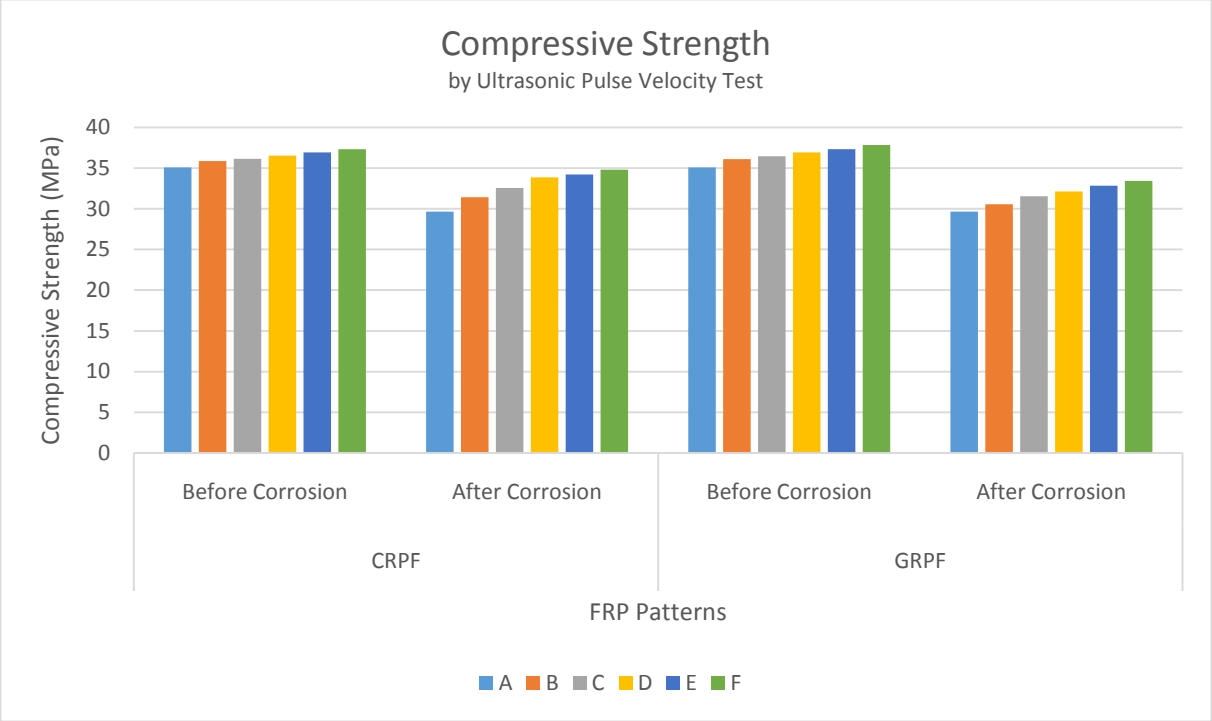


Figure 4: Compressive Strength by Ultra-sonic Pulse Velocity Test

Table 5: Specific Cube Strength of concrete by Ultra-sonic Pulse Velocity Test

Pattern	CRPF		GRPF	
	Before Corrosion (MPa)	After Corrosion (MPa)	Before Corrosion (MPa)	After Corrosion (MPa)
A	31.56	27.46	31.56	27.46
B	33.06	29.39	32.17	31.1
C	33.67	30.74	32.78	31.69
D	34.65	31.22	33.47	32.38
E	35.47	31.53	33.62	32.53
F	35.7	32.86	33.84	32.75

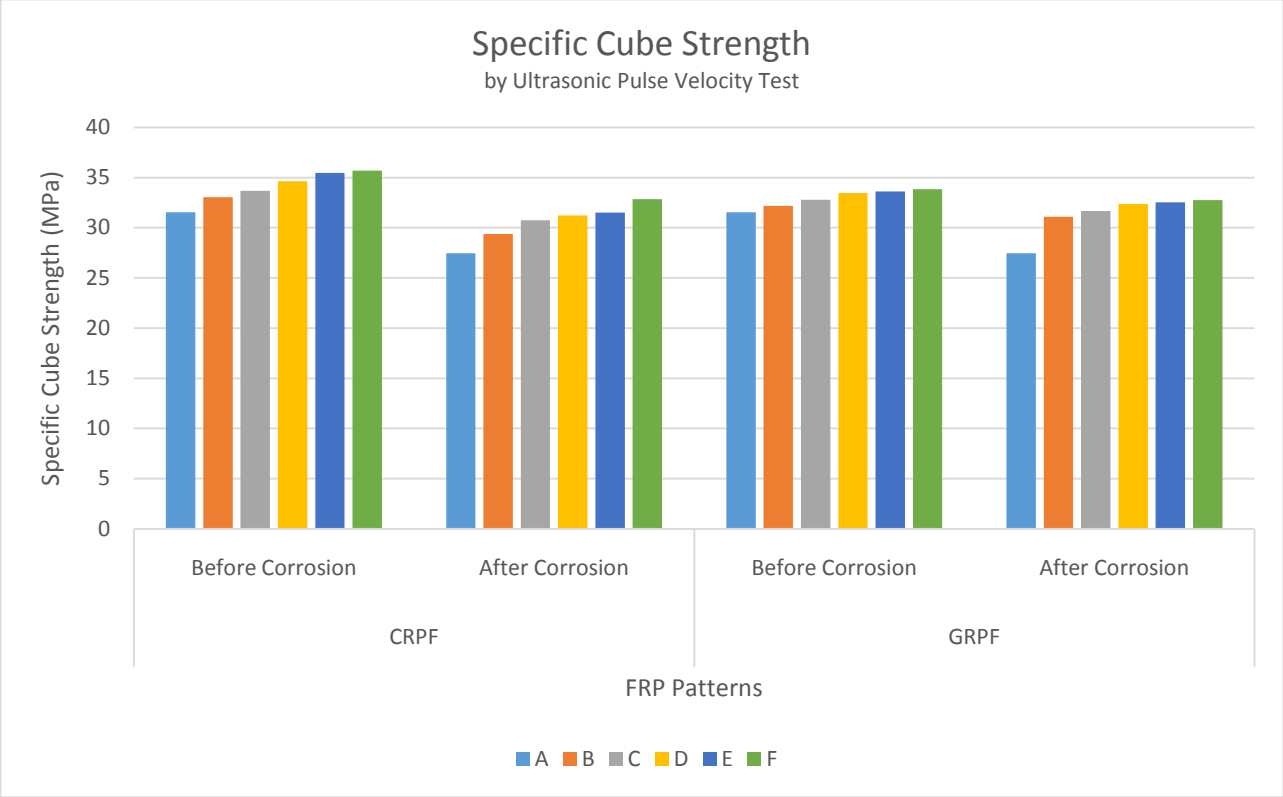


Figure 5: Specific Cube Strength of concrete by Ultra-sonic Pulse Velocity Test for different FRP Pattern

Table 6: Elastic modulus of Concrete by Ultra-sonic Pulse Velocity Test for different FRP Pattern

Pattern	CRPF		GRPF	
	Before Corrosion (kN/mm^2)	After Corrosion (kN/mm^2)	Before Corrosion (kN/mm^2)	After Corrosion (kN/mm^2)
A	3.05	2.15	3.05	2.15
B	3.21	2.49	3.1	2.21
C	3.58	2.86	3.19	2.36
D	3.97	3.22	3.36	2.59
E	4.13	3.6	3.65	2.86
F	4.25	3.79	3.79	3.12

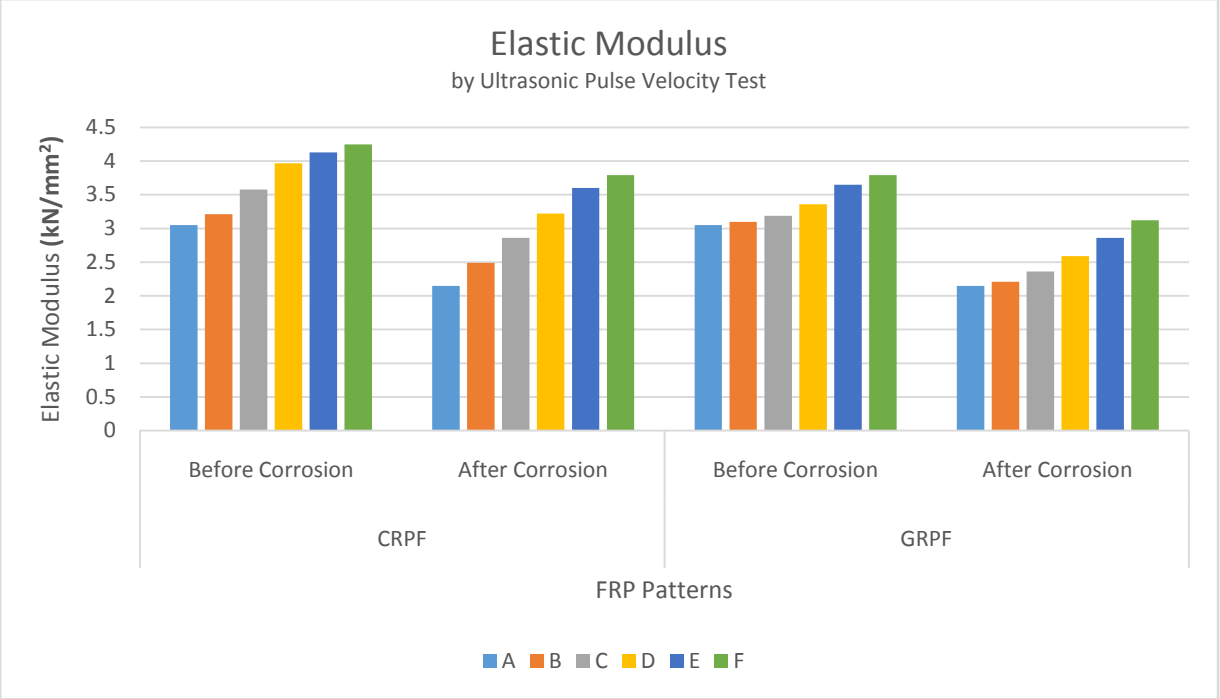


Figure 6 : Elastic modulus of Concrete by Ultra-sonic Pulse Velocity Test for different FRP Pattern

D. Flexural Strength

Flexural strength of beam have been checked by three point loading system for different patterns of FRP Sheets (Glass and Carbon) and it has been observed that carbon FRP Sheets possess better flexural strength when it is compares to Glass FRP sheets and it is also observed that flexural strength is decreased after corrosion is introduced in it, result also shows that when bottom is covered with FRP sheets, beam attain better flexural strength as compare to other FRP pattern of beam. Result of flexural strength for different FRP pattern (glass and carbon) before and after corrosion is given in table 7 – figure 7 and table 8 – figure 8 shows percentage decrease in flexural strength after corrosion.

Table 3: Flexural strength of beams for different FRP patterns.

Pattern	GFRP		CFRP	
Name	Before Corrosion	After Corrosion	Before Corrosion	After Corrosion
A	5.2	4.56	5.2	4.56
B	8.92	6.74	9.15	7.42
C	6.55	5.23	7.24	5.37
D	9.41	8.88	10.02	9.25
E	9.87	8.95	10.67	9.67
F	10.36	9.77	11.08	10.23

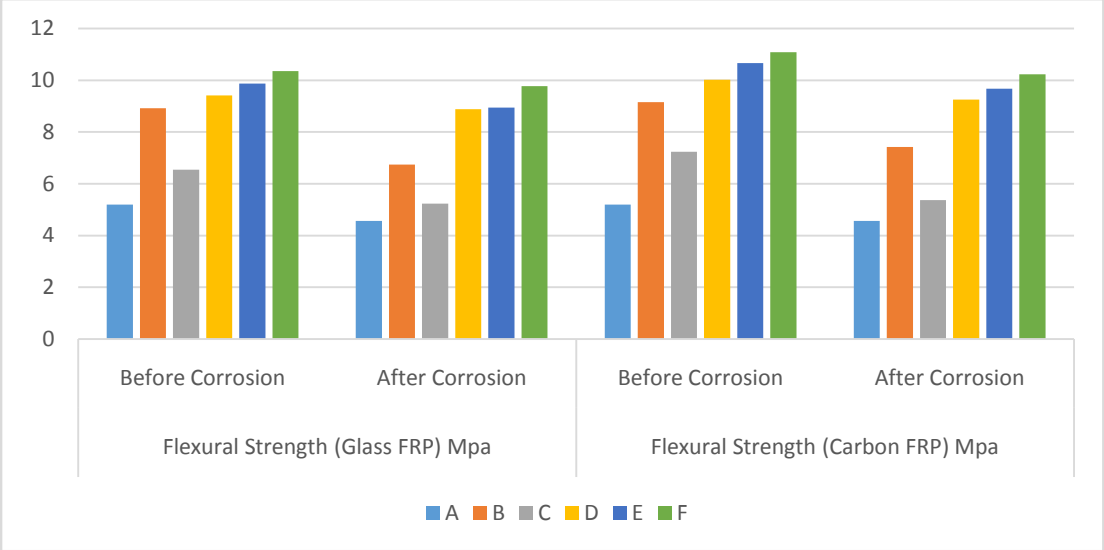


Figure 7: Flexural Strength of Beams with Glass and Carbon FRP Sheets

Table 8: Percentage decrease in flexural strength after corrosion.

Pattern Name	% Decrease after corrosion (Glass FRP)	% Decrease after corrosion (Carbon FRP)
A	12.31	12.31
B	24.44	18.91
C	20.15	25.83
D	5.63	7.68
E	9.32	9.37
F	5.69	7.67

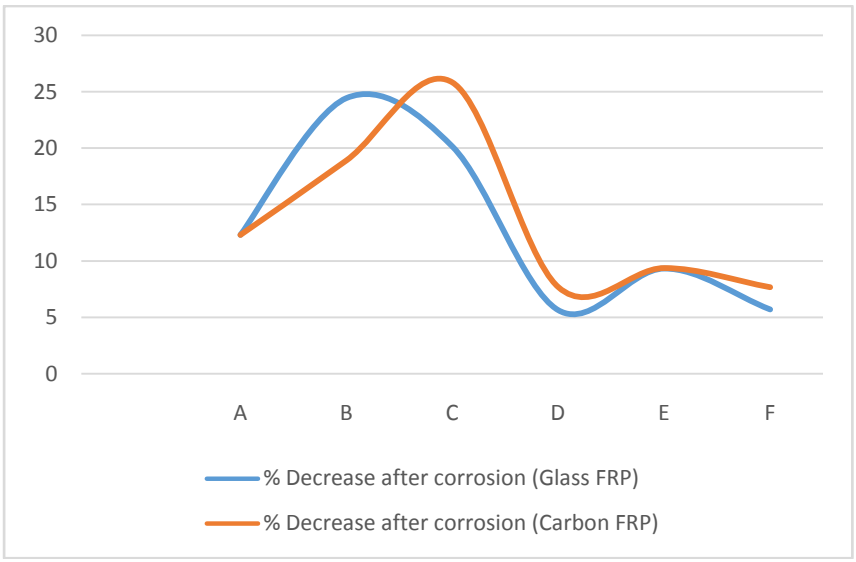


Figure 8: Percentage decrease in flexural strength after corrosion.

CONCLUSIONS

Study has been carried out on RC beams laminates with Glass and Carbon FRP Sheets, Conclusion drawn from current study is given below:

1. Result shows that when beams have been laminated by Glass FRP sheets, it has been observed that when beam is fully covered with bottom and sides have less effect of corrosion when it is compared to other patterns, like wise effect corrosion of bottom

completely covered and sides is partially covered beams is also less. Carbon FRP sheets also shows result in same format. Result also shows that Glass FRP is better in inhibiting corrosion as compare to Carbon FRP sheets, because corrosion of Glass FRP sheets reported less than Carbon FRP sheets.

2. Compressive Strength, Specific Cube Strength, Flexural strength and Elastic Modulus of laminated beams is evaluated before and after corrosion, result shows that Compressive Strength, Specific Cube Strength, Flexural strength and Elastic Modulus of the beam is decreased after introduction of corrosion in beams, but still after corrosion Carbon FRP sheets gives better result in Compressive Strength, Specific Cube Strength, Flexural strength and Elastic Modulus as compare to Glass FRP sheets. Likewise, before corrosion Carbon FRP sheets is better and strengthen beams. When it comes to pattern both FRP sheets fully covered beam with bottom and sides possess better Compressive Strength, Specific Cube Strength, Flexural strength and Elastic Modulus when it is compared to other patterns, like wise flexural strength of bottom completely covered and sides is partially covered beams is also better.

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EVOLUTION OF CONCRETE USING BRINE SLUDGE AND HYPO SLUDGE

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ABSTRACT
Portland cement is the most important ingredient of concrete and is a versatile and relatively high cost material. Large scale use of cement is causing environmental problems on one hand and depletion of natural resources on other hand. This work examines the possibility of using waste paper sludge (hypo sludge and brine sludge) to produce a low cost concrete by blending various ratios of cement and fine aggregate (sand) with hypo sludge and brine sludge and to reduce land disposal problems due to waste paper sludge. The innovative use of hypo sludge and brine sludge in concrete as a supplementary cementitious and fine aggregate material was tested as an alternative to concrete. In this study hypo sludge and brine sludge was partially replaced from 3%, 5%, 7%, 10% and 12% in cement to get optimum for M30 and M40 grades of concrete and also hypo sludge and brine sludge was partially replaced from 5%, 10%, 15% and 20% in fine aggregate to get optimum point in concrete for M40 mix and tested for its compressive strength, up to 28 days of strength and compared with conventional concrete. This study was based on the comparison of hypo sludge concrete and brine sludge concrete with partial replacement of cement and fine aggregate respectively. The study results indicate that use of waste paper sludge ash in concrete has improved the performance of concrete in strength and economic aspect. The optimum strength was achieved at 7% when replaced with cement and at 15% when replaced with fine aggregate.

KEY WORDS: Hypo Sludge, Brine sludge, Partial Replacement, Compressive Strength.

INTRODUCTION
Natural resources are not unlimited therefore, they must be optimally consumed. In order to make concrete industry sustainable, the use of waste materials in place of natural resources is one of the best approaches. Industrial waste is produced by industrial activity which includes any material that is rendered useless during a manufacturing process such as that of factories, mills and mines. This shall help not only to control degradation of environment but conserve them also for the use of future generation. This can be achieved by the process of recycling and making use of industrial wastes, disposal of which otherwise is a serious problem. Hypo sludge is such an industrial waste produced in plenty by paper mills and Brine sludge is the waste generated in the process of whitening of paper. This study presents the physical and chemical analysis of hypo sludge and brine sludge and its use in cement concrete as a partial substitute of cement and fine aggregate which economizes the cost of concrete [12].
Concrete is a composite construction material composed of cement, aggregate (generally a coarse aggregate made of gravels or crushed rocks such as limestone, or granite, plus a fine aggregate such as sand), water, and/or admixtures [12].
Concrete is made by mixing: Cement, water, coarse and fine aggregates. The objectives are to mix these materials traditionally to make concrete that is easy to: Transport, place, compact, finish and to give a strong and durable product. The main objective of the work is to experimentally study the effect of the replacement of cement with Hypo sludge and Brine sludge for M30 and M40 mixtures of concrete to obtain the optimum percentage of replacement of cement in concrete and also to study the effect of the replacement of fine aggregate with Hypo sludge and Brine sludge for M40 grade of concrete to obtain the

optimum percentage of replacement of fine aggregate in concrete. The aim of the research is to study the effect of hypo sludge and brine sludge on compressive strength of concrete at the age of 14 and 28 days.

LITERATURE REVIEW

A. Research Background

Pitroda et.al studied that to produce low cost concrete by blending various ratios of cement with paper waste & to reduce disposal and pollution problems due to hypo sludge it is most essential to develop profitable building materials from hypo sludge. To make good Quality paper limited number of times recycled Paper fibers can be used which produces a large amount of solid waste. The ingenious use of hypo sludge in concrete formulations as a supplementary cementitious material was tested as an alternative to traditional concrete. The cement has been replaced by waste paper sludge accordingly in the range of 0% (without Hypo sludge), 10%, 20%, 30% & 40% by weight for M-25 and M-40 mix. Concrete mixtures were manufactured, checked and compared in terms of strength with the conventional concrete. He carried out tests to evaluate the mechanical properties like compressive strength up to 28 days and split strength for 56 days are taken. He concluded that the compressive increased up to 10% addition of hypo sludge and further increased in hypo sludge reduces the strengths gradually [1].

Balamurugan (2014) et.al investigated that to produce low cost concrete by blending various ratios of cement with hypo sludge & to reduce disposal and pollution problems due to hypo sludge it is most essential to develop profitable building materials from hypo sludge. To make good quality paper limited number of times recycled Paper fibers can be used which produces a large amount of solid waste. He tested Hypo Sludge as an alternative to traditional concrete. He experimented and carried out tests to evaluate the mechanical properties like compressive strength up to 28 days. As a result, the compressive increased up to 10% addition of hypo sludge and further increased in hypo sludge reduces the strengths gradually[2].

Sumit A Balwaik S P Raut (2017) et.al explained that the use of paper-mill pulp in concrete formulations was investigated as an alternative to landfill disposal. In his study cement has been replaced by waste paper sludge accordingly in the range of 5% to 20% by weight for M-20 and M-30 mix. By using adequate amount of the waste paper pulp and water, concrete mixtures were produced and compared in terms of slump and strength with the conventional concrete. The concrete specimens were tested in three series of test as compression test, splitting tensile test and flexural test. He carried out these tests to evaluate the mechanical properties for up to 28 days. As a result, the compressive, splitting tensile and flexural strength increased up to 10% addition of waste paper pulp and further increased in waste paper pulp reduces the strengths gradually. According to him the research on use of paper sludge can be further carried out in concrete manufacturing as a new recycled material [3].

Mounika Ch., Asif Ali Sk.(2014) et.al examined the possibility of using waste paper sludge ash to produce a low cost concrete by blending various ratios of cement with paper sludge ash and to reduce disposal and pollution problems due to waste paper sludge ash. She innovatively used waste paper sludge ash in concrete as a supplementary cementitious material and tested it as an alternative to fibre reinforced concrete. In her study waste paper sludge ash was partially replaced from 5%, 10%, 15% in cement to get optimum point and from this optimum point addition of glass fibers with different proportions i.e. from 0.1%,

0.2%, 0.3%, 0.4% and also steel fibers with different proportions i.e. from 0.5%, 1%, 1.5%, 2% are used in concrete for M25 mix and tested for its compressive strength, splitting tensile strength and flexural strength up to 28 days of strength and compared with conventional concrete. By replacing waste paper sludge ash to the concrete, the optimum is obtained at 10%. By adding glass fibers, the optimum is obtained at 0.3% [4].

N. Sivakumar (2017) et.al studied to produce low cast concrete by blending various ratios of cement with hypo sludge. This project is concerned with experimental investigation on strength of concrete and optimum percentage of the partial replacement by replacing cement via 10%, 20%, 30%, 40%, 50%, 60% and 70% of Hypo Sludge. From this study, replacement of cement with this waste of hypo- sludge material provides maximum compressive strength at 30% replacement [5].

P. Venkata Sarath (2017) et.al used Hypo sludge as a partial replacement of cement in concrete. Hypo sludge is the waste produced from the paper industry. In this paper we utilized 5%, 10% of Hypo sludge as a partial replacement of cement. At 5% replacement we have noticed that the strength of concrete has increased by 6% to the nominal mix of M-30. On further increment in percentage of Hypo sludge the strength of concrete decreases [6].

Seyyedeh Fatemeh Seyyedalipour (2014) et.al investigated that using of pulp and paper industry wastes in various concrete mixes containing various contents of the waste can reduce the occurrence of environmental effects by these wastes disposal. The discussion included pulp and paper industry waste management which had received considerable attention and considers grit, dregs, ash, and fiber. He also discussed the advantages and disadvantages of the use of pulp and paper industry wastes in concrete formulations as an alternative to landfill disposal [8].

E Gyan Swaroop Raj (2016) et.al discussed that various efforts that can help to improve the environmental friendliness of concrete to make it suitable as a “Green Building” material. Foremost and most successful in this regard is the use suitable substitutes for Portland cement, especially those that are byproducts of industrial processes, like fly ash, ground granulated blast furnace slag, and silica fume. The combination of different binders and modifiers to produce cheaper and more durable building materials will solve to some extent the environmental and ecological problems [9].

Apurv Kadu (2016) et.al in his research deals with the waste produced from the paper mill industry in terms of Hypo Sludge Ash (HSA) as a replacement to cement. This Hypo Sludge Ash can be used as a replacement to cement in varying proportions such as 0%, 5%, 10%, 15%, 20% and very convincing results can be obtained. Grade concrete is used as per the requirement and accordingly mix design is prepared. Different tests are performed for testing the efficiency of concrete. It was found that compressive strength and flexural strength increased at 5%, 10% replacement of cement with Hypo Sludge Ash [10].

K. Hari kishan et.al worked with M20 grade which was developed using IS method of mix design. Specimens of dimensions of 150 x 300mm cylinders for split tensile strength and dimensions of 100 x 100 x 500mm prisms for flexure strength and 150*150*150mm cubes were cast, with and without hypo sludge and tested under axial compression to justify the compressive strength for 7 and 28 days. Standard cubes were immersed in 5%HCL, 5% H2SO4 inspecting the durability properties. It is concluded that hypo sludge concrete had better mechanical properties and durability properties compared to normal concrete. Test

results indicate that the use of hypo sludge in concrete has improved the performance of concrete from strength as well as durability aspects. The split tensile strength is less in hypo sludge concrete compared to normal concrete [11].

Jeeva D and Dr Lenin Sundar M attempt had been made to Study the mechanical properties of concrete in which hypo sludge as a replacement material for cement. The tests were carried out to evaluate the mechanical properties like compressive strength up to 28 days. As a result, the compressive increased up to 10% addition of hypo sludge and further increase in hypo sludge reduces the strengths gradually. His research work is concerned with experimental investigation on strength of concrete and optimum percentage of the partial replacement by replacing cement via 5%, 10%, 15%, and 20% of hypo sludge. The mix design of concrete was arrived as per IS: 10262-2009. The project casting of conventional cement concrete cubes has been done casting of concrete cube added with industrial waste (Hypo sludge) has also been done comparison of results has been done testing of concrete cubes with various methods like compression and split tensile test has been done for both cubes.
His results concluded that for 10% of hypo sludge concrete, the compression strength and split tensile strength has been increased, so up to 10% cement has been replaced by hypo sludge by replacement of hypo sludge the cost of construction should be minimized by effective utilization of waste product into concrete to also reduce the environmental effects [12].

Joshi Rohit R., Kerai Harish M., Kerai Manish G., Varsani Dhiren S. and Prof. Jigar Zala, present study work in which they replaced cement with waste paper sludge at 7%, and 10% respectively. This experimental investigation is carry out on different cubes, cylinders, and beams (such as normal cubes, cylinders, and beams and WPS replaced cubes, cylinders, and beams) which have standard size of cube 15cm x 15cm x 15cm, cylinder 15cm dia. x 30cm, beam 70cm x 15cm x 15cm and compare the results to check compressive strength, Split tensile strength and flexural strength test after 7 days, 14 days & 28 days of curing respectively. The following conclusions are obtained from above study, during the process of replacement, sludge is the waste material, therefore strength will be not increased with compared to cement. Water absorption capacity of waste paper sludge is more than cement, therefore bonding process of waste paper sludge with concrete is very poor and it is also affect on the strength of concrete. Split tensile strength test results for cylinders are approximately same with results which are required after 28 days curing period [13].

R. Srinivasan, K. Sathiya and M. Palanisamy, project is concerned with experimental investigation on strength of concrete and optimum percentage of the partial replacement by replacing cement via10%, 20%, 30%, 40%, 50%, 60% and 70% of hypo sludge. Based on limited experimental investigation concerning the compressive and split tensile strength of concrete, observations are made regarding the resistance of partially replaced hypo sludge. Compressive strength of the concrete should be increased when the percentage of replacement is increased up to 40% and replacement increased compressive strength become reduced. The split tensile strength should be decreased when the percentage of the replacement is increased. From this level, replacement of cement with this waste of hypo-sludge material provides maximum compressive strength at 30% replacement [14].

G. Nagendha Reddy, S. K. Subhan Alisha, K. Suseela and S. Neeraja, investigated that the cement is replaced by 20 percent of hypo sludge and fine aggregate by manufactured sand at 75 percent replacement. The tests were carried out to evaluate the mechanical

properties like compressive strength and split tensile strength at 7 days and 28 days for the concrete mix M35 in which cement replaced by hypo sludge via 5%, 10%, 15%, and 20% and at the same time for the concrete mix M35 in which cement is replaced by hypo sludge via 5%, 10%, 15% and 20% and fine aggregate by manufactured sand via 15%, 30%, 45%, 60% and 75%. He concluded that, the compressive strength of concrete cube specimens in which cement is replaced by hypo sludge via 5%,10%,15%,20% gave maximum strength at 10% replacement and the increase in strength is 8.94% compared to conventional mix. The compressive strength of concrete cube specimens in which cement is replaced by hypo sludge via 5%, 10%, 15%, 20%and fine aggregate replaced by manufacture send via15%, 30%, 45%, 60% and 75% gave maximum strength at 20% and 60% replacement. The strength incremented 20 and 60 replacement is 25.57% when compared to conventional mix [16].

Anil Kumar and Devika Rani, attempts to study the strength parameters such as compressive and tensile strength of Paper Sludge Ash (5%,10%, and 15%) as a partial replacement of cement and Foundry Sand (20%, 40% and 60%) as a partial replacement of fine aggregate for a design mix of M25. Based on the compressive strength of specimen with different replacement level these conclusions were found that for the grade of concrete considered for the study, FS2 i.e. the ratio of 60:40 of Conventional Sand, Foundry Sand has proved to be having optimum ratio which gives maximum Compressive strength of all ratios. Similarly for the same grade of concrete PSA1 the ratio of 95:5 of cement. Paper Sludge Ash has proved to have an optimum ratio which gives maximum compressive strength. Specimen of Conventional Concrete bearing the designation CC resulted in highest compressive strength at 7days, 14days and 28days of curing period, of all the mixes considered for the study. At the higher replacement levels i.e. at 60% foundry sand replacement, 28 days strength is 8.16% greater than that of CC concrete mix. From the compressive strength of specimen with 60% replacement by Foundry Sand and 15% Paper Sludge Ash sample, showed least strength for all the curing period considered for the study.

Mansi Rudrawar and SangmeshGhale examined the wood ash 5% as constant and further 0%, 5%, 10%,15% and 20% replacement of hypo sludge in pace of cement in concrete. Compressive strength initially increases when cement replaced hypo sludge and wood ash as we step by step increase the percentage from 0% up to 5%. As hypo sludge and wood ash percentage increases compressive strength. Workability of concrete mix decreases with increase in hypo sludge and wood ash content. They concluded that Environmental effects from wastes and residual amount of cement production can be reduced through this research. This research concludes that hypo sludge and wood ash can be inventive supplementary cementation construction material. Use of hypo sludge and wood ash in concrete can minimize disposal problems and useful concrete for construction [18].

Ashish Pandya and Tejas Joshi, study examines 5%, 10%, and 15% replacement of hypo sludge in place of cement in concrete for M20 and then tested for its compressive strength. The result shows that the strength of the concrete increases by adding hypo sludge up to 10%. However, further increase in hypo sludge decreases the compressive strength. The aim of this study is to make eco-friendly building material and to see how the concrete behaves on replacing the cement with the hypo sludge in various proportions. This study also examines the possibility of utilization of hypo sludge as a partial replacement material and it can be further used as cementitious material.

They concluded the rate of the concrete decreases on replacing the cement with hypo sludge. It also helps with the disposal and pollution problems associated with paper industry waste [23].

Summary

After studying most the literature paper on the paper industry wastes, we investigate that there are various types of paper waste used in construction material such as hypo sludge, brine sludge, paper mill waste ash and lime sludge, but most commonly used paper mill waste was hypo sludge. The researcher were used these sludge as replacement of cement and fine aggregate in concrete at various percentage of replacement such as from 5%, 7.5%, 10% and up to 40% in nominal mix or low grade such as M20 and M25. The various strength properties were tested on the paper sludge concrete for fresh and hardened state such as slump cone test, compressive strength test on cubes and cylinders, flexural strength test on beams & cylinders and split tensile strength on beams. In most of the researches, it was concluded that the compressive strength, flexural strength and split tensile strength of the concrete cubes, cylinders and beams made by replacing cement and fine aggregate with these sludge increases to a certain extent then starts decreasing. So to produce the economical concrete used in construction work, these sludge should be used smartly.

Research Thrust

In the research papers which we studied for this investigation work, it was seen that many replacement were used as cement which is a conventionally produced material but few of were used as substitute material for fine aggregate which is naturally available material and were found to be scare in future if not used wisely. The literature paper also shows that most of the researcher used paper sludge in low grade concrete mix such as M20 & M25. So, in this study we compared two types of wastes partially replaced with cement as well as fine aggregate also at a higher grade of concrete i.e. M30 and M40 when cement is replaced at 3%,5%,7%,10% and 12% whereas fine aggregate was replaced at 5%,10%,15%,20% for M40 grade of concrete.

METHODOLOGY

A. Raw Materials

The properties of materials used in the concrete are discussed below:

(1) Cement

Ordinary Portland cement of 43 grades conforming to IS12269:1987 available in local market was used. The specific gravity of cement that was taken was 3.12.

(2) Fine Aggregate

The sand sieved through 4.75 mm sieve is used having specific gravity of 2.67. The fine aggregates conforming to IS: 383-1997 was used.

(3) Coarse Aggregate

Locally available coarse aggregate having the maximum size of 20 mm is used. The coarse aggregate conforming to IS 2386 was used. The specific gravity of coarse aggregate that was taken was 2.73.

(4) Hypo Sludge

Hypo Sludge is a paper industry waste generated at the time formation of paper. Hypo Sludge is used as a replacement for cement. The specific gravity of hypo sludge that was used for cement replacement i.e., passing through 90 micron sieve was 3.10 whereas the specific gravity of hypo sludge that was used for fine aggregate replacement i.e., passing through 4.75 mm sieve was 2.17.

(5) Brine Sludge

Brine Sludge is a paper industry waste which is generated at the time of whitening of paper. Brine Sludge was used as a replacement for cement and fine aggregate. The specific gravity of brine sludge that was used for cement replacement i.e., passing through 90 micron sieve was 3.25 whereas the specific gravity of brine sludge that was used for fine aggregate replacement i.e., passing through 4.75 mm sieve was 2.42.

(6) Water

Clean water free from impurities and chemicals was used in this study conforming to IS 456:2000.

MIX DESIGN

Mix design for this study is done by using IS 10262 and Mix Names of various concretes is given in table

Table 1 Mix Names when Cement replaced with hypo sludge

Cement	Hypo Sludge	Mix	Name
		M30	M40
100%	0%	CC30	CC40
97%	3%	HC30A	HC40A
95%	5%	HC30B	HC40B
93%	7%	HC30C	HC40C
90%	10%	HC30D	HC40D
88%	12%	HC30E	HC40E

Table 2 Mix Names when Cement replaced with Brine Sludge

Cement	Brine Sludge	Mix	Name
		M30	M40
100%	0%	CC30	CC40
97%	3%	BC30A	BC40A
95%	5%	BC30B	BC40B
93%	7%	BC30C	BC40C
90%	10%	BC30D	BC40D
88%	12%	BC30E	BC40E

Table 3 Mix Names when Fine Aggregate replaced with Hypo and Brine Sludge

Fine Agg.	HS	Mix Name	Fine Agg.	BS	Mix Name
		M40			M40
100%	0%	CC40	100%	0%	CC40
95%	5%	HF40A	95%	5%	BF40A
90%	10%	HF40B	90%	10%	BF40B
85%	15%	HF40C	85%	15%	BF40C
80%	20%	HF40D	80%	20%	BF40D

RESULTS AND DISCUSSIONS

In this investigation the hardened properties of Hypo sludge concrete and Brine sludge concrete for various Partial replacement percentages of Hypo sludge and brine sludge with cement and fine aggregate are determined

A. Comparison of Hypo Sludge and Brine Sludge when Replaced with Cement

(1) Test on Fresh Concrete (Slump Cone test)

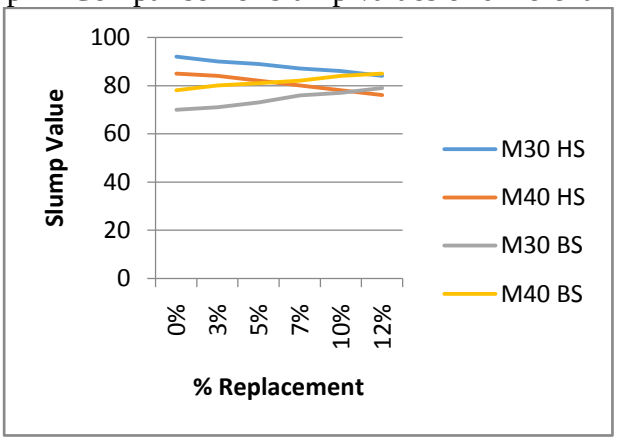
Table 4 Slump values of different mix when cement replaced with Hypo Sludge

Mix Name	Slump Value	Mix Name	Slump Value
CC30	92	CC40	85
HC30A	90	HC40A	84
HC30B	89	HC40B	82
HC30C	87	HC40C	80
HC30D	86	HC40D	78
HC30E	84	HC40E	76

Table 5 Slump values of different mix when cement replaced with Brine Sludge

Mix Name	Slump Value	Mix Name	M40 Grade
CC30	70	CC40	78
BC30A	71	BC40A	80
BC30B	73	BC40B	81
BC30C	76	BC40C	82
BC30D	77	BC40D	84
BC30E	79	BC40E	85

Graph 1 Comparison of slump values of different mix.



(2)Test on Hardened Concrete (Compressive Strength Test)

In order to find the mechanical properties Compressive strength tests were conducted at 7 and 28 days of cube (150 × 150 × 150 mm) specimens. For each combination, three specimens were tested and the average strength values reported in this paper. Compressive strength tests were performed on compression testing machine using cube samples.

(i) Hypo Sludge Concrete

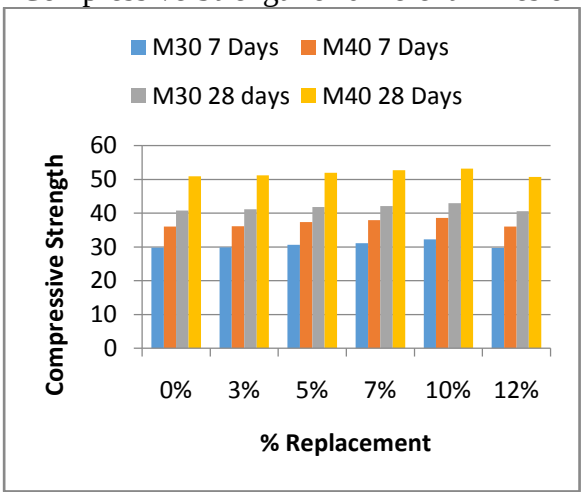
Table 6: 7 Day Compressive strength of hypo sludge concrete

Mix Name	Comp. strength (N/mm^2) M30	Mix Name	Comp. strength (N/mm^2) M40
CC30	29.80	CC40	36.05
HC30A	29.90	HC40A	36.09
HC30B	30.60	HC40B	37.30
HC30C	31.10	HC40C	37.90
HC30D	32.20	HC40D	38.60
HC30E	29.70	HC40E	36.04

Table 7: 28 Day Compressive strength of hypo sludge concrete

Mix Name	Comp. strength (N/mm^2)M30	Mix Name	Comp. strength (N/mm^2)M40
CC30	40.74	CC40	50.89
HC30A	41.10	HC40A	51.20
HC30B	41.80	HC40B	51.90
HC30C	42.10	HC40C	52.70
HC30D	42.90	HC40D	53.20
HC30E	40.60	HC40E	50.70

Graph 2 Comparison of Compressive Strength of different mixes of Hypo Sludge Concrete



For Hypo Sludge at 12 % the compressive strength of concrete is 40.6 N/mm2 which slightly less than that of conventional concrete i.e., 40.74 N/mm2 which shows that the compressive strength decreases at 12% replacement for M30 grade of Concrete. For M40 grade of concrete At 12 % the compressive strength of concrete is 51.70 N/mm2 which was slightly less than that of conventional concrete i.e., 51.89 N/mm2 which shows that strength will further decreased as cement was replaced. So, on comparing results it is clear that 10% replacement of cement with Hypo sludge is optimum for this study.

(ii) Brine Sludge Concrete

Table 8: 7 Day Compressive strength of hypo sludge concrete

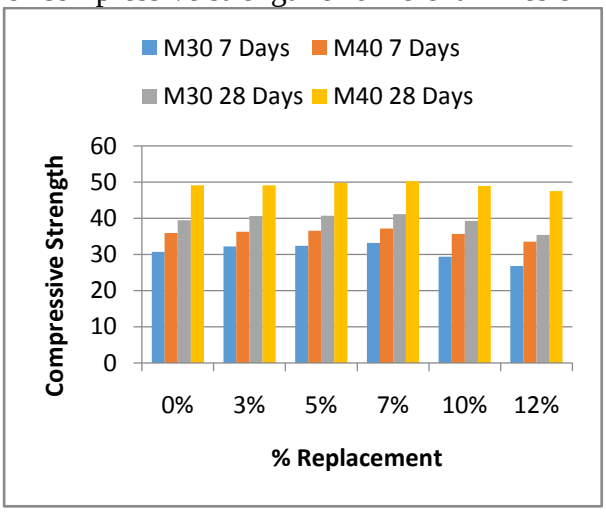
Mix Name	Comp. strength (N/mm^2) M30	Mix Name	Comp. strength (N/mm^2) M40
CC30	30.70	CC40	35.92
BC30A	32.18	BC40A	36.26
BC30B	32.38	BC40B	36.56
BC30C	33.20	BC40C	37.18
BC30D	29.36	BC40D	35.70
BC30E	26.84	BC40E	33.52

Table 9: 28 Day Compressive strength of hypo sludge concrete

Mix Name	Comp. strength (N/mm^2)	Mix Name	Comp. strength (N/mm^2)

	M30		M40
CC30	39.50	CC40	49.1
BC30A	40.65	BC40A	49.13
BC30B	40.68	BC40B	49.83
BC30C	41.19	BC40C	50.23
BC30D	39.33	BC40D	48.91
BC30E	35.42	BC40E	47.53

Graph 3: Comparison of compressive strength of different mixes of Brine Sludge Concrete



For Brine Sludge the Compressive Strength of M30 Grade of concrete reached to 41.19 N/mm^2 at 7% cement replacement with Brine Sludge.At 10 % the compressive strength of concrete is 39.33 N/mm^2 which nearly same to that of conventional concrete i.e., 39.5 N/mm^2 the compressive strength decreases further when 12% cement was replaced.The Compressive Strength of M40 Grade of concrete reached to 50.23 N/mm^2 at 7% cement replacement with Brine Sludge.At 10 % the compressive strength of concrete is 48.91 N/mm^2 which nearly same to that of conventional concrete i.e., 49.1 N/mm^2 the compressive strength decreases further when 12% cement was replaced.So, on comparing results it is clear that 7% replacement of cement with brine sludge is optimum for this study.

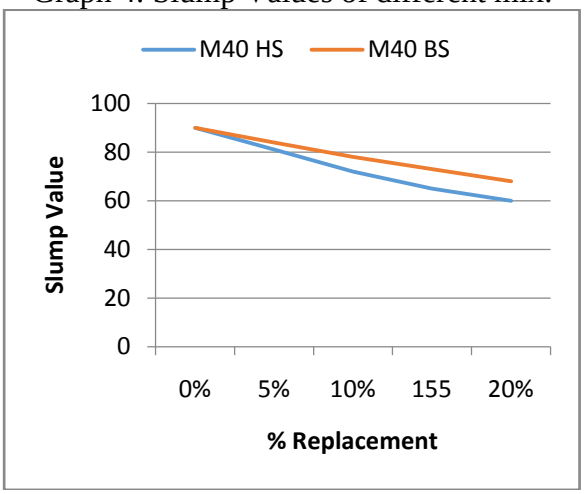
Comparison of Hypo Sludge and Brine Sludge when Replaced with Fine Aggregate

i. (1)Test on Fresh Concrete (Slump Cone test)

Table 10: Slump values of different mix when fine aggregate replaced with Brine Sludge and Hypo Sludge

Mix Name	Slump Value(mm)	Mix Name	Slump Value(mm)
CC40	90	CC40	90
BF40A	84	HF40A	81
BF40B	78	HF40B	72
BF40C	73	HF40C	65
BF40D	68	HF40D	60

Graph 4: Slump Values of different mix.



Test on Hardened Concrete

In order to find the mechanical properties Compressive strength tests were conducted at 7 and 28 days of cube (150 × 150 × 150 mm) specimens. For each combination, three specimens were tested.

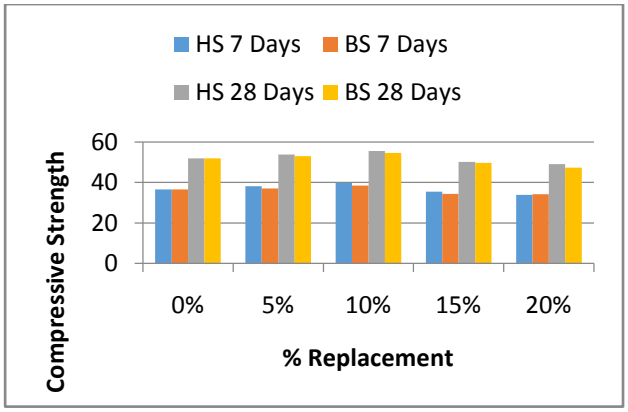
Table 11: 7 Days Compressive Strength

Mix Name	Comp. strength (N/mm^2) M40	Mix Name	Comp. strength (N/mm^2) M40
CC40	36.55	CC40	36.55
HF40A	38.1	BF40A	37.09
HF40B	39.89	BF40B	38.49
HF40C	35.4	BF40C	34.4
HF40D	33.8	BF40D	34.26

Table 12: 28 Days Compressive Strength

Mix Name	Comp. strength (N/mm^2) M40	Mix Name	Comp. strength (N/mm^2) M40
CC40	51.89	CC40	51.89
HF40A	53.9	BF40A	53.10
HF40B	55.5	BF40B	54.6
HF40C	50.20	BF40C	49.68
HF40D	49.15	BF40D	47.30

Graph 5: Comparison of Hypo Sludge concrete and Brine Sludge concrete



The graph shows that the strength of brine sludge concrete cubes increases from 0% replacement to 5% and up to 10% replacement i.e. at 0% - 51.89 N/ mm^2, at 5% - 53.1 N/ mm^2, at 10% - 54.6 N/ mm^2. But further increase in replacement percentage i.e. 15% and 20% the compressive strength was decreased i.e. at 15% - 49.68 N/ mm^2 and at 20% - 47.3 N/ mm^2. The optimum compressive strength of brine sludge concrete cubes was at 10% replacement and is 54.6 N/ mm^2. The graph shows that at 0 % replacement the strength of concrete is similar to the mean target strength of M40 grade of concrete but as fine aggregate was replaced by hypo sludge the strength increased till 10 % replacement. But further increase in hypo sludge content i.e. at 15% the compressive strength was decreased and was similar to the mean target strength but at 20% replacement of hypo sludge resulted into further decreased compressive strength. So, the optimum compressive strength of hypo sludge concrete was at 10% replacement.

CONCLUSIONS

The Compressive Strength of Concrete increases when the replacement of Cement with Hypo Sludge up to 10% replacement by weight of Cement. The Compressive Strength of Concrete increases when the replacement of Cement with Brine Sludge up to 7% replacement by weight of Cement.

The Compressive Strength of Concrete increases when the replacement of Fine Aggregate with Hypo Sludge up to 10% replacement by weight of Sand. The Compressive Strength of Concrete increases when the replacement of Fine Aggregate with Brine Sludge up to 10% replacement by weight of Fine Aggregate. Comparatively Hypo Sludge give more compressive strength till 10 % than Brine Sludge but when replaced with fine aggregate the optimum replacement for both sludge is 10%. Hypo Sludge and Brine Sludge are a better innovative supplementary cementitious construction material which is used in concrete, so it can save the paper industries waste disposal costs and produce a greener concrete for construction. This research concludes that hypo sludge and brine sludge can be innovative supplementary cementitious and fine aggregate Construction Material in Concrete up to certain extent.

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SEISMIC INVISIBILITY CLOAK – A REVIEW

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ABSTRACT
Earthquakes have been one of the major problems since ancient civilisation, raising havoc in the wake of its destruction. Over the years it has caused immense damage and loss of life and property. Even with the advent of modern technology, earthquakes still cause considerate damage to structures around them. For sensitive structures such as nuclear power plants, dams, hazardous chemical factories, where even the slightest crack in the structure can cause a major catastrophe, current earthquake mitigation measures are inadequate. Earthquake energy dissipation devices can be in cooperated in new constructions but it is challenging to carry out such measure on existing old heritage structures. In such cases it is only possible to protect the structure by creating barriers around it. It is thus, the duty of engineers to come up with ways to minimise earthquake damage as much as possible. The emergence of metamaterials paved the path to the possibility of altering the direction of wave travel. Research is being conducted in developing such devices that will change the path of propagation of waves. Other than man-made devices, it was also found that forests act as natural metamaterials in dissipating the wave energy. Thus, the idea of seismic invisibility cloaking is basically rendering the structure invisible to the path of seismic waves. Researchers all over the world are now focused on carrying this concept of seismic metamaterial to a whole new level. This concept has a wide prospect and so this paper aims at throwing light on some of the new and emerging methods that are still under investigation, and are currently being researched for actual application

KEYWORDS: Seismic metamaterial, buried mass resonators, negative refractive index, split ring resonator, left-handed material, bandgap, metastructures.

INTRODUCTION
A metamaterial can be defined as a manmade or engineered material of nanometre or micrometre scale, structured in the form of periodic arrays of resonant structures and that exhibit peculiar characteristics namely a negative refractive index. Due to this property a metamaterial is able to produce phenomenon such as invisibility cloaking, dispersion and manipulating the path of propagation of incident waves. This property has been explored in the field of optics as well as acoustics. It goes without further saying that these characteristic properties of metamaterials which make it such a diverse material could be used in the field of earthquake engineering.
The principle concept of metamaterials was envisioned first by Veselago (1964), as substances that have simultaneously negative values of dielectric constant and magnetic permeability. These materials were called left-handed material and displayed a negative refractive index. Figure 1 shows the refraction in left handed materials.

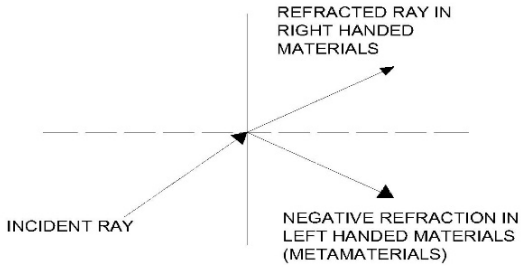


Fig. 1: Refraction in left handed metamaterials

Since such materials were non-existent in natural environment, it was difficult to understand the vast applicability which would soon change the way we see the areas of optics and acoustics. Soon after physicists such as Pendry et al. (1999)and Smith et al. (2000) constructed materials that showed the above-mentioned properties. Metamaterials soon came into existence and took over the field of optics. Later this concept of negative refractive index was applied to acoustics in dispersing and diffracting acoustic waves. With the wide scale application of metamaterials in optics and acoustics, physicists and researchers questioned the possibility of using this phenomenon on elastic waves or in other words seismic waves. Various experiments were conducted to prove the applicability of metamaterials in the field of earthquake engineering. Since the seismic waves are dispersed or diffracted around the structure to be protected, the structure remains unhindered by the earthquake and in a sense remains invisible to the waves. Thus, we use the term 'Seismic Invisibility Cloak' to describe the working of the seismic metamaterials.

LITERATURE REVIEW

Although the vast differences between electrodynamic waves, acoustic waves from elastic or seismic waves, the application of metamaterials in this area proved to be possible. In earthquake engineering the wave of major concern is the Rayleigh wave with frequency range of 0.1 to 10 Hz. Thus, seismic metamaterials would have to be designed to attenuate waves in this frequency range. Since metamaterials in large scale applications like that in civil field require attenuation of waves over a large area it is not possible to use metamaterials of the scale recommended in optics and acoustics. Accordingly, metamaterials in earthquake engineering takes the form of large structured soils with inclusions that are arranged in a particular array to diffract, disperse or absorb the wave energy. These inclusions vary from hollow cylindrical bore holes to resonators embedded in the soil.Kim and Das (2012) used an array of cylindrical shells with 4 side hole openings, termed as metacylinders, to demonstrate the attenuation of seismic waves caused by these cylinders when embedded in soil. The metacylinders created stop bands for the seismic waves by converting the wave into evanescent wave of some frequency range. The study was mainly derived from the principles of acoustic metamaterials.

The last decade has seen some major advancement in the application of metamaterials in seismic cloaking of structures. Large-scale experiments have been done by a group of

physicists in Fresnel where on field application of metamaterial in the form of an array of hollow cylindrical bore holes, is tested using an artificial wave source (Brule et al., 2013).
Seismic metamaterials in the form of an array of cylindrical bore holes were used to demonstrate practically the behavior of seismic wave when passing through the array of bore holes (Brûlé et al., 2014). It was observed that the attenuations for a wave of 50 Hz frequency.
One would argue over the effect of forests on the propagation of waves, and whether the interaction of seismic waves with a forest of trees would cause any desirable changes in wave frequencies. This was demonstrated at a geophysical scale by Colombi, et al. (2016).In this geophysical experiment, it was demonstrated that the longitudinal resonance of the vertical resonator was inversely proportional to the height of the tree. This experiment showed minor attenuation of waves, not good enough to prevent destruction by Rayleigh waves.Achaoui et al. (2017) explored the zero frequency stop band media through concrete columns clamped at the base to the bedrock.
Given the limitations in practicality of this study it provides research data which is a step further in the field of metamaterials. Using a cluster of sub-wavelength rods, Colombiet al. (2017a) demonstrated that the metamaterial parameters such as rod height and periodicity play an important role in the effect of wave attenuation.Colombi et al.(2017b)demonstrated that the longitudinal resonance of the vertical resonator was inversely proportional to the height of the tree. This experiment showed minor attenuation of waves, not good enough to prevent destruction by Rayleigh waves. Figure 2 shows the experimental demonstration of forest as a natural metamaterial.
Haupt et al. (2017) designed an array of boreholes and trenches to divert the waves from the protection zones and demonstrated this study experimentally on a scaled model. Since normal borehole array arrangements showed attenuation of waves in frequency range of 50 Hz and above, this research was advancement to the previous works. it was also important to note that the barrier was designed to not only divert the waves in aerial plane, but includes vertical V-shaped muffler structure in order to prevent the seismic waves passing through the boreholes from diffracting and travelling in the vertical plane. The results obtained were promising.

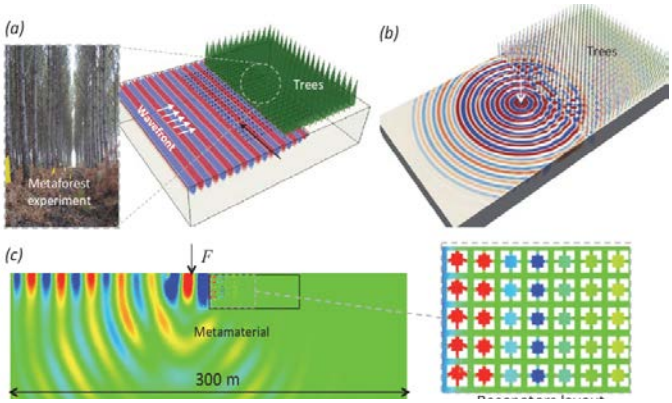


Fig. 2: Experimental demonstration of forest as a natural metamaterial (Colombi et al., 2017b)

Brule et al. (2017) highlighted the concept of metacity and its interaction with seismic waves. Cities arranged in an array of closely spaced buildings resemble metamaterials positioned in soil. These structures can be assumed to behave like a set of clamped resonators in a common base slab.Brûlé et al. (2019) briefly sums up the progress made in the field of seismic metamaterial including the various types of metamaterials developed to attenuate seismic

waves with regards to their placing and arrangement in the soil.In a most recent research**,** Ungureanu (2019)explored the efficiency of a split ball resonator as an advancement to his previous research. These resonators form the class of buried mass resonators and consist of steel spheres split in 8 pieces pieced together by rubber and embedded in concrete block with steel ligaments. The spheres displayed stop band frequencies in the range of 7 Hz. Thus, closing the gap towards the desirable frequency range of Rayleigh waves.

WORKING OF SEISMIC METAMATERIAL

Seismic metamaterials work in a way similar to acoustic metamaterials and the cloaking is as a result of destructive interference. Since seismic metamaterial are arranged in in periodic arrays of resonant structuressuch as cylindrical bore holes, the elastic waves that comes in contact with each of these structures, scatters away and the scattered waves from each of these structures in turn cancel out each other's effect. In other words, the periodic array arrangement produces bandgaps that are in the range of wavelength of the incident waves which results in attenuation of the incident ray. This is the working principle behind seismic metamaterials. However, so far experiments to reflect waves in the frequency range of 50 Hz have been conducted and have proven to show successful attenuation of the waves. Since Rayleigh wave have a frequency range of about 0.1 to 10 Hz, experiments in this frequency range are yet to be undertaken.

Array of Cylindrical Bore Holes as Seismic Metamaterials

Brule et al. (2014) conducted on field experiments to demonstrate the practical feasibility as well as the working of seismic metamaterials. These seismic metamaterials were introduced in the form of an array of hollow cylindrical intrusions. The diameter of the cylindrical bore holes was taken as 0.32 m, and were drilled 5m deep. The boreholes were spaced at 1.73 m distance. With the help of a vibroprobe, vibrations in the range of 50 Hz were introduced at a distance of 1.5 m from the first line of the boreholes. The geometric parameters were chosen based on numerical simulations while assuming an elastic plate model for simplification. Figure 3 shows the diagrammatic representation of on field experimental setup adopted by Brule et al. (2014).The results produced from the experiment showed wave attenuation by a factor of 2. This experiment not only provided practical evidence of wave attenuation but also showed the field applications of metamaterials with room for further investigation in narrower frequency range.

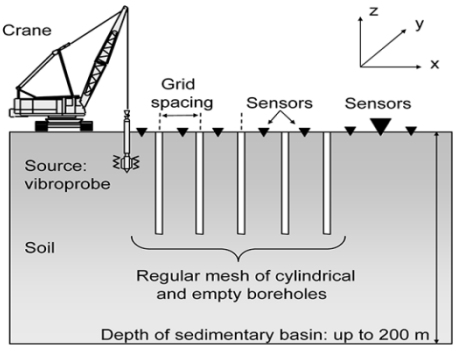


Fig. 3: Diagrammatic representation of on field experimental setup (Brûlé et al., 2014)

A major fall back of such cloaking devices is the fact that the waves are redirected away from the structure it is protecting and hence could in turn affect adjacent structures that were in the path of the redirected wave and could thus worsen the situation if not carefully planned and investigated.

CONCLUSIONS

Through the research carried out in the scope of this paper it is evident that seismic metamaterials are not only studied by numerical, analytical models but also on a model scale and large on field scale. The growing research in this area is a testimony to the fact that metamaterials are yet to make major advancements in the field of earthquake engineering. Seismic metamaterials have contributed to the new advents in the modern age and can be further explored for its capabilities. This only proves that such concepts that were once regarded as unimaginable are now possible thanks to the unyielding determination and growing contribution of engineers and physicists all around the world.

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COMPARATIVE STUDY OF POST-TENSIONED SLAB AND CONVENTIONAL RCC SLAB

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ABSTRACT

By tradition the construction of a building is done by RCC but in today's world, construction of high rise buildings is done by the method of prestressing. Buildings with post-tensioned slab are increasing widely and have gained more popularity than conventional RCC buildings due to its advantages and application. The use of post tensioned slab provide good advantages over reinforced concrete in terms of speed up of construction, reduction in steel quantity, smaller floor to floor height and larger spans and hence are preferred for residential, commercial and industrial buildings. Post-tensioned floors have already been in use in countries like USA, Australia, Singapore and Europe due to increase in urban population which has led to a rise in demand for livable space, resulting in increasing cost. In a developing country like India the benefits and usage of post-tensioning are yet to be recognized. In the present study, an attempt is made to review the experience available in the field of post-tensioning in building construction and to discuss the difference in design of post tensioned slabs with respect to reinforced concrete slabs and then compare the results.

KEYWORDS: Post-tensioning, Slabs, Reinforced concrete slabs, Design.

INTRODUCTION

Due to population magnification the need of land for livelihood has grown rapidly resulting in increase in cost. This has led to the development of advance technology in building construction. Owing to increase in demand, construction of multi-storeyed structures is becoming an essential part of our living style. Nowadays the use of flat slab is becoming popular and it is widely used. Buildings with post-tensioned flat slab are gaining more popularity in construction industry than conventional RCC buildings due to its wide ranging advantages.

Post tensioning (PT) method gives many strong points such as it gives thinner sections which increases the clear span, reduces floor loads resulting in lighter structures, reduced deflection, speedy construction, resistance to fatigue failure etc.Some of its application includes parking structures, commercial and industrial buildings, bridges etc.In countries like U.S.A, Thailand, South Africa, U.K, Australia, post tensioning methods have been already constructed and primarily used for floor system. It offers advantages over reinforced concrete systems are listed as follows:

- More economical structures as the cost of post tensioned members is less than that of normal RCC type structures.
- The usage of material in the construction is minimum in post tensioned as compared to reinforced concrete.
- Deflections due to loads can be easily controlled in post tensioning method than in the normal RCC structures.
- It offers large spans and greater slenderness resulting reduction in dead load and thus overall height of the building reduces.
- Good crack behaviour and permanent protection of steel against corrosion.

For residential buildings with smaller spans, RCC slab is better option when compared with post tensioned slabs as it proves to be cheaper and economical but for larger spans, RCC slab proves to be uneconomical resulting in large deflection and hence post tensioned methods are mainly used. In post tensioning, tendons are being stressed after concrete has been poured and gained 75% strength making the concrete stronger in tension zone. After attaining desired strength, tendons are stressed with the help of hydraulic jacks. Post tensioning method can be done by two systems: bonded system and unbonded systems. In bonded systems, cement grouting is carried out inside the duct where the tendons are fixed preventing the tendons from corrosion whereas in unbonded system, the bonding of tendon is prevented and thus tendons can move freely to the concrete.

LITERATURE REVIEW

Bahoria (2010)presented a study on comparison design of RCC and post tensioned flat slabs. The design methodology is done by two methods. An office building is considered as a case study and he designed by four different cases. Further comparative study was carried out based on economy point of view. It was reported that post tensioned flat slab is more economical when compared with other cases and the quantity of steel and concrete required for floor slab is lesser than that of normal RCC slab.

Thayapraba (2014) made an attempt to compare the cost of post tensioned slab to that of reinforced slab. A (G+8) building was considered. The results shows that post tensioned slabs are cheaper than RCC slabs for different range of spans. Along with this, cost quantities of steel were calculated showing the steel required for post tensioned slab is much lesser than normal RCC slab.Reddy and Pradeep (2017) performed studies on the cost effectiveness of post-tensioned slab with respect to reinforced concrete slab. The model having basement, ground and four floors is considered with dimensions 38.13 x 28.85 m. It was observed that concrete used for R.C.C slab construction is more than Post tensioned slab. The cost of steel required is less in case of post-tensioning slab than in the R.C.C Slab. The thickness of the slab is also lesser than the R.C.C slab. Use of PT Slab is more advisable for a commercial building than using a R.C.C Slab as the dead load gets reduced.Prawatwong etal. (2008) conducted an experimental study on the seismic performance of post-tensioned interior slab-column models with and without drop panel. Results included that no punching was observed when each model acted like linear elastic system with low dissipation of energy and it also concluded that the model without drop panel suddenly failed by punching shear shortly after reaching to its maximum lateral strength at 2.0% drift, whereas the improved model undergone a saturation of lateral peak load from about 2.5% to 6.0% drift before to punching shear.

Nethravathi and Prasad (2017) reported a study of un-bonded post tensioned cast-in-place parking floor subjected to various arrangements of tendon layout based on FEM analysis. It concluded that moment distributions across the slab panels may differ significantly between different tendon layouts, resulting in different signs at critical sections at service limit states. Total deflection including temperature effect, creep and shrinkage, the deflection values are also within the permissible limit.

Koti andVanakudre (2018) considered a residential structure with post-tensioned flat slab with drop considering seismic effect for the analysis and moments, stresses, time period, frequency and behaviour of flat slab under seismic effect were determined for Equivalent Frame method and strip base method. Seismic analysis was carried out showing natural periods and frequencies of PT slab. The study concluded that PT slab with drops help to reduce the stress concentration in the slab and column junctions. Moment calculated for PT flat plate slab is less when compared to moment calculated for RCC flat plate slab and it was concluded that the stresses calculated by strip method are greater than stresses calculated by

equivalent frame method.Dhawale etal. (2019) studied the analysis for Post-tensioning slab and made a comparison with conventional RCC slab on various criteria like base shear, storey drift, storey shear and stiffness etc. It also compared the behaviour of PT flat slab and RCC flab slab structure under the constant seismic loading. Gupta et al. (2018) reported that post tensioned flat slab is the most economical than the conventional two way slab as it provides good aesthetic architectural view without beams and hence reduces the floor height.

GENERAL ARRANGEMENT DETAILS

Figure 1 shows the first-floor slab plan of an industrial project considered for case study in the present paper. The slab (hatched) as shown in Fig. 1 has been analysed and designed and the slab panel is compared with an RCC slab having similar properties. Figure 2 shows the tendon layout for the first-floor plan. The RCC slab considered has dimensions 6.585 m by 10.7 m and beams of 300 mm by 500 mm depth have been considered. The loads considered are 5kN/m^2live load,2kN/m^2 of super imposed dead load and self-weight.

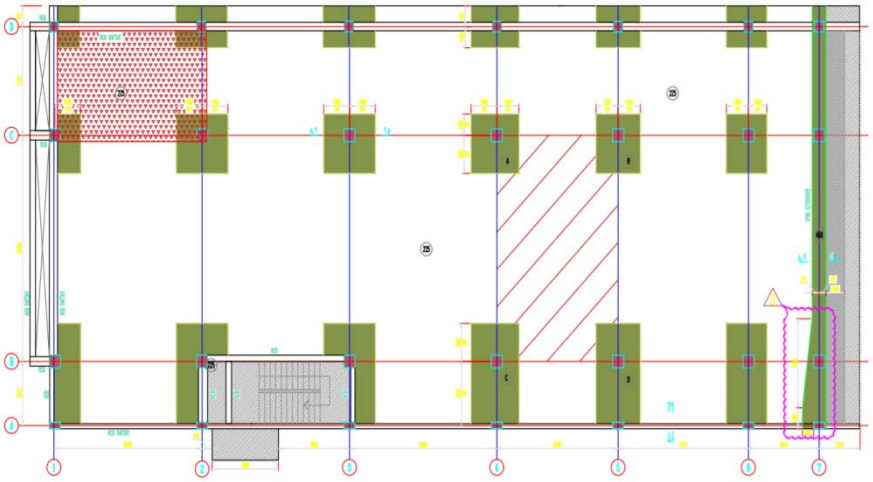


Fig. 1: First Floor Plan

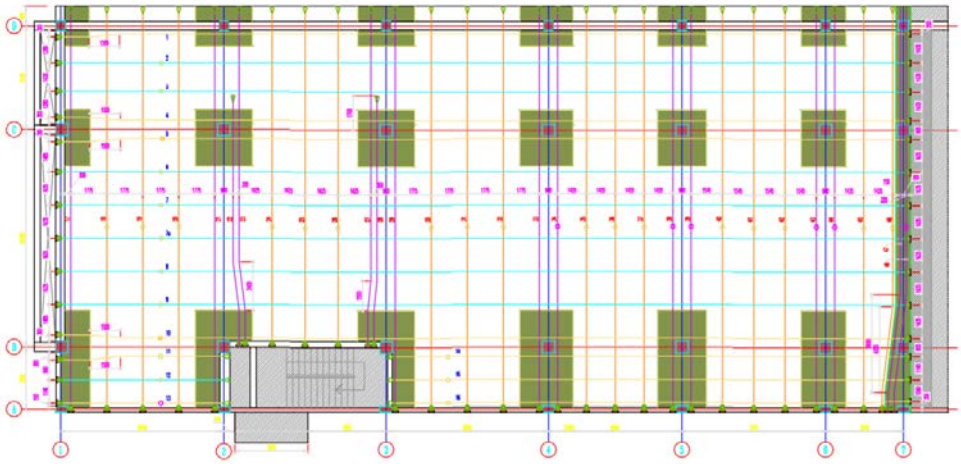


Fig. 2: Tendon Layout of the First Floor Slab

RESULTS

In this case study, a post tensioned slab having drop panels have been considered and compared for the cost with that of a reinforced concrete slab panel with beams. The slab

dimensions taken are 10.7 m by 6.585 m. The drop panel dimensions taken for the post tensioned slab are as shown in Table 1 below.

Table 1 Drop Panel Dimensions

Drop panel	Length (m)	Width (m)	Depth (m)
A	2.8	2.6	0.225
B	2.8	2.4	0.225
C	4.75	2.6	0.225
D	4.75	2.6	0.225

The materials considered for the post tensioned slab are as follows. Table 2 gives the PT strand properties adopted.

Table 2 PT Strands Properties

Strands	0.5''
Nominal Area	100 mm^2
Nominal weight	7.85 N/m
Tensile Strength	1860 N/mm^2
Modulus of elasticity	195 kN/mm^2
Minimum breaking load of strand	183.7 kN
Nominal diameter	12.7 mm

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The sheathings for the bonded tendons are spirally wound with galvanised steel to be made from galvanised steel strips of 0.25-0.3 m thickness.The grade of concrete considered is M35 and grade of steel considered is Fe500.The analysis have been carrried out for the post tensioned slabs and the reinforced concrete slab and the results are as obtained in the table below and they have been compared with respect to the cost of steel and concrete as shown in Table 3.

Table 3 Comparison of PT slab and RCC slab

	Concrete	Reinforcement	Prestressing steel
PT slab	24.56 m^3	0.5T	0.6T
RCC slab (with beams)	30.23 m^3	5T	-

From the above table, it can be seen that the quantity of steel for PT slabs is less then that for RCC slab .So also the concrete quantity is more for RCC slab with beams for the same dimensions as compared to that of PT slabs .It can be seen that the total cost of materials for RCC slab with the same dimensions is 38 percent more than that of the PT slabs.

CONCLUSIONS

From the analysis conducted for the industrial building as considered or the case study, it can be concluded that the cost of materials for s post tensioned slab with drop panels is less as compared to that of the RCC slab with similar properties and dimensions cost of materials is around 38 percent more for a RCC slab as compared to PT slab and hence PT slab can be considered to be more economical compared to RCC slab for larger spans.

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PRACTICAL APPLICATIONS OF FIBRE REINFORCED POLYMERS IN CONCRETE STRUCTURES – A REVIEW

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ABSTRACT
Over the few decades the Civil Engineering field has generously welcomed modern technologies and alternative materials in the construction industry. One such very popular alternative material in concrete technology is Fibre-Reinforced Polymer (FRP). A non-corrosive Fibre-Reinforced Polymer (FRP) has become a very popular subject of research and a great alternative to traditional steel bars. Corrosion of steel reinforcement is a major problem in reinforced concrete structures, which result in reduction in the strength of the structure leading to its failure. FRP have proved to possess excellent corrosion resistance, high tensile strength, light weight and good non-magnetization properties hence in the last few years the use of FRP has increased rapidly. FRPs are made of fibres embedded in polymeric resin, these fibres include Glass, Carbon and Aramid. Due to the increasing popularity and technological advancement, the FRP bars have been successfully installed in different applications like bridges, water tanks, parking garages, tunnels and marine structures where corrosion of steel reinforcements is a major concern. In the present study, a detailed review of practical applications of FRP as reinforcement in concrete structures, behaviour and material properties is presented.

KEYWORDS - Fibre Reinforced Polymer, reinforcement, Codes, material properties.

INTRODUCTION
For over the decades, the civil industry has religiously used steel bars as reinforcement in concrete structures however any material has its limitations. One such limitation of the conventional steel bar is corrosion. Corrosion of steel bars is a major problem in steel reinforced concrete structures. It is seen that corrosion rate is faster in marine environment. Corrosion of steel reinforcements in concrete structures may result in reduction of strength of the structure and if adequate measures are not taken to restore the strength, it may ultimately result in collapse of the structure. Therefore, an alternative material or repair method is to be adopted to ensure the safety of the structure. One such alternative material which has gained a lot of popularity in the last 10 years is Fibre Reinforced Concrete (FRP). The main attractive quality of this composite material is that it is non-corrosive and therefore it can be used in harsh marine environments where steel bars are most prone to corrosion. Other qualities of FRP are high tensile strength, light weight and good non-magnetization properties. However, the drawback of this alternative material is that unlike steel it is brittle which means that it will show no yielding before abrupt rupture, low modulus of elasticity and comparative higher cost compared to the traditional material.

LITERATURE REVIEW
In this literature survey is divided into two categories namely, Theoretical studies and experimental investigations.

Theoretical Studies
Gudonis et al. (2013) carried out a detailed study of physical and mechanical properties, manufacturing process, structural applications and design recommendations of Fibre

Reinforced Polymer (FRP). It was concluded that long term degradation of mechanical properties, proper selection of FRP material under extreme environmental conditions and the bond properties are the governing criteria for deformational analysis.Rathod and Vora (2015) observed that FRP composites can be used as an alternative to steel reinforcing material with a drawback as a brittle material and further investigations should be carried out on fire resistance and shear capacity of FRP.Benmokrane et al. (2016)concluded that is an excellent application in case of concrete bridges, parking garage structures, soft eye-tunnel, water tank and continuously reinforced-concrete-pavement.Tarafder and Swain (2016)concluded that strength of the FRP composites is directly proportional to the fibre length which means that slight decrease in fibre length can increase the internal adhesion strength of FRP composites.Soundarya and NandiniDevi (2017)observed that the deflection in GFRP reinforced beam was 20% less than the conventional beam and the GFRP reinforced slab resisted load 20 times longer than the steel reinforced slab. It was also noted that there was no evidence of debonding between GFRP and concrete. In most of the cases tensile stiffness reduction in GFRP ranged from 0% to 20% was observed. Observations were also made that the alkali attack could not detain GFRP reinforcement material and GFRP in alkaline and water solution exhibited better residual tensile strength.Shrivastava (2018) reported that even though FRP has many advantageous properties as compared to conventional materials, it still remains as a specialty item in many countries and also suggested that it should be included into teaching curricula and Government-Industry-Institute partnership should be encouraged.

Experimental Investigations

Sharbatdar and Saatcioglu (2004) performed an experimental study on beams and columns reinforced with CFRP bars in longitudinal direction and CFR grids in transverse direction. It was reported that in earthquake resistant structures, the columns and beams can be reinforced by FRP and can be designed to satisfy strength and deformability criteria. Mogy et al. (2010) reported that continuous beam reinforced with FRP gave ample warnings before failure. It wasnoted that the CSA code could predict failure load for GFRP beam.Masmoudi et al. (2011) studied the long term deflection of FRP reinforced concrete beams. It was observed that under frequent loading condition the steel reinforced RC beam deflected much lesser then FRP reinforced beam however under long term loading the steel beams deflected more than that of RC beams with FRP bars.Manjubashini et al. (2016) carried a comparative study on two beams, one was a conventional concrete beam and the other was beam reinforced with GFRP bars along the tension zone. It was concluded that the GFRP reinforced concrete beam showed 20% lesser deflection than the steel reinforced beam.George and Parappattu (2017) reported that in the steel reinforced beam, the initial cracks originated at the mid-span of tension zone and propagated towards the centre of beam whereas in case of GFRP reinforced beam, the initial crack originated near the supports and propagated towards the loaded points. It was concluded that the area of reinforcement required for steel reinforced beam was higher than that of GFRP with same moment capacity.Brozda et al. (2017) observed that that highest flexural strength was obtained for beam reinforced with CFRP, while GFRP has lowest value of proportion between beam length and deflection whereas CFRP and AFRP have similar value.Yang (2018) observedthat the bonding between FRP ribs and concrete were very poor compared to normal steel bars. It was concluded that FRP ribs attain better bonding with concrete after they are treated on the surface and the chemical gluing force helps to keep the bond between FRP tendons and concrete.

MATERIAL PROPERTIES

FRP Composite

Fibre Reinforced Polymer (FRP) is a polymer based material made by a combination of two types of component: (i) High performance fibres such as Glass, Carbon and Aramid (ii) Polymer resin matrix, these polymer resin matrix may either be thermosetting which means that they cannot be melted and recycled as the polymer chains are cross-linked or they may be thermoplastic which means that due to its non-linked composition they can be melted and recycled. When these high performance fibres are embedded into polymer resin matrix, they bind together and form Fibre Reinforced Polymer (FRP).

Classification

Depending on the fibres used the FRP are classified into three types, Glass, Carbon and Aramid. If the composite material is glass fibre then it is called Glass Fibre Reinforced Polymer (GFRP), Carbon or Aramid then it is called (CFRP) and (AFRP) respectively. GFRPs are mostly used as they are economical compared to CFRP and AFRP whose cost is on a higher side. CFRP is said to have the highest strength among the rest FRPs primarily due to its carbon nature and provides excellent resistance against creep and fatigue failures. AFRP on the other hand is not a common reinforcement material due to its low compressive strength perpendicular to fibre direction but exhibits great impact absorption characteristics.

Behaviour and Material Properties

Fibre Reinforced Polymer (FRP) shows excellent resistance to corrosion, good tensile strength, high Young's modulus, high strength to weight ratio, non-metallic nature, high fatigue resistance, low thermal conductivity, light weight (approx. $1/4^{th}$ of the weight of steel) therefore it can be easily transported and installed. However there are a few limitations like low modulus of elasticity, non-yielding characteristics, uneconomical and brittle nature.

Manufacturing Process

Fibre Reinforced Polymers (FRP's) are composite plastics made using a matrix or a binding agent reinforced by a fibre material. Thus the FRP manufacturing process comprises of 2 steps, making the fibrous material and bonding the material with a polymeric plastic. The Fibre manufacturing process involves Weaving, Braiding, Knitting and Stitching are the processes through which fibre preforms are manufactured. The second part of the FRP manufacturing process which involves bonding the fibre and polymeric plastic which can be achieved through various processes like wet layup, compression moulding, bladder moulding, chopper gun, autoclave and vacuum bag, mandrel moulding, filament winding and pultrusion.

PRACTICAL APPLICATIONS OF FRP AS REINFORCEMENT IN CONCRETE STRUCTURES AND CASE STUDIES

FRP bars have gained worldwide popularity in countries like USA, Canada, Japan, and Europe and have been achieved as an internal reinforcement in bridges, parking garages, water tanks, tunnels and marine structures giving commendable competition to steel bars in areas where corrosion resistance is a major task. Some of the practical on site applications of FRP bars as an internal reinforcement in concrete structures and related case studies are as follows:

Bridges

Fibre Reinforced Polymer (FRP) bars being light weight, corrosion resistant and less maintenance is an ideal application to bridges and have been successfully installed in footbridges and cable-stayed bridges up to a span of 10-40m, designed to carry pedestrian loads as well as the weight of tractors and trailers. In most cases, GFRP and CFRP bars are

used as main reinforcement in deck of the bridges. The deck slab of the bridge is designed to sustain significant axial compression force arising from cables and bending moment as resulted from live loads and dead loads. Some practical examples are:

- Cable- Stayed Storchen Bridge, Switzerland where 22 steel-stay cables and 35m long two Carbon FRP-stay cables are installed,
- Footbridge in Denmark where the deck of the bridge is post tensioned with 6 CFRP tendons, 12.5mm 7-wire strands and CFRP bars and stirrups are used and
- Nipigon Cable-Stayed Bridge, Canada whose deck is comprised of concrete panels reinforced totally using GFRP bars and supported on transverse steel beams.

Other than Bridges, FRP bars have been successfully applied toconcrete water tanks in Canada, parking garages, etc.

CONCLUSIONS

From the above study, it is concluded that FRP bars are an excellent emerging alternative to traditional steel bars. It has been practically used and tested on site in many countries like USA and Canada in various applications like bridges, tunnels, water tanks, parking garages, highways etc. In India, the FRP bars have not yet gained full applicability due to lack of awareness, less production and demand, non-existence of proper design codes and traditional mentality of consumers in construction industry on using conventional steel bars as reinforcement in concrete structures. FRP is a wonder material and encourages researchers, students and manufacturers to promote the use of FRP bars in construction in India.

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DEVELOPMENT OF MICROBIAL AMENDED GEOPOLYMER

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ABSTRACT
Due to increase in green house gases specially Carbon di-oxide in the atmosphere which is produced during the production of cement, the researchers are trying to replace cement with flyash and other substitutes as much as possible. Moreover large quantities of fly ash is also being produced by the thermal power plants which is very harmful for environment as it occupies a large amount of cultivable land, causes air pollution related health hazards and ground water contamination. A geopolymer mortar is being designed by the amendment of genetically-enriched *Bacillus subtilis* bacterial cells (10^5 cells/ml) to minimize the environmental pollution to atleast some extent. This paper discusses about the compressive strength and durability of microbe amended geopolymer mortar and its self-healing attributes. Heat curing of the flyash based geopolymer is an important factor for polymerization and in order to gain high mechanical strength. For the development of conventional heat-cured geopolymer samples, the bacterial cells along with the activator solution was being mixed uniformly with flyash, sand and was casted followed by heat curing at 60 degree Celcius for 48 hours and room temperature curing until testing. Artificial cracks was formed by applying partial breaking load (50%). Increment in the compressive strength as well as durability of the microbial incorporated heat cured geopolymer mortars compared to the control samples (without genetically enriched microbe incorporation) is observed.

INTRODUCTION

Day by day there is an increase in the production of cement which is leading to a huge production of CO_2 and other gases causing global warming and other environmetal problems [1]. Due to increasing demand of construction the researchers are trying to replace cement with other alternatives like GGBS, rice husk ash, fly ash, metakaolin etc [2]. Cracks are one of the major causes of deteoriation and decrease in durability of cementitious materials [3]. Recently various methods are there to repair the cracks but it is being seen generally that these processes are very costly and it consumes a prolonged period of time to repair. Self-healing by bacteria is a novel approach for repairing of the micro-cracks as well as crack size which is more than 0.9mm that are little difficult to be reformed, can be healed by Microbiologically Induced Calcium Carbonate Precipitation (MICP) [4]. Micro-organism amended modified mortar not only have much more compressive strength but is also more durable compared to mortars without microbes [3]. Review of literature reveals that applications of several spore forming bacterias like *Bacillus subtilis* ,*Bacillus aerius* ,*Bacillus megaterium, Bacillus sphaericus, Sporoscarcina pasteurii* have showed increament in mechanical strengths and durability in concrete [2]. The self-bioremediation of genetically transformed bacterium (*Bacillus subtilis*) which is being transformed with a protein gene which is possessing the property of biosilicification from a facultative anaerobic hotspring bacteria named BKH2, incorporated sulphuric acid resistant geopolymer material has been analyzed in this work [3]. Our aim is to create genetically transformed microbe incorporated geopolymer for future construction objectives with self-healing properties.

LITERATURE REVIEW

Researchers have shown that incorporation of anaerobic hot spring bacteria in concrete leads to increament in mechanical strength as well as durability due to formation of new phases inside the cementitious matrices like gehlenite [4,5]. But the lifetime of these anaerobic bacteria is short inside the cementitious matrices which limits the use of these bacteria as potential self healers of concrete. Genetically enriched alkaliphilic spore forming bacterium, *Bacillus sp.* by the amendment of the bioremediase-like gene, has been seen to increase the mechanical strength and durability of cementitious mortar samples along with autonomous healing attributes [5]. Chatterjee et al. has shown a comparative analysis of transformed *Bacillus sp.* incorporated geopolymer mortars with high mechanical strength as well as thermal resistance [6]. In this work, a comparative investigation has been done on structural behaviour and self healing attributes of geopolymer bio-mortars prepared by with and without incorporating bacteria. This study demands that a low cost but high performance bacterium amended green cement- alternative can be developed for construction purposes by using 100% fly ash, which will not only reduce the hazardous effect of fly ash but also will reduce the concentration of CO_2 gas in the atmosphere.

METHODOLOGY ADOPTED

Materials

For making of geopolymers, low Calcium content Class F dry fly-ash is being used that is collected from the National Thermal Power Corporation Ltd in Farakka, India. Ordinary Portland Cement 43 grade [7] and standard Ennore sand [8] were used for the study. Commericial grade sodium hydroxide (NaOH) of 99% purity and liquid sodium silicate (Na_2SiO_3) (specific gravity of 1.53 gm/cc) were taken to make the activator solution. Transformed *Bacillus subtilis* is grown in our laboratory in a special mineral media which is having pH 10.0 [6].

Mix proportion and curing

The various categories of samples are firstly the control OPC mortar, then OPC mortar along with bacteria, third is control Fly ash geopolymers without bacteria and lastly Fly ash geopolymer with bacteria are prepared for the work . For the preparation of fly ash control and fly ash amended with microbe samples, the Fly ash and bacterial cells (10^5 cells /ml concentration) with the activator fluid were mixed uniformly for few minutes before casting . After casting, all the samples were kept at 60 degree Celcius for 48hours then dollowed by removal from their respective moulds and kept at room temperature(37˚C ± 2) for respective days until testing.

Mechanical strength and Ultrasonic pulse velocity (UPV) study

After respective curing for 28 days, the ultrasonic pulse velocity and compressive strength of all the mortars were determined. The ultrasonic pulse velocity was determined by using PUNDIT plus PC 1007 UPV machine, UK Make as per ASTM C597-02 [9]. The compressive strength of each UPV-tested sample was measured by using Digital Compression Test machine, India Make. After 28 days of respective curing, the average UPV and breaking load of each category mortars (n=10) were measured. After measuring the breaking load, 50% of the corresponding breaking load was applied to the rest bio-mortars (n=10) of each category samples. The maximum breaking load applied to the specimen was recorded to calculate

Durability test

Water absorption capacity was determined as per Neville's method to the 50% breaking load applied mortars [10] and sulphate and acid resistance test was carried out as per the guideline

of ASTM STP663 [11]. For rapid chloride ion permeability test (RCPT), test was done as per ASTM C1202 [12].

Statistical analysis

There were 4 categories samples (20 samples in each) as described in methodology section were prepared. Each experimental data were presented as average ± S.D. Each set of experiment was repeated at least three times.

RESULTS

Table 1 shows the increment in compressive strength after giving 50% load in bacteria applied cementitious mortars as well as geopolymer mortars. It has been seen that there is a increment in UPV test and durability tests also (which has been shown in tables 2,3,4,5) of the bacterium applied geopolymer samples with respect to the geopolymer mortar without bacteria and cement control.

Table 1: Compressive strengths of Bacteria incorporated mortars

Samples	Compressive Strength (MPa)					
	28 Days	50% Load	3 Days	7 Days	14 Days	28 Days
1.OPC	19.6 ± 1.2	9.8 ± 0.9	10.2 ± 1.4	11.00 ± 1.3	11.9 ± 1.2	12.6 ± 1.5
2 .OPC + Bacteria	23.2 ± 1.25(18.3↑)	11.25 ± 1.1	12.5 ± 0.96 (22.5↑)	14.6 ± 1.4 (32.7↑)	16.5 ± 1.7(38.6↑)	19.0 ± 1.4 (50.7↑)
3.Fly ash	32.5 ± 1.9	15.25 ± 1.6	17.0 ± 1.6	17.5 ± 1.0	18.0 ± 1.1	18.89 ± 1.0
4.Fly ash + Bacteria	38.11 ± 1.11 (17.2↑)	19.00 ± 0.78	21.5 ± 0.9 (26.4↑)	26.2 ± 1.6 (50.0↑)	29.9 ± 1.2 (66.1↑)	32.0 ± 1.9 (69.4↑)

.

Table 2: Ultrasonic Pulse Velocity Results of Bacteria incorporated mortars

Samples	Velocity (Km/s)					
	28 Days	50% Load	3 Days	7 Days	14 Days	28 Days
1.OPC	3.5 ± 0.2	1.62 ± 0.8	1.8 ± 0.8	2.1 ± 0.9	2.2 ± 1.6	2.3 ± 1.2
2 .OPC + Bacteria	3.7 ± 0.45 (5.7↑)	1.8 ± 0.2	2.1 ± 1.0 (16.6↑)	2.4 ± 1.0 (14.2↑)	2.6 ± 0.8 (18.1↑)	3.1 ± 0.9 (34.7↑)
3.Fly ash	4.0 ± 0.9	2.0 ± 0.4	2.6 ± 0.9	2.75 ± 0.9	2.9 ± 1.2	3.0 ± 1.3
4.Fly ash + Bacteria	4.4 ± 0.9 (10.0↑)	2.2 ± 0.7	3.2 ± 1.0 (23.0↑)	3.7 ± 0.7 (34.5↑)	3.9 ± 1.0 (34.48↑)	4.1 ± 0.95 (36.6↑)

Table 3: Water Absorption Test of Bacteria incorporated mortars

Samples	Mass (g)				
	Initial mass	Mass after 30 minutes	% increase	Mass after 24 hours	% increase

1.OPC	738 ± 4.67	750 ± 5.6	1.6	761 ± 4.2	3.1
2 .OPC + Bacteria	725 ± 5.3	736± 6.2	1.5	745 ± 3.6	2.7
3.Fly ash	729 ± 5.2	741 ± 4.3	1.6	751 ± 8.1	3.0
4.Fly ash + Bacteria	718 ± 6.9	726 ± 3.5	1.1	733 ± 5.2	2.0

Table 4: Sulphate Resistance Test of Bacteria incorporated mortars

Samples	Mass (g)		
	Initial mass	Mass at 120 Days	% Increase of mass
1.OPC	739 ± 3.6	765 ± 5.6	3.5
2 .OPC + Bacteria	723 ± 5.3	745 ± 5.6	3.0
3.Fly ash	730 ± 4.2	756 ± 5.1	3.5
4.Fly ash + Bacteria	717 ± 5.9	730 ± 5.4	1.8

Table 5: Rapid Chloride Permeability Test of Bacteria incorporated mortars

Samples	Charge (C)
1.OPC	4499 ± 60.90
2 .OPC + Bacteria	2700 ± 77.50 (↓39.7)
3.Fly ash	2467 ± 53.41
4.Fly ash + Bacteria	1529 ± 62.65 (↓38.0)

CONCLUSIONS

It has been seen that greater strength and more durable geopolymers can be designed by amending transformed *Bacillus subtilis* cells in concentration of 10^5 cells/ml. The spore forming aerobic transformed bacteria incorporated geopolymer with autonomous bioremediation property can be used as cement-alternative for future construction purposes(road construction specially). The endospore forming capacity of the bacterium is an added advantage as it helps in remaining active within the geopolymer matrices for years. This study will provide an eco-friendly, pollution free, non-hazardous biotechnologically improved way for fulfilling the mostly desired self healing geopolymers. Thus it can be concluded that bacteria applied geopolymer may be used as green cement alternative in construction due to its enhanced compressive, tensile and textural strengths, better durability in near future

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PERFORMANCE ENHANCEMENT OF FLAT SLAB BUILDINGS UNDER INDEPENDENT AND CASCADING MULTI-HAZARD LOADING

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ABSTRACT

Flat-Slab building is very popular from the aesthetic and architectural point of view. From functional aspect a flat-slab building is more efficient than a R.C. frame building. So, construction of Flat-Slab building is increasing also in high seismic zone. Flat slabs are favored by both architects and clients because of their aesthetic and economic advantages.

When R. C. flat slab Building constructed in high seismic zone then it is vulnerability again Independent Multi Hazardous or Cascading Multi Hazardous, especially in the case of Nuclear Power Plant, Hospital, Heritage Structure, Educational Building and the building with National Importance.

Many researchers recommend that flat slab structure should be designed to resist not only gravity loads but also seismic load. If the flat slab structure is constructed at the high seismic zone with different topographical condition then the building is more susceptible against Independent multi hazardous and cascading multi hazardous. Punching shear is also a most popular cause of flat slab failure.

The Stiffness and flexibility are the two of the important factor and plays a vital role in the earthquake resistance design of the structure, for any structure the ratio of stiffness and flexibility should be appropriate.

Thus, this research is an attempt to study performed enhancement of flat slab structure under Independent and Cascading Multi-Hazard Loading with the help of bracing system. Static, Response Spectrum of ground motion, Nonlinear Static Analysis, Pushover for ductility and deformability, will be also performed to assess the performance of the flat slab buildings against independent and cascading multi-hazard loading. The present research deals with the study and scope of bracing system to develop earthquake resistant and fire resistance structure especially in the case of flat slab concrete structure. The costs of construction of the buildings will also compare. An extensive study on the lumped plasticity model as per FEMA 356 with the varying position of the plastic hinge and its effect on the pushover curve will also discuss in the research.

KEYWORDS: Flat slab, Nonlinear Static Analysis, ductility, Lumped plasticity model, Pushover analysis, Bracing System. Independent Multi-Hazard Loading, Cascading Multi-Hazard Loading

INTRODUCTION

Common practice of design and construction is to support the slab by beam and beams by column. This may be called as beam-column construction. The beam reduces the available net clear ceiling height. The aesthetically this type of construction is poor but performance of those buildings are quite good. In recent practice slabs are directly put on the column for aesthetic and architectural point of view. The load transmission path changes due to deletion

of beams. But the safety of those building is to be checked. Seismic codes are also silent on the design of flat slab. But from the past history it can be understood that the flat is very vulnerable in earthquake point of view. Figure 1.1 shows the flat slab failure in an earthquake.



(a) (b)

Fig-1.1 Flat slab failure in earthquake at Tropicana casino parking

Keeping the failure in mind in this dissertation the performance of the flat slab building over a similar conventional building is estimated. Frequency analysis, response spectrum analysis, non-linear pushover analysis and non-linear time history analysis has performed to ensure the stability of flat slab building.

In this paper the feasibility of flat slab in high seismic zone is checked. The cost effectiveness of flat slab over a conventional building is also estimated. The comparison of the response parameters of flat slab and conventional building by using IS:1893-1984 and euro code method is also included in the scope of the paper. The comparison of deflection of flat slab building and conventional building is also presented here.

Flat Slab Building

General

Flat slab are preferential by both architects and clients because of their aesthetic and economic advantages. Though this form of reinforced concrete construction gives several advantages over frame structure, they also present some disadvantages because of brittle punching failure and large deformation. Many researches propose that flat slab should be designed to resist only gravity loads when used in higher seismic zones and lateral loads should be carried by lateral resist in system.

Slab-column connections are the first point of yielding in case of building without braces or stud rail. The shear capacity of this connection reduces drastically in the presence of lateral loading and hence, in such type of buildings should be carefully designed. Flat slab can be supported by column and by a column capital and drop panel. The flat slab system has been adopted in many buildings constructed recently due to the advantage of reduced floor heights to meet the economical and architecture demands. The design of flat slabs is typically governed by the punching shear strength at failure.

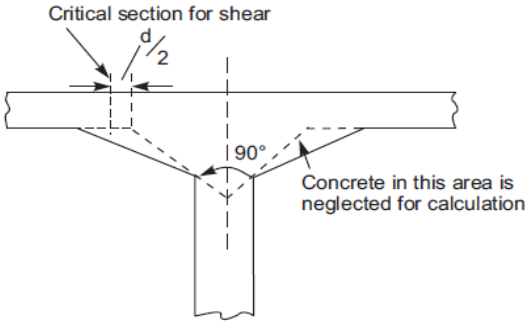
Types of Flat Slab

Flat slabs can be classified as per the slab column junction. There are four types of flat slabs are commonly used in buildings. They are as follows

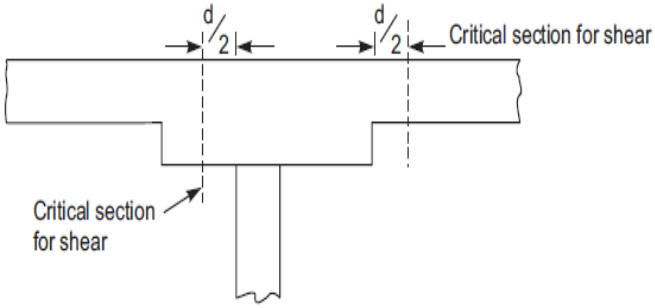
- Slab without drop and column with column head
- Slab with drop and column without column head.
- Slab without drop and column with column head.

- Slab without drop and column head

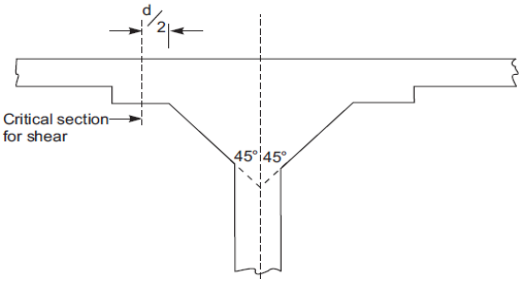
The various flat slabs are shown in Fig-2.1.



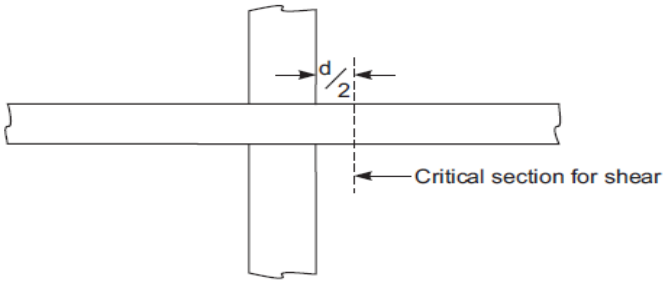
(a) Slab without drop and column with column head



(b) Slab with drop and column without column head



(c) Slab without drop and column with column head



(d) Slab without drop column head

Fig-2.1 Different types of flat slab

Building data

1. Grade of concrete used is M-20 and grade of steel used is Fe-415
2. Floor to floor height is 3.1m
3. Plinth height above ground level is 0.55m
4. Depth of foundation is 0.65m below ground level.
5. Slab thickness 200mm.
6. Size of Column is 500 X 500 mm
7. Live Load on floor is 3 kN/m^2 and Live Load on roof is 1.5 kN/m^2.
8. Site located in seismic Zone IV.
9. Building is resisting on medium soil.
10. Take Importance factor as 1
11. Building frame type is special moment resisting frame (SMRF)
12. Density of concrete is 25 kN/m^3.
13. Size of bracing is 250 x 250.

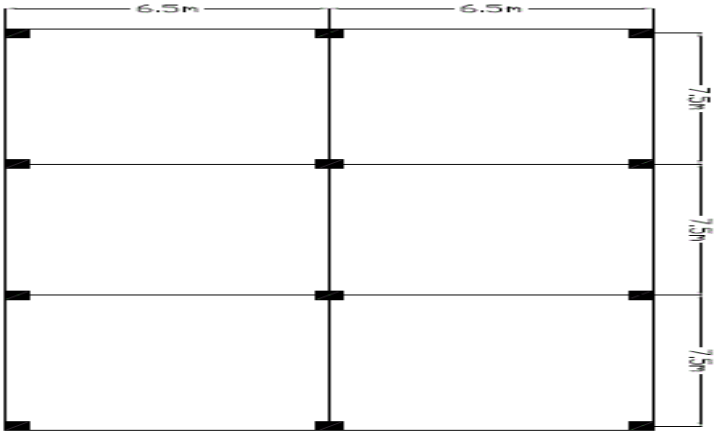


Fig-3.1 Floor plan of flat slab building

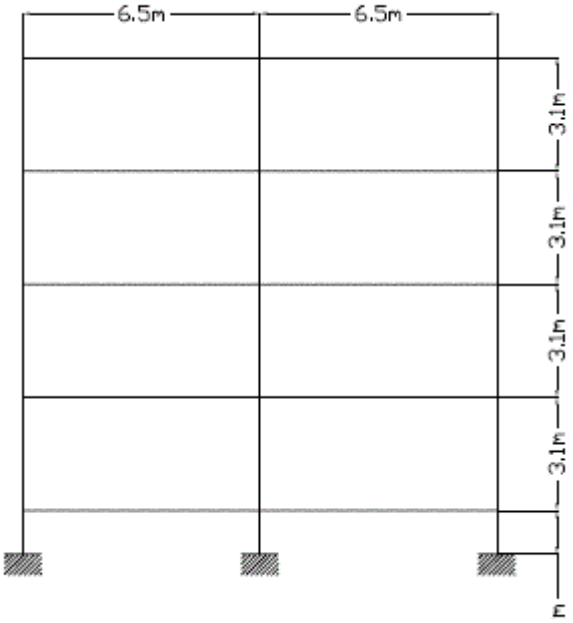


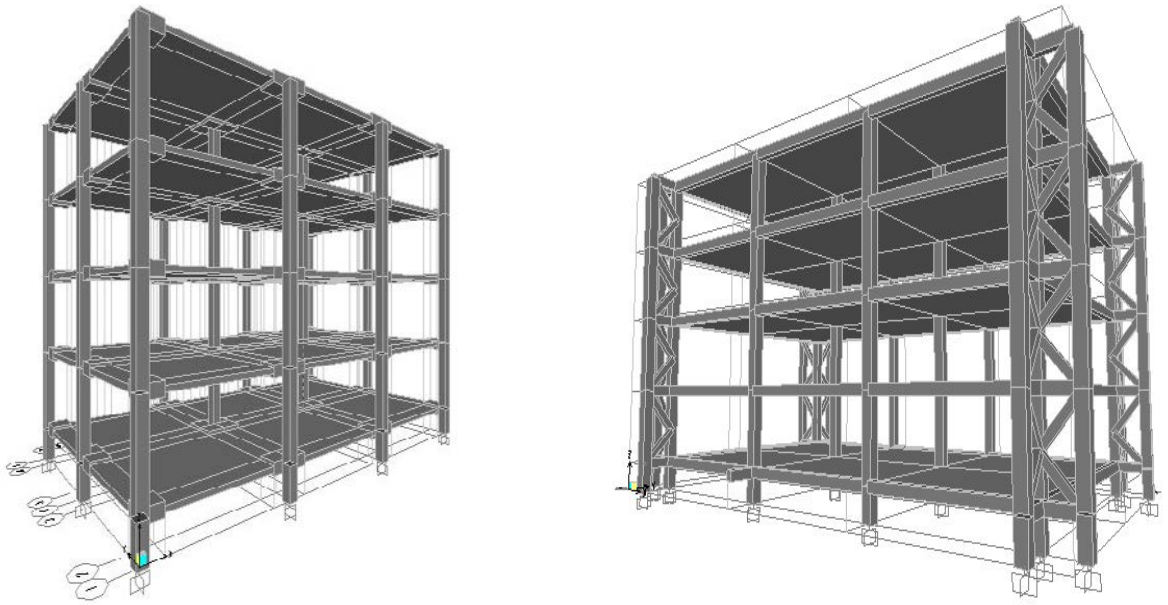
Fig-3.2 Elevation of flat slab building

The building is considered as a special moment resisting frame (SMRF).
For comparing the performance of the flat slab building with some different model of same building has been analyzed. The basic features of those buildings are given in Table-3.1.

Table-3.1 Dimensions of the components

Model no.	Slab thickness mm)	Beam	Drop Pane	Bracing
Model-1 (Conventional Flat Slab with Drop Panel)	200		2m x 2m x 250mm	
Model-2 (Flat Slab with Brace System)	200	300 x 450		250mm x 250mm

The flat slab with drop panel as well as flat slab building with bracing system models has shown in Fig-3.2.



(a) Flat slab building with drop panel (b) Conventional building with bracing system

Fig-3.2 Two building models

Load Combination

No	Load Combination
Comb-1	(1.3 DL + 1.6 LL)
Comb-2	(Comb-1 x 0.75 +EQ-X)
Comb-3	(Comb-1 x 0.75 +EQ-Y)
Comb-4	(Comb-1 x 0.75 +(-EQ-X))
Comb-5	(Comb-1 x 0.75 +(-EQ-Y))
Comb-6	Envelop of (Comb-1+ Comb-2+ Comb-3+ Comb-4+ Comb-5)

LITERATURE REVIEW

Seismic response of flat slab building has been a subject of discussion since many decades. A lot of research work has taken place in this field addressing all relevant issues pertaining to the modeling, analysis and construction of flat slab structures. Literature survey for seismic safety of flat slab buildings has also been covered.

Park et al. (2008) found that Equivalent Frame method is not appropriate in accurately predicting the response of two-way slab systems under lateral loads. The equivalent frame method was for two-way slab as a simple approximate method. Currently design code, ACI 31805[2.1] permit the EFM for the analysis of two-way slab system under gravity loads s well as lateral loads such as seismic loads.

Subramanian (2005) has found that to enhance the punching shear strength of lat slab, the shear stud reinforcement is found to provide economic and aesthetic solution. They not enhance the shear capacity but also result in flexural failure of the slab and thus increasing the ductility of flat slab, which is very important in earthquake prone zone.

Carl Erik Broms (1990) has proposed a method for the prediction of punching shear strength of the flat slab. This method accurately predicts the failure loads and the deformation in the whole range of failure modes, from the overall flexure failure to brittle punching failure. This method also account for the size effect, increasing brittleness at high strength, symmetrical and unsymmetrical loading and reinforcement ratio. He concluded flat

slab has two disadvantages i) poor capacity of slab to distribute the negative support moment and ii) the risk of brittle punching failure. But this disadvantage can be overcome by using shear reinforcement with the slab in the accurate fashion.

METHODOLOGY ADOPTED

There are three type of multi hazardous, Independent multi hazardous, cascading multi hazardous, concurrent multi hazardous. Independent hazardous may be Earthquake, High wind, Strom, Flood, Landslide, Volcanic eruption, War civil strike (Bomb Blast).

Cascading multi hazardous refer to that due to one hazardous another hazardous has occur, for example, due to earthquake building has caught fire.

Concurrent multi hazardous is the hazardous when three or more hazardous occurs at same time. The performance of structure depends on the vulnerability of the structure and the seismic hazard of the region and type of multi hazardous hit to the structure. Vulnerability has been defined as the degree to which a system, or part of it, may react adversely during the occurrence of a hazardous event. This concept of vulnerability implies a measure of risk associated with the physical, social and economic aspects. Increasing the resilience of a structure through specific strengthening measures to reduce their probability of collapse. As resilience increases, the degree of damage for a given intensity hazard decreases.

Stiffness and flexibility are two of the important factor for the earthquake resistant design of the structure. The value of Stiffness and flexibility for any earthquake resistant structure neither be so high nor be very less. The ratio of Stiffness and Flexibility should be appropriate.

In the case of stiffness, it plays a vital role in the earthquake-resistant design of the structure. For example, if the column is resisting member and if we increase the stiffness of the column then it will change the fundamental time period of the structure as stiffness is inversely proportional to time period.

$$\text{Stiffness} \propto \frac{1}{Time\ Period} \qquad \text{Eq. 1.1}$$

If, we increase the stiffness of any column, then for the same deflection that column will resist more earthquake forces.

$$F\ (Earthquake\ Force) = k.x \qquad \text{Eq. 1.2}$$

$$F_t = m\ \ddot{x} + c\dot{x} + \text{k x} \qquad \text{Eq.1.3}$$

Where, k is the stiffness of the member and x is the deflection during Earthquake, $\ddot{x}$ is the acceleration and $\dot{x}$ is the velocity.

Therefore, the increase of stiffness for the same deflection, that critical member will resist more Earthquake Forces, but stiffness should be in appropriate limit.

If the stiffness of the building is very less then there will be a Second-Order failure.

Keeping these principal in mind in this research the performance of the Earthquake Resistant Building over a similar conventional building is estimated. Frequency analysis, response spectrum analysis, non-linear pushover analysis and non-linear time history analysis will be performed to ensure the stability of the building.

In this research, the feasibility of building in the high seismic zone will be checked. The cost-effectiveness of Earthquake Resistant Building over a Conventional Building will also estimate. The comparison of the response parameters of Earthquake Resistant Building and Conventional Building by using IS:1893-1984 and Eurocode-8 method is also included in the scope of the research. The comparison of the deflection of the Earthquake Resistant Building and The conventional Building will also estimate.

Effects of High Temperature on Concrete

While it is important to understand the performance of individual concrete members during a fire, the behavior of the same structural elements within the context of a complete structure can vary widely from their independent responses.
This is small for a concrete section and it is uncertain whether this data would be scalable to larger members or applicable to members with normal-strength concrete. The study concluded that further work is required in this area.

Microbiologically induced calcium carbonate perecipitation (MICP) consisting of a series of complex biochemical reactions are summarized as under (Stocks-Fischer et al.,1999; Achal et al.,2010 and De Muynck et al.,2010.

$$Ca^{2+} + Cell \rightarrow Cell - Ca^{2+} \quad \text{Eq. 1.4}$$

$$Cl^{-} + HCO_3 + NH_3 \rightarrow NH_4Cl + CO_3^{2-} \quad \text{Eq. 1.5}$$

$$Cell + Ca^{2+} + CO_3^{2-} \rightarrow Cell - CaCO_3 \quad \text{Eq. 1.6}$$

Cracks thus healed by biologically producing limestone precipitating microorganisms would make the concrete more durable as well as more sustainable in future. These organisms are sometimes embedded in concrete matrix after immobilization on diatomaceous earth. It will start the precipitation of $CaCO_3$, which will fill the cracks and also form a film of calcium carbonate around cover of concrete to reduce the effect of high temperature, as calcium carbonate starts to evaporate around 1000 °C.

RESULTS

Static Analysis Results and Discussion

In this analysis wind load is not consider. Support conditions are considered fully fixed. The ground motion can be applied in one, two or three direction, individually or simultaneously.
The building has been modeled as 3D space frame with six degree of freedom at each node using SAP 2000 software
In static analysis, the value of torsion, moment and shear force in flat slab building with drop panel is high as compare to flat slab building with brace frame system. Only self-weight and live load of the building are considered. Maximum values of torsion are comparing Fig-4.1

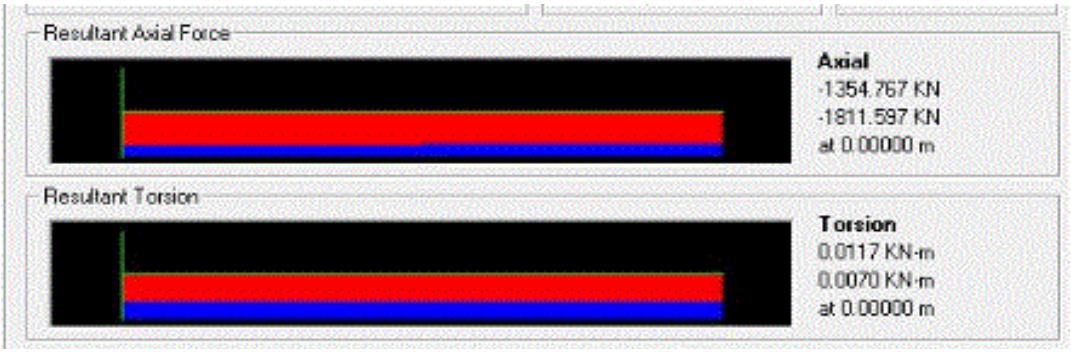


Fig- 4.1 Maximum torsional moment in flat slab with braced frame (max 0.0117 kN-m)

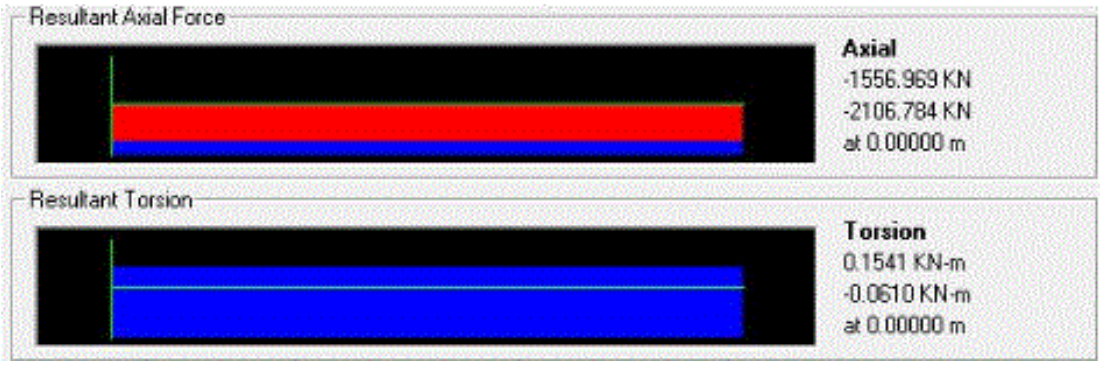


Fig- 4.2 Maximum torsional moment in flat slab with drop panel (max 0.1541 kN-m)

The maximum shear force in the flat slab with bracing system is 169.148 kN is observed

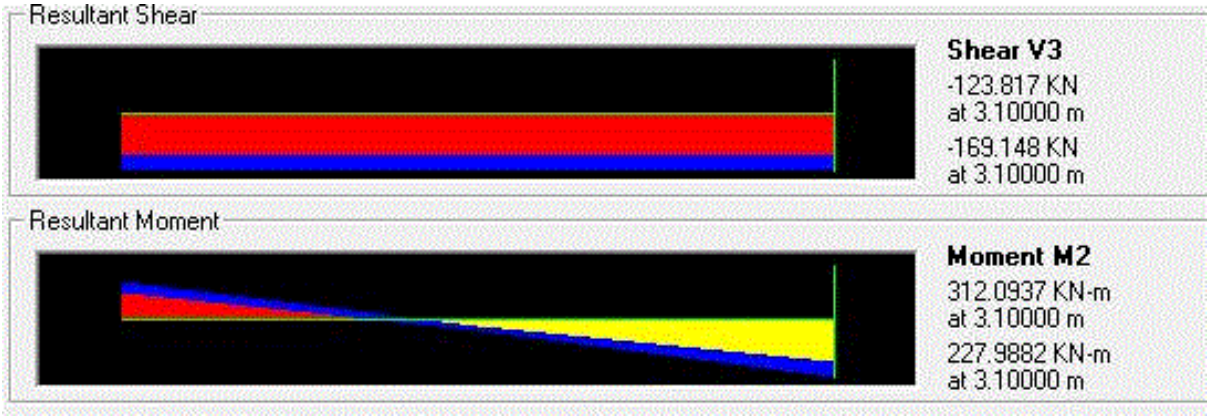


Fig-4.3 Maximum shear force flat slab with bracing system (max 169.148 kN)

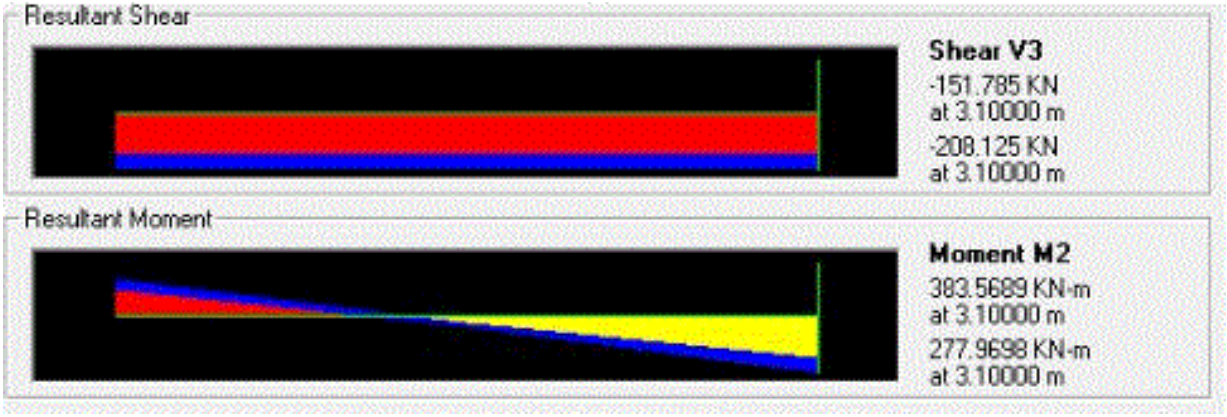


Fig-4.4 Maximum shear force flat slab with drop panel system (max 208.125 kN)

The maximum vertical deflection on the flat slab with drop panel is high as compare to flat slab with bracing system the deformations of the deflection are shown in Fig-4.2.

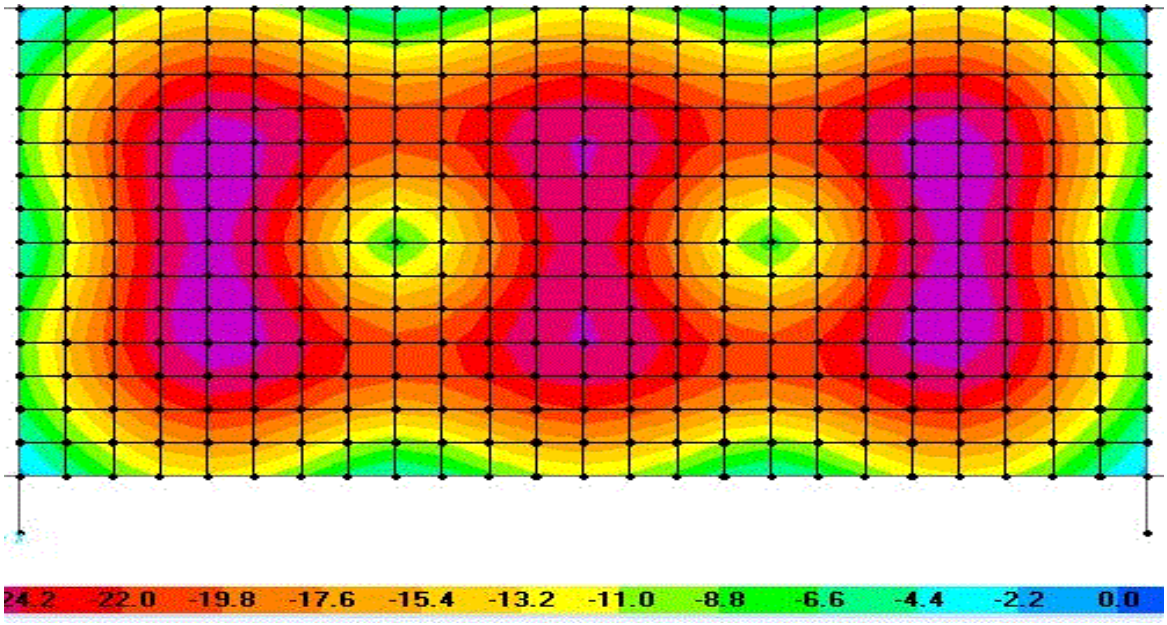


Fig-4.5 max deflection is 22 mm

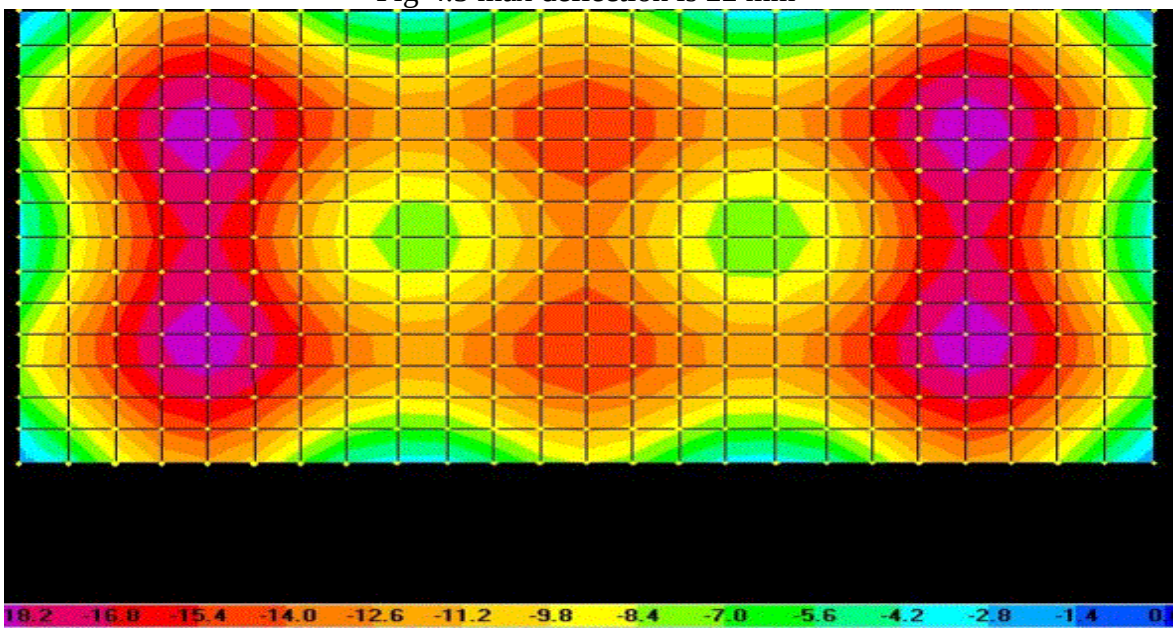


Fig-4.6 max deflection is 16 mm

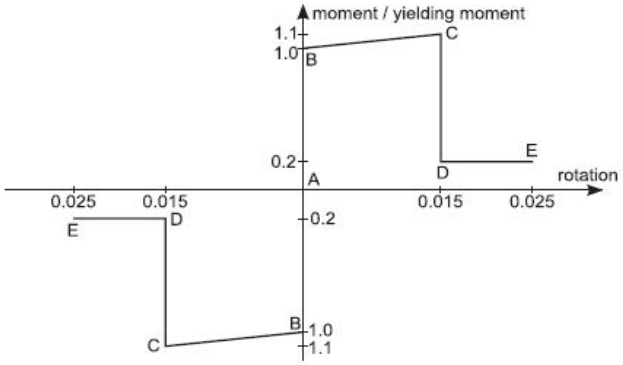
Push-Over Analysis Results and Discussion

Plastic hinge length (LP) is given by the equation.

$$LP = .08l_0 + .022 f_y d_b$$

where LP is the length of plastic hinge, l_0 =L/2 in m, f_y is the grade of steel in MPa, d_b is the diameter of the bar in m.

The moment curvature for the plastic hinge is plotted in Fig-4.6. To assign the moment curvature at the plastic hinge auto hinge is used in SAP. In the strain hardening region the performance points have to be assigned. IO, LS and CP are assigned in 0.003, 0.012 and 0.015 curvature (ASCE 41, 2007).



The pushover analysis for the flat slab buildings with bracing system and flat slab building with drop panel has performed.

The pushover curve of the two buildings has plotted in fig- 4.7.

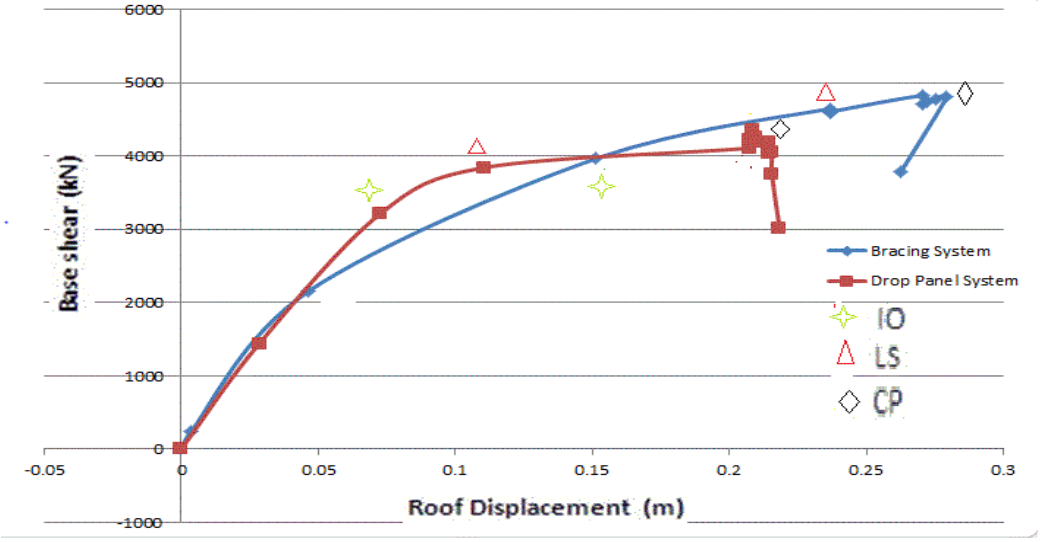


Fig-4.7 Pushover curve

Due to high punching shear the slab has fail after 30 mm displacement only. The performance of flat slab with bracing system is better than flat slab with drop panel. Flat slab with bracing system performance is better because in this building we provide bracing system at outer column, due to this bracing system the stiffness of outer column is increase. Due to this high stiffness outer column attract more earthquake forces and flat slab is less affected by earthquake force and give better performance. So the performance of the flat slab building with bracing system is better than drop panel system. It can carry 48600 kN force with a displacement of 30mm. Ductility of bracing system is also more then drop panel system.

Flat slab building can carry baser shear then flat slab building with drop panel

CONCLUSIONS

1. In this paper we compare two flat slab building with different lateral load resisting system.
2. The structural cost of flat slab building with bracing system and drop panel system are 4755686 birr and 4707310 birr. The cost of flat slab with bracing system is 1.15 % more than the flat slab with drop panel system

3. The value of drift in flat slab with bracing system is 2.5 time lower as compare to flat slab with drop panel system
4. The bending moment of bracing system flat slab is lower than flat slab with drop panel.
5. The torsional moment of flat slab with bracing system is lower than flat slab with drop panel.
6. The shear force in flat slab with bracing system is about 1.62 times lower as compared to flat slab with drop panel.
7. The later displacement in the column for flat slab with bracing system is 2.25 times lower as compare to flat slab with drop panel.
8. The deflection in the slab for flat slab with bracing system is 1.45 time lower as compare to flat slab with drop panel.
9. The ductility of flat slab with bracing system is more than flat slab with drop panel.
10. Flat slab with bracing system can resist more earthquake force as compare to flat slab with drop panel.
11. The performance of flat slab building with bracing system is better than flat slab building with drop panel against seismic forces.

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EFFECT OF SOLID65 ELEMENT IN FINITE ELEMENT MODELING OF REINFORCED CONCRETE BEAM

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ABSTRACT
Reinforced concrete has become one of most widely used engineering materials nowadays for any building application. It is important to study the behavior and failure of any material so that proper measures can be implemented for the construction of structural components and overall development of an efficient and safe structure. Finite Element Analysis (FEA) is a popular tool to analyze the structural components due to its capability to simulate the non-linear behavior and provide the response using mathematical modeling. SOLID65 is a legacy model in ANSYS used for three-dimensional modeling of solids such as concrete with or without reinforcing bars. The solid element has the ability to predict cracking in tension and crushing in compression which may be used to model the concrete while the rebar capability is available for modeling reinforcement either in discrete or smeared form. LINK180 is a 3D spar element in ANSYS library used to model steel reinforcement in discrete form while also being capable of plastic deformation. The analysis on reinforced concrete beam having various reinforcement sizes has been performed using ANSYS software and their results were compared for various parameters such as deformation, stresses, strain, cracking/crushing, load-deflection curve.

KEYWORDS: SOLID65, ANSYS, Reinforced Concrete Beam, Discrete reinforcement.

INTRODUCTION
Reinforced concrete structures are heavily being employed in engineering practices in many situations and applications. These structures are designed usingcode based predefined guidelines obtained from experimental data. One of the major problems in using finite element analysis is defining nonlinear material properties. The concrete material due to its brittle nature has a complicated stress-strain behavior which has various parameters affecting its response hence making it a very difficult and time-consuming work to understand its nonlinear response against different loading conditions [1]. ANSYS consists of a three dimensional eight noded element SOLID65 in its library capable of plastic deformation, cracking in three perpendicular directions, and crushing. This element has the ability to determine cracking in concrete due to tension and crushing of concrete due to compressive loads. The element can be embedded with reinforcement in smeared or discrete form however the latter has been preferred in order to simulate real life experiments and to account for the behavior of reinforcement. LINK180 element is used to model discrete reinforcement along with SOLID65 which is a 3-D spar element useful in a variety of engineering applications such as trusses, cables, links, springs etc. The element is a uniaxial tension-compression element with three degrees of freedom at each node: translations in the nodal x, y, and z directions [2].
In this research, firstly a model consisting only of concrete material is analyzed to study the material behavior and its deformation is compared with standard empirical formulas and then the concrete is reinforced using different tensile reinforcement to see the behavior of steel.

MATERIAL MODEL AND DEFINITION

Concrete

SOLID65 is a solid element used to define concrete in ANSYS, the solid can be defined to view cracking in tension and crushing in compression. The failure model developed by William and Warnke is used as concrete material model in ANSYS for multi-axial stress state. The cracking and crushing of concrete are defined through these failure criteria. Concrete material should consist of elastic properties and concrete material definitions. In Elastic definition, the modulus of elasticity and Poisson's ratio are required. For Concrete definition, axial tension strength of concrete and shear transfer coefficients between crack surfaces for open and closed cracks are required, the values can be varied as required. The tension behavior of concrete cannot be defined using any other material model. An additional stress-strain relationship for compressive behavior can be defined through a hardening model such as Multilinear Isotropic Hardening [3], in such cases the modulus of elasticity must be the same as the slope of the initial tangent of the defined stress-strain curve.The concrete model requires material properties such as elastic properties, material hardening, etc which are essential for simulations obtained from standard equation provided by code [4]. The Multilinear isotropic hardening model is used to describe the stress-strain behavior of unconfined concrete.

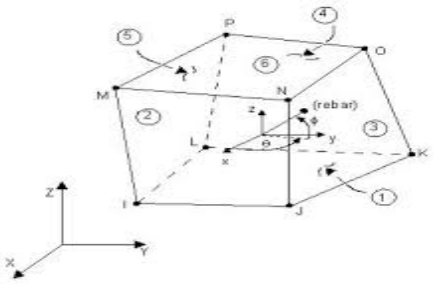


Figure 1: SOLID65 element

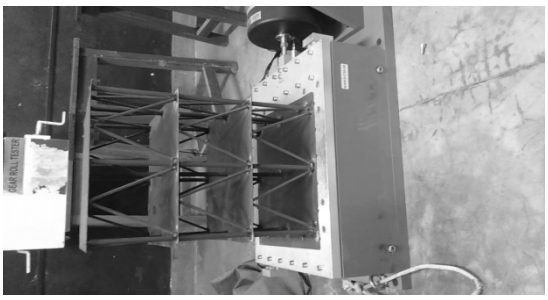


Figure 2: Concrete stress strain model

Table 1:SOLID65 material properties

Modulus of elasticity, E	25000 MPa
Poisson Ratio, v	0.2
Open shear transfer coefficient	0.3
Close shear transfer coefficient	0.8
Uniaxial tensile cracking stress	3.5 MPa
Uniaxial crushing stress	25 MPa
Tensile crack factor	0.6

Table 2: Steel material properties

Modulus of elasticity, E	2e5MPa
Poisson Ratio, v	0.30
Yield stress, f_s	400 MPa
Tangent modulus	0

Steel

The LINK180 element is used for discrete modeling of reinforcement inside the concrete. An elastic bilinear behavior is assumed for the reinforcing steel bar which is applied using bilinear isotropic hardening model defined using yield strength and tangent modulus of hardening. The steel bars show a linear elastic strength up to yield strength after which the

strain hardening occurs and then stresses drops till facture occurs [3]. The behavior is same in compression and tension loading. Along with the hardening model, incorporation of modulus of elasticity, Poisson ratio is necessary for defining the material model. The supports and loading impactor are modeled using SOLID185 elements having similar material definition to neglect the effect of stress concentration as done in previous studies [5].The LINK180 elements are modeled using line bodies having linear elastic isotropic property by providing modulus of elasticity and Poisson ratio. For the hardening properties, bilinear isotropic hardening parameters are defined using yield strength and tangent modulus. Similar to the reinforcing bars, the impactor and support pads are modeled using default structural steel element present in ANSYS library and only the modulus of elasticity and Poisson ratio is needed to be provided.

MODEL CONFIGURATION

A simple rectangular concrete beam, simply supported at the two ends is modeled and loaded at the center of the span to determine its deflection, stresses, strain and cracking behavior. The span of the beam is 3300 mm with dimensions 200mm x 400mm. The concrete grade M-25 is used for all the models. The first model has no reinforcement while the second model has varying tensile reinforcement of 12mm, 16mm, and 20mm.

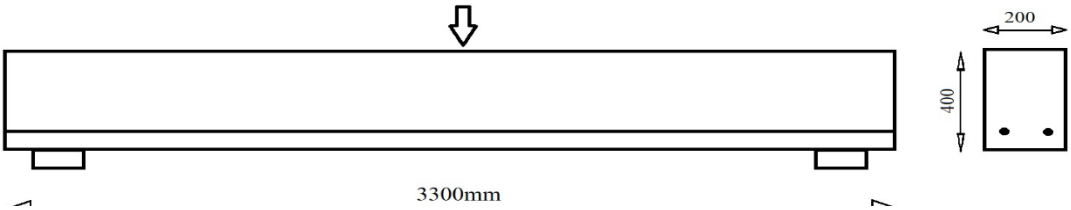


Figure 3: Beam dimensions

FINITE ELEMENT ANALYSIS

In order to check the accuracy of the results obtained from finite element analysis, the deflections from empirical formulas and ANSYS are compared which are shown in figure below for the plain concrete model. For further analysis, the results of reinforced concrete beam were plotted and compared with different tensile reinforcements in terms of deflection, stress-strain, and crack pattern at a constant load as well as at steel yielding stress.



Figure 4: Finite element model of beam

Meshing

The mesh size should be selected such that the nodes of concrete and steel bar coincide with each other, this help in linking the concrete with steel and hence they can deform together [5]. The meshing was done with linear setting to remove mid side nodes as the SOLID65 element does not allow for quadratic meshing.

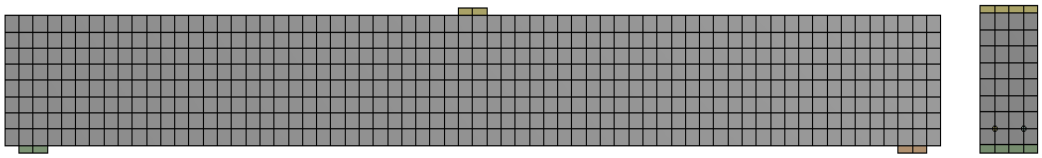


Figure 5: Meshing of beam

RESULTS AND DISCUSSION

Calibration of model

In order to verify the effect of loading and test the boundary conditions, the model was analyzed by application of force at the mid span and then the deflection was compared using empirical formula. Further, the load was applied through a steel plate as done in previous studies to neglect the effect of stress concentration at the top of the beam [3].

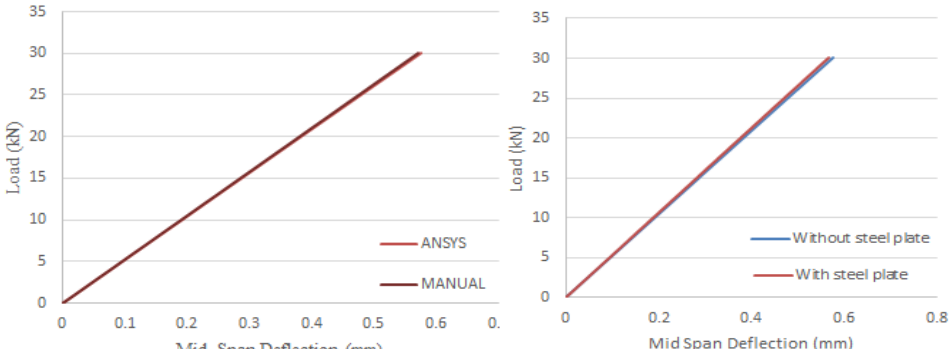


Figure6: Force displacement curves

The mid span deflections obtained from ANSYS (0.574 mm) matched the results of manual calculation (0.571 mm), hence our model was calibrated for further study. Instead of applying force directly on the beam surface, a steel plate was introduced between load and beam surface, which resulted in a deflection of (0.566 mm) however it helped in reducing the stresses at top of the beam, even distribution of load and also increases the convergence rate.

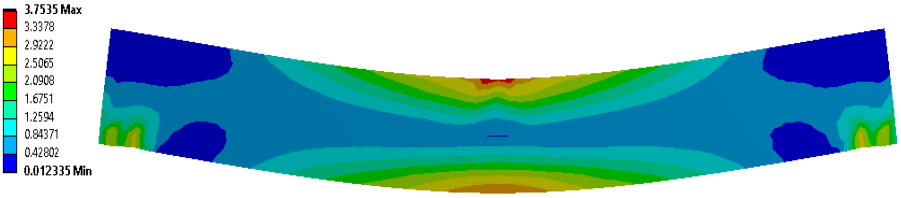


Figure 7: Stress concentration at top due to force

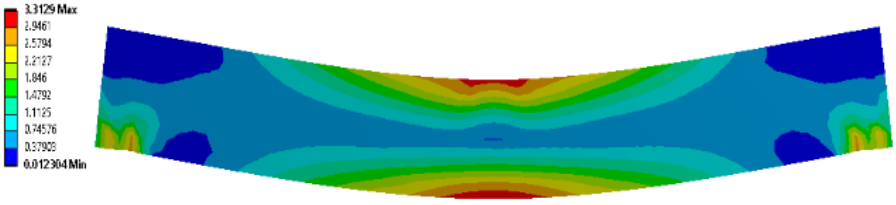


Figure8: Even stress distribution using steel plate

Analysis of reinforced concrete beam

The reinforced concrete beam having different tensile reinforcement was loaded at mid span and the results were obtained. Firstly, a constant load of 50kN was applied on all the three models to see the effect of reinforcement on the beam in terms of deflection, stresses in steel and crack/crush pattern. Further, the load was applied till the reinforcement yielding occurred and the results were plotted.

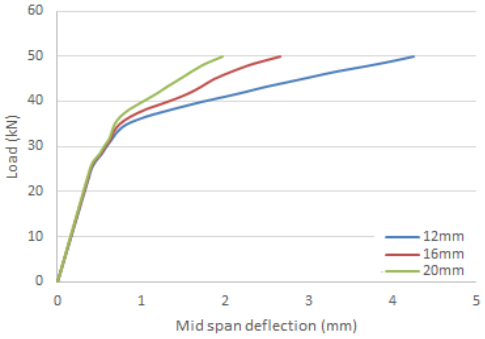


Figure9: Load deflection curve

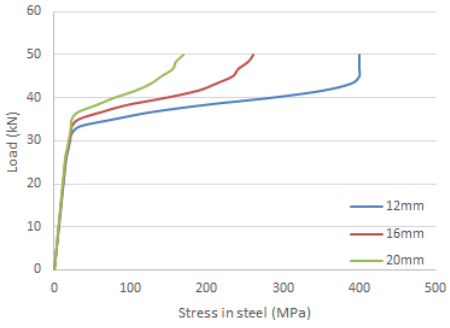


Figure10: Stresses in steel at 50kN load

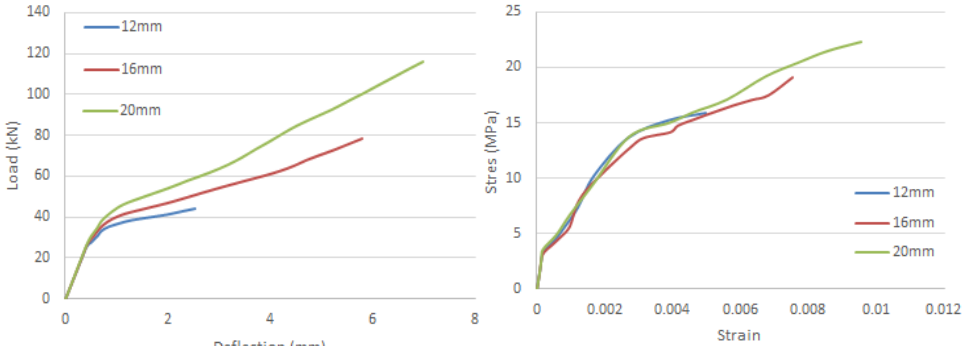


Figure11: Load deflection curve and stress strainof concrete at steel yielding

It was observed that the increase in the reinforcement size decreased the mid span deflection as the reinforcement provided additional stiffness to the beam, Also the stresses in the steel reduced gradually with the increase in tensile reinforcement. The crack/crush plots were also obtained for the beams and it was observed that the crushing in concrete was similar for all the three beams however as the size increased, there were some reduction due to increased capacity of the beams.The beams were loaded till steel yielding to determine the capacity of beams and it was found that the 20mm reinforcement beam had the highest capacityfollowed by 16mm and 12mm bars.Similiarly for the stress strain, it was seen that the higher reinforcement bars had more strain as compared to lower reinforcement bars because as the reinforcement increases, the section becomes over reinforced.

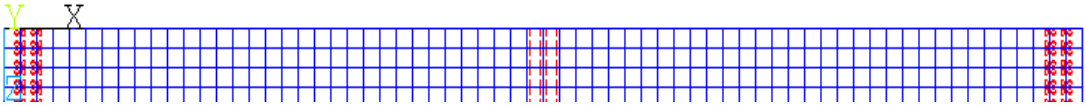


Figure 12: First crack in beam

Crushing and cracking were also determined for all the beams, first crack occurred at 30.8kN for 12mm beam, 33.68 kN for 16mm and 33 kN for 20mm bar. The final cracking pattern can be seen below at steel yielding.

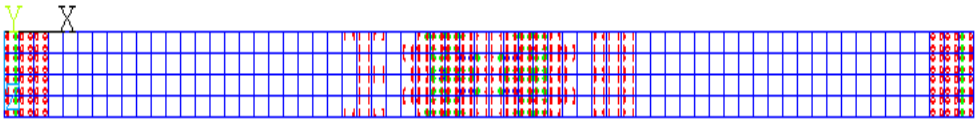


Figure 13: Crack pattern for 12mm rebar at 44 kN

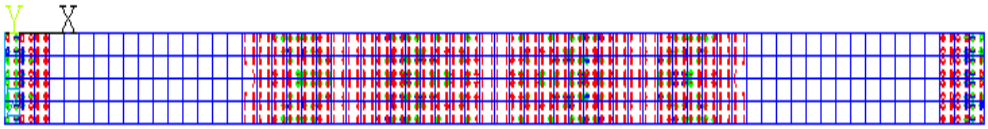


Figure 14: Crack pattern for 16mm rebar at 78 kN

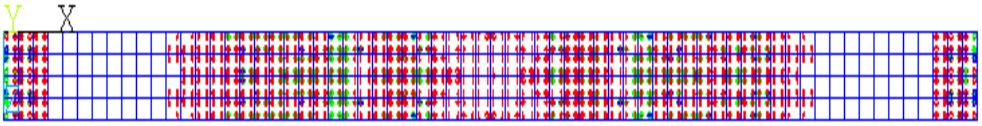


Figure 15: Crack pattern for 20mm rebar at 116 kN

From the crack patterns, it was observed that with the increase in tensile reinforcement, the load capacity of the beam is increased and able to resist crushing for significantly higher loads.

CONCLUSIONS

1. Reinforced concrete beams can be modelled using ANSYS using SOLID65 and LINK180 elements to determine deflections, stresses, crack/crush pattern etc.
2. The higher reinforcement sizes leads to decrease in the mid span deflection of the beam and also increase the load carrying capacity.
3. The yield stresses in steel decreases with the increase in the size of reinforcement and the crushing is more dominant with larger steel sizes due to higher capacity of the beam.
4. The SOLID65 is highly mesh dependent and will produce varying results for different mesh sizes and hence requires careful observation.
5. The material properties must be carefully assigned specially for concrete as the material has high nonlinearity.

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EFFECT OF CONFINEMENT IN REINFORCED CONCRETE BEAM USING ANSYS

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ABSTRACT
Concrete confined by stirrups has a greater ductility than unconfined concrete which is widely used in reinforced concrete (RC) structures and its behavior is a classic topic of discussion. It is very important to understand the response and failures of beams subjected to loads in order to save lives and reduce the damage to the building components. One of the most important functions of transverse reinforcement in reinforced concrete beams is to prevent the deformation of longitudinal bars and transfer of stresses from concrete to steel. ANSYS is widely used finite element analysis (FEA) software which has the tools to analyze structural elements and their non-linearbehavior. This is because ANSYS is capable of analyzing the non-linearbehavior of elements based on the finite element procedure. With this, researchers or design engineers can confidently predict in advance the actual response of various confined concrete beams not only in the linear-elastic region but additionally in the non-linear post-elastic region. In this study, reinforced concrete beams were modeled having various percentages of steel reinforcement subjected to bending tests and their behavior is studied in terms of deformation, stress, strain, crushing/cracking and force-displacement curves.

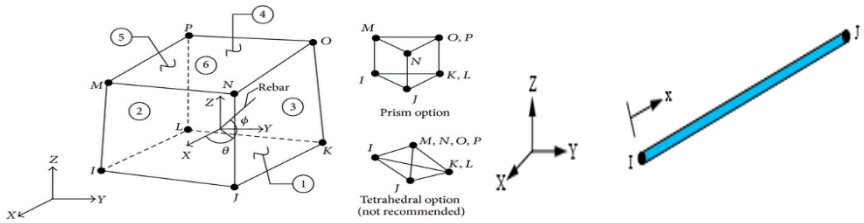
KEYWORDS: Reinforced concrete beam, Ductility, ANSYS, FEM

INTRODUCTION
Ductility is defined as the ratio of post-yield deformation to yield deformation of steel in reinforced concrete (RC) structures. Ductility is especially vital in structures because it ensures safety when structures are subjected to accidental, impact or seismic loads. In these cases, a ductile structure permits the stress redistribution and occurrence of plastic hinges within the critical parts of the structure without severe loss of resistance capacity. It eventually avoids the total collapse of the structure by issuing a warning of impending failure. There are various methods for modelling concrete structures through both analytical and numerical approaches. Finite element (FE) analysis is a numerical approach that is widely applied to the concrete structures based on the use of the non-linear behavior of materials. In reality, most of the problems are non-linear in nature, hence non-linear analysis is an effective tool to obtain an exact solution. FEA provides a tool that may simulate and predict the responses of reinforced and prestressed concrete members. It simulates the exact behavior of the material to evaluate strength in inelastic range and to identify the potential of the high load-carrying capacity of the components through redistribution, tensile and shear strength. Non-linear behavior of reinforced concrete beams is complex due to various parameters because of involvement of heterogenic material properties and cracking behavior of concrete, the non-linear behavior of reinforced concrete (RC) beams until the ultimate failure could be a complicated phenomenon [1]. The failure prediction of reinforced concrete elements is usually carried out using experimental testing, and the observations are recorded only at critical locations due to restriction at the cost of testing equipment and accessories. To avoid the destructive testing, cost-cutting of materials and labor, the behavior prediction of RC beams is typically carried out using numerical methods like Finite Element Method

which responds well to non-linear analysis as each component possesses different stress-strain behavior [2]. These things are efficiently covered by the software ANSYS which provides several elements for modelling of materials and apply loads to calculate the response [3].Though enormous research is already carried out on the non-linear finite element analysis (FEA) of RC beams, a substantial effort is made in this study to effectively utilize the ANSYS to generate a wide range of graphical display of the results, which may be even difficult to ascertain by experimental testing due to the reasons stated above. The use of SOLID65 concrete element along with 3D spar element LINK180 present in the ANSYS library has been done for discrete modeling of reinforced concrete beam and analysis has been done to obtain the effect of steel and concrete for various conditions.

MATERIAL AND MODELLING

Concrete: For modelling of concrete the ANSYS utilizes an eight-node element named SOLID65, which is a non-linear model of brittle material similar to concrete. It allows the presence of four different materials within each element; one matrix material (e.g. concrete) and a maximum of three independent reinforcing materials.SOLID65 is used for the 3D modelling of solids with or without reinforcement bars (rebar). It is a solid iso-parametric element with three degrees of freedom at each node-translations in the node locations x, y, and z directions. SOLID65is capable of plastic deformation, directional integration point cracking in tension in three orthogonal directions, and crushing in compression crushing besides incorporating plastic and creep behavior. The reinforcement (which alsoincorporates creep and plasticity) has uniaxial stiffness only and is assumed to be smeared throughout the element.



(a) (b)

Figure 1:(a)SOLID65: 3D reinforced concrete solid (b) LINK180: 3D spar element.

Steel: For the modelling of steel ANSYS provides auniaxial tension-compression shear element named as Link180 There were two ways to use it one was smeared and the other is discrete, discrete was considered to be more convergent as it subtracts the area of steel from total concrete which was the actual scenario whereas in smeared, the nodes are smeared as reinforcement in the concrete and behaved as one unit which is not practical [4]. A perfect bond between steel and concrete is considered in this research.

Failure Criteria for Concrete: The model developed using ANSYS is capable of predicting failure for concrete materials in which both cracking and crushing failure modes are considered. The two input strength parameters which are ultimate uniaxial tensile and compressive strengths are needed to define a failure surface for the concrete. So, a failure criterion of the concrete due to a multiaxial stress state was calculated by William and Warnke's (1975) constitutive model for multiaxial stresses. The cracking or crushing occurs when the principle stresses in any direction reaches more than the limiting stress. There are nine parameters in William Warnke's model described in literature however only five parameters are required to be input for basic analysis of reinforced concrete such as open and close shear coefficient, tensile and compressive stresses and tensile crack factor.

BEAM MODEL CONFIGURATION

The geometry of the full-size beam is 5500mm x 250mm x 400mm. The span between the two supports is 5200mm. Beam is simply supported and a uniformly distributed load has been applied on the whole beam. M-25 grade concrete and Fe-415 grade steel is used in this study. All the bars have same diameter of 8mm and the variation in stirrup spacing has been done as 300mm c/c, 200mm c/c and 100mm c/c. The geometric details of the RC beams are as shown in Fig.2.

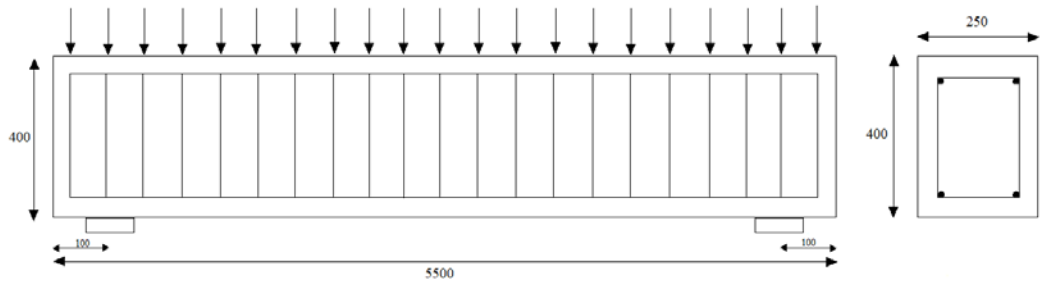


Figure 2: RCC beam model.

MATERIAL PROPERTIES

Steel Reinforcement

The steel reinforcement for the finite element models has been assumed to be of grade Fe-415. It is an elastic-perfectly plastic material and identical in tension and compression. Elastic modulus equal to 200000 MPa, tangent modulus of 0 MPa and Poisson's ratio of 0.3 has been used for all the reinforcing bars in this study.

Concrete Properties

Modelling an element for the behavior of concrete is a challenging task. Concrete is a quasi-brittle material and has exhibit different behavior in compression and tension. The tensile strength of concrete is typically about 10% of the compressive strength. The Poisson's ratio of 0.2 has been used and modulus of elasticity of concrete is calculated as 25000MPa as per IS 456:2000 [6]. The shear transfer coefficient for open crack and closed crack are 0.3 and 0.8 respectively taken from previous studies. Uniaxial tensile cracking stress is obtained using IS 456:2000. The stress strain curve of concrete is defined using multilinear isotropic hardening.

Table 1:SOLID65 material properties

Modulus of elasticity, E	25000 MPa
Poisson Ratio, v	0.2
Open shear transfer coefficient	0.3
Close shear transfer coefficient	0.8
Uniaxial tensile cracking stress	3.5 MPa
Uniaxial crushing stress	25 MPa
Tensile crack factor	0.6

Table 2: Steel material properties

Modulus of elasticity, E	2e5MPa
Poisson Ratio, v	0.30
Yield stress, f_s	400 MPa
Tangent modulus	0

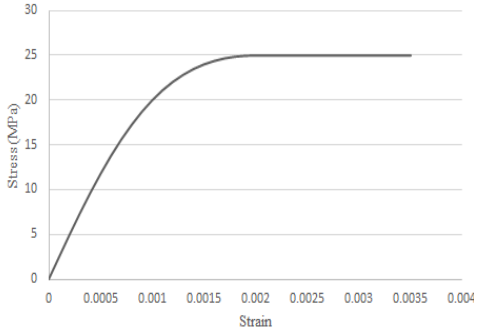


Figure 3: MISO hardening for concrete

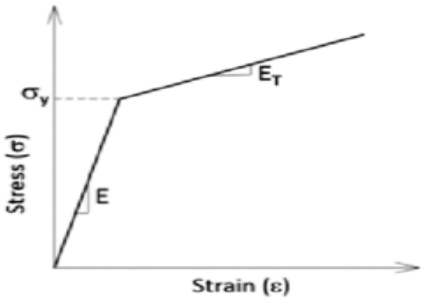


Figure 4: Steel hardening model

FINITE ELEMENT ANALYSIS

In this study, at first the unreinforced concrete beam with simple supports was loaded until non-convergence in ANSYS. Then that same load was applied on other reinforced concrete beams with varying stirrups spacing, having same longitudinal reinforcement. Symmetry was used to model half of the beam. All the beams were analyzed for deflection, stresses in steel and crack patterns.ANSYS has a function to separate the load into sub steps through which the load can be divided into smaller parts so that the results can be obtained for various intervals of loading instead of just at the end., which helps in convergence and also for application of time dependent loads.

Finite element modelling



(a) (b)

(c) (d)

Figure 5: (a) Unreinforced beam (b) 300mm c/c (c) 200 mm c/c (d) 100 mm c/c

Meshing and Boundary Conditions

In order to obtain the actual results from the Solid65 element, a mesh was formed using square elements of size 50mm asshown in Fig.4. The SOLID65 element supports linear meshing only since it is an 8 noded element and does not provide quadratic meshing. The mesh size must be such that it merges the steel and concrete nodes together so as to have a proper bonded effect. Boundary conditions are required to get a proper solution for the model. The symmetric boundary condition is set first to take advantage of faster solve time and results.

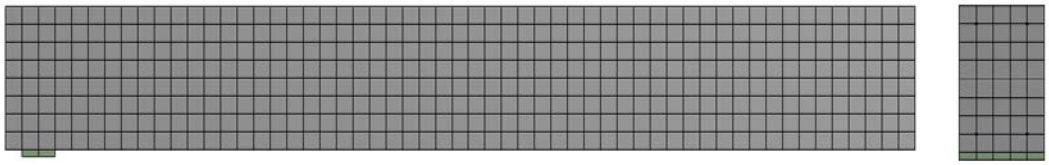
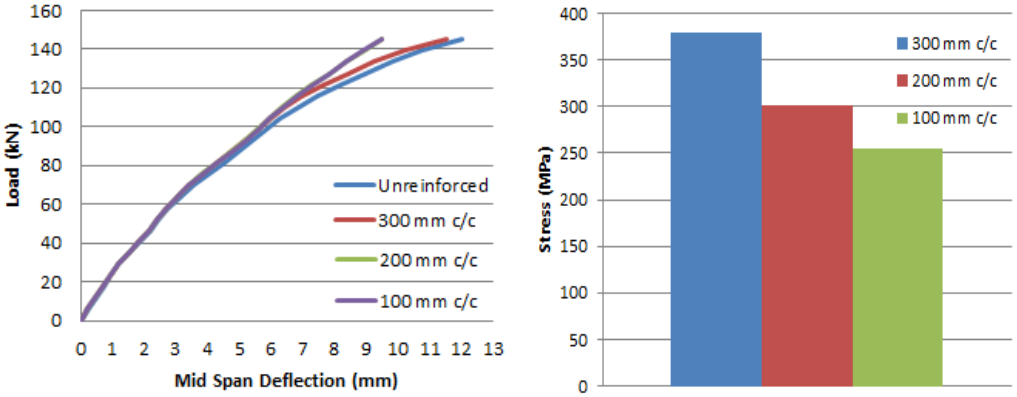


Figure 6: FEA model of beam withmeshing

RESULTS AND DISCUSSIONS

Comparison of Load-Deflection curves

The unreinforced beam was first loaded till non convergence in ANSYS. The same load was applied to the various reinforced concrete beams having different stirrups configuration. For further analyses, the reinforced beams were loaded till steel yielding and their load deflection curveswere plotted and the mid-span deflection at peak load is obtained for each beam. The spacings adopted for beam are 100mm,200mm and 300mmcenter-to-center.



(a) (b)

Figure7: Load deflection(a) and stresses in steel (b) at 140kN load

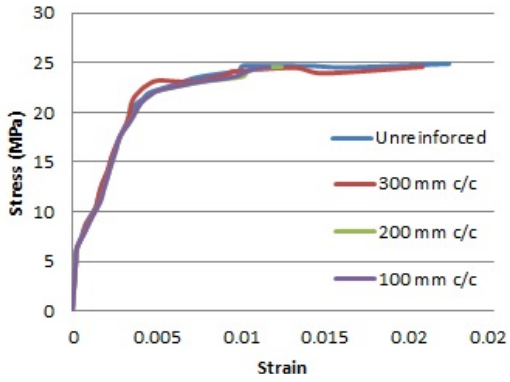


Figure8: Stress-Strain in concrete

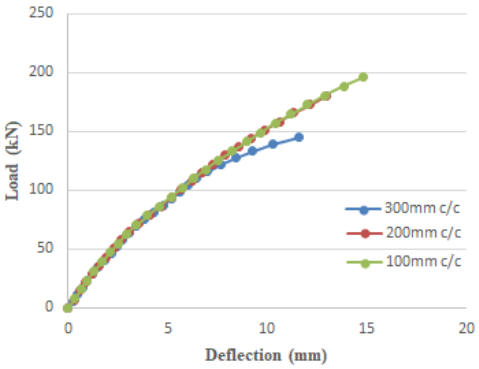


Figure 9 : Load-Deflection at steel yield

Comparison of Load-Deflection for all the stirrupsspacing are shown in figure 8 above.It was observed that the mid span deflection of beam decreased with higher spacing because of increase in stiffness resulting from stirrup densification. It is observed that as spacing of the beam increases, the stresses in tensile steel decreases as the stirrups contribute to the counter the loading effect.As for the stress strain in concrete, all the models reached the peak of 25MPa at 140kN load however the strain in concrete decreased with the increase in spacing of reinforcement. To find out the ultimate yield point for the beams, the beams were loaded further and the 300 mm c/c beam yielded at 150kN, followed by 180kN for 200mm c/c and 196kN for 100mm c/c beam which concludes that the ultimate capacity of beam is increased with closer spaced stirrups.

Comparison of crack patterns

The initial cracking behavior of the beam is not much influenced by the reinforcement percentage as the first crack occurred at about 58kN, 61.8kN, 61.8kN for 300, 200, 100 mm spacing respectively. However, it has more impact in the post cracking stage of the beam.

Thus, the ultimate capacity of the beam can be varied by varying the reinforcement percentage

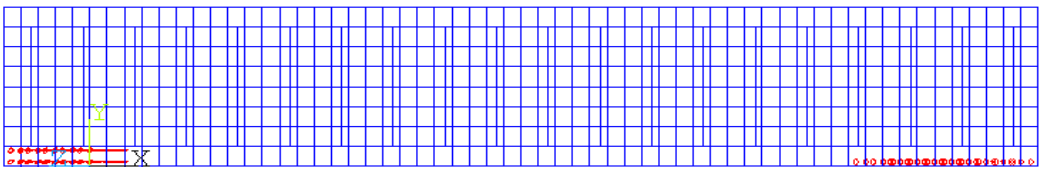


Figure 10: First crack in beam

The final crack pattern were obtained at peak load of 140kN, it was observed that the beam with 300mm stirrup spacing had the most crushed concrete followed by 200mm and 100mm c/c beam due to higher confinement binds the steel with concrete and does not allow the concrete to crush hence increasing the crushing capacity of the beam.

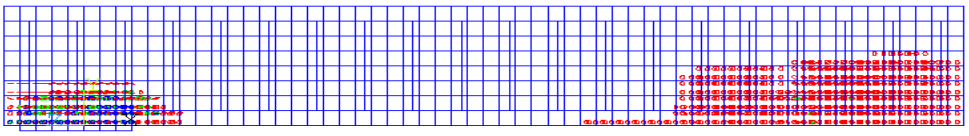


Figure11: Crack pattern for 300mm c/c beam at 140kN load

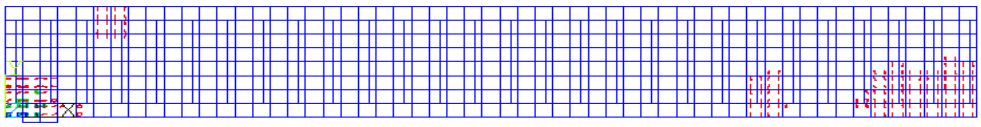


Figure 12:Crack pattern for 200mm c/c beam at 140kN load

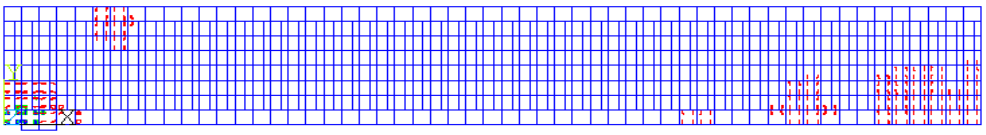


Figure 13: Crack pattern for 100mm c/c beam at 140kN load

CONCLUSIONS

From the finite element analysis of reinforced concrete beam, following conclusions were obtained.

1. Reinforced concrete beam can be modeled with SOLID65 and LINK180 elements and analyzed using ANSYS software to obtain accurate results such as stresses, deflection, etc.
2. Meshing plays a very crucial role in modelling of reinforced concrete beam, as the nodes must be connected to each other in order for the steel and concrete to deform together,
3. As the shear reinforcement of the beam is increased, the load carrying capacity increases and the stresses in tensile reinforcement decreased along with mid span deflection.
4. Due to the confinement effect in concrete, the crushing is reduced since the load is transferred to steel hence the ultimate capacity of beam is increased.

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SUITABILITY OF ULTRASONIC PULSE VELOCITY TEST IN LIFE OF CONCRETE STRUCTURE

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ABSTRACT
The civil infrastructure of most countries constitutes of mainly concrete structures. To ensure the safety of the structure, testing and quality check-ups are important. There are many variables such as type of cement used, type and size of aggregate, humidity, w/c ratio which affect the compressive strength and quality of concrete. Many other factors also cause deterioration of a structure over time. Non-Destructive Testing method such as Ultrasonic Pulse Velocity test is used to obtain information about a structure without damaging it. This study discusses the suitability ofUltrasonic Pulse Velocity method on a life of a structure. In this study, CLC slabs and Pervious beams were used. UPV test measurements were obtained on different points on the specimens. Indirect measurements were taken on sides of the specimen. Results were discussed of these specimens in this paper.

KEYWORDS: Ultrasonic Pulse Velocity, Non-Destructive Testing.

INTRODUCTION
The important part of the civil infrastructures across the world is concrete constructions. Reliability and safety are important factors considered in construction. There is an increasing need to assess the condition of a structure, at different stages of its life, to ensure the reliability and safety. Evaluating the quality of concrete with traditional method, has its disadvantages.

NDT methods, as the name suggests, provides information about the internal condition of the structure, without damaging it. Variation of amount of compaction, exposure condition, curing condition or mixing conditions, between the test specimen and real structure, fail to calculate characteristics of the structure correctly. NDT methods are used to assess the strength properties, durability, density, moisture content, elastic properties and condition of reinforcement of concrete. Themost widely used NDT methods is Ultrasonic Pulse Velocity Method.

Ultrasonic Pulse Velocity Method
This method is used to assess the homogeneity of concrete, presence of cracks, voids and other imperfections. Electro-Acoustical transducers are used to transmit ultrasonic pulse through the concrete. When a pulse is induced into the concrete, a system of stress waves is developed. These includes longitudinal (compressional), shear (transverse) and surface (Rayleigh) waves. Commonly used transducers are of frequencies ranges 50KHz to 100KHz. There are 3 ways of measuring pulse velocity: direct transmission (cross probing), semidirect transmission, and indirect transmission (surface probing).

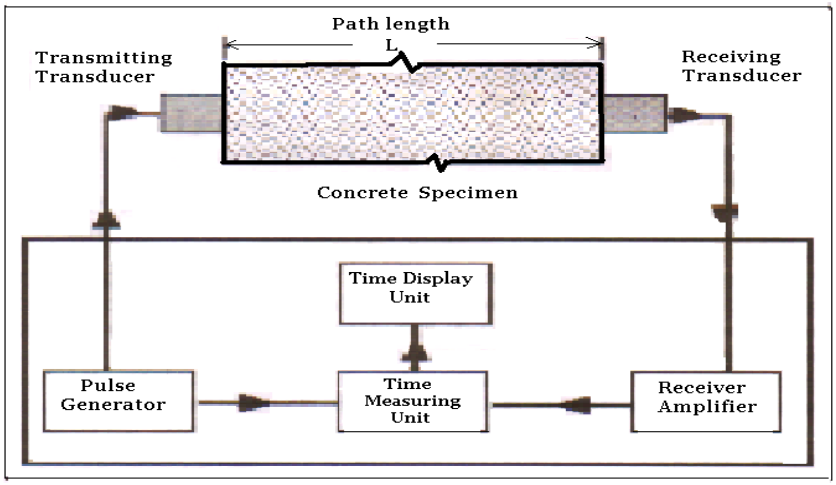


Figure 1: Schematic Diagram of Ultrasonic Pulse Velocity Method

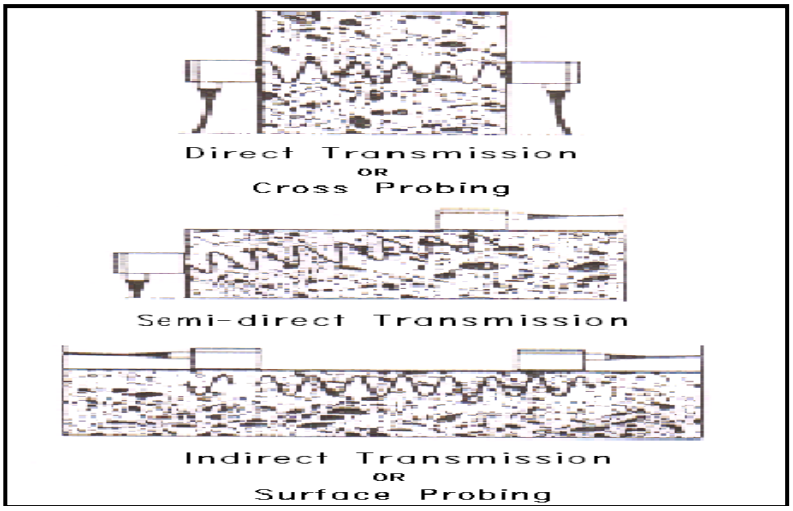


Figure 2: Different Methods of Propagating Ultrasonic Pulses through Concrete

Table 1 :Velocity Criteria For Concrete Quality Grading(As per Table 2 of IS 13311 (Part 1) : 1992)

Sr. No.	Pulse Velocity by Cross Probing (km/sec)	Concrete Quality Grading
1.	Above 4.5	Excellent
2.	3.5 to 4.5	Good
3.	3.0 to 3.5	Medium
4.	Below 3.0	Doubtful

METHODOLOGY ADOPTED

In the present study, UPV method is used as per IS code (IS-13311-1992 (part 1)). Indirect measurements were taken on test CLC slabs and test pervious beam. The data obtained on from test beam are analyzed to find out the total minimum number of readings required for proper assessment. Using the data obtained from the test CLC slab, surface contour map is prepared of the concrete surface. Using the maps and data, the size, type and location of defects are predicted.

PROCEDURE ADOPTED

Test Slab

The present experimental activity aims to assess the efficiency of indirect UPV measurements on slabs for predicting the defects. Slab surface was provided with grid points on it, and the measurements were taken on these respective points. Six grid points were taken on the slabs.

Dimensions of Test Slab

Length=50cm, Breadth=31.5cm, Depth=7.5cm

Grid size used: 15cm × 15cm

Distance of first grid point along length is 10cm from side.

Distance of first grid point along breadth is 8cm from side.

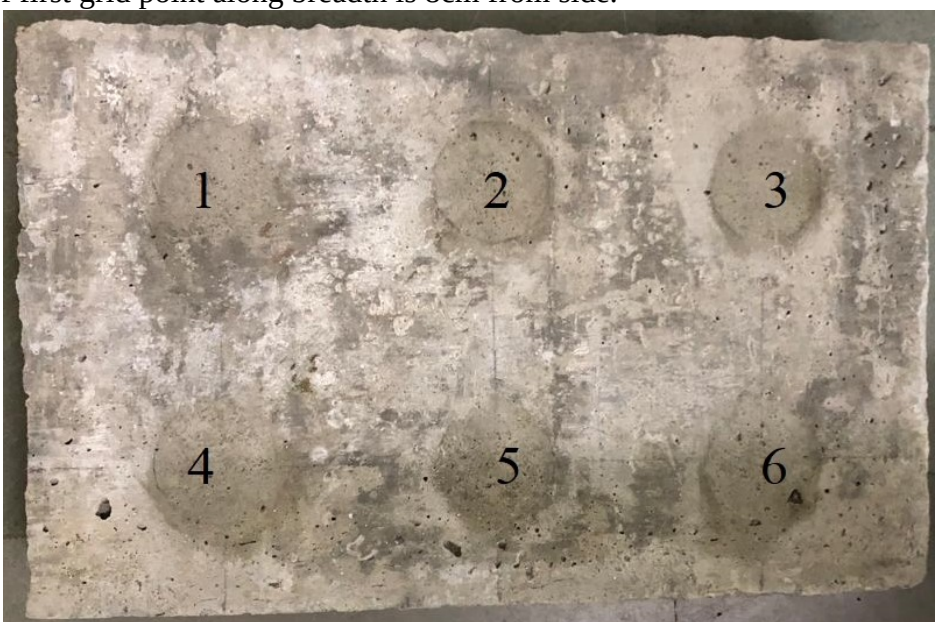


Figure 3: Test Slab

At each measurement points, the readings were taken by keeping the transmitter at that point and receiver at adjacent 3 points. For example, when measurements are takin at point 1, then the transmitter is placed at point 1 and the receiver at 2, 4 and 5. Readings are taken by placing the receiver on the points present on the right side. For the extreme right measurement points, the readings were taken as: (for example, if transmitter is placed at point 3, then receiver is placed at 6 and 5).The no of readings were increased by using interpolation, for developing a proper contour map of slab surface.

Test Beam

The experimental activity aims to stablish the number of readings required for better results. It shows the optimum number of readings required.

Dimensions of Test Beam

Length=63cm, Breadth=25cm, Depth=15cm

Grid size used 15cm × 15cm

Distance of first grid point along length is 9cm from side.

Distance of first grid point along breadth is 5cm from side.



Figure 4: Test Beam

In this study, UPV data is analyzed to determine the least number of measurement points that can be used to evaluate acceptable variability. Readings were taken for A10, B10, A40 and B40. On the grid, readings were taken at every 50mm distance. Minimum distance at which the receiver is placed is 100mm, since the wavelength of ultrasonic wave at 50KHz is approximately 90mm. for example if measurements are taken at A10, then the transmitter is placed at A10, and the readings were taken by moving the receiver from 100mm to 450mm with 50mm increment with each measurement.

RESULTS

Results of Test Slab

The following readings were obtained:

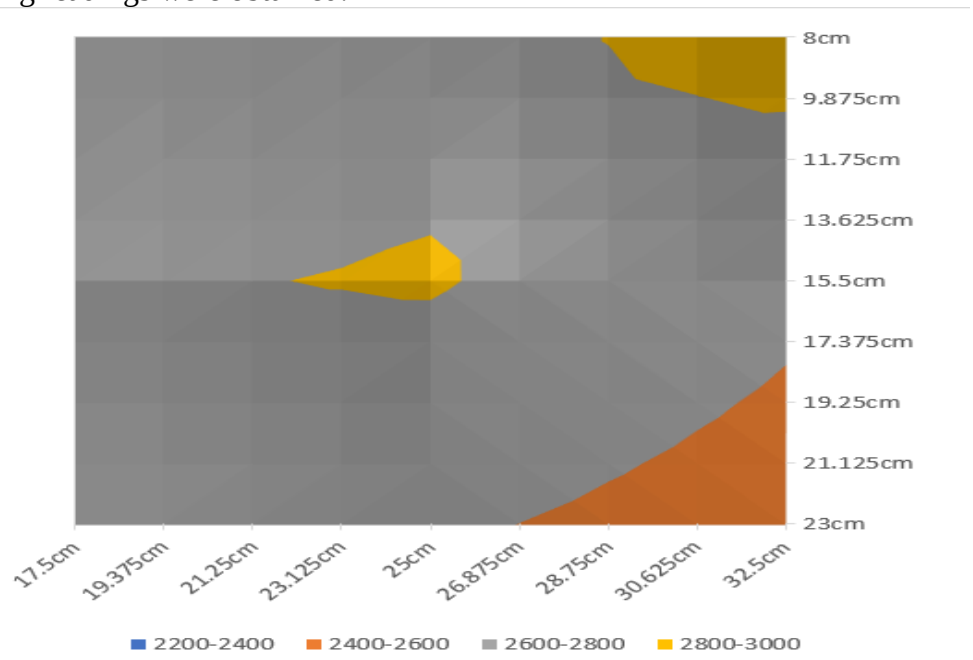


Figure 5: Contour Map of Test Slab

Analysis of UPV results

The surface contour maps created using UPV data distinguished the weaker portion of concrete from the other. In this map, orange color indicates the presence of defects or weaker portion. Grey and Yellow color represents the higher values of UPV. Majority of UPV values

are of the range 2600-2800. The distribution of these values is clearly visible. Light grey to dark grey (lower to higher values) color represents the distribution of UPV values in this range. Higher range of UPV values is shown as yellow color.

Results of Test Beam

The following readings were obtained:

Table 2: UPV readings of Test Beam

X	A10	B10	A40	B40
Y				
100	1883.24	1745.2	1831.5	2155.17
150	1662.97	1758.5	1815.98	1943.01
200	1860.47	1779.36	1746.72	1951.22
250	1708.82	1734.91	1779.36	1835.54
300	1815.98	1747.23	1783.59	1831.5
350	1760.56	1754.39	1751.75	1816.29
400	1657	1759.79	1755.93	1817.36
450	1738.79	1753.02	1719.53	1741.49

Analysis of UPV results

Coefficient of Variance (COV) of UPV values were taken to assess the variability of UPV values with different number of measurements. The COV value reduces from 7.37 to 3.47 as the number of measurement points were increased from 2 to 8. With increase in number of measurement points, more consistent readings are obtained. Decrease in variability is not significant when more than 5 readings are taken.

CONCLUSIONS

The conclusions drawn from the present work shows that the results of the test slab clearly distinguish the portion of the slab with possible defects. This study shows the capability of UPV to detect defects in concrete. In surface contour map orange color represents the weaker portion of concrete. Results of the test beam shows that, as more number of readings at a given location on concrete is taken, more consistent results are obtained. Variability of UPV values reduces with increase in number of measurement points. According to the results, when more than 5 numbers of readings are taken, the change is not significant.

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SEISMIC ANALYSIS OF MASONRY WALL BUILDING BY USING SHAKE TABLE

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ABSTRACT
To test the response of structures and to verify their seismic performance, shake table can be used. This paper presents the seismic performance of masonry building on the shake table. Experimental work is performed on a prototype model of unreinforced single storey masonry building with a flexible diaphragm. The building is subjected to base excitation on a shake table for seismic performance evaluation to an extended series of various displacements and frequencies on servo controller. Seismic performance of the structure is evaluated. The results show the crack pattern obtained throughout the test.

KEYWORDS: seismic performance, shake table, flexible diaphragm, servo controller.

INTRODUCTION
In order to obtain actual earthquake data, single degree of freedom shaking table is important. Shake table may be a very complex electro-hydraulic system, which is extremely expensive and requires high maintenance and operational costs. the random earthquake ground motions, Seismic analysis of structure means to provide equivalent distributed lateral force acting at various lumped level of structure above ground based on the ground motion at that site. Thus, rather than stepping into complex nature of study, the behavior of structure is analyzed when horizontal ground shaking occurs Uniaxial shake table is of 5 ft x 3 ft in size with amplitude displacement variation of 0 to 50 mm and frequency varying from 0 to 25 Hz. This shake tables will satisfy the essential requirement for testing models falling during this frequency range. A single degree of freedom model was tested on this shake table to gauge the performance of Shake Table and instrumentation setup.

LITERATURE REVIEW
Simsir et al. (2004): studied the out-of-plane behavior of unreinforced masonry buildings subjected to ground motions. From the results of shake table test the walls performed very well in contempt of the slenderness of the wall and the intensity of base motion. Further, the experimental results were compared using SDOF and MDOF computational models.

Meguro et al. (2005): performed a shake table test using 1/4-scale two-story masonry house models with and without retrofitting by the PP-band method. The test was carried out using a sinusoidal input wave.

Hanazato et al. (2008): studied the structures damaged in the earthquake in under developing countries.Shake table test were conducted on a masonry house 3m x 3m x 3m. Observing the results, a new numerical method was developed using the Extended Distinct Element Method (EDEM) to estimate the seismic performance of masonry structures.

Lihong et al. (2008): investigated three models of eight-storey masonry building with or without frame-shear walls. The results of seismic response are compared and a nonlinear response analysis was conducted to examine the reliability.

Bothara et al. (2009): analyzed the seismic performance of two storey brick masonry building by testing under, on a shake table first in the longitudinal direction and then in the transverse direction. The building suffered an increased level of damages as the excitations become severe.

Ersubasi1 and Korkmaz (2010): studied the strengthening methods of masonry building situated on an active earthquake region. Experimental work is carried out on a total of nine specimens. The test results were compared and assessed.

Mendes et al. (2013): assessed the seismic performance of an existing masonry building with flexible floors. A shake table test was performed on two models strengthened and non-strengthened. The results revealed that the strengthening technique which was adopted is much effective for reducing seismic vulnerability.

Meoni et al. (2019): presented the application of smart bricks used for detecting earthquake-induced damage in an unreinforced masonry building subjected to shake table tests. The results are interpreted using a 3D finite element model. Overall the results demonstrated that the smart bricks effectively reveal native permanent changes in structural conditions following progressive harm, thus being apt for earthquake-induced damage detection and localization.

METHODOLOGIES ADOPTED

Typical brick masonry single-story building is presented. In order to evaluate the dynamic behavior of a structure, reduced scaled model, the geometric and materials properties as well as loading on the prototype structure must be simulated by laws of similarities.

Properties of prototype brick

- **Size of bricks:** The size of prototype bricks is 95 x 45 x 45 mm. as shown in Fig.3.1.1



Fig. 3.1.1 Prototype Bricks.

- **Compressive strength:** As per code IS 3495-1992 part-1 compressive strength of brick masonry units is carried out using the Universal Compressive strength testing machine. In this study compressive strength of full unit is determined as shown in fig 3.1.2. Load is applied perpendicular to the bed face (length x width). Compressive strength of prototype bricks 3.977-5.501 N/mm²



Fig.3.1.2 Compressive Strength test on Prototype Bricks

Instrumentation: Uniaxial shake table is of 5 ft x 3 ft in size with amplitude displacement variation of 0 to 50 mm and frequency varying from 0 to 25 Hz is used. The shake table satisfied basic requirement for testing model falling in the frequency range. A prototype brick masonry model is prepared and tested. Figure 3.2.1.Shows Uni-axial high frequency shake table testing machine with servo controller.



Fig. 3.2.1 Shake table with Servo Controller

The Salient features of Shake Table are:

- Uni-axial motorized electro-mechanical Shake Table.
- Size 5 ft x 3 ft table platform for fixing model.
- Operated with motor as well.
- Amplitude range of Shake table is from 0 to 50 mm.
- Harmonic and random motion simulation.
- Frequency of simulation - 0 to 25 Hz.
- Payload capacity: 500 kg.

NUMERICAL STUDIES

To determine the behavior of typical masonry building it is easy to test a reduced scale masonry building as compared to original scale of masonry building. The present experimental work discussed about strength and comparison dynamic test of prototype masonry building. The model is one-fourth scale unreinforced masonry with flexible

diaphragm of roof is prepared as shown in Figure 4.2. The size of model in X-direction is 890 mm, in y-direction is 890 mm and in Z direction is 815 mm.

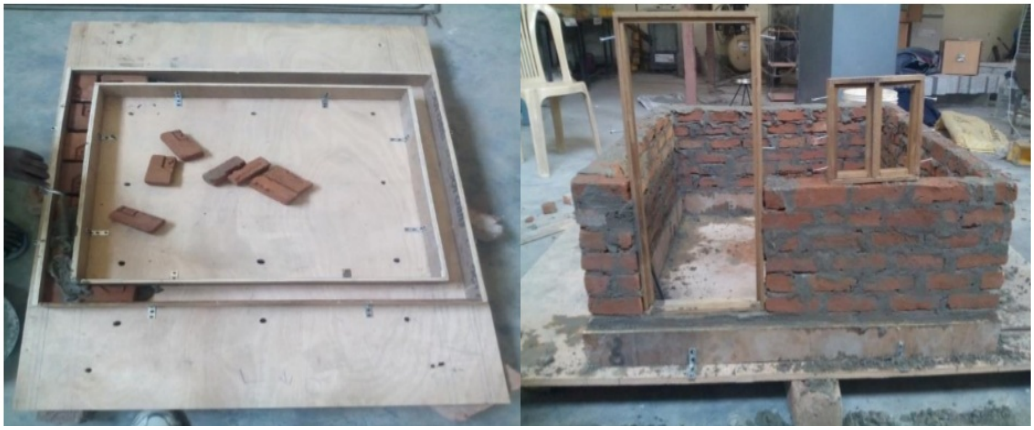


Fig. 4.1 Preparation of Model.



Fig. 4.2 Model of building.

RESULTS

The buildings are subjected to an extended series of different displacement and frequency on shake table servo controller. The maximum and minimum displacement of shake table are ± 1 mm and ± 50 mm respectively while the maximum and minimum frequency that can be given are 1 Hz and 25 Hz respectively.

Table 5.1 Observation Table for test model

Input of cycles	Observations
1st	No changes
2nd	No changes
3rd	Fine cracks observed
4th	Crack width increased
5th	More cracks observed
6th	Model about to Fail
7th	Failure of Model



Fig 5.1 Crack Pattern Observed on Model.

CONCLUSIONS

From the results drawn from shake table test it can be observed that up to 2nd cycle there was no damage in model, afterwards the crack started increasing. At 5th cycle more crack pattern was observed. Finally, the model failed at 7th cycle.

FUTURE SCOPE OF THE STUDY

- After this experimental investigation it is suggested that more tests can be carried out in the area of characterization of masonry properties by using shake table mechanics.
- The input velocities can be derived from the inelastic versus elastic building models.
- Use of masonry in tapered beams and columns of variable thickness and length.
- Actual time histories application.

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DEVELOPMENT OF GEO-POLYMER CONCRETE USING INDUSTRIAL BY-PRODUCTS

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ABSTRACT

Concrete has been, is and will always be the most popular construction material, prepared by mixing cement, sand, coarse aggregate and water in predetermined proportions. Cement is one crucial ingredient which acts as a binding agent in preparation of concrete. However, the production of ordinary portland cement (OPC) results in release of Carbon Dioxide gas (CO_2), which accounts for 6-7% of that generated by human activities.It is the need of the hour to look for alternative pozzolanic material which can replace cement in concrete without compromising on its strength and durability parameters. Researchers have found a new material i.e., Geopolymer concrete (GPC), which is a cement free concrete. In GPC, we use some pozzolanic materials like fly ash, silica fumes, red mud, ground granulated blast furnace slag (GGBS) etc. In this study, GGBS based GPC has been prepared for M25 grade of concrete. GGBSpossesses high silica and alumina content. Thus it can be used as a replacement of cement. GGBS has to be activated by alkali hydroxide and silicates in order to give GPC binding properties. Mix design of GGBS based GPC has been done in this research. The effect of accelerated curing and ambient curing on GGBS based GPC has been compared.28 days compressive strength of GGBS was found satisfactory. This proves that GGBS can completely replace cement and GPC can be made successfully using it. A set of samples were subjected to accelerated curing and another setwas kept in ambient temperature. It was found that both the sets achieved target strength. Samples with accelerated curing at elevated temperatures achieved 28 days compressive strength of 33.08 N/mm^2 and samples with ambient curing achieved 28 days compressive strength of 32.22. Thus it can be concluded that the research results have come out with a mix proportion in which theGGBS based GPC does not need any type of curing to attain the target strength of 31.6 N/mm^2 for M25 grade of concrete. This makes it easier to use GPC as it is very challenging to cure concrete at elevated temperature in practical applications.

KEYWORDS: Granulated Blast Furnace Slag (GBS), ground granulated blast furnace slag (GGBS), geopolymer concrete (GPC), Ordinary Portland Cement (OPC).

INTRODUCTION

Conventional concrete consists of cement, fine aggregates, coarse aggregates and water, where cement is the binding agent which is responsible for producing 6-7% of total carbon dioxide produced by human activities. In current scenario it is very crucial that the materialsused for making concrete structures are eco-friendly. There are many pozzolanic materials available, like fly ash, ground granulated blast furnace slag (GGBS), silica fumes, rice husk ash, etc. for replacement of cement. The use of these pozzolanic materials in making concrete will lead to substantial reduction in greenhouse gas emission and minimize the land required to dispose the industrial wastes.

GGBS is a pozzolanic material which can be usedfor producing concrete with required strength characteristics. Utilization of GGBS in making concrete will contribute in managing industrial waste sustainably, which has both environmental and economic benefits. GGBS is a waste product generated from blast furnace during the extraction of iron. The waste generated from blast furnace is in molten state which is cooled rapidly under high pressure to produce granules. These granules are then ground in ball mills to the fineness of cement and the product so generated is called GGBS. GGBS is used in manufacture of Portland slag cement in which 25-65% of slag can be used in its production as per the recommendations of IS 455:1989.

The term geopolymer was discovered by Davidovits in 1978. Geopolymer concrete is a cement free concrete, which makes use of materials rich in Silica (Si) and Alumina (Al). The basic ingredients of geopolymer concrete are pozzolanic materials, alkali hydroxide and silicate solution which binds the coarse and fine aggregates. Geopolymerization is a process in which amorphous alumino-silicates react with alkali hydroxide and silicate solution to forminorganic alumino-silicate polymeric gel.Any pozzolanic material rich in Silica (Si) and Alumina (Al), activated by highly alkaline solution and soluble silicate liquidssubjected toaccelerated curing at elevated temperatures (30-90oC) for a period of 24-48 hours, yields binders Si-O-Al (geopolymers) which is similar to C-S-H bond inconventional concrete. Geopolymer concrete needs higher temperatures for curingin order to achieve required strength which makes it difficult to bring it in practical use.

Hence, to surmount this issue research has been done, to develop a mix design for geopolymer concrete which can attain required strength using GGBS, without accelerated curing. The present study explores the use of GGBS in manufacturing M25 grade of concrete by replacing cement completely and making a GGBS based geopolymer. Also with the use of GGBS the industrial waste is utilized and its use can curtail the cost of construction which is incurred in making conventional concrete using cement.

LITERATURE REVIEW

Some of the research papers related to this study have been discussed below:

Gao, Yu, Yu, & Brouwers, (2017) investigated the performances of alkali activated slag-fly ash composites that are reinforced by long and/or short steel fibers, and the mortar samples were designed. The study concluded that the addition of long and short fibers (up to 1% by volume) decreases the slump flow from 25.9 to 20.6 and 23.1 cm, respectively. The compressive strength was increased by 10.3%/16.8% when short/long steel fiber is incorporated, respectively. The utilization of long and short steel fiber significantly reduces the 28 days drying shrinkage up to 27.6 and 31.9%, respectively.

Bhikshma & Kumar, (2016) studied the mechanical properties of fly ash based geopolymer concrete with varying the percentage of GGBS from 9%, 20%, 27.5%, 38% to 43% under ambient temperature (27°C) curing for M20, M30, M40, M50, M60 grades of concrete. It was concluded that the average density, workability, flexural and split tensile strength of geopolymer concrete prepared was similar to conventional concrete whereas the 28 days compressive strength was more than the values recommended by IS 456-2000.

Laskar & Talukdar, (2016) reported the results of laboratory experiments performed on geopolymer mortars prepared from ultrafine ground granulated blast-furnace slag with alkali activators composed of (1) sodium silicate and sodium hydroxide, and (2) only a sodium hydroxide solution to find examine various properties, using fly ash and superplasticizer. The experimentation concluded that sodium silicate-free mortars were better than sodium silicate-containing ones in terms of setting, workability, and strength-gain performance. Addition of fly ash and superplasticizer in mixes retards the early-setting property and improves the workability.

Nagalia, Abolmaali, & Aswath, (2016) examined the characteristics of various alkali hydroxide [NaOH, KOH, $Ba(OH)_2$, and LiOH] and its concentration on the compression strength of fly ash– based geopolymer concrete cured under different experimental conditions.The experimentation shows that higher concentrations of CaO in the fly ash resulted in significantly higher strengths in the finished geopolymer. NaOH was the only alkali hydroxide that resulted in compressive strength that was comparable to portland cement. Higher curing temperatures for longer curing duration resulted in a significant increase in compressive strength.

Inti, Sharma, & Tandon, (2016) carried out experimentation using GGBS and rice husk ash (RHA) to replace flyash in geopolymer concrete. Compressive strength test was performed on geopolymer concrete by varying percentages of RHA and GGBS. Results indicated that complete replacement of flyash in geololymer concrete with RHA and GGBS was not feasible. However, GGBS up to 50% can be used as replacement of flyash to attain desired compressive strength and 5-10% of RHA can be used to achieve similar strength.

Saavedra, Angulo, & Gutierrez, (2016) evaluated the performance of GPC based on a mixture of fly ash (FA) with blast furnace slag (GBFS) in an 80:20 ratio, activated with a mixture of sodium silicate and sodium hydroxide. FA/GBFS and OPC concretes were immersed in 5% by weight sodium sulfate and magnesium sulfate solutions. The performance of FA/GBFS based geopolymer in Na_2SO_4 and $MgSO_4$ solution of 50 g/L concentration exhibiting a minimal expansion percentage in both environments and a lower reduction in mechanical resistance in the presence of $MgSO_4$.These concretes should be considered as materials with high resistance against sulfates as compared to conventional concrete with OPC.

Deb & Sarker, (2016) presented the effects of ultrafine fly ash on setting, strength and porosity of geopolymer paste specimens cured at 20°C. Addition of ultrafine fly ash reduced setting time, increased compressive strength and reduced porosity of geopolymers based on fly ash only or fly ash blended with 15% GGBFS or 10% OPC.

Vora & Dave, (2013) illustrated that compressive strength increases with increase in curing time, curing temperature, rest period, concentration of sodium hydroxide solution and decreases with increase in the ratio of water to geopolymer solids by mass & admixture dosage, respectively. The addition of naphthalene based superplasticizer improves the workability of fresh geopolymer concrete. It was further observed that the water content in the geopolymer concrete mix plays significant role in achieving the desired compressive strength.

Patil & Allouche, (2013) found through an experimental investigation the relationof alkali silica reaction (ASR) between reactive aggregates and the geopolymer matrix. Specimens were prepared using i) Class C and ii) Class F fly ash. Results suggest that the extent of ASR reaction due to the presence of reactive aggregates in fly ash-based GPC is substantially lower than in the case of ordinary portland cement-based concrete. Utilizing ASR-vulnerable aggregates in the production of GPC products could contribute to the economic demand and sustainability of geopolymer binders in regions that suffer from insufficient local supply of high quality aggregates.

Reddy, Edouard, & Sobhan, (2013) evaluated the durability characteristics of low-calcium fly ash–based geopolymer concrete that was subjected to a corrosive marine environment. A series of GPC beams, which contained fly ash with 8-M and 14-M concentrations of NaOH and SiO_2/Na_2O solutions, respectively, were centrally reinforced with a 13-mm rebar and tested for accelerated corrosion artificial seawater environment. The test results showedadmirable resistance of GPC to chloride attack, with a longer time to corrosion cracking with respect to conventional concrete.

Li & Liu, (2007) utilized slag as an additive for fly ash-based geopolymers and discovered that the incorporation of slag could significantly increase the compressive strength of the geopolymer. The compressive strength of geopolymer with 4.0% slag reached 50 and 70 MPa when cured for 14 days at 30 and 70°C which improved significantly.

Outcome of the literature review:

It can be concluded from the research done by various researchers that GPC has improved strength, durability parameters and resistance to chemical attack as compared to conventional concrete. Till now the researchers have developed fly ash based GPC with or without partial replacement of fly ash with GGBS. This research aims at exploring the feasibility of developing a GGBS based GPC.

METHODOLOGY

It has been found from the literature review that GPC can be prepared from materials rich in alumina and silica. Table 1 shows the comparison of chemical composition of GGBSwithOPC, which is used in making GPC.It can be seen that the GGBS used to carry out the study is rich in alumina and silica. Hence, GGBS is fit for making GPC. To activate alumina and silica so that they yield binders Si-O-Al, the chemicals used are sodium hydroxide and sodium silicate.

Table 1 Chemical composition of OPC and GGBS

S.No.	Ingredients (% by mass)	GGBS (%)	OPC (%)
1.	Sulphuric Anhydride (SO_3)	0.28	1-2
2.	Magnesia (MgO)	6.58	0.5-4
3.	Silicon Oxide (SiO_2)	33.89	17-25
4.	Aluminium Oxide (Al_2O_3)	20.43	3.5-9
5.	Calcium Oxide (CaO)	36.94	60-65
6.	Ferric Oxide (Fe_2O_3)	0.18	0.5-6
7.	Sulphur (S)	0.55	-
8.	Chloride content	0.016	-

Table 2 shows the comparison of results obtained from the tests performed on GGBSand OPC. It can be seen that there is slight difference in the various properties like fineness, consistency and soundness. The specific gravity of GGBS is 2.74 which show a difference of 0.41 when compared with OPC. Also it has been observed that GGBS's initial and final setting time is less than that of OPC, giving GGBS an added advantage of quick setting material.

Table 2 Properties of Cement and GGBS

S. No.	Properties	GGBS	OPC
1.	Fineness	4.0 %	4.0 %
2.	Consistency	32.0 %	33.0 %
3.	Soundness	1.00 mm	2.00 mm
4.	Initial Setting time	63 minutes	93 minutes
5.	Final setting time	142 minutes	220 minutes
6.	Specific gravity	2.74	3.15

The mix design has been done with reference to the research done by various researchers in making GPC and the results obtained from material testing. After carrying out the chemical analysis and material testing the following procedure was adopted in making GGBS based GPC:

In the present study M25 grade of concrete has been casted. Locally available river sand confirming to zone II of IS 383:1970 has been used along with 20mm coarse aggregates. The binding agent in the geopolymer is GGBS and the chemical solutions of sodium hydroxide and sodium silicate are used. From the research done by various researchers, twelve molar sodium hydroxide solution was optimized, has been prepared in making GPC. Also sodium silicate was added to the sodium hydroxide solution, already prepared. The ratio of sodium silicate to sodium hydroxide by weight has been taken as two. The ratio of chemical solution to GGBS was taken as 0.45.

The mix design has been done in accordance with IS 10262:2009. After few trials the mix design was adopted and it consists of 75% of Aggregates (26% fine aggregates and 49% coarse aggregates), 17% of GGBS and 8% of chemical solution.After mixing the above quantities, the mix was not workable. So, additional 0.125% of water was added by weight. All the ingredients in above ratios were mixed and casted in 150mm cube moulds. The moulds were vibrated on table vibrator for a period of 50-80 seconds and were left in moulds for 24 hours. After 24 hours the concrete cubes were demolded and three specimens were kept in oven for accelerated curing at 70°C for 24 hours and the other three specimens were kept in ambient environment.

Study has been carried out in making GGBS based GPC and it was observed that concrete containing GGBS has poor early strength (7days) but as the concrete ages the strength gain is remarkable at 28 days. Because of this finding, 28 days compressive strength of the GPC has been checked.

RESULTS AND DISCUSSION

The results of 28 days compressive strength of geopolymer concrete are shown in Table 3. Target strength of M25 grade of concrete is 31.6MPa. The specimens A, B and C were subjected to accelerated curing of 70°C for 24 hours and then the specimens were subjected to compressive strength test. The specimens gained 31.37, 33.59, 34.28 N/mm^2 compressive strength after 28 days. On the other hand specimens P, Q and R were left at ambient temperature curing and then the specimens were subjected to a compressive strength test. The specimens gained 30.43, 32.87, 33.36 N/mm^2 compressive strength after 28 days. The average 28 days strength of specimen A, B, C is 33.08 N/mm^2 and of P, Q, R is 32.22 N/mm^2. Both the specimens i.e. with accelerated curing and without accelerated curing achieved the target strength (31.6 N/mm^2) for M25 grade of concrete.

The results show that there is no significant effect of temperatureon the compressive strength of GGBS based geopolymer.

Table 3 28 Days compressive strength of the geopolymer concrete

S. No.	Specimen Designation	Curing @ 70°C for 24 hours	28 Days Compressive Strength (N/mm^2)	Average Compressive Strength (N/mm^2)
1.	A	✓	31.37	33.08
2.	B	✓	33.59	
3.	C	✓	34.28	
4.	P	×	30.43	32.22
5.	Q	×	32.87	
6.	R	×	33.36	

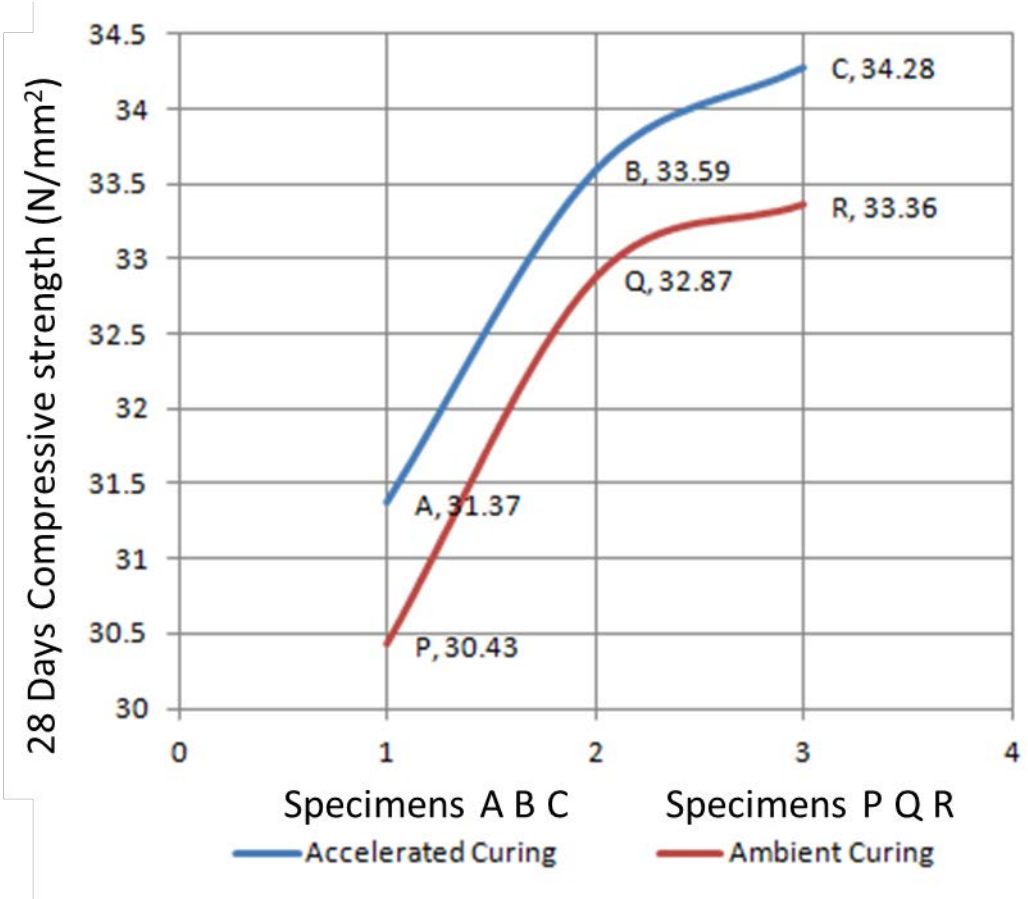


Fig. 1 28 Days Compressive Strength of GGBS based Geopolymer Concrete

CONCLUSIONS AND FUTURE SCOPE

The study has following conclusions:

1. GGBS can be used to replace cement from concrete and has proved that GGBS based GPC can be prepared and used in place of conventional concrete.
2. Use of GGBS based GPC will reduce the use of OPC which is responsible for greenhouse gas emission.
3. The study has developed a green technology for making concrete economical and eco-friendly.
4. The GGBS based GPC has attained an average strength of 33.08 N/mm^2 and 32.22 N/mm^2 in 28 dayswith accelerated curing and without accelerated curing respectively, which satisfies the strength criteria for M25 grade of concrete.
5. In the experimentation it was found that the effect of accelerated curing at the rate of 70°C had no significant change in the strength of GPC. Hence it can be concluded that the GGBS based GPC which is designed, develops its strength at ambient temperature also. This makes it easy to use in construction.
6. When the mixing was done, the concrete mix had limitations of poor workability and started setting very quickly when alkali activators were used.

This study can be extended by using some admixtures to improve the workability of the GPC and the quick setting of the mix can be checked by the use of superplasticizers. Also the various tests can be performed to check the strength and durability aspects of the GGBS based GPC.

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LIFE CYCLE ENERGY ANALYSIS OF MULTI- STORYED RCC BUILDINGS

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ABSTRACT

The buildings are constructed for residential, office and commercial purposes worldwide. These building are the major contributors to social & economic development of a country and utilize enormous portion of energy and available natural resources. Globally, 30-40% of entire elementary energy is used for buildings and they are responsible for 40-50% of green house gas emissions (GHGE). It is therefore essential for the building construction industry to achieve sustainable development in the society. Sustainable development can be achieved by adopting a multi-disciplinary approach covering a number of features such as energy efficiency, improvement in use of materials including water, reuse and recycling of materials and emissions control. Life cycle energy analysis of buildings has greater importance for formulating strategies to achieve reduction in primary energy use of the buildings and controlled emissions. In the present work the life cycle analysis of a G+1 RCC residential buildings different building materials in terms of embodied energy, operating energy and demolition energy are investigated. It is observed that electricity consumption of the building can be switched to sustainable source to decrease the life cycle energy of the building.

INTRODUCTION

Life cycle energy analysis is an approach that accounts for all energy inputs to a building in its life cycle. The boundary of this analysis incorporates the energy use of the following phases: construction, usage, and demolition. Constructional stage incorporates assembling and transportation of structure materials and specialized establishments utilized in erection and remodel of the structures. Operation stage characterizes all exercises identified with the utilization of the structures, over its life expectancy. These exercises incorporate keeping up solace condition inside the structures, water use and controlling apparatuses. At last, pulverization stage incorporates decimation of the structure and transportation of disassembled materials to landfill locales as well as reusing plants. Energy use in each phase is discussed in the next chapter. Energy simulation software widely used are -ENERGY-PLUS, VISUAL DOE, eQUEST, DESIGN BUILDER, ENORM, TRNSYS, ECOTECT, SUNCODE, etc. In a building the energy or work done is in terms of warm (heat), brilliant (light), movement (dynamic), put away (potential), auxiliary (for example power), compound, mechanical, etc. In the construction industry, the term 'vitality' is commonly utilized with regards to creating heat, fueling gear, making items and materials, transportation, etc. In case of a residential building, vitality utilization is frequently credited in terms of the following:

i. Space warming
ii. High temperature water
iii. Cooling and refrigeration
iv. Lighting
v. Washing and drying
vi. Cooking
vii. Other electric burdens

LITERATURE REVIEW

The studies on the life cycle analysis of building materials are discussed as below:

As per USEIA," The buildings and buildings construction sectors combined are responsible for 36% of global final energy consumption and nearly 40% of total direct and indirect CO_2 emissions. Energy demand from buildings and buildings construction continues to rise, driven by improved access to energy in developing countries, greater ownership and use of energy-consuming devices, and rapid growth in global buildings floor area, at nearly 3% per year."

Debnath et al. (1995) examined the vitality required for significant structure materials for single and double story residential structure with load-bearing walls and four storey residential structures with fortified concrete construction in India. For the aggregate floor territory of 50-200 sq. m, complete vitality utilization per unit of floor region diminishes, from 5 to 4.1 GJ for single story, from 4.2 to 3.7 for double storey, and from 4.3 to 3.1 GJ for four storey structures. Adalberth (1997) considered three different single unit dwellings built in Sweden in 1991-92, in order to calculate energy use throughout their life cycle. External wall and joist elements were factory made, which were then transported to the site. Winther and Hestnes (1999) carried out a study of a wooden row house in terms of embodied energy and operating energy in its lifetime, the building located at South of the Norway features passive as well as active solar measures. The result was indicating that solar buildings have low overall energy consumption, when both operational as well as embodied energy was considered. Treloar et al. (2000) investigated the life cycle energy of an Australian residential building. The study focused on the life cycle energy analysis that usually comprises of embodied energy in the manufacturing of building materials, energy usage in operational period along with periodic maintenance. Fay et al. (2000) carried out a case study on an Australian residential building to study its use in evaluating alternative strategies for designing an energy efficient building. It was found that on adding higher insulation levels in the buildings in Australia, it paid back the initial embodied energy in terms of life cycle energy

Reddy and Jagadish (2003) studied the epitomized vitality in structures especially in the Indian setting. Vitality of a multi-storied structure, a heap bearing brickwork building and a soil-bond square structure utilizing elective structure materials has been looked at. It has appeared all out epitomized vitality of burden bearing load-bearing masonry structures can be diminished by half when vitality proficient/elective structure materials are utilized. Asif et al. (2007) studied an actual existence cycle evaluation (LCA) of a 3-bedroom semi-separated house in Scotland. It was discovered that concrete, timber and fired tiles are the three noteworthy vitality costly materials included. It has been determined that concrete alone expends 65% of the absolute epitomized vitality of the home while a lot of ecological effects is even progressively critical. Utama and Gheewala (2008) studied the building envelopes contribute 10-half of the total structure cost and 14-17% of the summed up material mass. The active, as well as passive influence of the envelope materials, assumes a significant job in the structure life cycle vitality. Single landed houses, the commonly built houses in Indonesia, have been decided for this investigation. Shukla et al. (2009) studied the vitality of the adobe house at Solar Energy Park, Indian Institute of Technology Delhi, New Delhi and its impact on the earth. The unique element of the adobe house is that the entire house is built by utilizing low vitality serious materials like soil, sand, dairy animals' compost, and so forth. The adobe house is more naturally cordial house in contrast with ordinary structures. Ramesh et al. (2012) studied the life cycle vitality (LCE) request of a residential structure of usable floor region about 85.5 m^2 situated at Hyderabad (Andhra Pradesh), India. The house is demonstrated for five distinctive climatic zones of India, for example, hot and dry, warm

and muggy, composite, cold and moderate. Non-conventional wall materials alone (without protection) lessen It was observed that around 10 cm for composite, hot and dry, warm and muggy, and cold atmospheres and, 5 cm for the moderate atmosphere.
Ramesh et al. (2013) studied the life cycle vitality examination of a multifamily house arranged in Allahabad (U.P), India. The investigation covers vitality for development, task, support and obliteration periods of the structure. The results demonstrated that working vitality (89%) of the structure is the biggest supporter of life cycle vitality of the structure, trailed by epitomized vitality (11%).

METHODOLOGY ADOPTED

In the present dissertation work the life cycle analysis of a G+1 RCC residential building as shown in Fig.1 in terms of embodied energy, operating energy and demolition energy. The energy consumption for different building materials as mentioned below using eQUEST (QUick Energy Simulation Tool) are investigated: (a)Roof: R1: clay tile finish with orange color (medium) and exterior insulation of 2" polystyrene instead of 4" thick concrete without exterior finish with medium color, R2: aluminum finished uncolored roof surface and exterior insulation of 2" polystyrene, R3: film, mylar aluminized uncolored and exterior insulation of 2" polystyrene instead of 4" thick concrete w/o exterior finish with medium color. (b) Concrete masonry unit (CMU): CMU1: CMU (w/o finish, solid grouted), medium colored and exterior insulation of 2" polystyrene instead of 8" thick brick with red masonry, CMU2: vapor deposited low e-coating finish, uncolored colored, solid grouted and exterior insulation of 2" polystyrene instead of 8" thick brick with red masonry, CMU3: Steel galvanized (bright) uncolored colored, hollow and exterior insulation of 2" polystyrene instead of 8" thick brick with red masonry and (c) Ceilings and Floors: F1: lay in acoustic tile with R-11 batt insulation in ceilings, vinyl tile with 1+1/2" polystyrene insulation and 1.25" concrete cap in floors e instead of ceilings with plaster finish and floors w/o any insulation, F2: drywall finish with R-11 batt insulation in ceilings, vinyl tile with 1+1/2" polystyrene and 1.25" concrete cap in floor instead of ceilings with plaster finish and floors w/o any insulation, F3: plaster finish with R-11 batt insulation in ceilings, vinyl tile with 1+1/2" polystyrene insulation and 1.25" concrete cap in floor instead of ceiling with plaster finish and floors w/o any insulation.

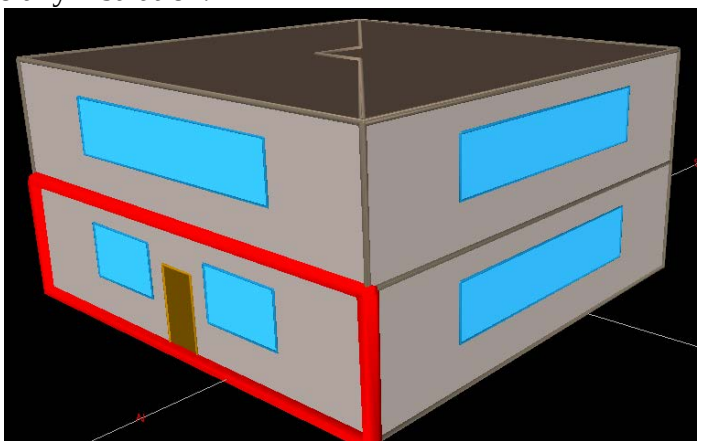


Fig. 1 3-D view of G+1 building model A1

Life Cycle Energy (LCE)

Life cycle vitality of the structure is assessed by summing up the vitality brought about for development (introductory embodied), task, upkeep (repeating encapsulated) lastly destruction of the structure toward an amazing finish. The life cycle energy comprises of

initial embodied energy, building operation, building maintenance and demolition energy. Life Cycle Energy (LCE) for a building it is given by:

LCE = IBE + RE + (OE × life span of building) + DE

where,

IBE = Initial Embodied Energy;

RE = Repeating Embodied Energy;

OE = Operating Energy per year;

DE = Demolition Energy.

This case study includes four phases i.e. structural material manufacturing, construction of building, operation – maintenance and demolition. Embodied energy corresponding to each building materials are taken from Indian Literature and the same are mentioned. The information so obtained about the type and quantity of structural materials for the building is estimated using various sources. The result so obtained shows the electrical consumption (in kWh) of the building for each month. From Fig. 2 it can be easily concluded that maximum energy consumption is due to heating and space cooling in the month of January (0.93) and May (1.78) respectively.

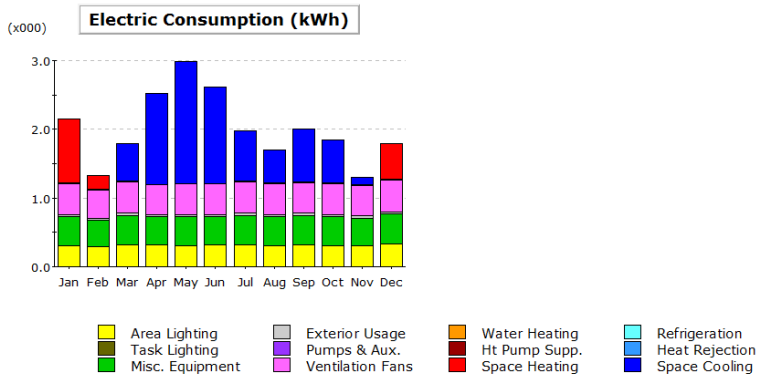


Fig. 2 Electric consumption for Model A1

Model R1, the result so obtained shows the electrical consumption (in kWh) of the building for each month. For the year, aggregate energy consumption is at 22.71 kWh. Fig. 3 shows the electric consumption for model R1. From January to December months. Similarly, results are obtained for other models.

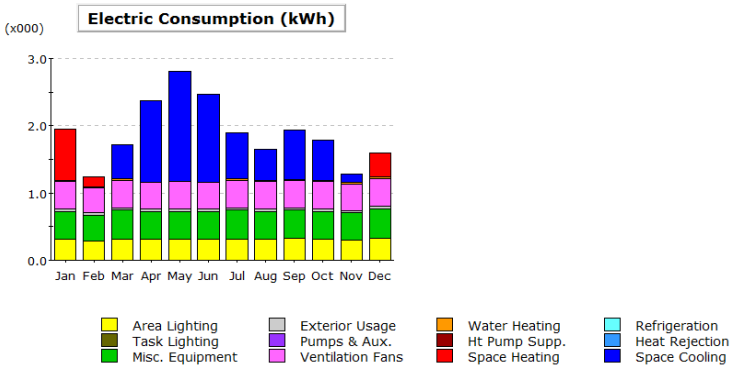


Fig.3 Electric consumption for Model R_1

RESULTS

Fig. 4 shows the comparison of the annual electric consumption in kWh for all the models considered in the present study. The horizontal axis refers the different models under

consideration and the vertical axis shows their electric consumption annually. Fig. 5 shows the comparison of annual utility charges for all the models. Table 1 gives the values of annual electric consumption for different models and also the percentage reduction in the electric consumption with respect to the model A1. It can be observed that the minimum reduction is in the case of model F1 1.08 % where lay in acoustic tile with R-11 batt insulation in ceilings, vinyl tile with 1+1/2" polystyrene insulation and 1.25" concrete cap in floors e instead of ceilings with plaster finish and floors w/o any insulation are used as flooring materials. The maximum reduction in energy consumption is in the case of model CMU2 12.91 % where vapor deposited low e-coating finish; uncolored colored, solid grouted and exterior insulation of 2" polystyrene instead of 8" thick brick with red masonry is used.

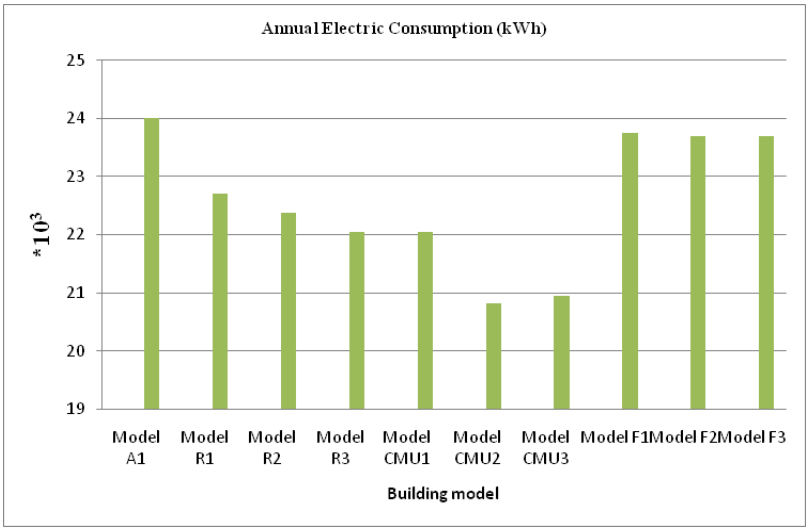


Fig. 4 Comparison of annual electric consumption for all the models

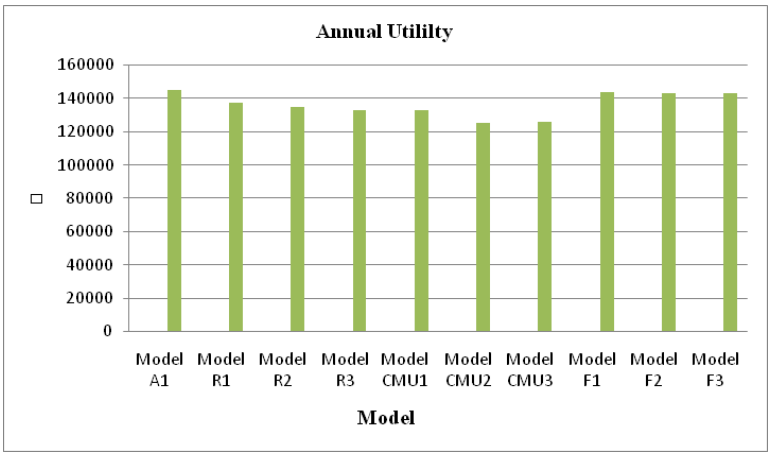


Fig 5 Comparison of annual utility charges for all the models

Table 1 Comparative electric consumption for different models

Building Model	Annual Energy Consumption(kWh)	(%) Reduction in Energy Consumption	Annual utility cost (Rs.)	(%)Reduction in utility cost (Rs.)
A_1	24.01×10^3	0	145,524.73	0
R_1	22.71×10^3	5.41	137,409.20	5.57
R_2	22.37×10^3	6.83	135,258.93	7.05
R_3	22.05×10^3	8.16	133,316.75	8.38

CMU_1	22.04×10^3	8.20	133,247.38	8.43
CMU_2	20.91×10^3	12.91	125,548.03	13.72
CMU_3	20.94×10^3	12.78	126,380.39	13.15
F_1	23.75×10^3	1.08	143,860.01	1.14
F_2	23.70×10^3	1.29	143,568.17	1.34
F_3	23.69×10^3	1.33	143,498.81	1.39

CONCLUSIONS

Based upon the result obtained from the present work, following conclusions can be made:

1) Major contributor to the life cycle energy analysis of the building is (75.4%) operating energy during the operating phase followed by the (21.8%) constructional phase and (2.71%) demolition phase.
2) It is observed that as on varying the comfortable indoor temperature, the operating phase can contribute further to the life cycle energy analysis.
3) Large contributors to the initial embodies energy were steel, cement and bricks. The combination alone contribute to 62% of the total initial embodies energy.
4) The energy consumption is found to be 107.6 kWh/m^2annually.
5) The life cycle energy intensity calculated is ~36 GJ/m^2.
6) On replacing the building materials it has been found that Model CMU_2 has the (12.91%) maximum reduction in operating phase followed by (12.78%) Model CMU_3.Further on assuming that the introduction of these replaced building materials would not increase the energy consumption in construction phase to this extent; they'll also reduce the life cycle energy of the building to some extent.
7) Electricity consumption of the building can be switched to sustainable source to decrease the life cycle energy of the building.
8) Finally using low embodied energy materials leaves a significant effect on the manufacturing phase.

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EXPERIMENTAL INVESTIGATION ON AGGREGATE AND CEMENT BY PARTIAL REPLACEMENT OF KOTA STONE AND KOTA STONE SLUDGE IN CONCRETE

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ABSTRACT

Concrete is the most important component used in the construction works throughout the world, where the coarse aggregate is generally natural aggregate. The use of natural aggregate in construction behavior results in the excessive mining. Due to excessive mining, natural resources are decreases day by day. Thus, it is becoming necessary to use the substitute material in concrete. Kota stone is one of the important materials used in the construction industry. The study concerns mainly on the possible use of stone waste in the construction industry, which would reduce both environmental impacts and the production cost. The main objective of this study is to make concrete eco-friendly and economical for construction work by partial replacement of concrete material with stone waste. The study concern on concrete work in which industrial waste like small pieces of Kota stone and Kota stone sludge are partially replaced by aggregates and cement in concrete respectively. Results concluded that the 20% replacement of aggregate, cement with Kota stone and Kota stone sludge achieved compressive strength 27 N/mm^2 of M-20 concrete which is 7.5% more than the conventional concrete. It is reasonable to utilize Kota stone waste as completely coarse or fractional in concrete with Kota stone sludge.

KEYWORDS: Kota stone; Kota stone sludge; M-20; compressive strength; conventional concrete

INTRODUCTION

Concrete is the most crucial material in modern day construction. It is known for versatile properties like high compressive strength and long lasting durability. Due to speedy growth in construction activity, the consumption of concrete is rising every year. This results in excessive exploitation of natural resources. In some places either natural sand may not be of good quality or good quality sand has to be transported from long distances, which adds to the cost of construction. Therefore, it is becoming inevitable to replace natural sand in concrete by an alternative material either partially or completely without affecting the quality of concrete. Large-scale efforts are required for reducing the usage of the raw materials that are currently available so that large replacement is using the various by-product materials that are easily accessible in present day. Use of steel industry waste by-products, marble sludge, and granite stone provides a great opportunity to be utilized as an alternative for aggregates.Therefore, many countries have still been working on how to re-use the waste materials. Use of stone waste in various engineering applications can solve the problem of disposal of stone waste and other purposes. Stone waste can be used in concrete to improve its strength and other durability factors. Stone waste can be used as a partial replacement of cement or replacement of fine aggregate and as a supplementary addition to achieving different properties of concrete.In past there are several experimental investigations, which analyze raw material as replaced with concrete ingredients.Dhanapandian and Gnanavel (2009) founded that the average strength of waste mixed brick obtained at 20 wt. % is higher

than that of other wt. %. Finally, granite and marble waste as an alternative raw material in brick production will induce a relief on waste disposal concerns.

Rai et al. (2011) stated that partial replacement of cement and fine aggregates by varying percentage of marble powder and marble granules reveals that increased waste marble powder or waste marble granule ratio result in increasedworkability and compressive strengths of the mortar and concrete.Shiruleet.al (2012) proved that the optimum percentage for replacement of marble powder with cement and it is almost 10% cement for both cubes and cylinders. Omar, et.al (2012) replaced proportion of sand with limestone waste, 25%, 50%, and 75% were practicedin the concrete mixes except in the concrete mix. Besides, proportions of 5%, 10% and 15% marblepowder were practiced in the concrete mixes. Nagpal et al. (2013) stated that crushed stone dust waste can be used effectively used to replace natural sand in concrete. In the experimental study of strength characteristics of concrete using crushed stone dust as a fine aggregate, it is found that there is an increase in compressive strength, flexural strength and tensile strength of concrete.

Soliman (2013) found that the Increasing the marble powder ratio replacement of cement led to the increase in the compressive strength by about (25% and 8%) for the marble powder replacement ratios (5% and 7.5%) compared to the control mix. Patel et al.(2014) reviewed that use of different waste material as a partial replacement of cement or fine aggregate or coarse aggregate and studied as many useful research papers in this field and trying to improve with locally available waste material, proved economical as well as least damage to surrounding environment.Kujur et al.(2014)was conducted experimental program using 30%, 40%, 50%, 60% and 70% partial replacement of fine aggregate with stone dust has been taken for concrete of M25 grade with 0.46 water cement ratio. In this study, set of cubes and beams were cast for compressive and split tensile strength respectively. It has been observed that 40% replacement of fine aggregate with stone dust is adaptable. Ali and Hashmi (2014) indicated that marble dusts incorporation results insignificant improvements in the compressive, flexural and split tensile strengths of concrete up to 10% of replacement and also the results of the present investigation indicate that stone dusts and marble dust incorporation results insignificant improvements in the compressive strengths of concrete up to 20% of SD and 10% of MP of replacement.Kushwah et al. (2015) studied that marble slurry in Cement Concrete replacing Sand is 30% which shows equal strength as of Control i.e. 1:2:4 Cement Concrete 0% Marble slurry. Marble slurry can be easily utilized in the construction industry in preparing Cement Concrete.Kushwah and Prakash(2016)determined different properties of marble slurry in the laboratory. Sp. gravity 2.61, Fineness modulus was found to be 0.91 and Utilization of marble slurry in Cement Mortar replacing Sand is 30% which shows equal strength as of Control i.e. 1:3 Cement Mortar 0% Marble slurry.

In the recent years, Raghvendra and Trivedi (2017)replacedMarble dust in the concretewith 5%, 10%, 15%, 20% & 25% of cement and then compared with Conventional concrete and then analyze the variation betweenvalues of results with conventional concrete. Afterwards Kumar et. al (2018) concluded that the partial replacement of cement with GGBS and complete replacement of sand with manufactured sand helped in improving the strength of the concrete substantially compared to normal mix concrete. Compressive strength test was carried out for 7, 14 and 28 days. The present study deals with economy of concrete, with inclusion of Kota stone waste concrete compared to conventional concrete

METHODOLOGY ADOPTED

In this study, we will be making 3 cubes of concrete with different percentages of cement andaggregate is replaced with Kota stonesludge and Kota stone respectively and compared with conventional concrete.

Experimental Material

Cement-Cement is the major requirement of concrete and it is a binding material in concrete so use of cement increased every day. For the experimental work OPC-53 grade cement is being used.

Fine Aggregates- Sand is an essential ingredient of concrete as a fine aggregate. It is an inert ingredient in the concrete mix; its role cannot be neglected. For the experimental work locally available natural river sandof Zone II(Having characteristics according toIS-383-1963) is being used.

Coarse Aggregates-Aggregates plays major role in the properties of both fresh and hardened concrete. Changes in size, gradation, specific gravity and unit weight can all modify the performance of concrete mix. For the experimental work 10-20mm size of aggregate (Having characteristics according to IS-383-1963) is being used.

Water-The strength of concrete directly depends on the quantity and quality of water used in the concrete mix and plays an important role in the mixing.For the experimental work water from IPS Academy is being used.

Kota Stone-Kota Stone is a fine grained variety of limestone quarried at Indore district, Madhya Pradesh, India. Kota Stone industry generates solid waste and stone sludge. For the experimental work Kota stone is being used and replaced with percentage variation of Aggregates.

Kota Stone Sludge- Kota Stone Sludge affects productivity of land due to decreased porosity, water absorption and percolation. For the experimental work Kota stone slurry is being used and replaced with percentage variation of Cement.

Tests on Materials

The properties of materials was tested according to relevant IS code provisions. Table 1 shows physical properties of Ordinary Portland Cement and Table 2 shows physical properties of Coarse Aggregate and Kota stone. The chemical properties of coarse Aggregate and Kota stone as shown in Table 3 and physical properties of Kota stone sludge and Fine Aggregates as shown in Table 4.

Table 1:Physical Properties of Ordinary Portland Cement (OPC 53 Grade)

Sr. No.	Physical properties of cement	Result	Requirements as per IS:8112-1989
1	Specific gravity	3.15	3.10-3.15
2	Standard consistency (%)	31%	30-35
3	Initial setting time (hours, min)	35 min	30 minimum
4	Final setting time (hours, min)	178 min	600 maximum

Table 2:Physical Properties of Coarse Aggregate and Kota stone

Sr.No	Physical properties of Coarse Aggregate and Kota stone	Aggregate (10-20mm)	Kota Stone
1	Specific Gravity	2.84	2.73
2	Impact Value	9.38%	9.6%
3	Angeles Abrasion Value	19.85%	37.416%
4	Crushing Value	22.67%	20.23%

Table 3:Chemical Properties of Coarse Aggregate and Kota stone

Sr No	Chemical Constituent	Percentage by WeightKota Stone
1	Calcium Oxide	38.86%

2	Magnesium Oxide	1.09%
3	Aluminium Oxide	2.20%
4	Silica	26.67%

Table 4:Physical Properties of Kota stone sludge and Fine Aggregates

Sr.No	Physical properties of Coarse Aggregate and Kota stone	Material Kota stone sludge	Fine Aggregate (Sand)
1	Specific Gravity	2.25	2.62
2	Water Absorption (%)	5.85	1.98
3	Fineness Modulus	-	2.01

MIX PROPORTIONS OF CONCRETE BLOCKS

As per concrete mix proportioning IS 10262:2009, firstly find the proportions of concrete (weight in kilograms) ingredients for 1 (no.) block as shown in Table 5, details of various types of blocks as shown in Table 6 and then determine the proportions of concrete (weight in kilograms) as per batch, each batch contain 6 blocks.

Table 5Mix Proportions for 1 block (By Weight in kilograms)

Concrete Block Mixes	Cement (Kg)	Fine Aggregate(Kg)	Coarse Aggregate 10-20mm(Kg)	Kota stone (Kg)	Kotastones sludge (Kg)
A	1.22	2.50	4.18	0	0
K1	0.976	2.50	3.314	0.836	0.244
K2	0.854	2.50	2.926	1.254	0.366
K3	0.732	2.50	2.508	1.672	0.488

Table 6Details of various types of blocks

Concrete Block Mixes	Description
A	Conventional block (as per manufacturer proportions)
K1	40(20+20)% replacement of the Aggregate and cement by kota stone andKota stone sludge by weight
K2	60(30+30)% replacement of the Aggregate and cement by kota stone andKota stone sludge by weight
K3	80(40+40)% replacement of the Aggregate and cement by kota stone andKota stone sludge by weight

RESULTS

Testing plays an important role in controlling the quality of cement concrete work. Systematic testing of the raw materials, fresh concrete and hardened concrete is an inseparable part of any quality control program for concrete, which helps to achieve higher efficiency of the materials used and greater assurance of the performance of the concrete in regard to both strength and durability.

Compressive Strength Test [IS: 2185 (Part II) -1983]

15cm × 15cm × 15cm concrete blocks specimens with aggregate (10-20mm) and Ordinary Portland cement (OPC) partially replaced by Kota stones and Kota stonesludge, accordingly at 20%, 30%, 40% level is cast, individually. During casting, the blocks are made in the hand press machine. After making the specimens are removed from the moulds and subjected to

water curing for 3 days. After curing, the specimens are tested for compressive strength using a calibrated compression testing machine of 2,000 KN capacities. The compression test is carried out the specimens at the end of 14 days and 28 days for concrete mix A,K1,K2 and K3 respectively as shown in Table-7.

Table 7: Compressive strength test results for A (Conventional) Concrete Block at 14, 28 Days

Concrete Block Mix	Average Compressive Strength N/mm^2	
	14 Days	28 Days
A (0 %)	15.91	25.8
K1(20 %)	15.9	27.4
K2(30 %)	14.8	24.4
K3(40 %)	13.73	22.16

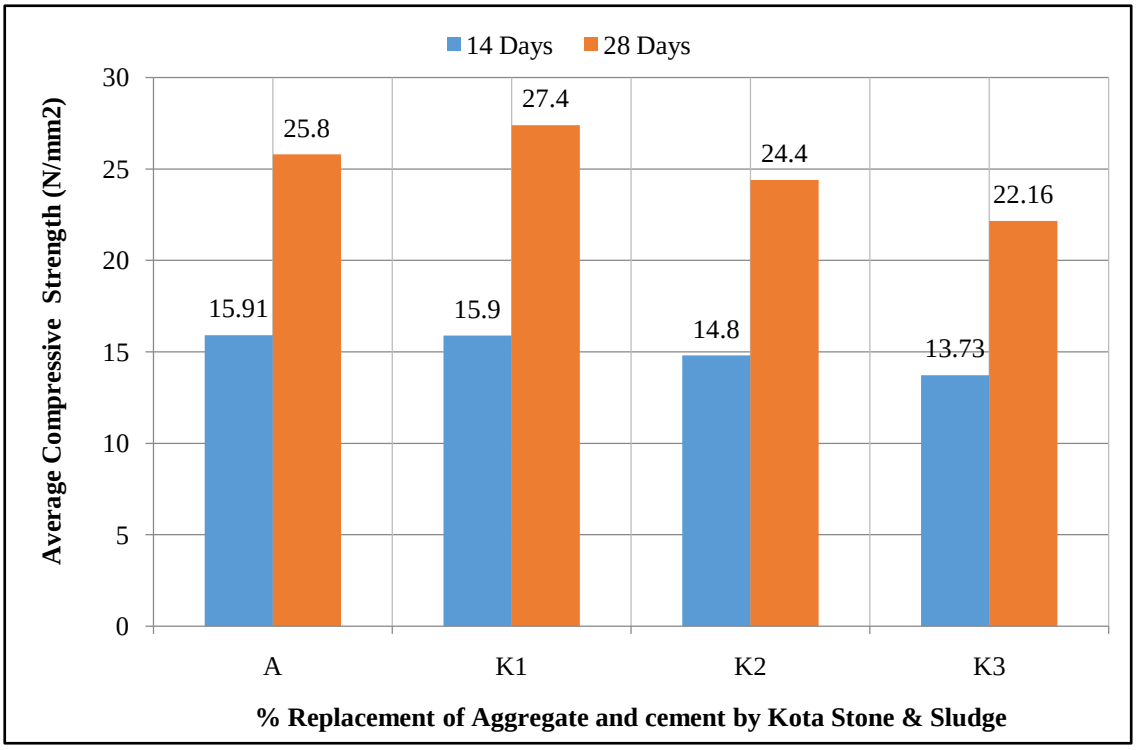


Fig.1: % Replacement of Aggregate and cement by Kota Stone & Sludge v/s Compressive Strength (N/mm^2) at 14, 28 Days (For A, K1, K2 and K3)

From above Fig.1,percentage replacement of Aggregate and Cement by Kota stone and Kota stone sludge v/s Compressive Strength (N/mm^2) at 14, 28 Days (For A, K1, K2 and K3).

Water Absorption Test [IS 2185 PART 1:2005]

Concrete blocks shall be completely immersed in clean water at roomtemperature for 24 hours. The blocks shall then be removed from the water and permitted to drain forone minute by placing them on a 10 mm wire mesh, water being removed with a damp cloth, the saturated and surface dry blocks immediately weighed. After weighing allblocks shall be dried in a oven at 100°c to 115°c for 24 hours and until twosuccessive weighing at intervals of 2 hours show an increment of loss not greater than 0.2 percent ofthe last previously determined the mass of the specimen.The water absorption calculates as given below:

Absorption, percent = (A-B) /B * 100
Where,

A= Wet mass of unit in kg.
B = Dry mass of a unit in kg.

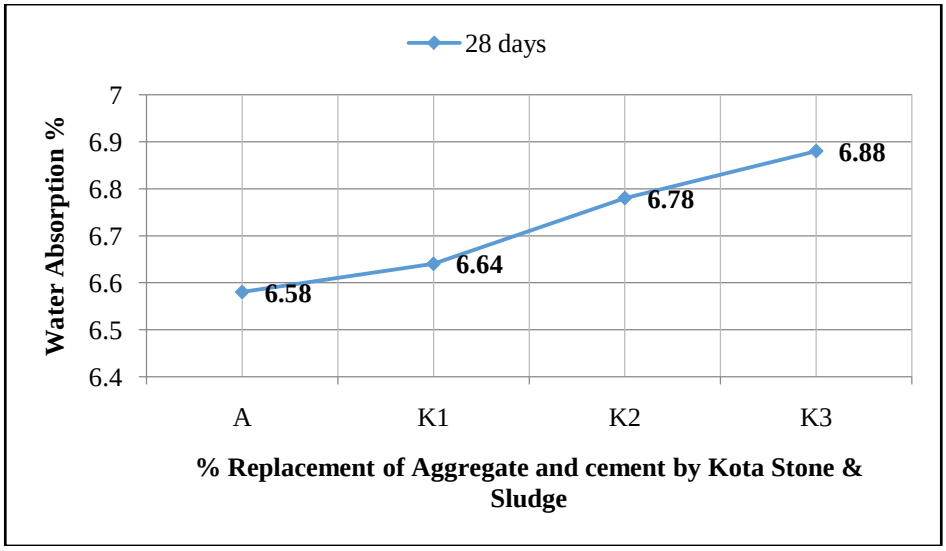


Fig. 2: % Replacement of Aggregate and cement by Kota Stone & Sludge vs Water Absorption % at 28 days for (For A, K1, K2 and K3).

From above Fig.2, water absorption of solid concrete block increased with the increase in percentages of Kota stones chips in solid concrete blocks. But for the 20% replacement of the Kota stones chips with aggregate and Kota stone sludge gives slightly higher water absorption to the conventional concrete block at 28 days. Also increased percentage of kota stone and kota stone sludge in solid concrete block gives more water absorption compared to the standard concrete block.

CONCLUSIONS

Based on the experimental test results obtained from the study, the following conclusions are drawn.

1. The compressive strength of concrete block increased with the increase in days. But for the K1 replacement of the Kota stones waste and Kota stone sludge by Aggregate and Cement gives higher compressive strength compared to the conventional concrete block at 28 days.
2. Compared to the K2and K3 replacement of Kota stone waste andKota stone sludge to the Aggregate and cement in concrete blocks,K1 gives higher compressive strengthat 28 days.
3. As K2 and K3 i.e. % replacement of Aggregate and Cement by Kota stone and Kota stone sludge increase, compressive strength decrease with respect to conventional concrete but it gives satisfactory results and this combination used in lighter construction.
4. Production of solid concrete blocks using Kota stone waste is cost-effective and eco-friendly.
5. It can also reduce the disposal problems of waste materials and also consume the demand of aggregates and cement used for making of Solid Concrete Block.

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ANALYSIS AND DESIGN OF REINFORCED CONCRETE BUNKER

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ABSTRACT

Bunker and silo are the most traditionally used bulk storage structures. These structures find an important application in the storage of wide variety of materials extending from industrial raw material such as coal to bulk storage of grains in the warehouses. The objective of the study is to develop a program for the analysis and design of a Reinforced Concrete Bunker, in accordance with Indian Standards, and validation of the results using existing guidelines from latest structure design software. The program developed is structured for the most economic section the aforesaid storage bin. The scheme used in this research follows the order of developing the program, manually checking the same for various capacities of the bin, running the design in structural software thereby obtaining stress contours and finallyreaching the validation of the obtained results. The program described herein facilitates the designing of bunkers of various capacities within a small time frame.

KEYWORDS: Bunker, Storage bin, Concrete, Stress contour**,** Economic design, Reinforced Concrete Bunker

INTRODUCTION

Bunkers and silos are used for large scale storage and upkeep of various bulk materials such as raw materials in industries, grains in warehouses etc.As per the technical specifications, a bunker and a silo can be differentiated appropriately. If the depth and breadth of the storage container is such that the plane of rupture meets the surface of the material before the plane of rupture hits the opposite side of the storage container, it is called a bunker. However, if the plane of rupture does not cross the surface level of the material, it is known as silo. In general terms, for a bin to be called as a silo, its depth should be greater than twice its breadth.Hence, other name given to bunker is shallow bin while the other name for silo is deep bin. These structures need to be designed carefully as they have to bear huge pressure on their walls due to static bulk material loads.This pressure fluctuates during the process of filling and emptying. It also varies with the location of the dischargeopening for the stored material.The actual analysis of pressure is very difficult due to a lot of variable factors. In bunkers, due to shallow depth, we assume that the friction between the walls of the bunker and the material filled in it is negligible. Here, the load of the material stored in it is considered to be supported by the floor of the bunker. Hence, the Bunkers are so designed that they can withstand live pressure acting on their inner walls along with their self-weight.

The work represented here is a result of thorough studies done on the various researches targeted by different scientists and researchers in this field.The research in the field of storage bins started with the onset of 1960s and is continuing till date. The first logical method for design of gravity mass-flow hoppers was issued in an approachable format by Jenike(1964).The design procedures of storage bins from different sources were elaborated to a merged method that can be applied to a larger class of reinforced concrete silos by Saleem et al. (2018).In this study, analysis and design mechanism were summarized and portrayed in

a simplified form.Throwing a light on the past researches again,Killion (1985) examined design pressures in circular storage structures. Bunker pressures were distinguished from silo pressures.Blight (1992) suggested that pressures and strains for deep bins could be non-uniform around the perimeter, and are dependent on thermal effects.Askari and Elwi(1998) presented a numerical exploration of wall pressures produced by bulk materials kept in hopper-silo combinations.As far as the loads acting on deep bins are considered, Bucklin et al. (2013) concluded that the facilities for grain storage should be designed to bear loads inflicted by the grains along with other environmental loads.Dogangun et al. (2009) conducted a case study, a review and discussion of the regular or special silo failures due to explosion and bursting, asymmetrical loads generated during filling or discharging, non-uniform soil pressure, corrosion of metal silos, deterioration of concrete silos because of silage acids, internal structural failure and thermal ratcheting.Meshram and Bhadke(2019) studied the most economical design of reinforced concrete storage structures with respect to different seismic zones in India.

Keeping in mind all the above prospects of researches already conducted in the concerned field, a program was developed for the analysis and design of a reinforced concrete square bunker used primarily for the storage of raw materials such as coal etc in thermal power plants. The program was built using spreadsheets and was validated using structural design software STAAD. Pro v8i. Meanwhile the stress contours were also drawn which helped in the validation of the same. While designing the aforesaid bunker, utmost care is taken to make the section as economical as possible.

METHODOLOGY

A program is developed for the design and analysis of a reinforced concrete bunker using spreadsheets in accordance with IS: 4995-1974 (part I and part II) and IS: 456-2000. The program is checked for varying capacities of the storage material in the bin.Stress contours are obtained for the designed bunker(s) using STAAD.PRO V8i. The validation of the results obtained using spreadsheet program is done with the help of stress contours obtained using STAAD.PRO V8i. Conclusions are drawn after observing the results of analysis and design of reinforced concrete bunker of different capacities by the program developed using spreadsheets and its stress contours obtained using STAAD.PRO V8i. The entire process carried out during the research work is represented in Fig. 1.

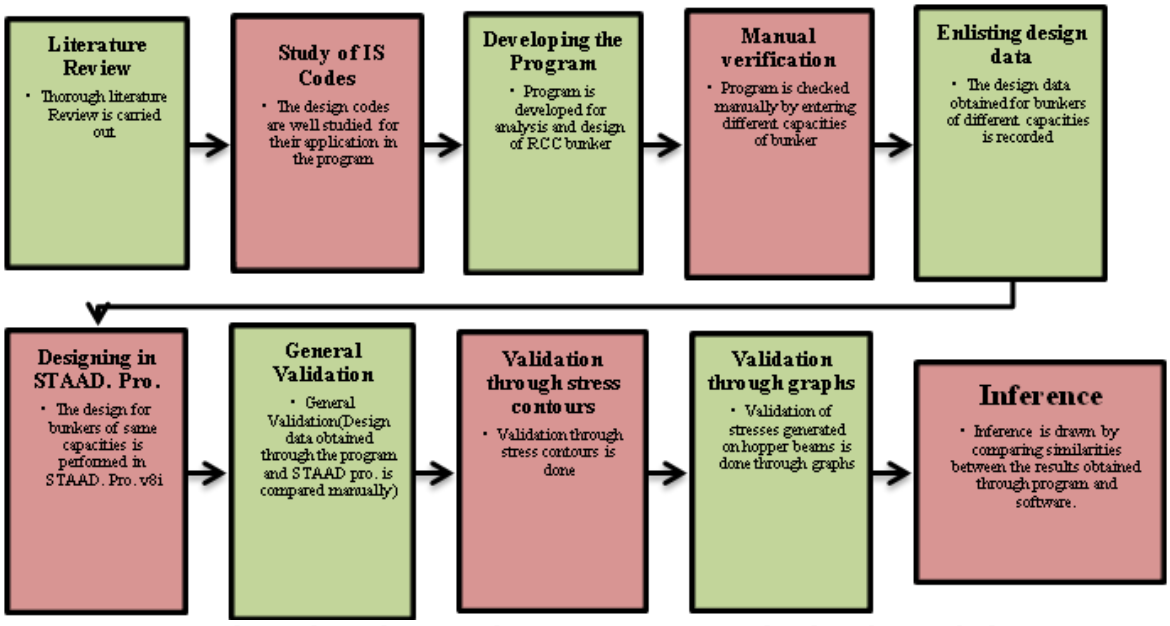


Fig. 1: Flow Chart showing the various steps involved in the work done

ANALYSIS AND DESIGN

The analysis and design of reinforced concrete bunker has been done using spreadsheets. A program is developed in MS-Excel for a square reinforced concrete bunker keeping in mind the codal provisions for the design and analysis of reinforced concrete storage bins as per Indian Standards. The program developed for reinforced concrete bunker serves the purpose of both analysis and design. Different capacities of bunkers have been considered and the program has been checked. The design of the bunker of a given capacity is done making all the sections as economic as possible. Three capacities of bunkers are taken, which are 300 KN, 800 KN and 1200 KN. All the three bunkers are analyzed and designed as per the provisions of IS: 4995-1974(Part1) (Criteria for design of Reinforced Concrete Bins for Storage of Granular and Powdery Materials), IS: 4995-1974(Part II) (Criteria for design of Reinforced Concrete Bins for Storage of Granular and Powdery Materials) through the program developed. A sample capacity was taken initially to design the structure of the program. Later on the program was generalized and was checked by inputting different capacities of the bunker.

The data generated from the design of the bunkers of three different capacities executed by the program is presented in Table 1.

Table 1 Dimensions and Design of bunker using program

S.	Dimension and Design	Value adopted/calculated
1.	For 300KN capacity bunker	
I	Dimensions	
a)	Cross section of bunker	3m x 3m
b)	Height of Hopper portion	1.25m
c)	Cross section of discharging hole	0.5m x 0.5m
d)	Total height	3.25m
e)	Storage capacity of the bunker	303 KN
II	Design of side walls	
a)	Horizontal pressure at any level	20329 N/m^2
b)	Direct tension in the wall	32323 N
c)	Effective depth	133 mm (provided effective depth = 140
d)	Area of steel at corners	1135 mm^2
e)	Area of steel at center	631 mm^2
f)	Total area of vertical reinforcement	435 mm^2
g)	Thickness of the wall	150.8 mm
h)	Area of steel at support (reinforcement	800 mm^2
i)	Area of steel at center (reinforcement	440 mm^2
III	Design of hopper bottom	
a)	Direct tension in hopper slab	127083.2 N
b)	Area of steel to bear direct tension	908 mm^2
c)	Effective depth	100.7 mm (provided effective depth = 130
d)	Area of steel required (at corners)	821 mm^2
e)	Area of steel required (in the middle	483 mm^2
2.	For 800KN capacity bunker	
I	Dimensions	
a)	Cross section of bunker	4m x 4m
b)	Height of Hopper portion	1.75m
c)	Cross section of discharging hole	0.5m x 0.5m
d)	Total height	5.00m

e)	Storage capacity of the bunker	807 KN
II	Design of side walls	
a)	Horizontal pressure at any level	31275 N/m^2
b)	Direct tension in the wall	65365 N
c)	Effective depth	213 mm (provided effective depth = 220
d)	Area of steel at corners	1941 mm^2
e)	Area of steel at center	1088 mm^2
f)	Total area of vertical reinforcement	435 mm^2
g)	Thickness of the wall	232.0 mm
h)	Area of steel at support (reinforcement	839 mm^2
i)	Area of steel at center (reinforcement	467 mm^2
III	Design of hopper bottom	
a)	Direct tension in hopper slab	321155.4 N
b)	Area of steel to bear direct tension	2294 mm^2
c)	Effective depth	160.4 mm (provided effective depth is
d)	Area of steel required (at corners)	1913 mm^2
e)	Area of steel required (in the middle	1181 mm^2
3.	For 1200 kN capacity bunker	
I	Dimensions	
a)	Cross section of bunker	5m x 5m
b)	Height of Hopper portion	2.25m
c)	Cross section of discharging hole	0.5m x 0.5m
d)	Total height	4.45 m
e)	Storage capacity of the bunker	1202 KN
II	Design of side walls	
a)	Horizontal pressure at any level	27835 N/m^2
b)	Direct tension in the wall	72092 N
c)	Effective depth	252 mm (provided effective depth = 270
d)	Area of steel at corners	2207 mm^2
e)	Area of steel at center	1256 mm^2
f)	Total area of vertical reinforcement	435 mm^2
g)	Thickness of the wall	272.5 mm
h)	Area of steel at support (reinforcement	1066 mm^2
i)	Area of steel at center (reinforcement	590 mm^2
III	Design of hopper bottom	
a)	Direct tension in hopper slab	480832.6 N
b)	Area of steel to bear direct tension	3435 mm^2
c)	Effective depth	196.5 mm (provided effective depth is
d)	Area of steel required (at corners)	2720 mm^2
e)	Area of steel required (in the middle	1523 mm^2

VALIDATION

General Validation

After carrying out the design using the program, bunkers of the same respective capacities are designed using the structural software STAAD. Pro. V8i. keeping all the considerations and assumptions taken to be exactly same as considered during the designing of bunkers using the program developed.

The data generated from the design of the bunkers of three different capacities executed by the program is presented in Table 2.

Table 2 Dimensions and design of bunker using STAAD. Pro.

S. No.	Dimension and Design	Value adopted/calculated
1.	For 300KN capacity bunker	
I	Dimensions	
a)	Cross section of bunker	3m x 3m
b)	Height of Hopper portion	1.25m
c)	Cross section of discharging hole	0.5m x 0.5m
d)	Total height	3.25m
e)	Storage capacity of the bunker	305 KN
II	Design of side walls	
a)	Horizontal pressure at any level	20334 N/m^2
b)	Direct tension in the wall	32325 N
c)	Effective depth	131 mm (provided effective depth = 140 mm)
d)	Area of steel at corners	1140 mm^2
e)	Area of steel at center	636 mm^2
f)	Total area of vertical reinforcement	435 mm^2
g)	Thickness of the wall	152mm
h)	Area of steel at support (reinforcement after curtailment)	808 mm^2
i)	Area of steel at center (reinforcement after curtailment)	445mm^2
III	Design of hopper bottom	
a)	Direct tension in hopper slab	127087.2 N
b)	Area of steel to bear direct tension	905 mm^2
c)	Effective depth	102mm (provided effective depth = 130 mm)
d)	Area of steel required (at corners)	830 mm^2
e)	Area of steel required (in the middle span)	485 mm^2
2.	For 800KN capacity bunker	
I	Dimensions	
a)	Cross section of bunker	4m x 4m
b)	Height of Hopper portion	1.75m
c)	Cross section of discharging hole	0.5m x 0.5m
d)	Total height	5.00m
e)	Storage capacity of the bunker	807 KN
II	Design of side walls	
a)	Horizontal pressure at any level	31278N/m^2
b)	Direct tension in the wall	65363 N
c)	Effective depth	216 mm (provided effective depth = 220 mm)
d)	Area of steel at corners	1945 mm^2
e)	Area of steel at center	1091 mm^2
f)	Total area of vertical reinforcement	435 mm^2
g)	Thickness of the wall	233 mm

h)	Area of steel at support (reinforcement after curtailment)	842 mm^2
i)	Area of steel at center (reinforcement after curtailment)	468 mm^2
III	Design of hopper bottom	
a)	Direct tension in hopper slab	321151.5 N
b)	Area of steel to bear direct tension	2291 mm^2
c)	Effective depth	160mm (provided effective depth is different)
d)	Area of steel required (at corners)	1910 mm^2
e)	Area of steel required (in the middle span)	1179 mm^2
3.	For 1200KN capacity bunker	
I	Dimensions	
a)	Cross section of bunker	5m x 5m
b)	Height of Hopper portion	2.25m
c)	Cross section of discharging hole	0.5m x 0.5m
d)	Total height	4.45 m
e)	Storage capacity of the bunker	1208 KN
II	Design of side walls	
a)	Horizontal pressure at any level	27832 N/m^2
b)	Direct tension in the wall	72094 N
c)	Effective depth	255 mm (provided effective depth = 270 mm)
d)	Area of steel at corners	2210 mm^2
e)	Area of steel at center	1252 mm^2
f)	Total area of vertical reinforcement	435 mm^2
g)	Thickness of the wall	274 mm
h)	Area of steel at support (reinforcement after curtailment)	1065 mm^2
i)	Area of steel at center (reinforcement after curtailment)	591 mm^2
III	Design of hopper bottom	
a)	Direct tension in hopper slab	480833 N
b)	Area of steel to bear direct tension	3432 mm^2
c)	Effective depth	196.5 mm (provided effective depth is different)
d)	Area of steel required (at corners)	2721 mm^2
e)	Area of steel required (in the middle span)	1520 mm^2

On comparing the different values found as per the design executed in the program and in STAAD. Pro for each respective bunker of given capacity, it has been concluded that the design of the structure through the program developed is in proper agreement with the design of the structure using standard design software.

Validation through Stress Contours

For validating the results obtained through the program, the actualstressescomputed using the program,are entered in the structural design software STAAD. Pro v8i and the stress contours

corresponding to each capacity that is, 300KN, 800KN and 1200KN are obtained in the form of fig. 2 and fig. 3. Here the design is not actually performed using the design software instead just used for validation of the results obtained through the program developed. Two types of stress contours are studied, namely, plate stress contours and beam stress contours. Both these stress contours reveal that the design performed using the program developed is well within permissible limits in terms of stresses generated on the plates of the bunker as well as on the beams of the same, due to static loading.

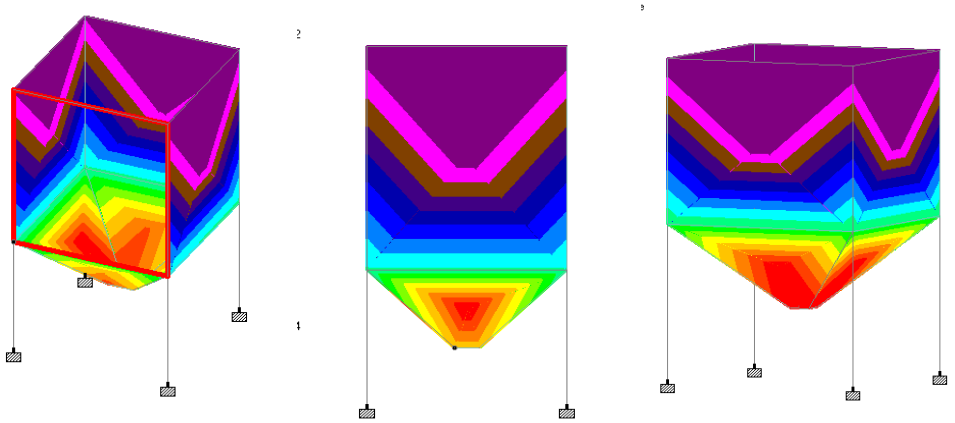


Fig. 2(a) Fig. 2(b) Fig. 2(c)

Fig. 2:Plate stress contours of bunkers with capacities (a)300KN,(b) 800KN,(c)1200KN

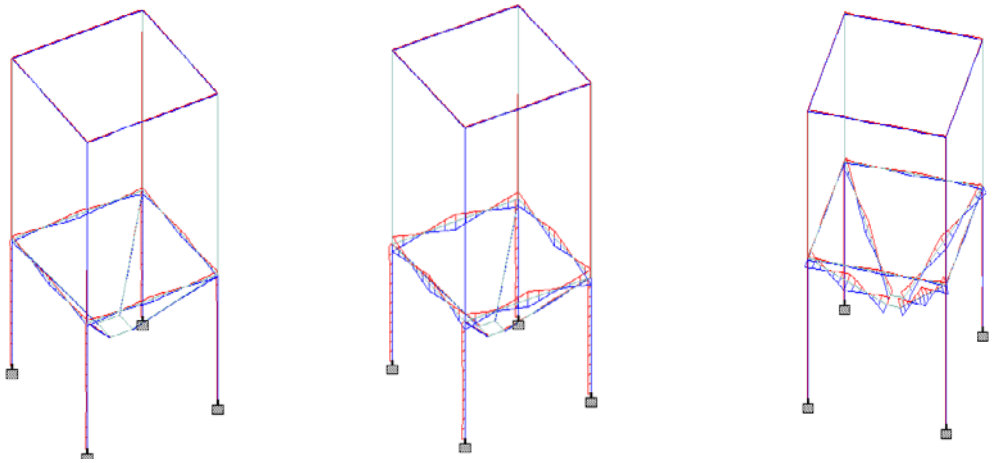


Fig. 3(a) Fig. 3(b) Fig. 3(c)

Fig. 3:Beam stress contours of bunkers with capacities (a) 300KN, (b) 800KN, (c)1200KN

Validation through Graphs

The aforesaid validation is just the validation for the hopper beams of the bunkers of capacities 300KN, 800KN, 1200KN. Graphs for hopper beams for the reinforced concrete bunker of respective capacities are generated in the design software STAAD. Pro. Since these graphs are generated for the design created by the programdeveloped, these also validate the same by showing that the values are well within proper limits. These graphs are represented in fig. 4. Here, the values of forces and stresses calculated using the program are applied virtually on the hopper beams in STAAD Pro. to obtain the graphs for each beam thereby verifying that all the loads and stresses acting are well within permissible limits.

The graphs for each capacity of bunker are shown for loads acting on any one hopper beam since the graphical pattern is same for all the four hopper beams for a particular capacity.

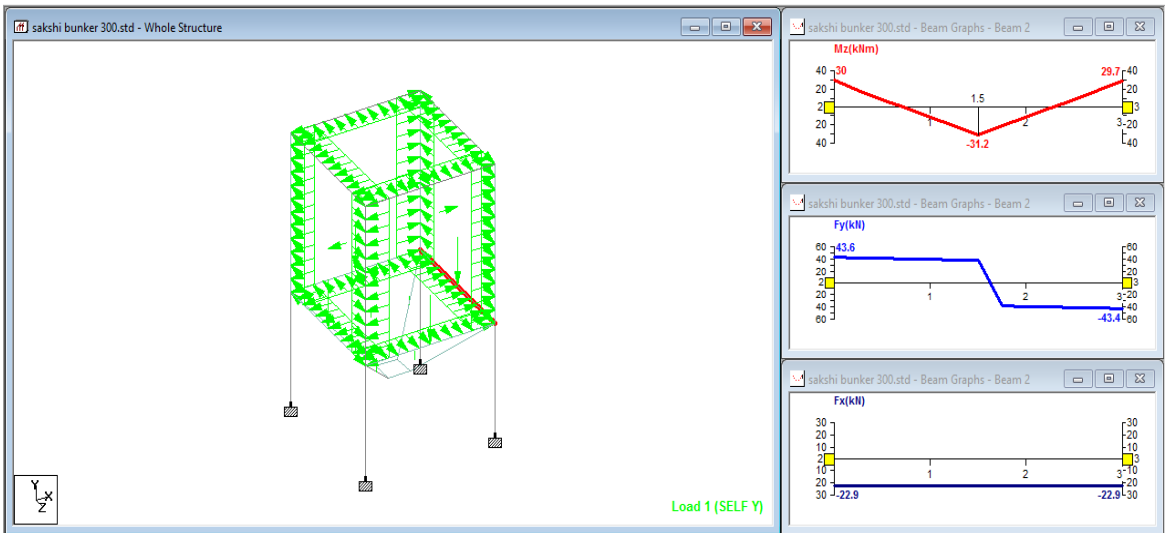


Fig. 4(a): Graphs showing the loads acting on a typical hopper beam of a bunker of capacity 300KN

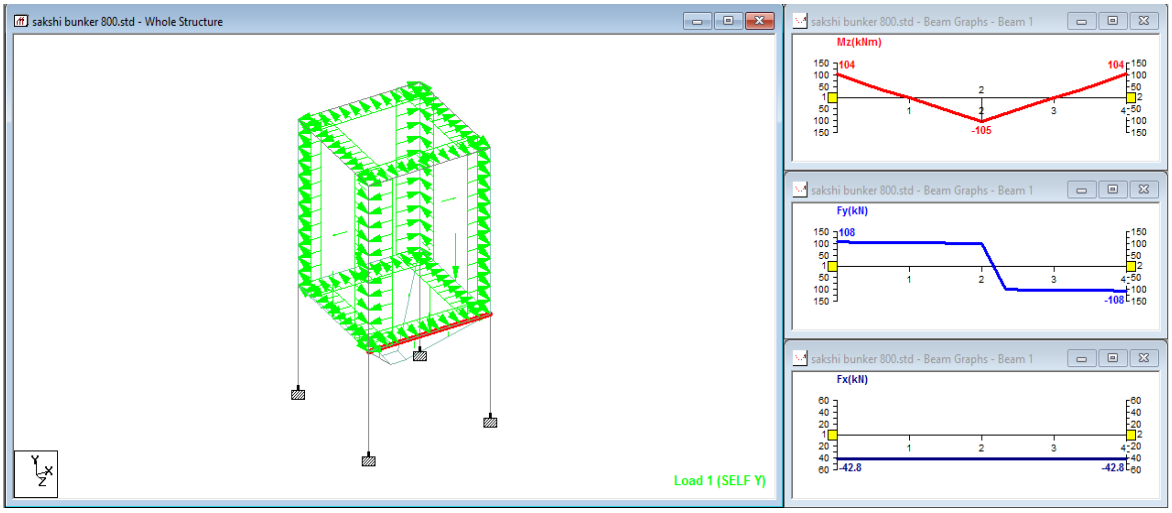


Fig. 4(b):Graphs showing the loads acting on a typical hopper beam of a bunker of capacity 800KN

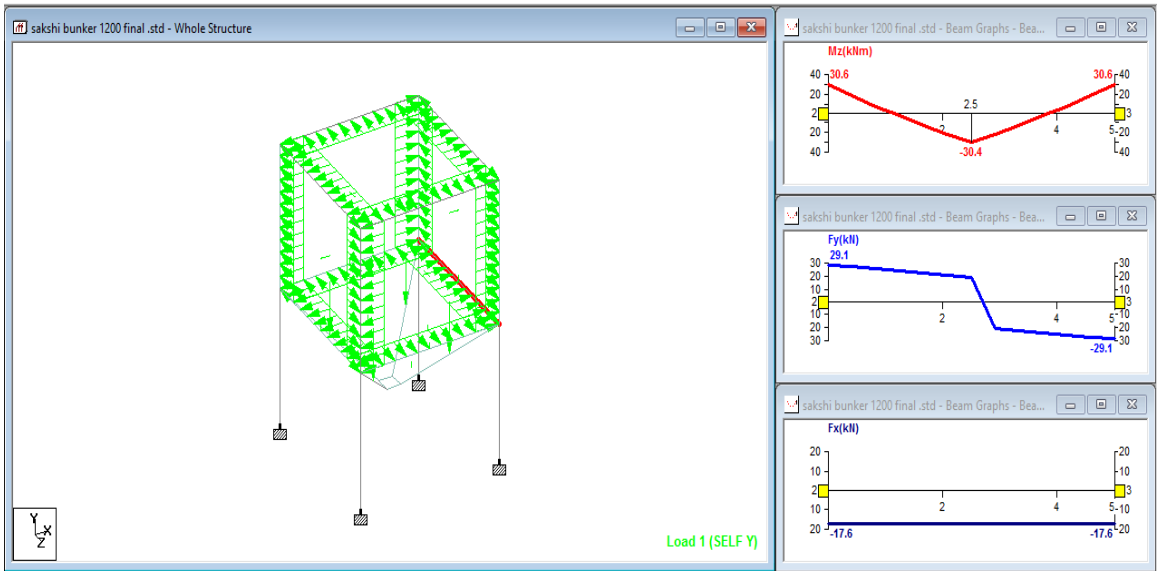


Fig. 4(c): Graphs showing the loads acting on a typical hopper beam of a bunker of capacity 1200KN

INFERENCE

The inference drawn by comparing the design data obtained using the program developed (Refer 3) and the design performed in design software STAAD Pro. (Refer 4.1) is thoroughly analyzed by comparing the values as per the different design considerations. On visual comparison of the values of different design aspects, it was found that the design values obtained by the program and by using STAAD. Pro.are almost same and hence both are in proper agreement.

CONCLUSION

In today's scenario, the importance of bulk storage facilities is well known; ranging from industries to warehouses. The research work conducted here helped in the development of a program for the analysis and design of reinforced concrete bunkers to be used as bulk storage facility in industries. From the study, it can be concluded that the program developed works brilliantly well for designing a square reinforced concrete bunker used for storing coal in thermal power plants. The program built herein is easy to work with and does not require any cumbersome technique or a high level skill set for its operation. Validation of the program is done by cross checking the results by carrying out the design in standard structural design software STAAD. Pro. v8i. The design values obtained through both the program and the software were compared and a very high level of similarity was found. The stress contours and graphs drawn for different capacities of bunker revealed that the designed structure was well within loading limits. Design of different capacities of bunker also revealed that the economy criterion was taken into consideration while developing the program. This program could prove really useful for the designers as it is easy to use, saves a lot of time, is accurate and enhances the economy criterion.

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HEALTH MONITORING OF AN EXISTING RC STRUCTURE CONSIDERING SEISMIC ANALYSIS USING ANALYSIS TOOLS STAAD

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ABSTRACT
Seismic retrofitting is mainly done to meet the seismic safety requirements. The planning of alterations to existing buildings differs from new planning through an important condition; the existing construction must be taken as the basis of all planning and building actions. India is one of the most earthquake prone countries in the world and has experienced several major or moderate earthquakes during the last 15 years.
About 50-60 % of the total area of the country is vulnerable to seismic activity of varying intensities. Many existing buildings do not meet the seismic strength requirement.
In this study we are performing health monitoring on existing old structure G+2, using rebound hammer to evaluate its present strength and analyze the structure using analysis tool staad.pro.
In this study we are performing time history analysis (el-centrino) with retrofitting over weak members.

KEYWORDSTime history, staad.pro, Non destructive test, rebound hammer, existing structure, retrofitting, analysis.

INTRODUCTION
Retrofitting of constructions susceptible to earthquakes is a problem of great political and social significance. Prevention of disasters due to earthquake has become more and more important in recent years. There has been much research on the topic of retrofitting of structures in modern years. Considerations has been focused on both building and bridge structures and with the extensive damage to older structures, owners are increasingly taking action to avert similar damage to existing structures in future earthquakes. Disaster avoidance includes the reduction of seismic risk through retrofitting present buildings in order to meet seismic safety necessities. Though, no such thing as fully earthquake proof structure can exist in real, proper retrofitting method can remarkably improve the seismic performance of a structure. Mostly column failures, which include shear failure and shear cracking, have been detected in a RC structure during the recent earthquakes.

This study shows comparative study of high-rise G+02 building R.C. frame considering seismic zone II with medium soil type Under the seismic effect (TIME HISTORY ELCENTRO) as per IS 1893(part I) -2016analysis. A comparison of analysis of results in terms of forces, moment, displacement and cost is presented in this study

Retrofitting of the Structure
The need for seismic retrofitting of an existing building can arise due to several reasons like: building not designed to code, subsequent updating of code and design practice, subsequent upgrading of seismic zone, deterioration of strength and aging, modification of existing structure, change in use of the building, etc. Seismic retrofit is primarily applied to achieve public safety, with various levels of structure and material survivability determined by economic considerations. In recent years, an increased urgency has been felt to strengthen the

deficient buildings, as part of active disaster mitigation, and to work out the modifications that may be made to an existing structure to improve the structural performance during an earthquake.

The need for retrofitting in existing building can arise due to any of the following reasons:

- Building not designed to code
- Subsequent updating of code and design practice
- Subsequent upgrading of seismic zone
- Deterioration of strength and aging
- Modification of existing structure
- Additional loads
- Change in use of the building, etc.

Rebound Hammer test is a Non-destructive testing method of concrete which provide a convenient and rapid indication of the compressive strength of the concrete. The rebound hammer is also called as Schmidt hammer that consist of a spring controlled mass that slides on a plunger within a tubular housing.
As per the Indian code IS: 13311(2)-1992, the rebound hammer determine the compressive strength of the concrete by relating the rebound index and the compressive strength with a principle that the hardness of concrete and rebound hammer reading can be correlated with compressive strength of concrete.

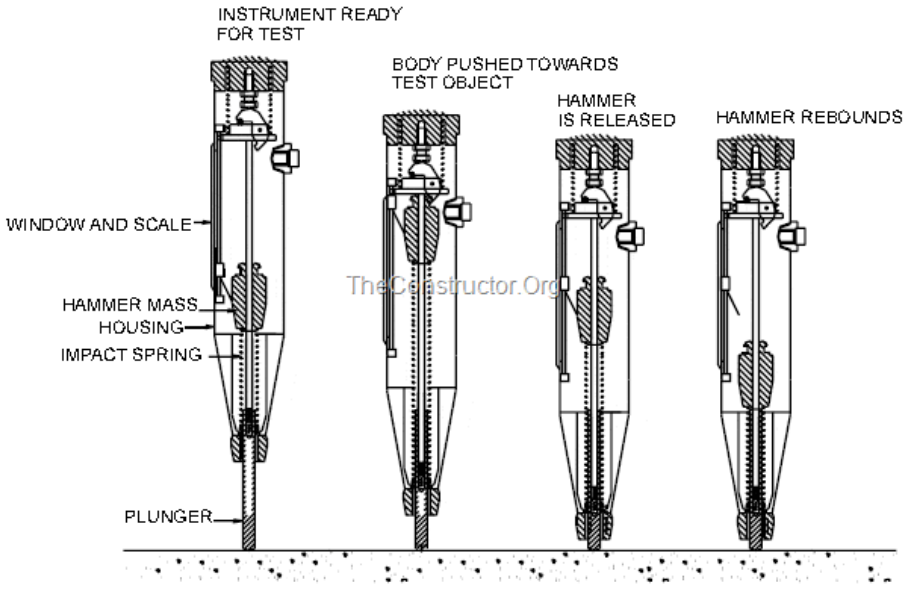


Fig 1: Rebound Hammer

LITERATURE REVIEW

Premalatha&Lakshmipriya (2018) An analytical study on seismic retrofitting of a reinforced concrete Beam-column joint was performed using FEM modeling . The main objective of this study is to increase the shear capacity and load carrying capacity of the structures using retrofitting techniques. In this study, the retrofitting was done by jacketing methods like carbon fibre reinforced polymer sheets (CFRP), Glass fibre reinforced polymer mesh, Sisalfibres along with crossed bars are carried out using the ANSYS Workbench. The wrapping of beam column joint was done by single, double, triple layer of CFRP, GFRP and Sisal fibres with different thickness. During the analysis one end of the column were fixed.

Cyclic loading was applied at the free end of the cantilever beam in Beam-column joint and Fixed load was applied at the top of the column. The load is applied up to the ultimate load to obtain the fatigue failure. This report discusses about the performance of the retrofitted beam column joint; and was compared with the conventional specimen.

Tsige and Zekaria (2018) analyzed a office medium rise building for earthquake force by considering three type of structural system. i.e. Bare Frame system, partially-infilled and fully- Infilled frame system. Effectiveness of masonry wall has been was studied with the help of five different models. Infills were modeled using the equivalent strut approach. Nonlinear static analyses for lateral loads were performed by using standard package ETABS, 2015 software. The comparison of these models for different earthquake response parameters like base shear vs roof displacement, Story displacement, Story shear and member forces are carried out, found that the seismic demand in the bare frame is considerably more when infill stiffness is not taken with larger displacements. It has been concluded that fully infilled frame is around 15% more compared to bare frame model; frame with 25% masonry wall decreased is nearly 10% more compared to the bare frame; frame with 50% of the masonry wall decreased is nearly 8% more compared to the bare frame and frame with 75% of the masonry wall decreased is about 5% more compared to the bare frame. This is because the bare frame models do not account the stiffness rendered by the infill panel, it gives significantly longer time period.

Tarek et al. (2017) have strengthened and tested the 3 beams with GFRP and two with CFRP composites using epoxy resins and results were compared with a control beam. 1 and 2 layers of 1 mm CFRP and 1, 2 and 4 numbers of layers of 1.3 mm GFRP were used for strengthening beams externally. All beams were reinforced with 3-10 mm diameter bars at tension zone and 1-6 mm diameter bar at compression zone and 2 legged 8 mm diameter stirrups at 100mm c/c. The equations developed are based on ACI code for moment capacity of strengthened beams, thickness of FRP is theoretically verified with experimental values. Two point loads with spacing 200 mm were applied for the test. All beams failed by concrete crushing at compression zone. It was determined that the flexural strength of strengthened beams using FRP laminates at tension zone, more than that of control beam.

METHODOLOGY ADOPTED

Analysis of building with given dimensions has been considered for the parametric analysis of critical load position as per superimposed loading standard which are analyses with the help of staad pro software. Proposed steps are as followings:

Step 1: Selection of the geometry of superstructure by using coordinate system in STAAD Pro or plot over the AUTO CAD, which can be import in Staad-Pro as per dimension of beams, c/c distance of columns, expansion to expansion distance and no. of diaphragm etc. Schematicsketch of the superstructure.

Step 2: building as per existing strength is modelled and one model with retrofitting is prepared of same dimension and same loadings as per Indian standards. finite element modeling of the model considering the above parameters.

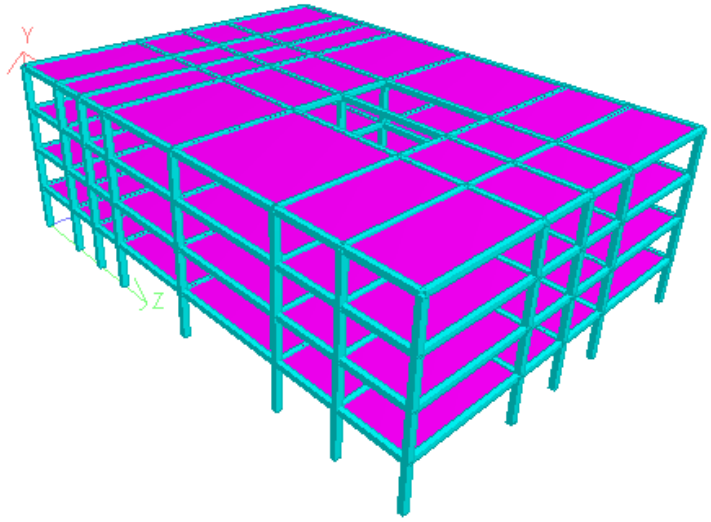


Fig 2: Existing Structure

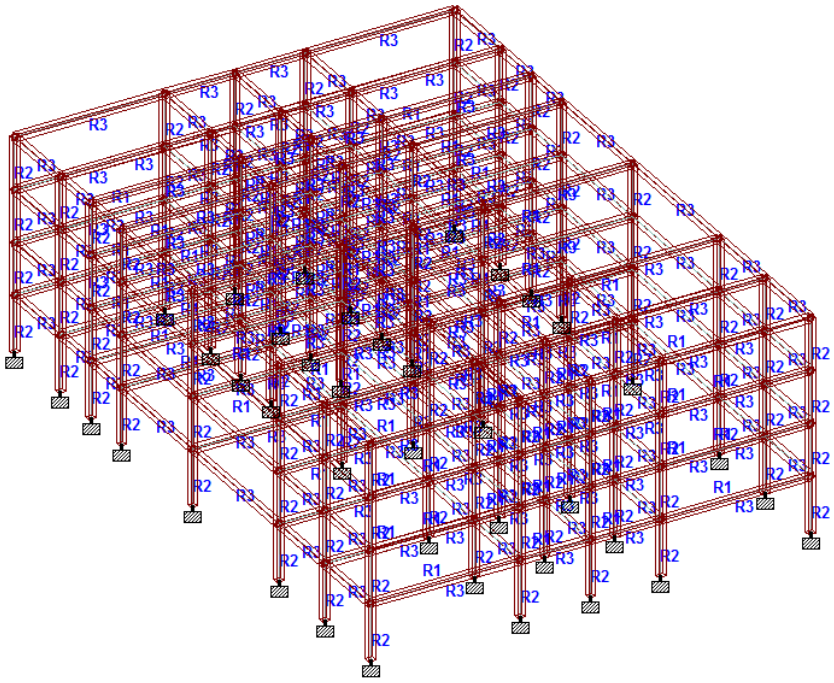


Fig 3: Building after Retrofitting (Steel Casing)

Step 3: Computation of existing culvert strength by N.D.T. rebound hammer method to determine its present condition.

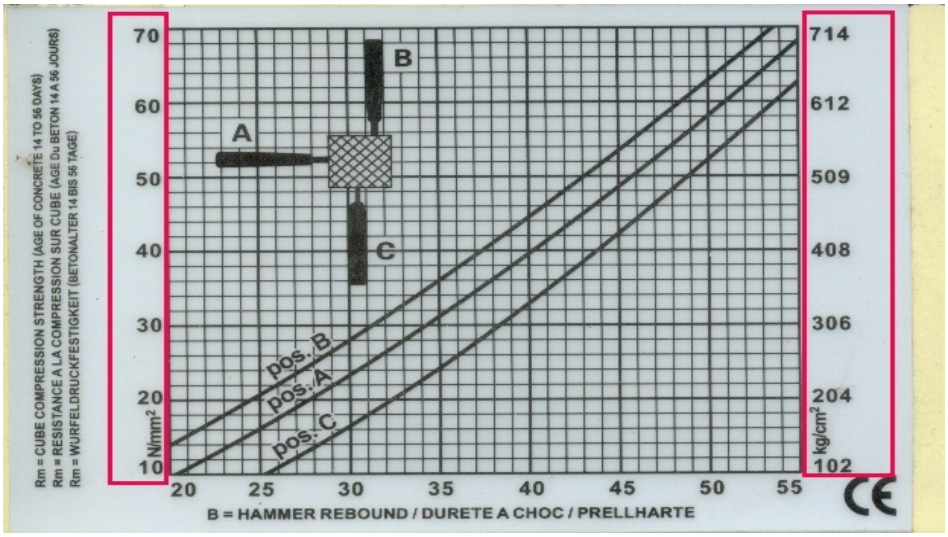


Fig 4: NDT Strength Graph

Step 5: After apply the support condition, now the next step to be considered for the Dead Load of the superstructure i.e. “self weight”.
Step 6: After apply the Dead Load, now the next step to be considered for the Superimposed load.
Step-7Selection of Seismic zones (Zone II) and medium type soil as per IS- 1893(part I) - 2016.
Step-8 load combination as per 875-part-V
Step-9 Analysis of building frames considering Time history Analysis (ELCENTRO CASE) method for seismic forces in X & Z direction and gravity load as shown in figure below.

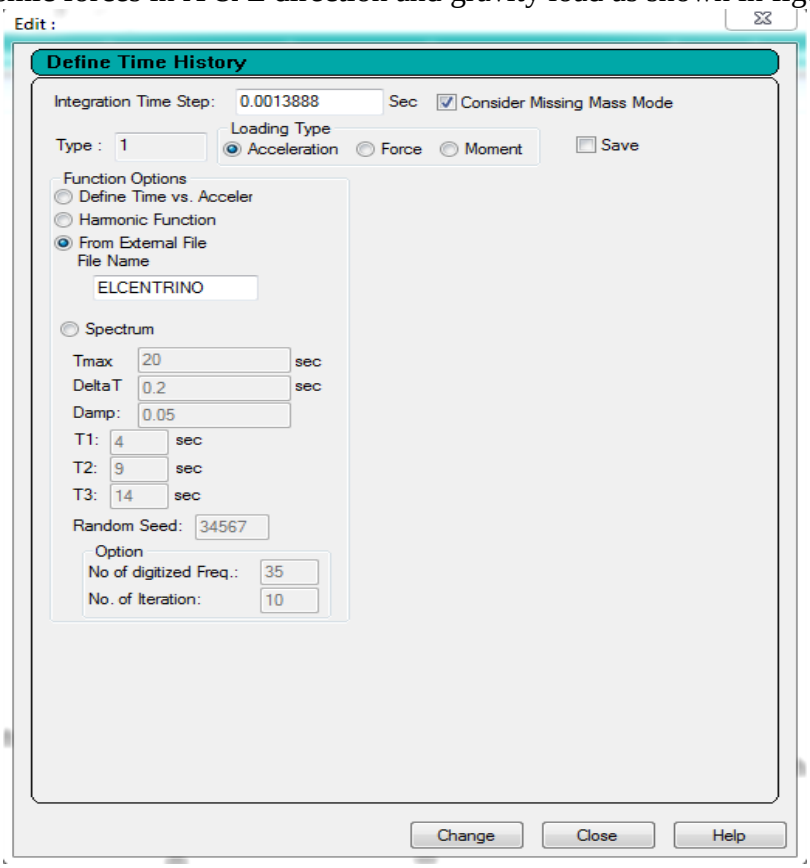


Fig 5: Assigning El-Centro (time history)

Step-10 Designing structures as per I.S.456:2000 to determine the amount of reinforcement required in both the cases.
Step-11 Cost analysis of material quantity i.e. concrete in cubic meter and reinforcement in kilo newton using S.O.R. M.P.P.W.D. 2014.
Step 12: After applied all the boundary condition and forces, now the model has to be "Analyze" for getting the results i.e. Axial force, shear force, deflection and support reactions etc.

Table 1: Description of existing structure

S. No.	Description	Value
1	Area of building	
2	Length	24 m
3	Breadth	17 m
4	Storey height	3.5 m
5	Height of the column below plinth level	1.5 m
6	Size of the column	300 mm x 300 mm
7 (a)	Size of beam for 5m span	200 mm x 500 mm
7 (b)	Size of beam for 4m span	200 mm x 500 mm
8	Thickness of slab	150 mm
9	Thickness of outer walls	200 mm
10	Thickness of inner walls	100 mm
11	Support condition	fixed

OBJECTIVES

These following are the primary objective of our research work:

- To determine the strength of an existing old structure using NDT (rebound hammer).
- To determine the effect of composite member on an old structure strengthening.
- To perform Non-linear (Time history analysis) over an strengthened old structure.
- To determine the cost of retrofitting as per SOR (CPWD).

ANALYSIS RESULTS

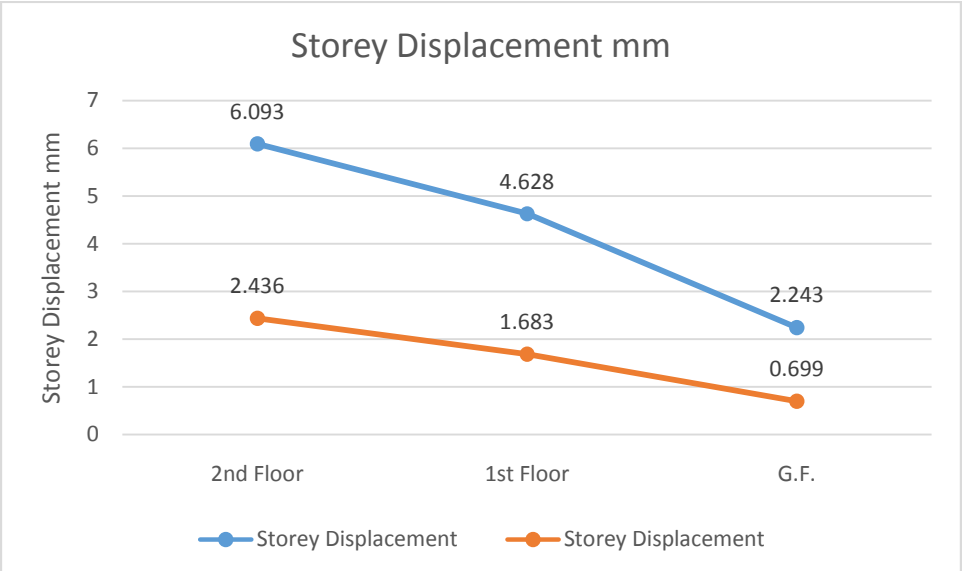


Fig 6: Displacement (mm)

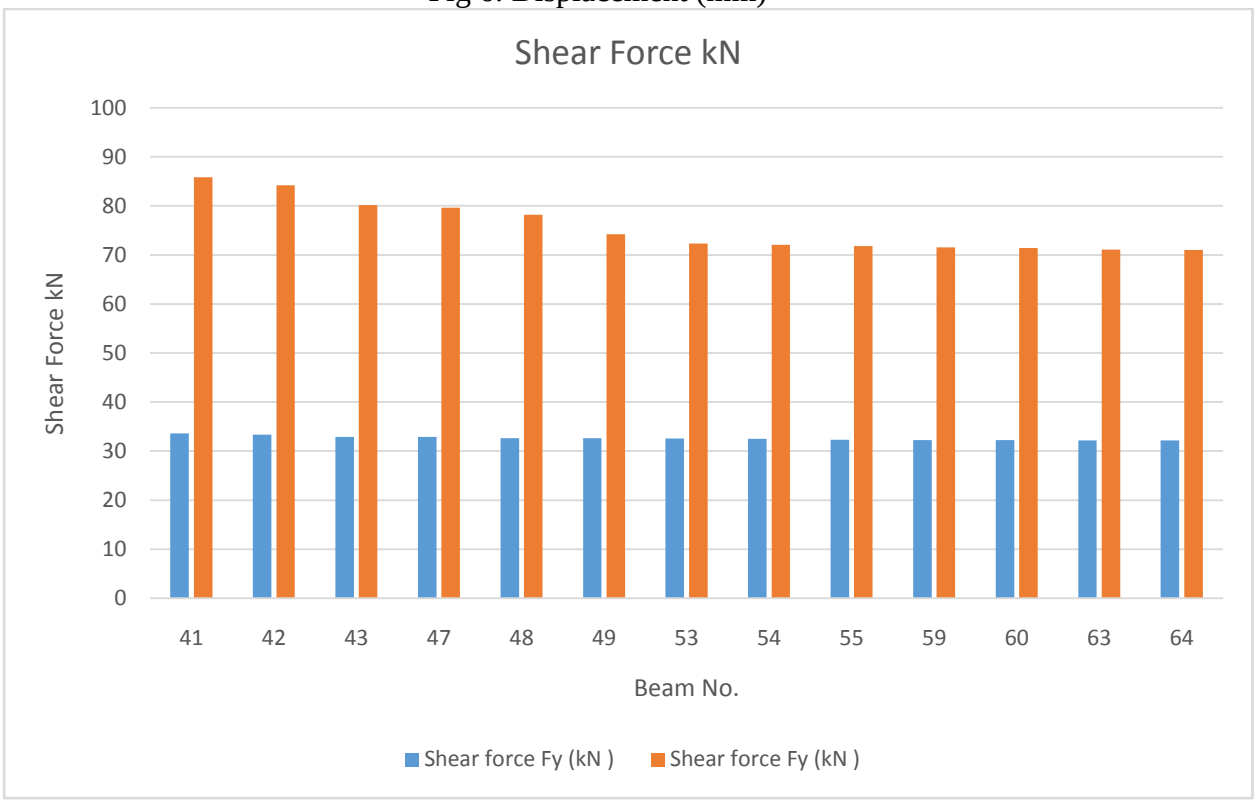


Fig 7: Shear Force KN

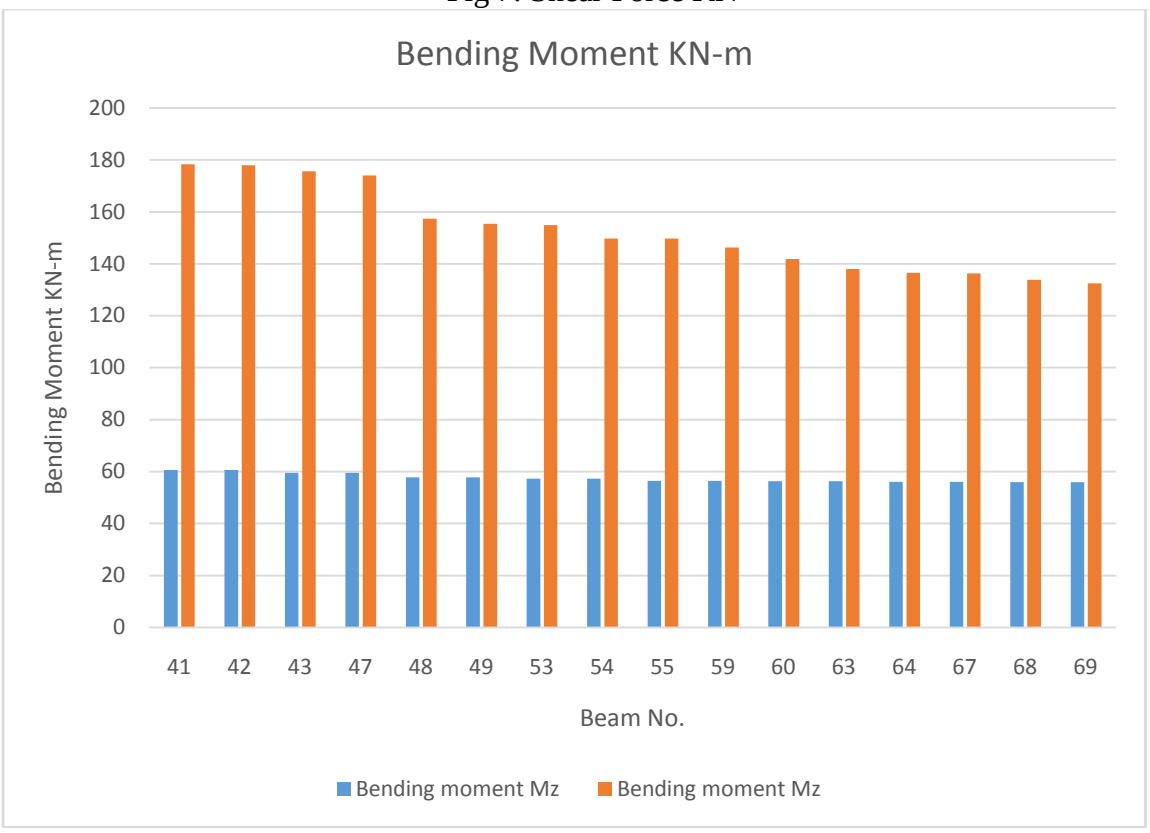


Fig 8: Bending Moment kN-m

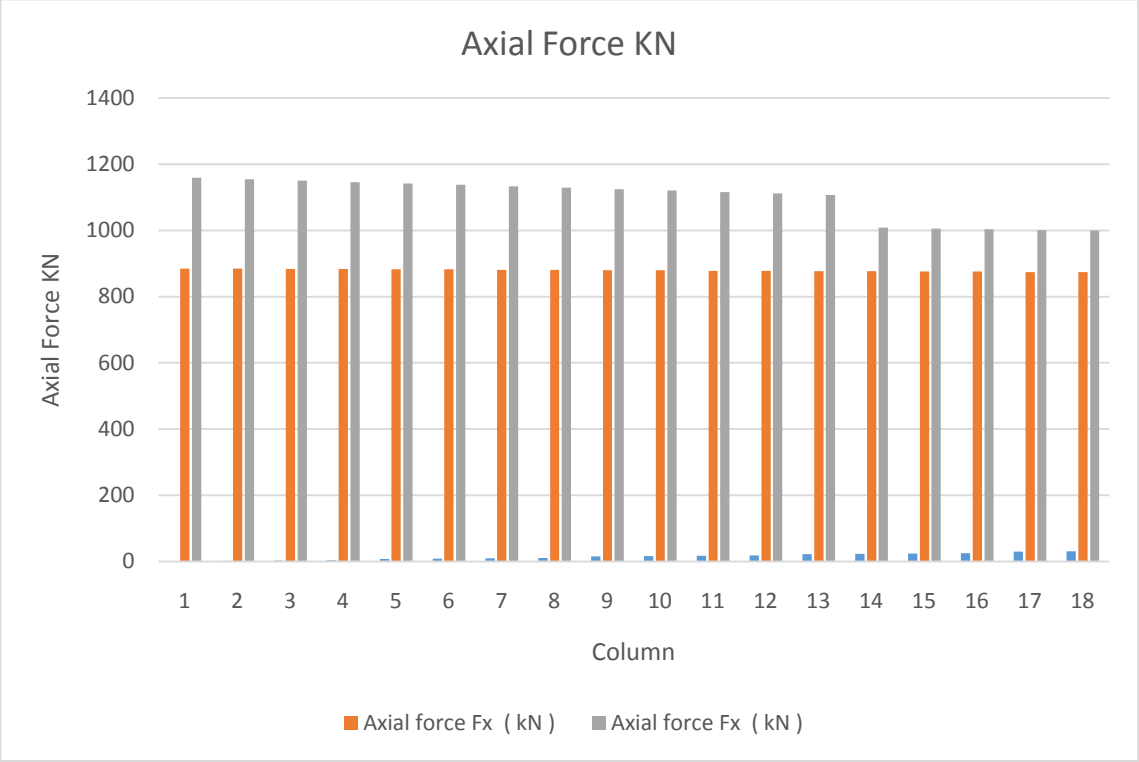


Fig 9: Axial Force kN

Table 2: Cost Analysis

Material	S.O.R. Rate	Quantity	Total Rate
Steel Casing	68 / kg	4154.91	2,82,533.608/-
Concrete	5091 / cu. M.	30.08	1,53,137.28/-

CONCLUSION

Following are the ends according to the examination

- In this investigation, it is seen that with the procedure of retrofitting, the soundness of a structure can be recovered without disassembling the structure utilizing fortifying steady individuals.
- It is seen that the retrofitting method can be 65% cost effective than destroying and developing another structure.
- It can be reasoned that product examination and site test work can be joined for the advancement of the framework, As we did in this investigation where we decided the quality of the structure utilizing NDT (Non-destructive testing) though displaying and checking quality improvement should be possible utilizing investigation apparatus staad.pro.

Maximum bending moment

It is seen in section 5 that with retrofitting method bending moment in bar and segments are diminished by 23.5%.

Maximum shear force.

It is seen in chapter 5 that with retrofitting method shear force in bar and segments are diminished by 34.6%.

Maximum axial force

An axial force constrain is any power that explicitly follows up on the centre turn of an inquiry. These powers are regularly expanding power or weight compel, dependent upon heading. Additionally, when the power stack is even over the edge's geometric centre, it is

concentric, and when it is uneven, it is strange. Here outcomes demonstrate that pivotal powers are dispersing equitably.

Maximum Storey displacement.

It is observed that after retrofitting of the sectional members with steel casing, storeydisplacemet has been reduced to 42.3 % which is in its permissible limit. Hence structure is now stable and stiff to bear and distribute load.

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COMPARATIVE STUDY OF OMRF AND SMRF

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ABSTRACT
It is well known that masonry infills, although non-engineered and considered as non-structural, may provide most of the earthquake resistance and prevent collapse of relatively flexible and weak RC structures. In this paper the effect of masonry infills on the performance of reinforced concrete frames subjected to earthquake ground motions is explained. To this purpose an equivalent discrete shear-type model with and without infills was used for the evaluation of elastic and inelastic response of multi-story frame structures. The masonry-infilled modeled by means of equivalent strut elements, which can only carry compressive loads, characterized by an idealized degrading hysteretic behavior.

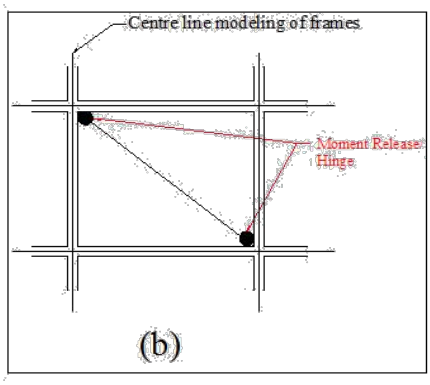
KEYWORDS: Bare frame, Infill Walls, Seismic analysis,Strut frame

INTRODUCTION
.In this paper the infill panels were modeled by equivalent diagonal struts, which carry loads only in compression are discussed. The diagonal strut model, representing the infill panels were therefore implemented in an equivalent discrete shear-type model, that allowed to perform a large number of dynamic analysis on a set of reinforced concrete two-bay bare and infilled frames subjected to earthquake, in order to establish response spectra of the most significant parameters characterizing the behavior of multi-degree-of-freedom systems.

LITERATURE REVIEW
M. Leipold and J. Schwarz [1]in their paper they presented modeling techniques forR/C-frame systems with infills. In their work, selected modeling techniques for masonry infills and reinforced concrete frame members was discussed.
Goutam Mondal and Sudhir K. Jain[2]In their study they proposed a reduction factorfor effective width of diagonal strut over that of the solid reinforced concrete (RC) infilled frame to calculate its initial lateral stiffness when a central window opening is present.
T.C. Liaum [3]In his study a structural system against earthquake as represented byvarious forms of infilled frames. Theoretical study in relation with various practical parameters.

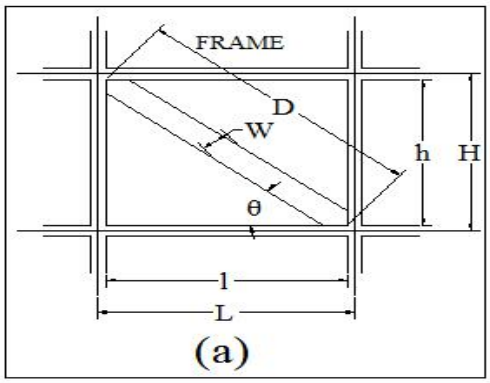
.

Equivalent Diagonal strut Method
In conventional analysis of infilled frame systems, the masonry infill is modelled using equivalent strut model in Figure. The former is simple and computationally attractive but is theoretically weak. First, identifying the equivalent nonlinear stiffness of the infill masonry using diagonal struts is not straightforward, especially when there exist some openings, such as doors or windows, in the wall. Furthermore, it is also not possible to predicate the damaged area of masonry either. The diagonal strut method based on continuum model can provide an accurate computational representation of both material and geometry aspects, if the properties and the sources of nonlinearity of the masonry carefully defined (Hao, Ma and Lu, 2002).
The strut area A_e was given by the following expression:



$A_e = W\ t$ where., $W = 0.175\ (\lambda H)^{-0.4}\ D$

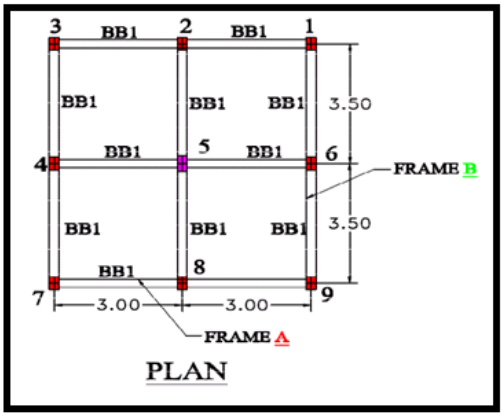
$$\lambda = \sqrt[4]{\frac{E_i\ t\ \sin(2\theta)}{4\ E_f\ I_c\ h}}$$

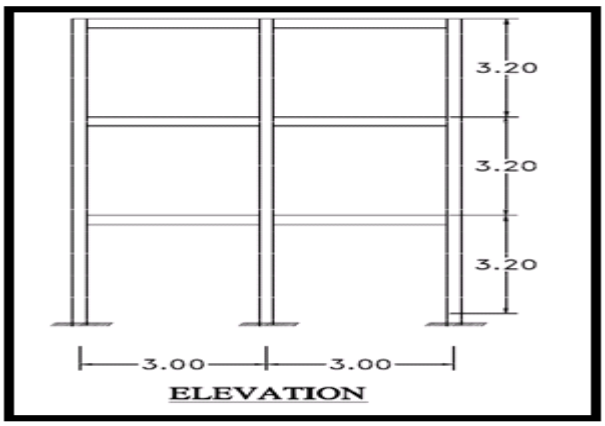


Where,

E_i = the modules of elasticity of the infill material, N/mm^2
E_f = the modules of elasticity of the frame material, N/mm^2
I_c = the moment of inertia of column, mm^4
t = the thickness of infill, mm
h = the height of infill
L =the centre line width of frames
l = the width of infill
D = the diagonal length of infill panel
θ = the slope of infill diagonal to the horizontal

4 Structural Details

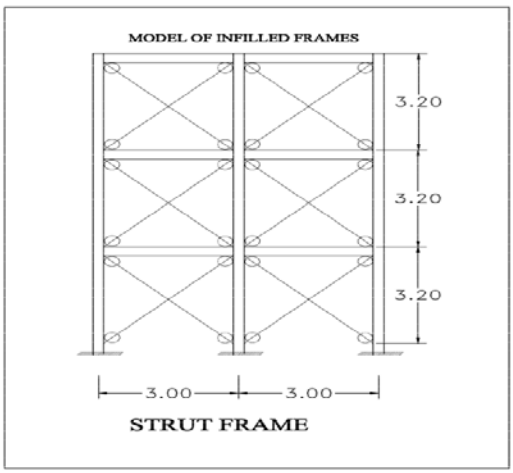


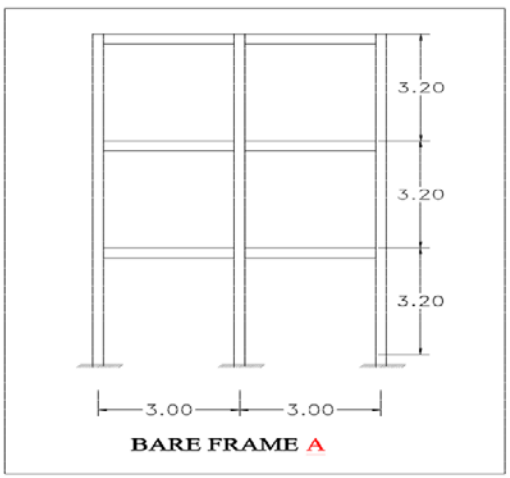


Concrete: Characteristic compressive strength (f_{ck}) = 20 MPa
Poissons Ratio = 0.3
Density = 25 kN/m^3
Modulus of Elasticity (E) = 5000 x $\sqrt{}$ f_{ck} = 22360.67 MPa
Size of Column 300×500mm
Size of Beam B1= 230×450
Equivalent Statics Method Seismic Analysis

Steel : Fe 415 grade of steel = 415 MPa
Modulus of Elasticity (E) = 2×10^5 MPa

Masonry infill Clay burnt brick, Class A, confined unreinforced masonry
Compressive strength of brick, fm = 10 MPa
Modulus of Elasticity of masonry (E_i) = 550 × fm = 5500MPa
Poissons ratio = 0.15
Dead load = 3.5 kN/m^2 Floor finish = 1.5 kN/m^2
Live load = 3 kN/m^2 Live load at terrace = 5 kN/m^2





Result of bare frame column no.1

No.	Height of Building	Deflection in mm	Shear in KN	Moment KNm	Ast mm^2
1	3.2	10.5	22.56	15.71	1489
2	6.4	20.77	19.53	14.72	1112

Result of bare frame column no.5

No.	Height of Building	Deflection in mm	Shear in KN	Moment KNm	Ast mm^2
1	3.2	10.88	29.78	50.84	1493
2	6.4	21.67	26.64	44.34	1199

Result of bare frame column no.6

No.	Height of Building	Deflection in mm	Shear in KN	Moment KNm	Ast mm^2
1	3.2	10.87	23.25	39.78	1136
2	6.4	21.66	21.93	37.12	927

Result of strut frame column no.1

No.	Height of Building	Deflection in mm	Shear in KN	Moment KNm	Ast mm^2
1	3.2	0.75	2.00	6.54	296
2	6.4	1.02	6.89	10.62	185

Result of strut frame column no.5

No.	Height of Building	Deflection in mm	Shear in KN	Moment KNm	Ast mm^2
1	3.2	0.73	0.167	0.67	452
2	6.4	1.29	0.666	1.3	272

Result of strut frame column no.6

No.	Height of Building	Deflection in mm	Shear in KN	Moment KNm	Ast mm^2
1	3.2	0.912	2.6	8.33	365
2	6.4	1.34	8.84	13.69	227

CONCLUSIONS

1. Deflection in case of bare frame is very large, in case of strut frame.
2. There is drastic decrease in case of strut model shear value & Bending Moment in columns.
3. Due to reduction of shear force and bending moment there is drastic reduction in requirement of steel in strut model.
4. Generally for normal building where room dimension are small and walls are provided, for up to G+2 structures DL + LL loads are calculated on columns, these axial loads are increased by certain % depending upon position of column (from 25% to 60%). For this equivalent axis loads column are designed. This empirical method is proved to be correct by considering the effect of infill walls.
5. The impact of the infills on the seismic behaviour of buildings may be positive or negative.

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COMPARATIVE STUDY OF WAREHOUSE STRUCTURE IN PRE ENGINEERED BUILDING WITH CONVENTIONAL STEEL BUILDING

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ABSTRACT
In present years, the concept of Pre-Engineered Building in creation format of structures has helped into optimizing a design. Steel industry sector is developing more hastily in almost throughout elements of the world. In a hazard of global warming the usage of steel structures is not only efficient but also sustainable at the time. In Pre- Engineered Buildings (PEB) steel structures (Pre-fabricated) time is being the maximum significant aspect is built in very short duration period. In any type of Industrial structures and Pre-Engineered Buildings (PEB) Maximum Span, Column free suitable structures are the most vital in fulfills this requirement at the aspect of decreased time and price in comparison to conventional structures. Here this study is achieved by planning 3D frame of an Industrial Warehouse constructing the use of the thoughts and studying the frames using appropriate assessment and design software after due validation. In this research, an industrial structure warehouse is analyzed and designed according to the Indian standards (IS 800-2007) and additionally through referring American Standard (AISC LRFD). The various loads like dead, live, wind, seismic and snow loads according as per IS codes are considered for the present work for relative study of Pre-Engineered Buildings (PEB) and Conventional Steel Building (CSB). To compare the consequences of the numerous parametric study to perform the variations in terms of shear force, support reaction, weight correlation and cost evaluation.

KEYWORDS: Pre-Engineered Building, Conventional Steel Building, Ware House, Tapered Section, STAAD Pro V8i

INTRODUCTION
In India the industry has fast development of steel building makes for exciting analyzing and it can have commenced all with the on location fabrication model. With the opening up of the economy during the 1990s but, such things started to change. In the might of the PEB enterprise this became the cause for predominantly “manufacturing plant constructed” structures and to achieve economy growth. In PEB technology steel is one of the world’s most reused material and toward the finish of any steel product’s life, practically 100% of it tends to be reused without losing its engineering properties, preferred and ecological construction methods. The tropical nation like in India to give great protection impact and would be highly suitable for a PEB structures. PEBs is an ideal construction sector for in remote & hilly areas. A recently survey through the Metal Building Associations (MBMA) shows that nearby 50% to 60% of the non-residential near to the ground rises constructing in USA are pre-engineered buildings. This flexibility would seem to with ease provide itself to optimization of member cross-section shapes. In Industrial building systems, the partitions can be formed of steel columns with cladding which may be of profiled or plain sheets, Galvanized Iron sheets, roofing, precast concrete, or masonry work. The wall must be effectively in strong to resist the lateral pressure because of wind or seismic activity resistance. As the PEBs offers fastest technique to construct the warehouses structures as compared to conventional methods and its demand for a broad range activities of construction will growth over the afterward four years.

METHODOLOGY

In the research work, by using STAAD Pro V8i structural software three dissimilar types of 3D steel buildings are designed for static and vibrant forces. In this work, an industrial warehouse structure of length 60m with bay spacing at 6m along the length, 24m width and 8.402m eave height in which 2.1 m from ground level is used for brick work and remaining 6.3 m is used for cladding. The slope of roof is taken as 9.91° for both Pre-Engineered Steel Building and Conventional steel buildings and roofs are protected with GI sheet. The spacing of the purlins is maintained as 1.5m and girts is maintained as 2.1m. Pinned type of support condition can be used. The building layout plan of the proposed industrial warehouse structure considered for the study is as shown below in figure:

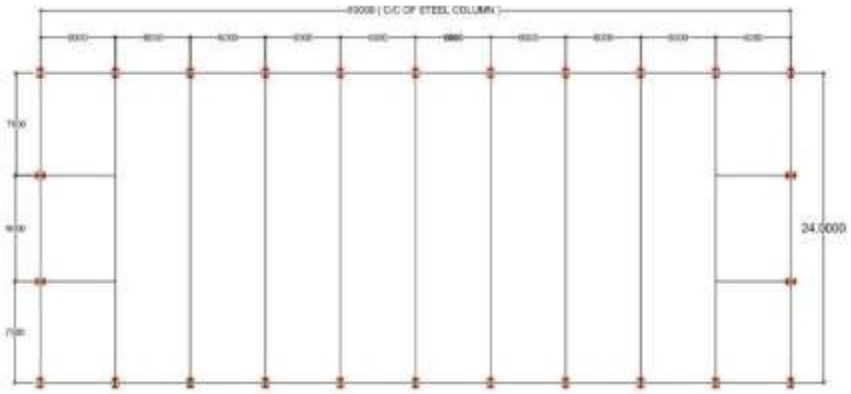


Fig 1. Plan Layout of Warehouse Structure

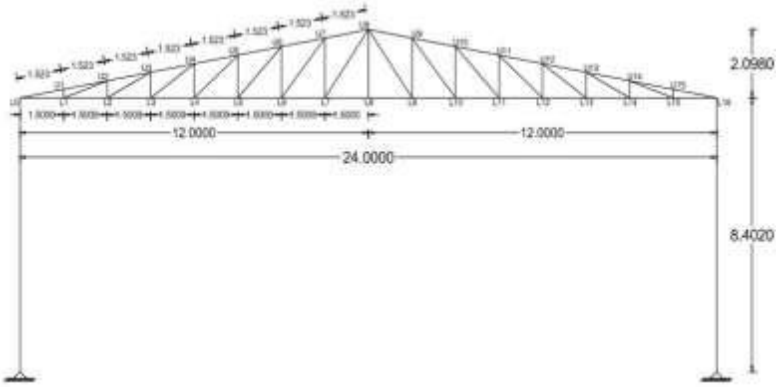


Fig. 2: Elevation of Warehouse Structure

LOADINGCALCULATIONS

Dead load calculation (IS 875-1987 Part-I):

Total load on purlin:

Weightofsheet =0.058kN/m^2 Weightoffixing = 0.025 kN/m^2

Weightofservices= 0.1 kN/m^2 Spacing of Purlin = 1.523m

Total weight on Purlin = 0.183 kN/m^2

Total weight on Purlin = weight (kN/m^2) × spacing of Purlin= 0.183 × 1.523= 0.278 kN/m

Assume weight of purlin=0.10 kN/m

Weight of truss=(Span/3+5)10= (24/3+5) ×10= 0.130 kN/m^2

= 0.103 × plan length= 0.103 ×1.5= 0.154 kN/m

Total dead load = 0.532kN/m

Live Load

= 750 – 20(9.91 – 10)= 0.751kN/m^2= 0.751 ×1.523= 1.143 kN/m

= 2/3 ×1.143= 0.762kN/m

Wind load calculation (IS 875-1987 Part-III):

Basic wind speed = Vb = 39 m/s

Designwindspeed = Vz = k_1 k_2 k_3V_b

Designwindpressur = Pz = 0.6 Vz^2 Wind pressure on roof = (C_{pe} - C_{pi})

Where,C_{pe} = Coefficient of external windpressure C_{pi} = Coefficient of internal wind pressure

$k_{1=}$ Riskcoefficient

$k_{2=}$ Terrain height and structure sizefactor

$k_{3=}$ Topographyfactor

For all general building and structure, Mean probable design life = 50 years Risk coefficient k_1 = 1.0

Terrain category= 3 (As height of building 10m)

ClassB (As horizontal or vertical dimension in between 20 to 50 m)

k_2 = 0.99

k_3 = topography factor k_3=1

V_z= 1.0 × 0.99 × 1.0 × 39= 38.61 m/s

Design wind pressure = Pz = 0.6 × Vz^2 = 0.6 × 38.61^2 = 894 N/m^2

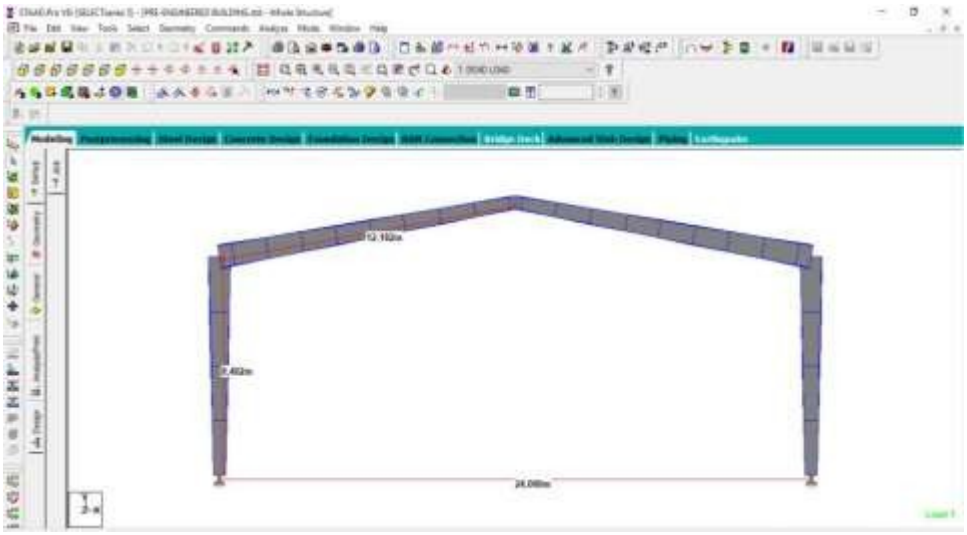


Fig 3: Assembled Tapered Section

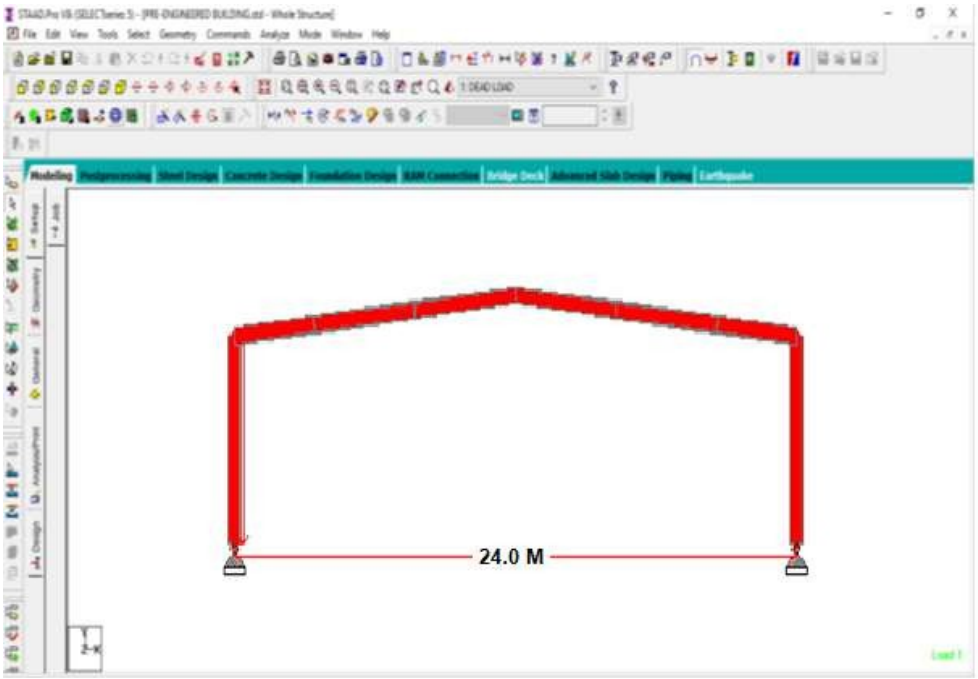


Fig 4: Assembled I-Section OF CSB Section

RESULTS AND PERFORMED ANALYSIS

The structural analysis and design of the structural constructing taken into consideration changed into executed using the Staad Pro software database which may be very user pleasant and powerful. Graphical representation shows results obtained from software as below:

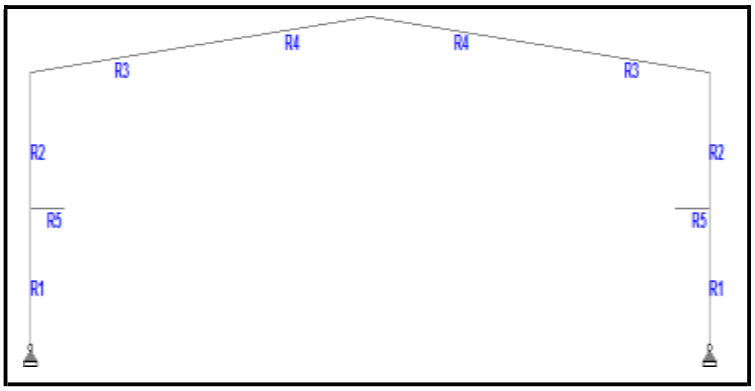


Fig 5 : Typical Section OF PEB-IS 800:2007 Frame

Table 1 : Sectional Details OF PEB-IS 800:2007 Frame

Profile	Length (m)	Weight (t)
Tapered Member No: 1	12	5.006
Tapered Member No: 2	37.68	26.925
Tapered Member No: 3	37.68	23.763
Tapered Member	12	6.616

No: 4		
ST ISMB300	7.5	3.243
ISA 75X75X6	502.65	8.150604
ISA 50X50X6	501.143	22.04178
ST 200Z60X2	3680	20.16215
		Total=115.9015

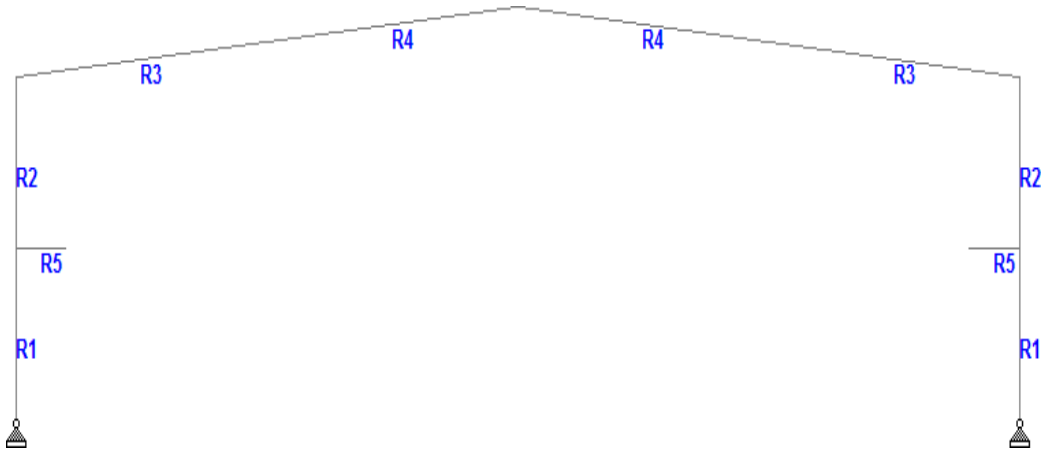


Fig 6: Typical Section OF PEB- AISC/LRFD Frame

Table 2 : Sectional Details of PEB-IS 800:2007

Profile	Length (m)	Weight (t)
Tapered Member No: 1	12	5.006
Tapered Member No: 2	28.68	20.49387
Tapered Member No: 3	28.68	18.08712
Tapered Member No: 4	12	6.616
ST ISMB300	7.5	3.243
ISA 75X75X6	506.851	8.218725
ISA 50X50X6	506.843	22.29248
	Total=	104.3854

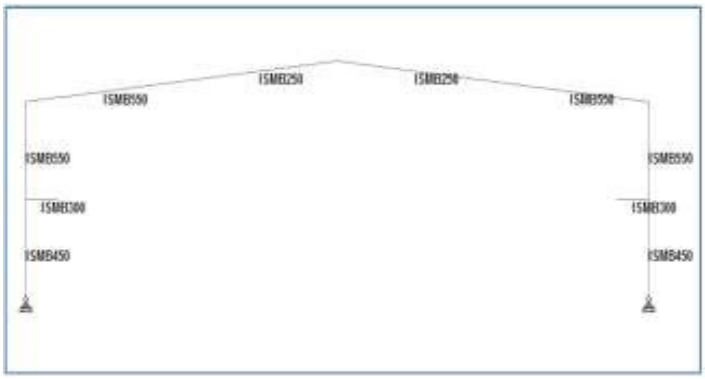


Fig. 10 : Typical Section of CSB IS 800:2007

Table 4 : Sectional Details of CSB-IS 800:2007 Frame

Profile	Length (m)	Weight (t)
ST ISMB250	48.1	17.554968
ST ISMB300	10	4.324
ST ISMB450	24	17.016
ST ISMB550	74.02	75.124717
ISA 75X75X6	583.03	38.78454
ISA 50X50X6	587.02	25.67551
ST 200Z60X2	6140	33.640115
	Total=	212.1185

Table 5 : COST ANALYSIS BETWEEN PEB AND CSB

Types of frame	Weight of frame in kN	Purlin weight in kN	Grit member weight in kN	Total weight in kN	Total weight in kg	Rate of steel in kg	Total cost
CSB FRAME IS 800:2007	1394.289	333.796	352.118	2080.2	2,12,118.30	55.00	1,16,66,506
PEB FRAME IS 800:2007	935.619	144.19	56.807	1136.62	1,15,900.73	55.00	63,74,540.3
PEB FRAME AISC LRFD	879.151	105.09	39.439	1023.68	1,04,384.65	55.00	57,41,155.7

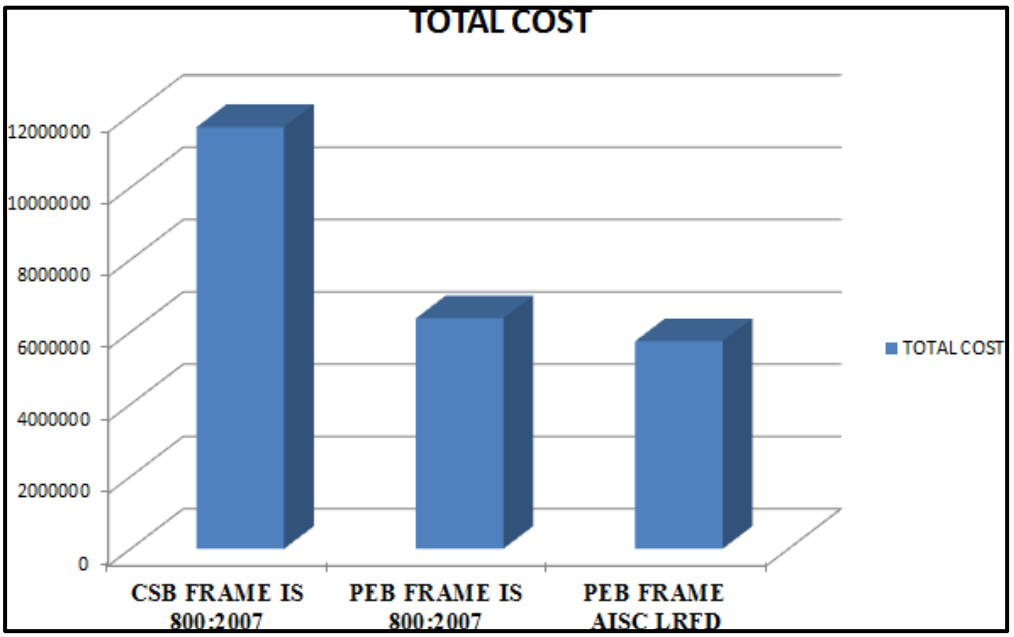


Fig. 7: Cost Analysis

In this work it is noted that the Cost Analysis of PEB as compared with Indian code structure by 45.36% and Cost Analysis of PEB as compared with American code structure by 50.78% lesser than the Cost Analysis of CSB structure.

CONCLUSIONS

1. In this work we observe of self-weight of the models indicated that the self-weight for PEB is much less than that of CSB in the equivalent geometry.
1. It is noted that the weight of PEB as compared with Indian code structure by 33% and weight of PEB as compared with American code structure by 37% lesser than the weight of CSBstructure.
2. It is noted that the Purlin weight of PEB as compared with Indian code structure by 56.80% and Purlin weight of PEB as compared with American code structure by 68.51% lesser than the weight of CSBstructure.
3. It is noted that the Grit Member weight of PEB as compared with Indian code structure by 83.86 % and Grit Member weight of PEB as compared with American code structure by 88.79% lesser than the Grit Member weight of CSBstructure.
4. It is noted that the Maximum Shear Force of PEB as compared with Indian code structure by 49.27% and Maximum Shear Force of PEB as compared with American code structure by 62.53% as compared to lesser than the Maximum Shear Force of CSBstructure.
5. It is noted that the Maximum Support Reaction of PEB as compared with Indian code structure by 17.33% and Maximum Support Reaction of PEB as compared with American code structure by 37.18 % lesser than the Maximum Support Reaction of CSBstructure.
6. It is noted that the Cost Analysis of PEB as compared with Indian code structure by 45.36% and Cost Analysis of PEB as compared with American code structure by 50.78% lesser than the Cost Analysis of CSBstructure.

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AMELIORATIVE EFFECT OF MIXED BACTERIAL CULTURE OF SOIL FROM COASTAL AREAS AND HOT SPRING IN CONCRETE

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ABSTRACT

Construction materials are deteriorating day by day, which needs to be checked soon. Recent studies have been reported on increment in strength and durability of bacteria isolated from hotsprings incorporated concrete. The coastal areas serve as rich source for bacteria capable of silica leaching. Soil bacteria of hotsprings and coastal areas generally possess urease gene which is capable of Calcite precipitation. Microbiologically induced calcite precipitation(MICP) helps in filling up the voids in the matrices of cementitious materials and thus helps in gaining high mechanical strength. Our aim is to compare the increment in the mechanical strength by the incorporation of mixed bacterial culture of soil from coastal areas and hotspring in cementitious materials.Concrete is very good material to resist the compressive load to a limit but if the load applied on the concrete is more than their limitof resisting load,it causes the strength reduction of concrete by producing the cracks in the concrete and the treatment of the cracks in very expensive.Some of the property like durability, permeability ad strength of the concrete structure is also decreases. Due to increase in the permeability of the concrete the water easily pass through the concrete and come in the contact with the reinforcement of the concrete structure and after some time corrosionstart due to this strength of the concrete structurewill decreasesso it will be necessary to repair the cracks [1].By introduce the bacteria in concrete it producing calcium carbonate crystals which block the micro cracks and pores in the concrete [2]. In concrete micro cracks are always avoided but to some extent they are responsible to their failure in strength.The selection of the bacteria is depend on the survive capability of bacteria in the alkaline environment. Most of the microorganisms diein an environment with pH value of 10 or above [3]. Concrete is very good material to resist the compressive load to a limit but if the load applied on the concrete is more than their limitof resisting load,it causes the strength reduction of concrete by producing the cracks in the concrete and the treatment of the cracks in very expensive.Some of the property like durability, permeability ad strength of the concrete structure is also decreases. Due to increase in the permeability of the concrete the water easily pass through the concrete and come in the contact with the reinforcement of the concrete structure and after some time corrosionstart due to this strength of the concrete structurewill decreasesso it will be necessary to repair the cracks [1].By introduce the bacteria in concrete it producing calcium carbonate crystals which block the micro cracks and pores in the concrete [2]. In concrete micro cracks are always avoided but to some extent they are responsible to their failure in strength.The selection of the bacteria is depend on the survive capability of bacteria in the alkaline environment. Most of the microorganisms diein an environment with pH value of 10 or above

INTRODUCTION

Concrete is the most commonly used construction materials as it is widely used in the construction of buildings, dams, storage and other various infrastructures [1]. Cement is the

most important part of the concrete material that binds the aggregates and fills the voids between coarse and fine particles [2]. Due to high compressive strength, availability, durability as well as compatible behavior with reinforcement bars, low price, simple preparation and possibility of molding in different shapes of choice and size, concrete is the most acceptable option for many applications. Besideadvantages, concrete has a high propensity of forming cracks allowing aggressive chemicals and water to permeate the structure [3]. Cracks are therefore the major causes of concrete deterioration and decrease in durability [4].If the load applied on the concrete is more than a certain limit, it causes strength reduction of concrete by producing cracks in the concrete and subsequently durability and strength of the concrete structure also decreases simultaneously [5].The treatment for repairingcracks is a very costly affair. Due to increase in the permeability of the concrete, water easily passes through the concrete and reaches the reinforcement of the concrete structure. Quite naturally, after some time corrosion starts and consequently strength of the concrete structure further decreases necessitating repair of the cracks. Reaction between bacteria and calcium lactate produces calcium carbonate crystals($CaCO_3$) which blocks the micro cracks and pores in the concrete. Most of the microorganisms fails to exist in an environment with pH >3. Different types of bacteria Like*Bacilluscohniiand Bacillus pasteurii*are used to precipitate $CaCO_3$.In our work, mixed culture of bacteria of soil of coastal areas of Digha, West Bengal and Hot spring of Birbhum district of West Bengal has been used. The aim is to develop mixed culture of soil bacteria incorporated cement mortars which can be used in future construction purposes.

LITERATURE REVIEW

Hot springs and coastal areas generally have conditions which are favourable for the development of microbes of different groups as it contains various minerals dissolved in the water [6]. Ghosh et al (2008), Majumdaret al.(2012), Sarkar et al. (2014,2015), Sarkar et al. (2019) have used bacteria isolated from hotsprings of Bakreshwar,West Bengal which can deposit gehlenite inside the cementitious matrices [6,7,8,9,10]. Chatterjee et al (2019) have used genetically transformed Bacillus sp. in geopolymer mortar and have shown that by the incorporation of bacteria the mechanical strength increases as well as durability [11].

METHODOLOGY ADOPTED

All analytical grade chemicals were purchased for the preparation of mineral media to grow bacteria .Ordinary Portland Cement 43 grade [12] and standard Ennore sand [13] were used for the study. The mixed bacterial culture of soil of respective areas was grown in a specific mineral media (pH 10.0). The various categories of samples are firstly control OPC mortar, then OPC mortar along with mixed culture of bacteria from soil of coastal areas of Digha in West Bengal and, third is OPC mortar along with mixed culture of bacteria from soil of hot spring of Birbhum district in West Bengal are prepared for the work . Standard mortar cubes (70.6 mm × 70.6 mm × 70.6 mm) were prepared by using cement-sand ratio 1:3 by weight. For the preparation of 2nd and 3rd categorysamples amended with microbe mixed population bacterial cells (10^7 cells /ml concentration) were used. After 24 hours of casting, all the samples were removed from their respective moulds and cured at water for respective days. After curing for respective category of sample for several days, compressive strength of all the samples were measured using Digital Compression Test machine, India Make [10].Water absorption capacity was being recorded according to Neville's process by weighing the mass of the samples at 30 minutes and after 24 hours of immersion in water [14].

RESULTS

Table 1 show the increment in compressive strength in mixed culture of bacteria applied cementitious mortars compared to the cement mortar without bacteria. There is an increment in durability tests also which has been shown in Table 2 showing maximum increment in the samples amended with mixed bacterial culture of bacteria of soil of hot spring.

Table 1: Compressive strengths of mixed population of bacteria incorporated mortars

Samples	Compressive Strength (MPa)					
	3 Days	7 Days	14 Days	28 Days	90 Days	120 Days
1.OPC Control without bacteria	5.9 ± 1.2	11.8 ± 0.9	16.2 ± 1.4	20.2 ± 1.3	27.0 ± 1.2	34.5 ± 1.5
2 .OPC + Mixed culture of Bacteria of soil of coastal area	7.0 ± 1.25	13.25 ± 1.1	18.5 ± 0.96	23.2 ± 1.4	31.2 ± 1.7	37.0 ± 1.4
3.OPC + Mixed culture of Bacteria of soil of Hotspring	9.0 ± 1.9	15.4 ± 1.6	20..0 ± 1.6	25.7 ± 1.0	34.2 ± 1.1	39.4 ± 1.0

.

Table 2: Water Absorption Test of Mixed population of bacteria incorporated mortars

Samples	Mass (g)		
	Initial mass	Mass after 30 minutes	Mass after 24 hours
1.OPC	745 ± 4.67	754 ± 5.6	768 ± 4.2
2 . OPC + Mixed culture of Bacteria of soil of coastal area	727 ± 5.3	735± 6.2	747 ± 3.6
3. OPC + Mixed culture of Bacteria of soil of Hotspring	724 ± 5.2	731 ± 4.3	740 ± 8.1

CONCLUSIONS

It has been seen that greater compressive strength and more durability in case of cement mortars incorporated with mixed bacterial culture of soil isolated from hotspring of West Bengal as compared to cement mortar without bacteria as well as cement mortar with mixed bacterial culture of soil from coastal areas of Digha, West Bengal. Lot of silica leaching, spore forming, calcite precipitating bacteria are generally found in hotsprings and coastal areas and hence there is an increment in strength of bacteria induced mortar sampleswhich will help in future construction purposes.

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FINITE ELEMENT ANALYSIS OF STRENGTHENING CONCRETE BEAM USING FIBER REINFORCED POLYMER

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ABSTRACT

Reinforcing polymers with fibers is a very popular method of improving load capacity of structural members. The most commonly available fiber reinforced polymers (FRP) are carbon (CFRP), glass (GFRP) and aramid (AFRP). Due to their lightweight, high strength, resistance to corrosion, speed and ease of application and on site formation into different shapes has made them referenced materials in strengthening application.In the present studythe behavior of concrete beam strengthened by fibre reinforced polymer (FRP) sheet is investigated. The finite element analysis is performed to determine the load carrying capacity of concrete beams. The load carryingcapacities are compared in order to study the performance of concrete beams with and without FRP sheets.

KEYWORDS: Concrete, Fibre polymer concrete, flexural strenght and Non-linearity of concrete.

INTRODUCTION

The degradation of infrastructure in and around anyzone requires consideration in the form of replacement, rehabilitation or repair. The full structure replacement might have determinate disadvantages such as high costs for material and labour, a stronger environmental impact and inconvenience due to interruption of the function of the structure e.g. traffic problems. When possible, it is often better to repair or upgrade the structure by retrofitting. In late decades, there have been various investigations for restrengthening and retrofitting the reinforced concrete structure.

Retrofittingis defined as to increase the mechanical strength (tensile strength, compressive strength, flexural strength, shear strength) and durability i.e. resistance to water absorption, permeability and chloride penetration.

Due to light weight, Non-corroded and high strength capacity Fiber reinforced polymer has been use all over the world. The FRP has been proved effective in improving the strength and stiffness of concrete structures, making positive contribution to control the crack expansion.

Further more work has been done on FRP retrofitting in concrete structure.

Jahami et-al (2019) examined the performance of reinforced concrete beams using fiber reinforced polymer when subjected to dynamic loading.Yi et al. (2009) carried out the three-point-bending tests of concrete beams wrapped with CFRP and proved that reinforcement brought about higher load capacity and ductility and there by smaller deflection as well as concrete expansion in concrete. Kezmane et-al (2016) used CFRP in reinforced concrete wall for the seismic reinforcement of buildings. He performed numerical simulation of squat reinforced concrete wall strengthened by FRP composite material.

Obaidat et-al (2010)has developed a nonlinear 3-D numerical model using the ABAQUS finite element program, and it was used to examine the shear behavior of beams retrofitted by CFRP.Mahaidi and Kalfat(2010)have used Carbonfiber for anchorage purpose.
The present study is aimed to evaluating the flexural performance of PCC beams retrofitted with polymer materials.

AIM and SCOPE

The overall aim of the present study is to investigate and improve the understanding of the behavior of reinforced concrete beam retrofitted with reinforced polymers. The ABAQUS program is used to develop finite element models for simulation of the behavior of beams. From the analyses the load-deflection relationships has obtained and compared to the experimental results. The models is used to study how different parameters affect retrofitted beam behavior and investigate how FRP should be applied in order to get maximum increase of load capacity.

RESEARCH OBJECTIVES

Based on the observations from the literature review, gaps there in, the following are the main objectives of the present research work:

1. To develop Finite element model of three points flexural strength test of PCC beam.
2. To check the Flexural strength of PCC beam
3. To develop Finite element model of three points flexural strength test of PCC beam strengthened by FRP sheets.
4. To compare load v/s deflection curve of PCC beam and FRP beam.

PROPOSED METHODOLOGY

To meet the above-mentioned objectives the following methodology is adopted.
A finite element model of reinforced concrete beam strengthened by FRP sheets has been created and analyzed in finite element software package ABAQUS.
Following arethe step in this methodology:
Geometry of PCC beam
A plain cement concrete beam having dimensions 100 mm x 200 mm x 1520mm [7] has been modeled. This beam is called control beam.

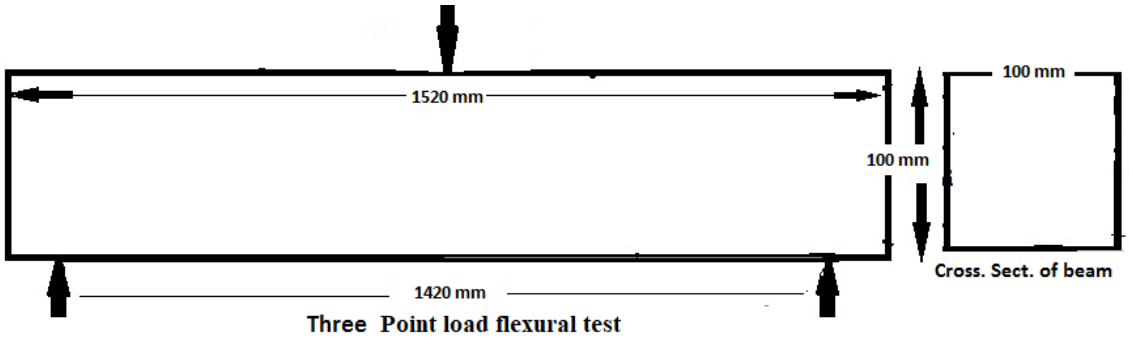


Figure 1. PCC beam

The beams are strengthened by FRP sheets 10mm thick, longitudinal side at bottom wrapped as shown below.

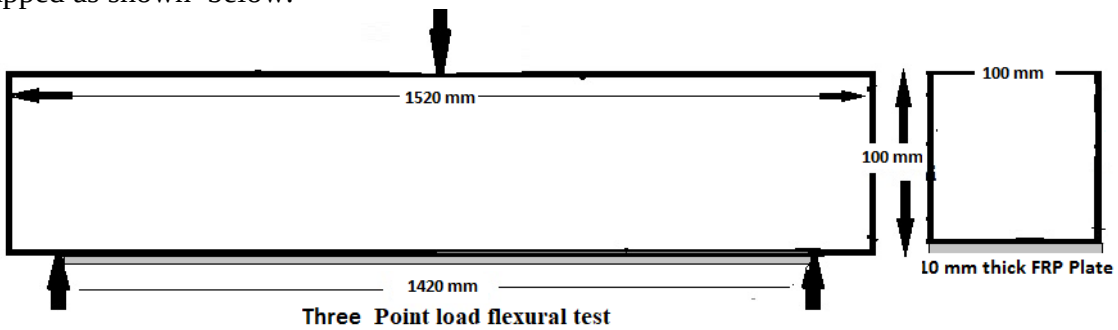


Figure 2. PCC Beamstrengthened by FRP sheets

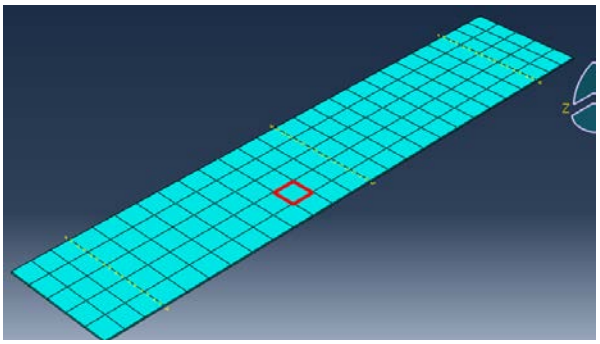


Figure 3. FRP sheet

MECHANICAL PROPERTIES OF MATERIALS

Table 13 Material Property

Material	Tensile strength (MPa)	Modulus of Elasticity (GPa)	Density kg/m^3	Poisson's ratio
FRP	1500-3700	160-540	1400-1700	0.25
Concrete	2.842	35.35	2400	0.20

Plasticity:This model is to simulate the nonlinear behavior of concrete.Load and three points load test isshown in Figure 4.

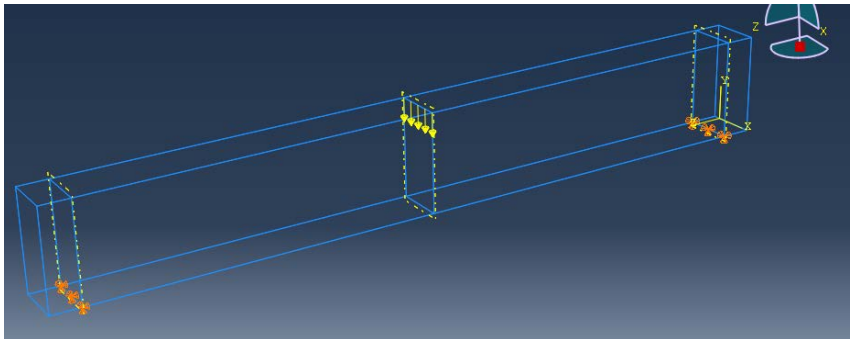


Figure 4.Simple supported Beam

Meshing of control beam is shown in Figure 5.

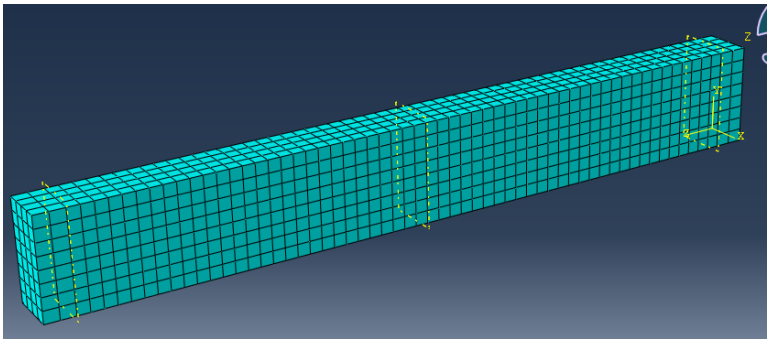


Figure 5. Mesh element

RESULTS

Maximum deflection at midspan of PCC beam increases when PCC beam is strengthned by FRP sheet as shown in Table:2 and in Figure 6

Table 14 Load & deflection

Material	Experimenatal analysis of PCC Beam	FEA Model of PCC Beam	FEA Model of FRP Beam
Size (mm)	100×200×1520	100×200×1520	100×1420×10
Load (kN)	17.36	17.357	44.931
Dispalcment (mm)	4.22	4.54	4.54

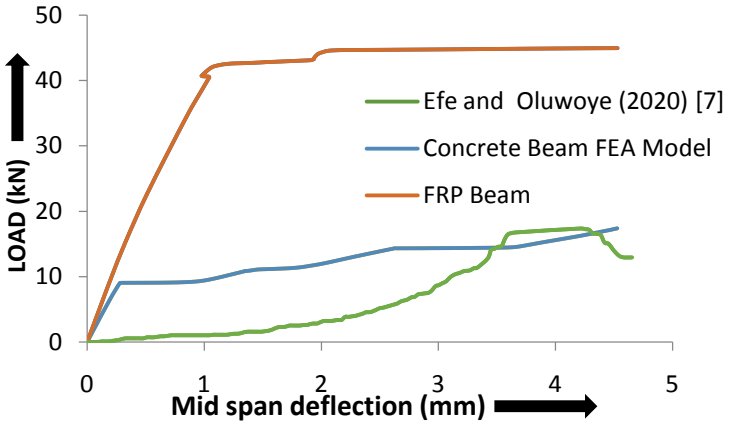


Figure 6 Load deflection curve for PCC Beam (experimental model and FEA model) and FEA Model of FRP Beam

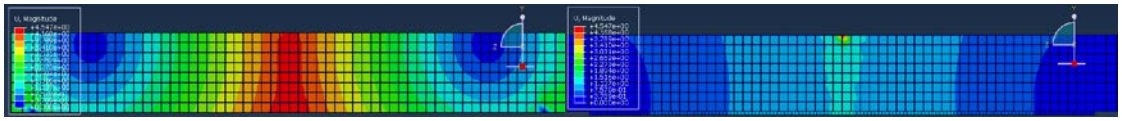


Figure 7Deform shape of PCC Beam (a) & FRP Beam (b).

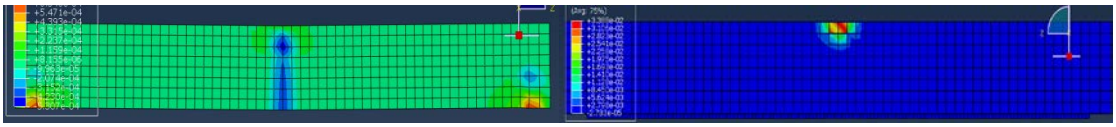


Figure 8Contour map of plastic strain in PCC Beam (a) & FRP Beam (b)

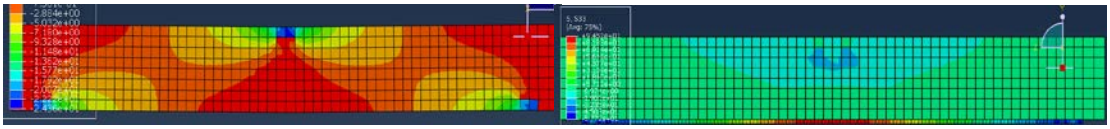


Figure 9. Contour map stressPCC Beam (a) & FRP Beam (b)

DISCUSSIONS

Result of the study exhibit the following points:

1. Experimental resultsof Efe& Oluwoye (2020)[7] have been validated sucessfully by FEA model of PCC beam as shown in Figure 6.
2. Load carrying capecity of plain cement concrete beam has been increased from 17.36 kN to 44.93kN when strenghten by FRP sheet.
3. Plastic behaviour of concrete beam and FRP beam is shown in Figure 8.
4. Also Propagation of crackbefore failure in beamswith and without FRP sheets clearly visible in Figure 8.
5. Failure of concrete beam with and without FRP sheets is due to bending stress is shown in Figure 9.
6. Deflection of plain cement concrete beam has been dicreased from 4.52 mm to 0.379 mm when strenghten by FRP sheet.

CONCLUSIONS

A 3D nonlinear FE model implemented with the general-purpose program ABAQUS has been presented in this for the simulation of behavoiur of PCC beam strenghtned with FRP.
The concrete in this model,is treated as a elasto-plastic material and fracture mechanics notions are used to define the failure.By using finite-element model, Load deflection curve is successfully simulated. PCC beam strengthened with FRP exhibited a higher load capacity compared to respective control beam.Load carrying capacity PCC beam can be increased by using FRP sheets.

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CONCRETE USING HYPO SLUDGE AND POLYPROPYLENE FIBER

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ABSTRACT
Reinforced concrete structure is the major construction typology prevailing worldwide which majorly uses concrete and steel, apart from this concrete is most extensively used material in civil industry for construction. Energy demand and raw materials for producing concrete results in Greenhouse gases (GHG) emission during production of raw material and production as well, cement industry leads to 5% emission of GHG, whereas concrete production contribute almost double of it. Reduction of its environmental impact and enhancing the sustainability of concrete is major concern and need to be addressed. Optimum usage or partial replacement of cement, without altering concrete's physical and mechanical properties can lead to reduction in GHG emissions, partial replacement using industrial wastes like fly ash, hypo sludge (HS), sugarcane bagasse etc. also helps in proper disposal.
HS is a waste material from paper industry and disposed in large amount in the form of slurry, disposal of these materials causes serious environmental pollution. Therefore, usage of HS as partial replacement of cement in concrete manufacturing can help in minimizing the environmental pollution issue. In present study, based on the past research, usage of HS as partial replacement of cement varying up to 20% for concrete manufacturing in combination with addition of polypropylene fiber (PF) up to 1% by volume of concrete is done, with objective to find acceptable optimum replacement of HS without significant reduction in physical and mechanical properties compared to normal concrete of same grade. Results concluded based on lab test for identification of mechanical properties like compressive strength, split tensile strength and flexural strength done after 7 days, 14 days and 28 days of curing. Concrete mix design and experiments were done using relevant Indian Standards. Result indicates the addition of both HS and PF increase the compressive, tensile and flexural strength of concrete at all curing ages up to certain point. The combination of 10% Hypo sludge and 0.5% polypropylene fiber shown the optimum performance in terms of mechanical strength, also Concrete production cost reduced up to 15%.

KEYWORD: Hypo sludge, Polypropylene fiber, Compressive strength, Split tensile strength, Flexural strength.

INTRODUCTION
Many research organizations are doing huge work on waste materials concerning. Paper mill sludge is a major environmental and economic issue for the board and paper industry. The material is a by-product of the de-inking and re-pulping of paper. The pulp and paper industry generates large volume of waste called Hypo sludge; which is technology-dependent. It is estimated around 18% of waste (sludge) is generated during the production of pulp.

Hypo sludge contributes advantageous properties to the concrete while helping to maintain the economy. Therefore, numerous contemporary research works have focused on the application of hypo sludge in cement and concrete production to attain sustainable development. Many researchers have investigated the feasibility of using the paper industry

waste in concrete production as partial replacement of cement. Disposal cost of paper industries can be reduced by using hypo sludge in concrete and produced the green concrete for construction.

Polypropylene is a synthetic hydrocarbon polymer, the fibre of which is made using processes of extrusion by hot drawing the material through a die. Polypropylene use enables consistent and effective utilization of tensile and flexural strength of the material along with significant reduction of plastic shrinkage, cracking and reducing of thermal cracking.

LITERATURE REVIEW

Sam S, Perararasan M and Suji D (2015):- Studied the residual properties of polypropylene fibre reinforced concrete under elevated temperatures. High temperature can cause the development of cracks which may eventually cause loss of structural integrity and shorting of service life. An experimental program was carried out using different volume fraction of 0%, 0.2%, 0.4%, and 0.6% of polypropylene fibre to find the fire resistance characteristics. Beams of size 500x100x100 mm size were used to find the flexural behavior change under 100 °C, 200 °C and 300 °C. Experiment shows that addition of polypropylene fibres enhances the flexural behavior of concrete compared to concrete without fibres at room temperature. But maximum loss in tensile strength and flexural strength was observed at 300^0C for exposure period of 6hrs.

Santosh Ahirwar and Rajeev Chandak (2018):- Paper making generally produces a large amount of solid waste. Paper fibers can be recycled only a limited number of times before they become too short or weak to make high quality paper. Concrete specimens were prepared with 7.5%, 10%, 12.5% and 15% hypo sludge as a replacement of cement weight, The most important mechanical property of concrete is compressive strength and it is evaluated on 150X150X150 mm cubes. The test was carried out obtain compressive strength of concrete at the age of 7 and 28 days. The cubes were tested using Compression Testing Machine (CTM) of capacity 2000KN available in structures lab. From Fig1 the compressive strength is up to 22.52 N/mm2 and 31.60 N/mm2 at 7 and 28 days. The maximum compressive strength is observed at 10% replacement of Hypo Sludge.

P.Velumani, C.Lakshmipriya, N.Poovitha & R.Jeevanram (2018):- Utilization or effectual disposal of waste materials from the industries or from the other sources plays a vital role in global level to reduce the environmental pollution as per the environment policies of the respective countries. Reuse of process finished materials leads in the reduction of unnecessary land filling / dumping. Hypo sludge is an industrial preliminary waste generated from the caustic soda process of paper making and other specific manufacturing industries. In this investigation, preliminary steps have been executed so as to confirm the doable utilization of hypo sludge as an ancillary cementitious material in suitable proportions. The compressive strength in sludge-cement blocks after 28 days of curing were tabulated and compared for 10, 20 % sludge addition with cement. The control gave the strength of 33.79 after 28 days of water curing. It was observed that the strength for mortar cubes at 10 % of replacement was increased. The replacement up to 10% of sludge gives satisfactory strength requirement as per IS 1489-1991. Water absorption of mortar increases as the percentage of addition of hypo sludge increases and ranges from 4.12 % to 5.06 %. Moreover the water absorption for all the combinations does not exceed around 5%.As far as block density was concerned, it varies from 2300 t0 2700 kg/m^3.

Mehtab Alam,Vebhav Berera:- For sustainable development, among others, protection of environment is one of the major challenges. In order to reduce the consumption of natural resources, which are limited, use of industrial wastes to replace fresh materials is

one of the viable options to control degradation of environment. Study shows some important parameters such as workability, cube strength, stress-Strain characteristics (cylinder), modulus of elasticity and failure patterns for M20 (1:1.5:3) mix with 10%, 20%, 30%, 40% cement replacement with hypo sludge and comparison with that of conventional cement concrete, In the result 20% replacement of cement by hypo sludge gives as much strength as pure cement concrete, 10% replacement of cement by hypo sludge gives about 11% higher strength than pure concrete.

P. Akhil Kumar, Venu Malagavelli, J.S.R Prasad (2019):- In this study total five concrete mixes were made with steel fiber in dosages of 2.5%, 2%, 1.5%, 1% and polypropylene fibers are in dosage 0%, 0.5%, 1%, 1.5% of the weight of concrete mix. The specimens were casted and all the specimens are tested for 7days and 28 days strength. The results have depicted a gradual increase in the strength of the concrete as the fiber content increased. The compressive strength values obtained for mix1,mix2, mix3 and mix4 is 55.8mpa,56.2mpa, 58.6mpa, and 59.8mpa respectively. Results depict that 1.45%, increase in its strength for mix1, 2.18%, increase in its strength for mix2, 6.54%, increase in its strength for mix3 and 8.72%, increase in its strength for mix4.

The split tensile strength obtained for conventional concrete at 28 days of curing is 3.42mpa, and the split tensile strength values obtained for mix1,mix2, mix3 and mix4 is 3.67mpa, 3.88mpa, 4.01mpa and 4.32mpa. Results depict that 7.3%, increase in its strength for mix1, 13.45%, increase in its strength for mix2, 17.25%, increase in its strength for mix3, and 26.31%, increase in its strength for mix4.when compared to conventional concrete.

The flexural strength obtained for conventional concrete at 28 days of curing is 4.82mpa, and the flexural strength values obtained for mix1,mix2 mix3 and mix4 is 5.01mpa ,5.26mpa, 5.69mpa and 5.88mpa . Results depict that 4.2%, increase in its strength for mix1, 9.12%, increase in its strength for mix2, 18.04%, increase in its strength for mix3, 21.99%, increase in its strength for mix4. When compared to conventional concrete.

MATERIAL USED

Cement: Itis a binding material which is formed by crushing, burning and then grinding of the stones clinkers these clinkers are formed by siliceous, argillaceous and calcareous stones. Cement provides binding property by the process of heat of hydration with the help of water, now a day's cement is widely used material on the earth. The various uses of cement are bonding of bricks, plastering of the walls, preparation of the base for the foundation, flooring work, grouting work and final finishing.

Aggregate: Aggregates typically constitute 75% of concrete volume. Hence, aggregate types and sizes play an essential role in modifying the concrete properties. The broken stone is generally used as a coarse aggregate. The nature of work decides the maximum size of the coarse aggregate .Locally available coarse aggregate having the maximum size of 20mm was used in the present work. The main purpose of the aggregates is to provide a rigid skeletal structure and reduce the space occupied by the cement paste. Coarse aggregates and fine aggregates are required but the proportions of different sizes of coarse aggregate will vary depending on the particular mix required for each individual en. Generally aggregates can be classified as normal weight aggregates, light weight aggregates and heavy weight aggregates. Normal weight aggregates can be further classified as natural aggregates and artificial aggregates.

Hypo sludge: The process of formation of paper from pulm includes the following process during which the hypo sludge is formed as waste by-product is purely a chemical waste and do not contain any bio-degradable element . The total percentage of the three combinations

of SiO_2, Al_2O_3 and Fe_2O_3 was 45% and expected to possess low Pozzolanic reactivity (50%). Hypo sludge is obtained from paper industry and used in construction work as partial replacement of cement. Hypo sludge buying from Adopt consultancy, Near Piplani Petrol Pump Bhopal MP was used in this study which is shown in figure 1



Figure 1: Hypo sludge

Polypropylene fibre: Polypropylene fibers are thermo plastics produced from Propylene gas. These are micro reinforcement fibers. They are 100% virgin homopolymer polypropylene graded monofilament fibers. The raw material of polypropylene is derived from monomeric C3H6 which is purely a hydrocarbon. These fibers have low density of 0.9 gm/cc. They are highly crystalline, with high stiffness and excellent resistance to chemical and bacterial attack. Perfectly Isotactic polypropylene has a melting point of 171°C while commercial Isotactic polypropylene has a melting point ranging from 160 to 166 °C. Polypropylene fibers are non-magnetic, rust free, alkali resistant, safe and easy to use. Addition of fibers increases ductility performance, pre-crack tensile strength, fracture strength, toughness, impact resistance, flexural strength resistance, fatigue performance etc. Polypropylene fibers are tough but have low tensile strength and modulus of elasticity Polypropylene fibers have been reported to reduce unrestrained plastic and drying shrinkage of concrete at fiber contents of 0.1 to 0.3% by volume. Polypropylene fibers buying from Khandelwal Fabrics,E-7,LIG-333,Mitramandal Society, Arera Colony, Bhopal MP were used in this study shown in Figure 2

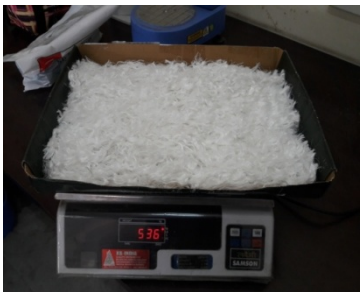


Figure 2: Polypropylene

Water: Fresh and clean tap water was used for casting the specimens in the present study. The water was relatively free from organic matter, silt, oil, sugar, chloride and acidic material**.**

METHODOLOGY

Workability of concrete as per IS: 1199-1959: Workability is the property of fresh cement concrete mix which shows the ability of ease in working like mixing, handling, placing, compacting and final finishing . Workability is not just based on the properties of the concrete, but also on the nature of the application. The strength and durability of hardened concrete, in addition to labour costs, depend on concrete having appropriate workability.

The basic steps involved in the concrete mix design can be summarized as follows:

i. The mould for the slump test is a frustum of a cone, 300 mm of height. The base is 200 mm in diameter and it has a smaller opening at the top of 100 mm.

ii. The base is placed on a flat surface and the slump cone is filled with concrete in three layers.Each layer is temped 25 times with a standard 16 mm diameter steel rod, rounded at the end.

iii. When the mold is completely filled with concrete, the top surface is leveled by means of screening and rolling movement of the temping rod.

iv. The decrease in the height of the center of the slumped concrete is called slump.

v. The slump is measured by placing the cone besides the slump concrete and the temping rod is placed over the cone so that it should also come over the area of slumped concrete. The reduce in height of concrete the mould is noted with scale (usually measured to the nearby 5 mm.)



Figure:-3 Slum Cone Test

Compressive strength of concrete as per IS: 516 -1959: Compressive strength of concrete was determined from cubes of 150 mm × 150 mm × 150 mm in size. Cube moulds were cleaned and oil was applied. Then the concrete was filled into the cube moulds. To ensure proper compaction, concrete moulds were vibrated.

Specimens were taken out from the curing tank at the ages of 7 days, 14 days and 28 days. Surface water was wiped off and specimens were immediately tested after removal from the curing tank. The compressive strength of concrete cubes was tested under Universal Testing Machine (CTM). The load was applied gradually without shock till the failure of the specimen occur and thus the compressive strength of concrete cubes was found

$$\textbf{Compressive strength} = \frac{\textbf{Load(KN)}}{\textbf{Area(mm2)}}$$



Figure 4:- Casted cube specimen & Curing of the casted cube specimens

Splitting Tensile Strength as per IS: 5816-1970: The magnitude of this tensile stress Fsp acting in a direction perpendicular to the line of action of applied loading is given by the formula (IS:5816-1970)

$$\textbf{Fsp (split tensile strength)} = 2P/\pi dl$$

where

P =Load at which the specimen fail (in kN)
D =Diameter of cylinder specimen (in mm)
l =Length of cylinder specimen (in mm)

The ratio of the split tensile strength to cylinder strength not only varies with the grade of the concrete but is also dependent on the age of concrete. The ratio is found to decrease with time after about a month. The air cured concrete gives lower tensile strength than that given by moist cured concrete.



Figure 5: Splitting tensile strength test

Flexural strength as per IS: 516-1959: The flexural strength test of beam, a specimen of size 700*150*150mm is placed over two point loading arrangement and the stress produced during breakage of specimen. The flexural strength is reported as Modulus of Rupture ft (N/mm)and calculated as

$$\text{Flexural strength Ft} = pl/bd^2$$

where
P=Load at which the beam specimen fails (in KN)
L=Effective length of the bam specimen (in mm)
b= Width of the beam specimen (in mm)
d=Depth of the beam specimen (in mm).



Figure 6: Flexural strength testing using UTM

RESULTS AND DISCUSSIONS

Test results for workability of concrete:-

Mix	Hypo sludge%	Polypropylene fibre%	Slump (mm)	Degree of workability
Mix1	0	0	100	Medium
Mix2	10	0	90	Medium
Mix3	20	0	70	Medium
Mix4	0	0.25	90	Medium
Mix5	10	0.25	80	Medium
Mix6	20	0.25	60	Medium
Mix7	0	0.50	84	Medium
Mix8	10	0.50	70	Medium
Mix9	20	0.50	54	Medium
Mix10	0	1.00	55	Medium
Mix11	10	1.00	45	Low

Mix12	20	1.00	40	Low

It was observed that the workability of concrete decreases as the content of hypo sludge and polypropylene fiber increases. The slump for the mix with 0% hypo sludge was 100mm, while slump for the mix with 20% hypo sludge was 70mm.The slump decreases when a higher amount of hypo sludge was replaced with cement. Due to this reason hypo sludge have high water absorption capability. When higher amount of hypo sludge was added in the mixture, the water demand of the mix rises which leads to the reduction in workability of concrete. Further, it was observed that the addition of polypropylene fiber reduces the workability of concrete. The slump for 0% polypropylene fiber was 100mm, while slump for 1% polypropylene fiber was 60mm. Workability reduces at higher dosage of fibers compared to initial dosage.

Test results for compressive strength of concrete at 7 days, 14 days and 28 days:-

Mix	Mix 1	Mix 2	Mix 3	Mix 4	Mix 5	Mix 6	Mix 7	Mix 8	Mix 9	Mix 10	Mix 11	Mix 12
Hypo sludge%	0	10	20	0	10	20	0	10	20	0	10	20
PolypropyleneFibre%	0	0	0	0.25	0.25	0.25	0.50	0.50	0.50	1.00	1.00	1.00
Compressive strength after 7 Days N/mm^2	14.34	17.61	12.22	16.55	18.90	14.56	18.68	19.33	15.22	13.87	15.47	9.15
Compressive strength after 14 Days N/mm^2	22.32	23.50	18.54	23.65	25.00	19.62	24.33	25.53	20.10	21.34	21.80	12.80
compressive strength after 28 Days N/mm^2	25.55	26.44	21.67	26.72	27.76	21.77	27.24	28.32	22.13	22.66	23.17	1335

From the compressive strength results given above the following observation have been done

1. It can be observed that the compressive strength of concrete increase up to 10% of hypo sludge. Then after, the compressive strength decreases. The same trend was observed for 0.25% and 1% of polypropylene fiber.
2. It can be seen that the change in compressive strength was minor up to 10% of hypo sludge but beyond 10% of hypo sludge, compressive strength due to water absorbing ability of hypo sludge which disturbed the water-cement ratio was significantly reduction.
3. At higher dosage of sludge, cement content decreases which leads to the weak bonds especially around sludge particles. This eventually caused early crack development during compressive test.
4. It was observed that the addition of polypropylene fiber up to 0.5% showed the increase in compressive strength of concrete.
5. The strength decreases for1% of polypropylene fiber. This is due to the fact that polypropylene fiber attributes towards a good cohesion with other aggregates. But at higher dosage of fiber due to the formation of air voids, the concrete becomes stiff and difficult to compact which tends to decrease the compressive strength.

The combination with 10% of hypo sludge (as the replacement of cement) and 0.50% polypropylene fiber (addition) is recommended for the use for the maximum compressive strength.

Graph1 Combine graph of Compressive Strength for all mixes

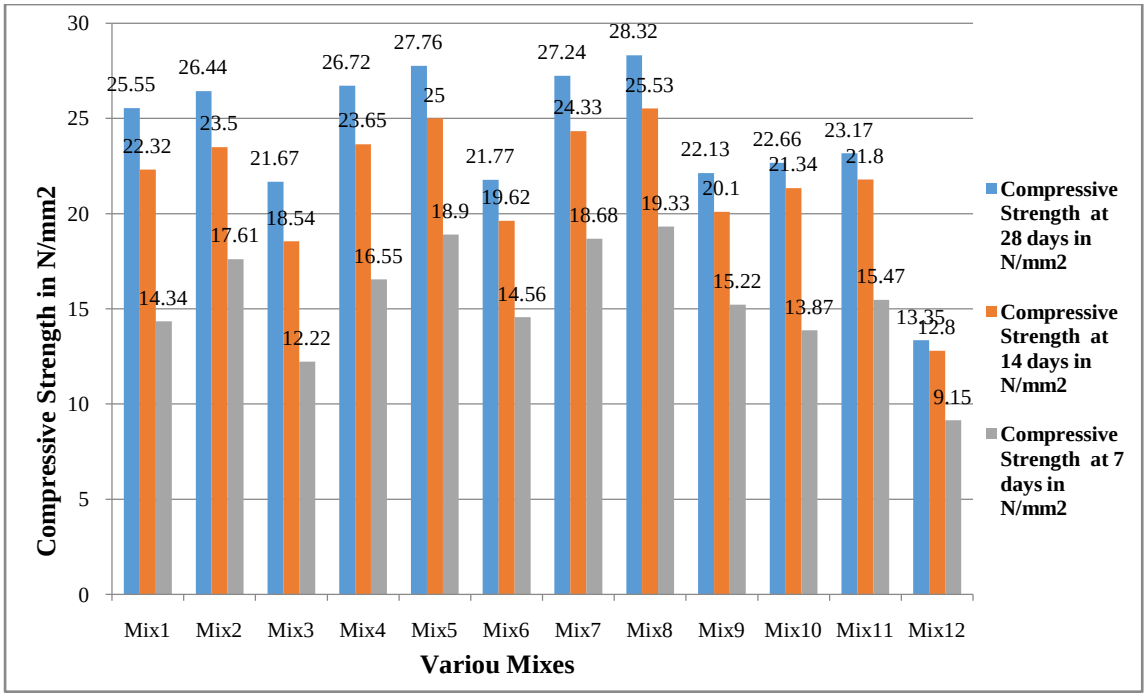


TABLE :5 Test result of Split tensile strength at 7 days, 14 days and 28 days:-

Mix	Mix 1	Mix 2	Mix 3	Mix 4	Mix 5	Mix 6	Mix 7	Mix 8	Mix 9	Mix1 0	Mix1 1	Mix1 2
Hypo sludge%	0	10	20	0	10	20	0	10	20	0	10	20
Polypropylene fibre%	0	0	0	0.25	0.25	0.25	0.50	0.50	0.50	1.00	1.00	1.00
Split tensile strength after 7 days (N/mm^2)	1.75	2.00	1.55	1.70	2.12	1.48	1.85	2.30	1.80	1.25	1.90	1.35
Split tensile strength after 14 days(N/mm^2)	2.70	3.10	2.30	3.16	3.40	2.50	3.32	4.12	2.80	2.25	2.70	1.80
Split tensile strength after 28 days (N/mm^2)	3.33	3.60	2.70	3.52	3.78	2.78	3.75	4.60	3.20	2.50	2.75	2.15

From the Split tensile strength results given above the following observation have been done

1. It can be observed that the splitting tensile strength of concrete increase up to 10% of hypo sludge. Then after, the spitting tensile strength decreases. The same trend was observed for 0.25%, 0.50% and 1% of polypropylene fiber.
2. It was observed that the addition of polypropylene fiber up to 0.5% showed the increase in splitting tensile strength of concrete. After that the strength decreases for 0.5% and 1% of polypropylene fiber.

From the above discussion it can be seen that the combination with 10% of hypo sludge (as the replacement of cement) and 0.50% polypropylene fiber (addition) is recommended for the use for the maximum splitting tensile strength.

Graph2 Combine graph of Split Tensile Strength for all mixes

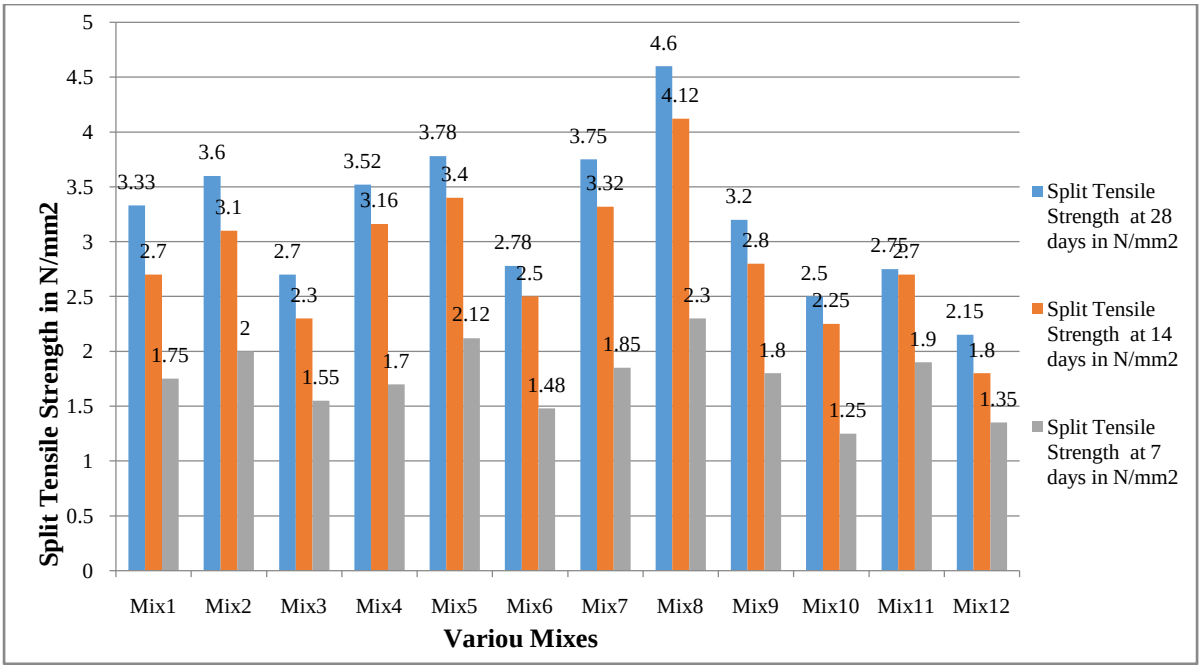


Table:6 Test result for Flexural strength at 7 days, 14 days and 28 days curing

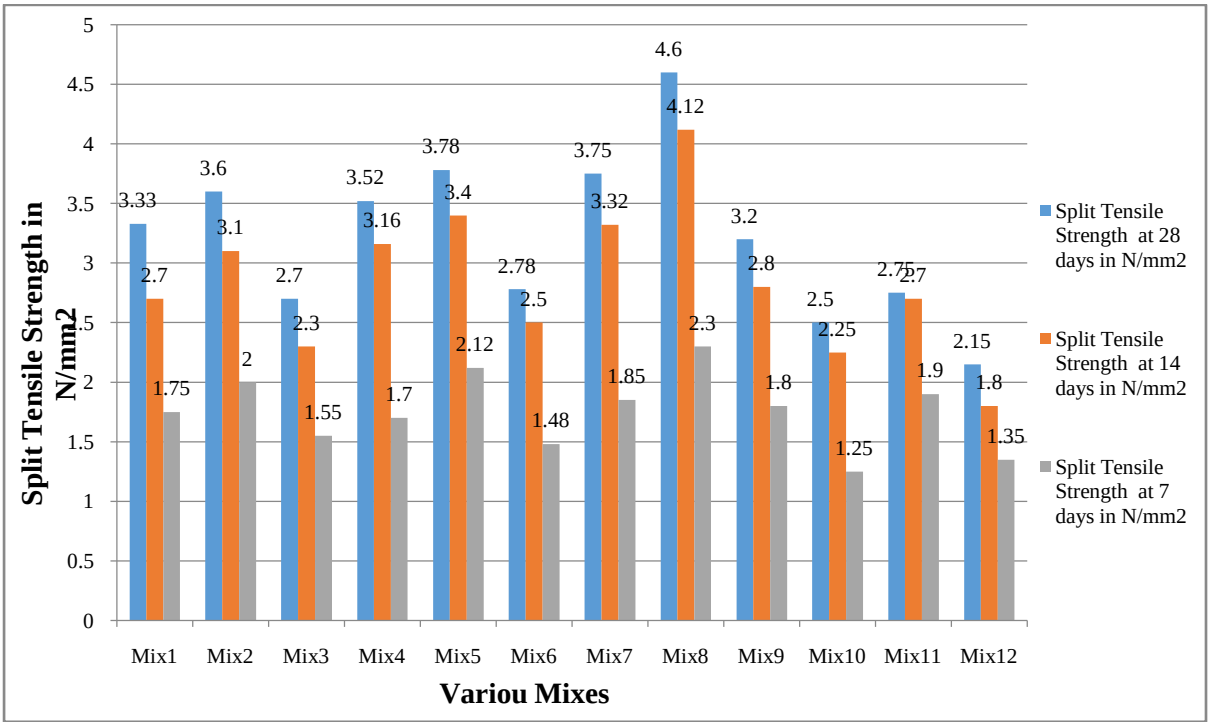
Mix	Mix 1	Mix 2	Mix 3	Mix 4	Mix 5	Mix 6	Mix 7	Mix 8	Mix 9	Mix 10	Mix 11	Mix 12
Hypo sludge%	0	10	20	0	10	20	0	10	20	0	10	20
Polypropylene fibre%	0	0	0	0.25	0.25	0.25	0.50	0.50	0.50	1.00	1.00	1.00
Flexural strength after 7 days (N/mm^2)	1.58	1.81	1.54	1.33	1.11	1.79	1.90	2.20	2.05	1.25	1.60	1.40
Flexural strength after 14 days (N/mm^2)	2.0	2.32	2.21	2.01	2.51	2.30	2.44	2.88	2.46	1.89	1.85	1.53
Flexural strength after 28 days (N/mm^2)	2.65	2.75	2.55	2.03	2.79	2.44	2.85	3.08	2.88	2.25	2.48	2.18

From the Flexural strength results given above the following observation have been done

1. It can be observed that the flexural strength of concrete increase up to 10% of hypo sludge. Then after, the flexural strength decreases. The same trend was observed for 0.25% and 1% of polypropylene fiber.
2. At higher dosage of sludge, cement content decreases which leads to the weak bonds especially around sludge particles. This eventually caused early crack development during flexural strength test.
3. It was observed that the addition of polypropylene fiber up to 0.5% showed the increase in flexural strength of concrete.

The combination with 10% of hypo sludge (as the replacement of cement) and 0.50% polypropylene fiber (addition) is recommended for the use for the maximum flexural strength

Graph3 Combine graph of Flexural Strength for all mixes



CONCLUSION AND FUTURE WORK

The objectives of the study were to study the effect of partial replacement of hypo sludge with cement, with polypropylene fibers used as additive in concrete. 72 concrete specimens were prepared by partial replacement of cement with hypo sludge 0% to 20% and with polypropylene fiber 0% to 1%. After having trial of mixes, the water-cement ratio selected was 0.55 and it was kept constant for all the mixes. The workability of concrete was tested immediately after preparing the concrete whereas the compressive strength, splitting tensile strength and flexural strength of concrete was tested after 7 and 28 days of curing.

From the experimental investigations, it is concluded that:

1. Polypropylene fibre addition improves the compressive, split tensile strength and flexural strength of concrete.
2. The addition of both hypo sludge and polypropylene fiber increases the strength of concrete for all curing ages up to certain point. After that there is an abrupt reduction in strength of the concrete. Because at higher dosage, concrete loses its ability to make a proper bond.
3. The gradual increase was seen in compressive strength and splitting tensile strength of concrete blended with 0% to 10% of hypo sludge content for all curing ages. Beyond that there is significant reduction in strength. This is due to water absorbing capability of hypo sludge which disturbed the water-cement ratio.
4. The gradual increase was seen in compressive strength, splitting tensile strength and flexural strength of concrete blended with 0% to 0.5% polypropylene fiber for all curing ages. Beyond that there is significant reduction in compressive strength. Because at higher dosage of fiber due to the formation of air voids, the concrete becomes stiff and difficult to compact which tend to decrease the strength.
5. The mix Mix8 with combination 10% hypo sludge and 0.5% polypropylene fiber has shown the highest compressive as well as splitting tensile strength. Therefore Mix8 is recommended for maximum strength.

6. Mix8 (10% hypo sludge and 0.5% polypropylene fiber) is best suited as the coefficient of brittleness (5.5) is minimum, whereas Mix10 (with the highest dosage of hypo sludge and polypropylene fiber) has the maximum coefficient of brittleness that is 9.06.
7. The cost feasibility indicates that with incorporation of hypo sludge decreases the cost of concrete, but at the same time strength also decreases. In general, use of the hypo sludge in concrete can save disposal costs of paper industry waste and produce an efficient concrete for construction. Also the environmental effects caused due to the production of cement can be minimized at the same time.

SCOPE FOR THE FUTURE

1. Further studies need to be conducted for the test of durability, soundness, thermal insulation, and water absorption of the concrete.
2. Further studies need to be conducted by using hybrid fibres (combination of artificial and natural fibres) as reinforcement with steel fibre could increases tensile strength of concrete.
3. If we increase the CaO in hypo sludge then we increase huge the strength in compressive strength and flexural strength.
4. Experiments can be performed to study the behaviour of soil reinforced with hypo sludge.

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BRICK MANUFACTURING USING DOLOMITE WASTE FROM BILASPUR REGION

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ABSTRACT
Bricks are the major building construction material used abundantly in construction. Conventional bricks are manufactured using clay and sand resulting in resource exhaustion and depletion, also intensive firing is being required for the proper burning of bricks leading to the environmental pollution. To overcome from these complications with conventional bricks, an alternate solution for manufacturing bricks need to be studied. Industrial wastes like Fly Ash, Hypo sludge, Dolomite etc. are available in plenty nearby production and mining areas, handing and proper disposal of such wastes is tedious and if not disposed properly may lead to pollution. Dolomite waste (DW) that is being produced in different region at Bilaspur district of Chhattisgarh by extraction of dolomite from dolomite mining, due to the improper storage and unavailability of site location for its disposal, it has covered approximately all the nearby areas of dolomite mines. So utilization of dolomite waste for some productive construction material will be beneficial from environmental and economical point of view.
In the present paper, optimum usage of DW as a raw material in the manufacturing of bricks without alteration in physical and mechanical properties compared to conventional bricks is done. DW bricks also show slight improvement in water absorption, efflorescence, compressive strength etc.

KEY WORDS: Industrial Waste, Dolomite Bricks, Cost Estimation

INTRODUCTION
Our world is full of waste materials which are used in different alternatives works as by using as by-product and by replaced. There is a lots materials used in construction industry. Now days most common waste material in India is fly ash which help in make the building economical in terms of using as bricks. India is developing country. Its economy and population both are increasing day by day and the demands of people also increasing so to meets the demands we raised the industrialization.

USES OF DOLOMITE IN INDUSTRY
Dolomite as a mineral has very few uses. In any case, dolostone has a gigantic number of employments on the grounds that it happens in stores that are sufficiently vast to mine. The most well-known use for dolostone is in the cconstruction industry. It is pounded and estimated for use as a street base material, a total in cement what's more, black-top, railroad counterweight, tear rap, or fill. It is additionally calcined in the generation of concrete and cut into squares of explicit size known as "measurementstone.

1. Industrial Applications

A few steel plant shave dispensed with the use of dolomite in blast furnaces and its use in the preparation of self-fluxing sinters is found adequate for blast-furnace charge. It is useful in the recovery of magnesia and also in the manufacture of magnesium metal; for the

manufacture of basic magnesium carbonate for other heat insulation, in pharmaceutical, rubber and chemical industries, and in the manufacture of paper, leather, glass, potteries and high-magnesium limes. In agriculture, it is used as a soil conditioner to correct acidity. It finds use as a filler in fertilizers , paints & varnishes, for suppression of dust in coalmines. It is not also as a building stone and in the making of flooring tiles as chips &powder.

LITERATURE REVIEW

1"Multi impregnating pitch-bonded Egyptian dolomite refractory brick for app in ladle furnace" by Mahmoud Rabah, Emad M. M. Ewais (2009*)* The preparation of multi impregnated pitch bonded Egyptian dolomite refractory brick for ladle furnace method is described herewith. Blend of calcined dolomite mineral & coal tar pitch were used for the preparation of Brick samples. The blend was pressed under a compression force up to 15 MPA and was hot mixed. A temperature of 1000°C for 2h was forced for the baking of green bricks .

2"Preparation of porous brick from diatomite and sugar filter mud at lower temperature" by Jianzong Man, Wenyuan Gao, Shuang Yan, Guishan Liu ,Hangshun Hao (2017) Low-grade diatomite and sugar channel mud (SFM) is utilized for the results of permeable blocks in an alternate extent somewhere in the range of 0 and 30 wt% were created at sintering temperatures extending from 700 to 900 °C. The Investigation is done on the impact on physical and mechanical properties of sintering temperature and including an extent of sugar channel mud. DSC-TG warm examination, XRD, SEM and all inclusive testing machine are physical and mechanical properties.

3."Dolomite brick with free lime" by G. S. Dolgina (1960) The creation of dolomite block was free lime. Unadulterated dolomite with a high substance of free calcium oxide and adequate free magnesium oxide and a low substance of alumina and sesquioxides, the last in adequate amount to guarantee sintering, ought to be utilized. In the wake of drying, anhydrous folio P can be utilized as the bond since it empowers solid green material. At 250-300° the green material ought to be dried in a drier with a separating fan. Water vapor ought notbe contained for warmth exchange. At 1600°, the partsought to be terminated for 5 hours the temperature ascends to 250° amid the primary hour.

MATERIALS AND METHODOLOGY

The materials used for manufacturing of brick are sand clay cement dolomite waste water and lime for brick production.

- **Manufacturing process of Dolomitebricks**
  Following procedure has been adopted for the manufacturing of Dolomite bricks Procurement of materials Preparation of mix sample Molding Drying and Burning.
- **MoldingProcess:**Bricksaremainlymoldedbytwo

MIX PROPORTIONING OF DOLOMITEBRICKS

Sample bricks were prepared by varying the percentage composition of DW and clay in the brick and keeping the percentage of sand, lime, and cement constant. Different samples prepared are tabulated below:

Table 1 - Proposed Mix Proportioning

S.No	Name of Sample	No. of Burnt Bricks	DW (%)	Clay (%)	Sand (%)	Lime	Cement
1	S1	5	50	20	10	10	10
2	S2	5	50	15	10	10	15
3	S3	5	50	10	10	10	20

4	S4	5	45	25	10	10	10
5	S5	5	45	20	10	10	15

Tests performed on Dolomitebricks

- ☐ Compressive Strength test: [IS:3495(Part 1)-1992]
- ☐ Water Absorption test: [IS: 3495(Part2)-1992]
- ☐ Efflorescence test: [IS: 3495(Part3)-1992]
- ☐ Determination of Warpage: [IS: 3495(Part4)-1992]

RESULTS OF COMPRESSIVE STRENGTH TEST

Variation in Compressive Strength of burnt bricks at 28 and 56 Days

S.No.	Specimen (BurntBricks)	Compressive Strength			
		Burnt Bricks			
		28 Days		56 Days	
		kN	N/mm^2	kN	N/mm^2
1	S1	156	6.44	166	6.85
2	S2	164	6.77	178	7.35
3	S3	178	7.35	198	8.10
4	S4	160	6.61	170	7.02
5	S5	172	7.10	186	7.68

Water Absorption Test results-

S.No.	Specimen (Burnt Bricks)	Observations		
		Mean Dry Weight, W1 (kg)	Mean Moist Weight, W2 (kg)	Water Absorption (%)
1	S1	2.876	3.254	13.1
2	S2	2.720	3.117	14.5
3	S3	2.729	3.121	14.3
4	S4	2.732	3.140	13.0
5	S5	2.826	3.212	13.6

☐ Variation in results of Water Absorption of Burnt bricks at28

Efflorescence Test results

S.No.	Specimen(Burnt Bricks)	Observations
1	S1	Slight
2	S2	Slight
3	S3	Slight
4	S4	Slight
5	S5	Slight

Warpage Test Results

S.No.	Concave (mm)	Convex (mm)
S1	3.7	4.78
S2	4.61	4.81
S3	4.69	4.87
S4	4.72	4.95
S5	4.98	5.10




CONCLUSIONS

1. Initially with increase in percentage of DW the compressive strength of brick increases upto its 50% DW addition, but beyond that i.e. on 55% and 60% addition of DW this value goes ondecreasing.
2. Best results for compressive strength for Dolomite burnt bricks is achieved in specimen (S3) having 50% Dolomite waste, 10% clay, lime, cement., and sand.
3. The water absorption capacity of Dolomite brick (S3) is reduced by 72% as compare to the maximum water absorption specified for third class brick in IS code. Water absorption of Dolomite brick decreases as the Dolomite waste percentage increases in thebricks.
4. Weight of Dolomite brick is 15% less as compared to conventional clay brick as specified in IScode.
5. Efflorescence in Dolomite brick is slight which indicates the brick is nearly free from salts. It has also pleasing appearance and gives good finishing to the masonry wall without salt deposition over itssurface.
6. Warping in bricks is minimum which implies that they will bind properly with mortar and will consume less mortar. It will impart good appearance tostructure.

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APPLICATION OF COCONUT FIBRES FOR INORGANIC CLAY OF INTERMEDIATE PLASTICITY

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ABSTRACT

Many a times, structures are to be constructed at sites which have soil of poor engineering properties. In such cases, engineers need to rely upon means of improving soil characteristics rather than replacing the poor soil as the latter option may not be viable one. Soil stabilization process is commonly used for improvement in desired properties of soil. Number of soil stabilization process has been developed and used for soil improvement. One of these methods is to mix fibres in soil that does not meet desired properties. The fibres in soil may be mixed in the form of layers with same orientation or may be randomly mixed. The fibres generally used are of synthetic. However, this experimental study deals with application of coconut fibers, a waste material, for improvement in engineering properties of inorganic clay of intermediate plasticity. Effect of (i) adding coconut fibres and (ii) length of coconut fibres is observed on clay properties namely (1) Optimum Moisture Content (OMC)/ Maximum Dry Density (MDD); (2) Shear parameters (c and φ); (3) Unconfined compressive strength (UCS); and (4) CBR values (soaked and unsoaked). Detailed experimental investigation was carried out through Standard Proctor test; Tri-axial shear test; Unconfined Compression Strength test and CBR tests (both soaked and unsoaked), performed on specimens of the clay before and after mixing of coconut fibres. It is observed that addition of coconut fibres leads to improved values of desired engineering properties. Thus, use of coconut fibres for stabilization of such weak soil, can be considered an eco-friendly method as it has potential for disposal of this waste material.

KEYWORDS: Inorganic Clay; Intermediate Plasticity; Engineering properties; Stabilization; Coconut fibres

INTRODUCTION

Increasing need of Infrastructure resulted into large budget allocation towards construction of roads. These roads are being made for improving transportation facilities in India through different Government schemes. Presence of soils, with poor engineering properties, significantly influences the infrastructure development of the region. The properties of weak soil are generally improved adopting soil stabilization methods. Some of the stabilization methods need addition of materials which decreases the compressibility and also enhances the strength of weak soils. Reinforced earth construction has become a viable technique for formation of subgrades of roads, railways, base of retaining walls etc. Of late, researchers has worked on addition of fibres, mostly the synthetic fibres viz. nylon, polypropylene, polyster etc. for improving the desired properties. These fibres may be laid in layers at certain orientation or mixed randomly in weak soil. Fibres, randomly mixed with soil, is one of the latest techniques which can result into improving engineering properties of soils uniformly in all the directions. However, the additional cost due to mixing of such synthetic fibres may make the stabilization process non-viable. Application of natural fibres like coconut fibres and jute fibres may be suitable replacement of synthetic fibres (Maity et al. 2018). Coconut is grown in most of the countries. India itself has large annual production of about coconut nuts

that are used by industries for different coconut based products. However, coconut husk and shell is also contributing to the pollution problem as it takes very long time for biodegradation. Application of this waste material for construction purpose may help in solving its disposal problem and thus may be considered as an eco-friendly solution. Use of waste coconut shell is being explored for concrete production (Kanojia and Jain, 2017). Also, use of coconut fibers has been proposed for improving certain desired properties of medium strength concrete (Khan and Ali, 2018). The major aim of this study is to determine the amount of improvement in properties of locally available inorganic clay of intermediate plasticity by randomly mixing coconut fibres. In this experimental investigation, Standard Proctor test; Unconfined Compressive Strength (UCS) test; Tri-axial test; and CBR test (both Soaked and Unsoaked) are performed on specimens of the inorganic clay of intermediate plasticity, before and after mixing the coconut fibres.

CHARACTERIZATION OF THE CLAY

The soil used in this experimental study was collected from Gwalior city, State of Madhya Pradesh. Grain size analysis of the soil sample was carried out [IS:2720 (Pt. 4)]. Percentage passing through various Indian Standard (IS) sieve size is shown in Table 1.

Table 1: Gradation of the Soil Sample

IS sieve size (μm)	4750	2360	850	600	425	212	150	75
Percentage passing	99.8	95.6	92.6	92.4	90.8	88.4	88.0	85.0

The above gradation shows that the soil is fine grained. Thumb penetration test indicates that the soil is cohesive in nature (Fig. 1). A series of tests were conducted on the soil for its classification and finding its Index and Engineering properties. The procedures followed are as prescribed by Indian Standards. The properties of the clay, as determined in the laboratory are summarized in Table 2.

Location of the soil on the Plasticity Chart (IS 1498-1970) is shown in Fig. 2. It is slightly above the A-line. The figure shows that the soil may be classified as Inorganic clay of intermediate plasticity.

Table 2: Index and Engineering Properties of the Clay

Property	Value
Liquid Limit	41%
Plastic Limit	23%
Unconfined compressive strength, c	1.03 kg/cm^2
Angle of internal friction, ϕ	8.35^0
Optimum Moisture Content	13%
Maximum Dry Density	1.89 g/cc
California Bearing Ratio at OMC (soaked)	3.72%
California Bearing Ratio (unsoaked)	7.32%



Fig. 1: The soil sample collected from Gwalior region

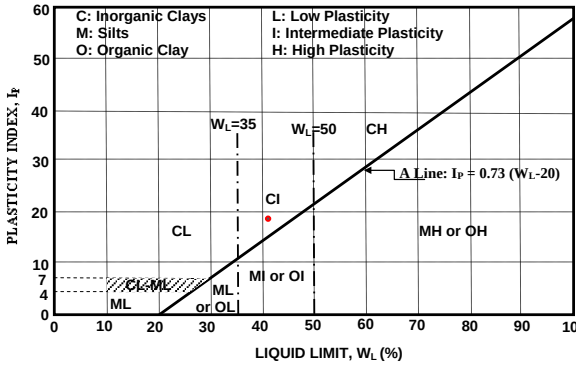


Fig. 2: Location of the Clay in the Plasticity Chart (Marked by•)

PROPERTIES OF COCONUT FIBRES

Coconut fibres are generally obtained by dehusking coconut and separating it by process of retting. Coconut fibre is generally of length ranging between 15cm to 28cm and diameter varying between 100µm to 500µm. It has been reported that for coconut fibres of diameter 100µm to 450µm, modulus of these fibres is 3.7 GN/m^2 to 5.8 GN/m^2; the ultimate tensile strength of these fibres may be as high as 106 N/mm^2 to 175 N/mm^2 and percentage elongation at break may be up to 17.8% to 47.3%. The modulus decrease with increase in diameter of the fibre; while the strength increase up to fibre diameter of 200 µm, after which it becomes practically constant [Satyanarayana et al. 1981]. Biodegradation of coconut fibers is a slow process and it takes centuries for biodegradation of these fibers. Also, among natural fibers, coconut fibers have the highest toughness (Khan and Ali, 2018).

PREPARATION OF TEST SPECIMENS

The shells of used coconut were brought from local temples. The coconut husk was removed manually from the shell. Collected coconut fibres was uniformly sun dried and then cleaned from dust. Then, they were segregated and cut into uniform sizes of length 10mm and 20 mm (Fig. 3). The experimental study was carried out for three conditions (i) clay without fibres; (ii) clay with coconut fibres of 10mm length and (iii) clay with coconut fibres of 20mm length respectively. Quantity of coconut fibres was kept as one percent of the dry weight of soil. The coconut fibres are than mixed manually in the clay with proper care at each stage of mixing for achieving uniform distribution of the fibres into the clay (Fig. 4).



Fig. 3 Preparation of coconut fibres



Fig. 4 Mixing of coconut fibres with clay

EXPERIMENTAL SETUP

This experimental study aims to investigate the influence of mixing coconut fibres for improvement in engineering properties of inorganic clay with intermediate plasticity. Effect of (i) adding coconut fibres and (ii) length of coconut fibres is observed on the clay through

various properties namely (1) Optimum Moisture Content (OMC)/ Maximum Dry Density (MDD), (2) Unconfined compressive strength, (3) Shear strength parameters (c & □□ values, and (4) CBR values (soaked and unsoaked). The properties are determined for the three cases namely (i) clay without fibres; (ii) clay with one percent coconut fibres of length 10mm; and (iii) clay with one percent coconut fibres of length 20mm. The test setups for different tests are shown in Fig. 5.

Optimum Moisture Content (OMC) and Maximum Dry Density (MDD) is estimated using Standard Proctor test as prescribed by Indian Standards [IS:2720 (Pt. 7)]. The values of OMC and MDD are determined for the above three cases. UCS test were performed to determine the unconfined compressive strength values. The test were conducted for all the three cases, following the method prescribed in Indian Standards [IS:2720 (Pt. 10)]. Tri-axial test, as recommended by Indian Standards [IS:2720 (Pt. 11)], was used to find out the shear strength parameters c and □ values for t were conducted following the procedure prescribed by Indian Standards [IS:2720 (Pt. 16)], to find out the CBR values for the three cases.



(a) Standard Proctor Compaction Test



(b) Liquid Limit test



(c) Tri-axial Shear Test

(d) Test for CBR Value

Fig. 5 : Experimental Setup and Equipment used

RESULTS AND DISCUSSION

With the aim of improving engineering properties of inorganic clay of intermediate plasticity, a systematic experimental investigation was undertaken to study the effectiveness of mixing coconut fibres available in plenty in these area. The results obtained on the basis of this experimental study, are shown in Table 3.

As observed from Table 3, the MDD of the clay decreases due to the addition of coconut fibres. Randomly mixing of 10mm long coconut fibers, 1% of dry weight of clay, results into

12.6% decrease in MDD and 4.4% increase in OMC. The decrease in dry density with the increase of fibre content is due to less specific gravity of the coconut fibres in comparison with the soil solids. The OMC value of clayey soil increases due to addition of coconut fibre. This may be due to the fact that the natural fibres have more water absorption capacity as compared to the clay. It is observed that the OMC and MDD values for the clay mixed with 10mm long coconut fibers decreases slightly when length of coconut fibers mixed is 20mm in place of 10mm long fibers.

Table 3: Effect of mixing coconut fibres on engineering properties of the clay

Soil	Proctor test values		CBR Value (%)		UCS q_u (kg/cm^2)	Shear parameters	
	OMC (%)	MDD (gm/cc)	Soaked	Unsoaked		c (kg/cm^2)	φ
Clay	13.6	1.90	3.72	7.32	2.11	0.83	7.20^0
Clay with 1% coconut fibers (10mm long)	14.2	1.66	4.39	8.19	2.45	1.26	8.35^0
Clay with 1% coconut fibers (20mm long)	14.0	1.55	4.19	7.98	2.58	1.41	6.33^0

There is an increase in both unsoaked and soaked CBR value for clayey soil when mixed with randomly distributed coconut fibres. Randomly mixing of 10mm long coconut fibers, 1% of dry weight of clay, leads to 18.0% increase in soaked CBR values and 11.9% increase in unsoaked CBR values. It is also observed from Table 3 that both, unsoaked and soaked, CBR values are more for fibre length of 10 mm as compared to fibre length of 20mm.

Table 3 shows that there is a considerable increase in the UCS value for the clay when mixed with the coconut fibers. The increase in UCS value is 16.1% when the clay was mixed with 10mm long coconut fibers for quantity 1% of dry weight of the clay. Further increase of 6.2% was observed when the 10mm coconut fibers got replaced by 20mm coconut fibers.

The results obtained from Tri-axial tests shows that the Shear strength parameters, c and φ, of the clay increase significantly when mixed with 1% coconut fibers of 10mm length. The increase in c value is about 50% and φ is 16.1%. The increase in length of fibers to 20mm leads to further increase in c value by 18.1% but decrease in the angle by 28.2%.

Thus, from the discussion, it can be inferred that addition of coconut fibers in inorganic clay of intermediate plasticity can be considered as good ground improvement technique specially in engineering projects founded on such soils.

CONCLUSIONS

In case, the engineering properties of soil at construction site are not matching with the desired standards, it becomes imperative to use soil improvement techniques. With this aim, a systematic experimental study was undertaken to investigate the efficacy of mixing waste coconut fibres randomly in the clay, to improve the desired properties of inorganic clay of intermediate plasticity. Though the scope of the study is not wide, following general conclusions may be drawn -

1. The OMC value of the clay increases and MDD value decreases when coconut fibres are mixed randomly in the clay.
2. There is an increase in both unsoaked and soaked CBR value of the clay soil when mixed with the randomly distributed coconut fibres. CBR value for the clay with longer coconut fibres is less as compared to the clay mixed with smaller fibres.

3. There is an increase in the UCS value of the clay due to mixing with the coconut fibres. The increase is more for coconut fibres of length 20mm as compared to fibres of length 10mm.
4. Both the shear parameters, c and φ of the clay increases significantly due to addition of 10mm long coconut fibres. However, only c value of the clay increases for 20mm long fibres when compared with 10mm long fibres, whereas φvalue decreases.

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OPTIMIZATION OF GROUND GRANULATED BLAST FURNACE SLAG CEMENT CONCRETE MIX

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ABSTRACT
The aim of this study is to formulate a numerical model using Mixture design of experiment technique and optimize the properties of GGBS-Cement concrete. Concrete properties such as mechanical strength, workability, durability, permeability etc. vary with the variation in the proportions of material in mixture. Therefore through experimental data a function can be developed which would help in optimization of concrete properties. Cement being a major constituent of concrete is widely used in construction area. But its production carries the burden of CO_2 emission, which hinders the sustainability. The use of industrial by-product such as GGBS, Silica fumes, Rice husk ash, Fly ash, High Reactive Metakaolin etc. as partial replacement for OPC, not only help to counter environmental problems but also increases the compressive strength, durability and decreases the permeability. In this study GGBS is used as partial replacement for cement to design the concrete of grade M40. The mix proportions of the concrete are statistically designed and its responses are obtained from lab0ratory experiments. Using mixture approach of design of experiments the responses are formulated as function of constituents of concrete.

KEYWORDS: Modelling, Optimization, GGBS, Design of experiment, Trial mix batch, Response surface methodology, etc.

INTRODUCTION: The strong structures witnessing the civil engineering expertise are artefact of concrete in which the major constituent is cement. Unluckily cement production leads to emission of carbon dioxide, a major greenhouse gas responsible for global warming. Cement industries account almost 5% of global emission of CO_2, of which 50% is from chemical process when calcium carbonate is thermally decomposed, 40% from burning of fuels for energy and rest 10% from transportation.The emission of CO_2 is directly proportional to cement used in concrete. Increase in utilization of cement will eventually increase CO_2 emission. Therefore the concern of environmental degradation has initiated many research works to discover an alternative or a supplementary material as adequate replacement of cement, which would not only be sustainable and but also eco-friendly.

LITERATURE REVIEW
Simon M.J. et al. (1997) investigated the optimization of concrete mix through statistical mixture design method. They stated that high performance concrete are mixture of many elements and it's complicated to optimize these mixture under several performance constraints. The paper described the feasibility to optimize concrete mixture through statistical experiment design and analysis method. The use of statistical mixture experiment design and analysis method was widely applied in many industries which produced gasoline, detergents, medical drugs and foods but in concrete industries it was hardly applied. The mixture design experiments were particularly developed to optimize concrete mixes in which relative composition of elements play a major role in determining the ultimate responses of

the mix. The study was concentrated on optimizing 6 component concrete mixture exposed to various performance constraints, using statistical experiment design and analysis method.
Marinela B. and Daniel L. (2008) utilized statistical approach in analysing data from mixture design experiments and developed regression models to calculate the effect on responses according to the variation in factors of polymer concrete. The concrete was prepared from epoxy resins, silica fumes and aggregate and their combination was designed through mixture design theory. They studied and analysed the effect of each combination on the properties of polymer concrete, through response surface methodology. The reported results were sufficiently executed in few runs.

METHODOLOGY

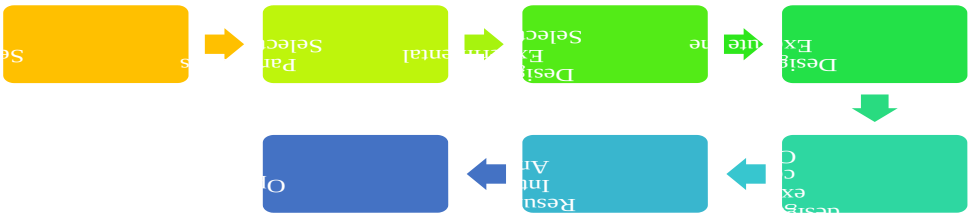


Figure 1: Steps of design of experiments

Extreme Vertices Designs

A mixture design in which only a small portion of the simplex is covered. This small space within the simplex is the result of lower and upper bound under which the proportion of ingredients are restricted. For instance a pancake is a mixture prepared by proper addition of several ingredient. The taste of pancake depends on the specific amount of ingredient which must be optimum for good tastes. The ultimate objective of extreme vertices design is to allow the experimenter to work over an adequate domain.
For instance, concrete is a mixture of 3 ingredient i.e. water (w), cement (c) and aggregate (a), here ‘w’, ‘c’ and ‘a’ respectively denotes the proportion of water, cement and aggregate volume. The sum of these proportion is one and domain of this mixture is a simplex. The vertex of triangle symbolises the pure ingredient like vertex ‘w’ is pure water mixture, vertex ‘c’ is pure cement mixture and vertex ‘a’ is pure aggregate mixture. The point at which 3 vertices join when extended is the centroid with $(\frac{1}{3},\frac{1}{3},\frac{1}{3})$. The entire region can also be utilized to investigate the characteristic of experiment by choosing simplex centroid design shown in Figure 2.



Figure 2: Simplex lattice and simplex centroid design for three components

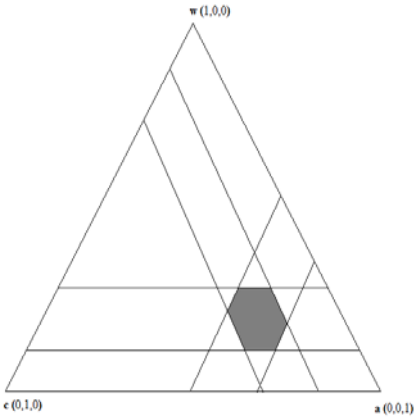


Figure 3: Extreme vertices design for 3 components

In centroid design all point like vertices, midpoints and centroid are included in design. But in entire region all point won't be feasible or practical to conduct the experiments.

Because all the points in the domain are either not feasible or practical, the small region within the triangle accumulating the practical point is designed restricting the value of 'w', 'c' and 'a'. This design is a different case than simplex design and is called extreme vertices design (Figure iii shows the extreme vertices design).

After region is defined, the design points are utilised to collect the results or responses. These responses are modelled as function of the components. From any forms of function normally polynomial function is used for models due to ease in working with it.

The linear model for three components is:

$$f(w,c,a) = \beta_0 + \beta_1 w + \beta_2 c + \beta_3 a + e$$

whereβ are constants and e is error term. The model is modified in mixture experiment since $w + c + a = 1$, therefore:

$$f(w,c,a) = b_w w + b_c c + b_a a + e$$

where$b_w = \beta_0+\beta_1, b_c = \beta_0+\beta_2$and $b_a = \beta_0+\beta_3$ using $\beta_0 = \beta_0*(w+c+a)$[Scheffe linear mixture polynomial].

Similarly, the quadratic model is developed and then with help of Analysis of variance (ANOVA) and least square methods model fitting and validation are done.

$$f(w,c,a) = b_w w + b_c c + b_a a + b_{wc} wc + b_{wa} wa + b_{ca} ca + e$$

In thid present study, scheffe's quadratic model for six variables (materials) is reduced to second degree polynomialand used to design numerical model for responsesas a function ofmaterial proportions.

OPTIMIZATION

After developing a fitting model, optimization of more than a few responses can be done concurrently. There are many techniques through which optimization can be conducted. Generally graphical and numerical methods are applied to optimize responses.

In numerical optimization, the desirability or objective functions are defined according to which the responses are optimized. One can use desirability approach for optimization. It is a method widely used in industries for quality optimization. In desirability approach certain value is assigned to the responsesand that combination of factors or propoetions is selected which maximise the assigned value. The individual desirability (d) defines the effect of proportions in optimization of a particular responses while composite desirability (D) define effect of proportion in optimization of a group of responses.

$$D = (d_1 * d_2 * d_3 * \ldots * d_4)^{\frac{1}{n}}$$

MATERIALS

For this study Ordinary Portland Cement (OPC) of grade 43, manufactured by Birla Cement is used in all mixes, all through the experiments. For physical properties, the cement is tested according to Indian standard codes and its results are listed in Table (ii). The ground granulated blast furnace slag is brought from Jindal Steel and Power Limited, Raigarh, India. Physical properties of GGBS in given in Table 1.

Table 1: GGBS	
Parameters (% by mass)	Results
Total loss of ignition	Nil
Insoluble residue	0.35
Sulphuric anhydride	0.45
Magnesia(MgO)	8.2
Silicon Oxide (SiO_2)	34.12
Aluminium oxide (Al_2O_3)	18.95
Calcium oxide (CaO)	35.46
Ferric oxide (Fe_2O_3)	0.23
Sulpher	0.76
(Source: Jindal Steel and Power Limited)	
#required minimum 275 m^2/kg as per BS 6699:1992)	

Table 2: OPC 43 grade	
Characteristics	Test Results
Specific gravity	3.15
Standard consistency	
Initial setting time	98 minutes
Final setting time	414 minutes
Compressive strength (7days)	36.2N/mm^2
Table (iii) : Physical properties of GGBS	
Characteristics	Results
Colour	Off white
Specific Gravity	2.9
Fineness	387# m^2/kg
Initial Setting Time	175 minutes
Residue (90µ)	0.86
Residue (45µ)	4.12
Compressive Strength (7 days)	25.12

Table 4: Properties of Coarse Aggregate,	
Property	Results
Size of aggregate	20 mm
Impact value	14.61%
Specific gravity	2.65
Water absorption	0.503%

Table 5: Properties of Fine Aggregate	
Property	Result
Zone	III
Fineness modulus	2.37
Specific Gravity	2.63
Water Absorption	0.91%

Conplast SP 500 is used as super plasticizer admixture manufactured by Fosroc. The admixture is based on Sulphonated Naphthalene Polymers to produce high performance workable concrete. It was capable of reducing water up to 25% with same workability.

ANALYSIS

This study is conducted using two method, i.e. analytical method and experimental method. Through analytical method, every run and the proportion of each run is varied randomly. Based on the obtained run, concrete samples are prepared and tested. Preparation and testing are done under experimental method.

Analytical Method

Analytical method involves the mixture design experiments to optimize the characteristics of GGBS concrete. The total mass or volume of mixture in mixture design is constrained to one. In same manner the batching of concrete is done for volume of 1 m^3.

Mix Proportioning and Constraints

With the help of mix design the materials are proportioned and its range of maximum minimum are defined. Initially the proportions of all 6 materials are taken as volume and later transformed into weight for batching. The maximum and minimum proportions by volume and weight are given in Table 4.6.

In this design the scheffe's quadratic model for 6 materials is used for each property. Total 31 runs were conducted to evaluate the model and out of total runs, 21 runs are required for term in the model, 5 runs are replicated to avoid uncertainty and 5 runs are added into experiment to check adequacy of the model.

Table 6: Maximum and Minimum proportions of material

Content	Maximum		Minimum	
	Mass (kg)	Vol. (m^3)	Mass (kg)	Vol. (m^3)
Cement	304.6105	0.0967	252	0.0800
GGBS	130.5474	0.0450	108	0.0372
Water	165.36	0.1654	136.8	0.1368
SP	4.351579	0.0035	3.6	0.0029
CA	893.2377	0.3371	828.7684	0.3127
FA	1067.861	0.4060	990.788	0.3767

Through commercial statistical software the experiments are designed and analysed. From the processed list of design points, 31 points are selected for experiments. Table 4.7 summarizes the mixture proportions selected for experiments.

RESULTS

Table 7: Optimization for compressive strength (optimal value of proportion in m^3)

Cement	GGBS	Water	Superplasticizer	Coarse aggregate	Fine aggregate
0.0967	0.0447	0.1742	0.0035	0.420	0.2709
Compressive strength 48.6 MPa					

Table 8: Optimization for Compacting factor (optimal value of proportion in $m^{3)}$

Cement	GGBS	Water	Superplasticizer	Coarse aggregate	Fine aggregate
0.080	0.0448	0.1654	0.00345	0.4369	0.2694
Compacting factor 0.905					

CONCLUSIONS

Theoptimization value of compressive strength and workability of M40 cement concrete blend with GGBS are determined through laboratory experiments and obtained results are then used to formulate response models of respective properties and finally optimization of GGBS cement concrete mix is done through mixture design of experiment.

The concrete was presented for 31 mixes, obtain through statistical software, varying proportion of each material in each mix and thereby varying water to binder ratio from 0.31 to 0.46 and replacing cement by GGBS from 26% to 34% of total cementitious material. The concrete mix proportion for optimum response is C:G:W::FA:CA = 1:0.37:0.56:3.81:2.53 determined. The optimum compressive strength of GGBS concrete is obtained when GGBS replaced cement by 30% of total cementitious material is 48.66 Mpa, slump 58 mm and compacting factor 0.883.

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ANALYSIS OF A BUILDING FRAME CONSIDERING WIND PRESSURE UNDER LATERAL LOAD RESISTING SYSTEM USING ANALYSIS TOOL ETABS

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ABSTRACT
The modern era construction has led to the use of post-tensioning method in every projects whether commercial or residential sectors. Looking over the flaws of the conventional method of construction, post-tensioning being an upgrade can overcome the deficiencies of the conventional method of construction, for example, more slender structural members, lighter in weight and smaller floor-to-floor heights. Post-tensioning assures every plan to quite economical and safe.
In this study we are considering a G+20 tall structure under wind pressure using post tensioning cables as a lateral load resisting member and comparing it with X typre bracing structure to determine its effectiveness over bracing system.

KEYWORDS: Structural Analysis, post tensioning cables, displacement, moment, forces, cost, stability.

INTRODUCTION
Generally a High rise structure have to resist forces created because of wind and earthquakes, but the designing of the structure varies for resisting both the loads. Earthquake forces hampers the structure at the base level whereas, nifty wind breezes affects the exposed area of the structure, which is termed as force-type loading. A raised structure is a tall construction, rather than a low-rising amassing and is described by its stature unmistakably in various areas. It is used as a private, spot of business, or various limits including hotel, retail, or with different purposes joined. Private raised structures are generally called apex squares and maybe insinuated as "MDUs", signifying "multi-staying unit". A tall raised structure is implied as an elevated structure.
Tall structures ended up possible with the production of the (lift) and progressively moderate, progressively endless structure materials. The materials used for the assistant course of action of tall structures are fortified concrete and steel. Most North American style tall structures have a steel layout, while private squares are ordinarily worked of concrete. There is no unquestionable differentiation between an apex square and a skyscraper, regardless of the way that working within any event fifty stories is generally viewed as a high rise.

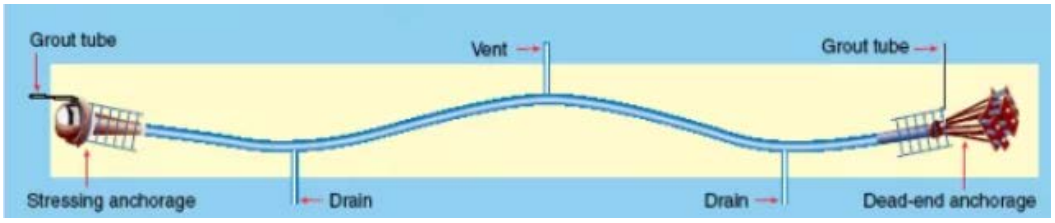


Fig 1: Post tensioning cables

WIND ANALYSIS
Wind approaching a building is a complex phenomenon where the flow pattern generated around a building is complicated by the distortion of the mean flow, the flow separation, the vortex formation, and the wake development. Large wind pressure fluctuations occur on the surface of a building due to the above mentioned characteristics. As a result, large

aerodynamic loads are imposed on the structural system and intense localised fluctuating forces act on the facades.
The aim of this study is to determine the stable method of lateral load resisting structure, here we will assign post tensioning cables at the outer edges of the building to determine its impact over the building under lateral forces.

LITERATURE SURVEY

VeeratSrilaxmiet. al. (2018) the author channelized the present days the pre-tensioning and post-tensioning frameworks are prevalently utilized in the significant developments of a structure. The Pre-tensioning and Post-tensioning the two strategies are utilized under the pre-focusing on the procedure. In which has few edges over the standard non-focused on structures like more noteworthy range to profundity proportion, higher minute and shear limit. These techniques were commonly received taking shape of PSC braces, sleepers, Bridges, Slabs in structures, Concrete Pile, Repair and Rehabilitation, Nuclear Forces Plant and so forth
Among the sorts of cement's, for example, by and large, typical concrete, strengthened bond concrete, pre-stressed cement was the best concrete for acquiring much quality in the significant developments of a structure and furthermore for getting more life expectancy of a structure.
Sridhar and Rose (2019) author looked at the trial results on the flexural conduct of post-tensioned solid pillars with fortified framework testing Four rectangular post-tensioned and further investigated the equivalent. The shafts were tried under single point monotonic stacking condition and two points monotonic stacking condition. The heap diversion conduct, stress-strain conduct and split examples are exhibited from the test outcomes. Post-pressure framework viably controlled redirection and split because of the nearness of ligaments notwithstanding the strengthening steel. All the four test results were looked at and they indicated great outcomes over the traditional bars.
The outcomes expressed that the prestressed solid pillar with high PT forces (64kN and 42kN) accomplished the most extreme load when contrasted with different shafts under two-point stacking and single point-stacking condition individually. The splits began to show up precisely under the stacking focuses, which unmistakably demonstrated that the bars bombed under flexure.
The redirection rate diminished by 24.17% when the PT forces is kept up at 150kg/cm2 contrasted with 120kg/cm2under two-point stacking condition and it diminished by 8.8% when PT forces is kept up at 100kg/cm2 contrasted with 70kg/cm2 under single point stacking condition.
Singh et. al. (2018) author accentuated on structuring a post-tensioned structure utilizing ETABS and SAFE. ETABS represents Extended Three-Dimensional Analysis of structure frameworks. The fundamental reason for this product is to plan the multi-storeyed structure in a precise procedure which will pursue Indian Standard structure codes.
The author venture managed the arrangement of tremor and wind opposition structure where the Minimum sizes of segment and bar gave were C500*500 and B300*500 and later Seismic investigation was finished by utilizing ETABS programming where whole individuals were passed in the plan. As the structure was posted tensioned one, it demonstrated to be efficient.

Outcome of the study are:

The writing audit has proposed that the utilization of a limited component displaying the structure. So it has been chosen to utilize ETABS for the Finite Element Modeling. With the assistance of this product investigation of structure, the structure has been finished. ETABS

additionally helps in Finite Element Modeling in perspective on that extraordinary kind of forcess can apply to get genuine outcomes.

OBJECTIVES

The primary objectives of the investigation are as per the following:

1. To decide the capacity of post-tensioning cables in a tall structure as a lateral load resisting members.
2. Estimation and costing of material according to S.O.R.
3. Dynamic investigation of the tall structure considering wind pressure.
4. Utilization of Advance diagnostic application ETABS for examination of horizontal load opposing structure.
5. To set up a reference study for the usage of post-tensioning and Bracing members in the Indian district according to wind pressure as per I.S. 875-III 2015.

METHODOLOGY

Step-1: Modelling of Selected plan in etabs.

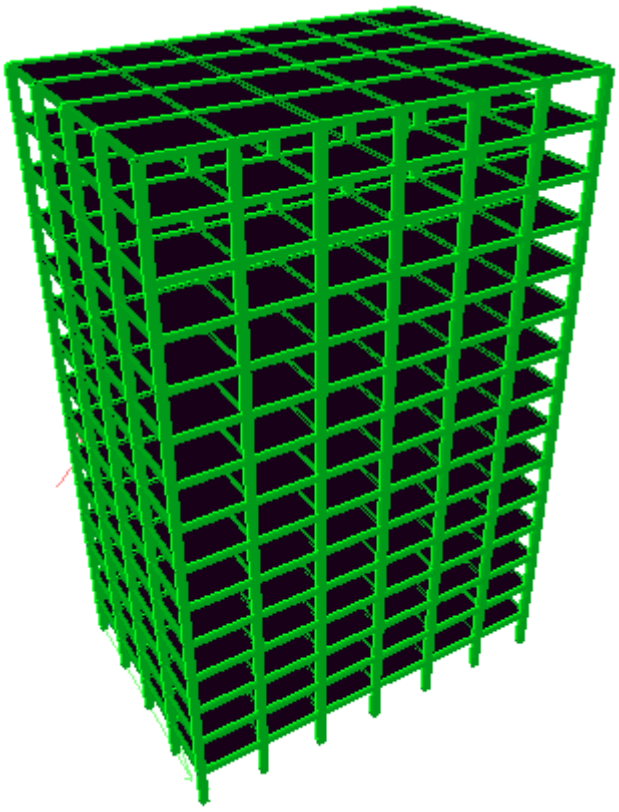


Fig 2: Bare Frame

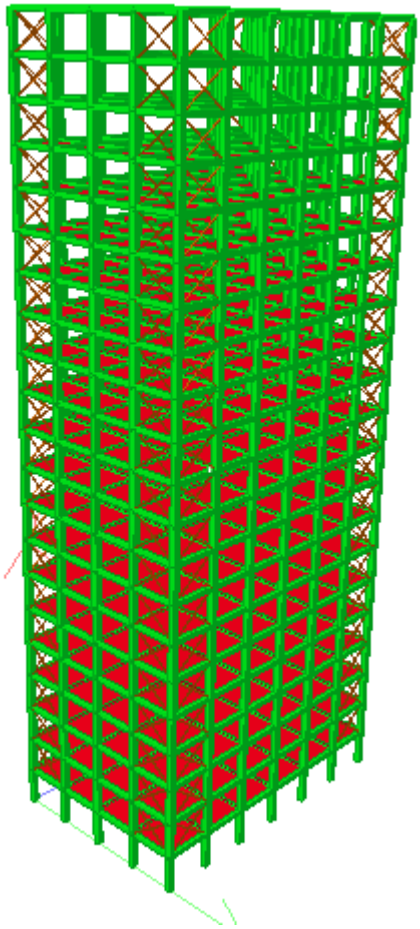


Fig 3: Post tensioning frame

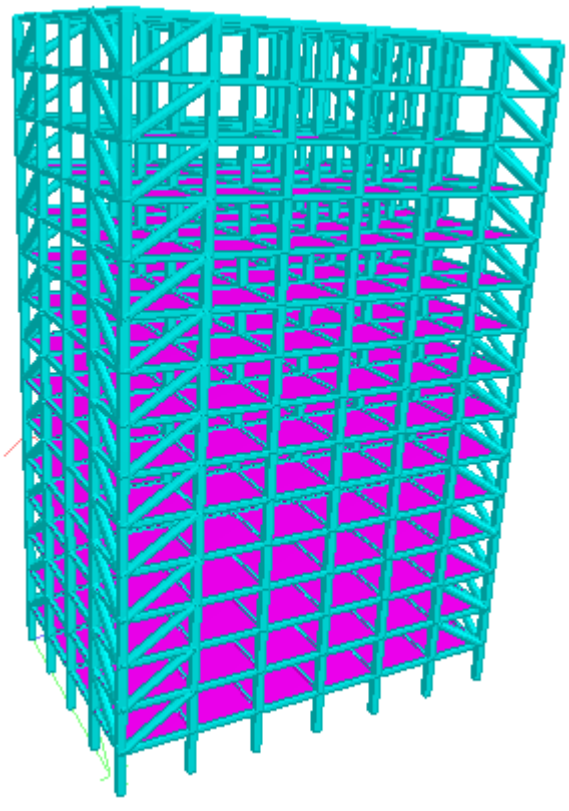


Fig 4: X bracing Frame

Step-2: Defining and assigning property.

Step-3: Calculating load conditions i.e. dead laod, live load and wind pressure as per I.S. 875-III.

Step-4: assigning post tesnioni member in post tensioning case. Bracing system in bracing case.

Step-5: Analyzing the structure considering Finite element method.

Step-6: Preparing comparative study.

Table 1: Geometrical details

Design data of building	Dimension
Plan dimension	30× 20 m
No. of bay in X direction	6 Bay
No. of bay in Y direction	4 Bay
No. of storey	G + 20
Typical storey height	3.5 m
Bottom storey height	2.0 m
Column size	500 × 500
Beam size	350 × 300
Thickness of slab	125 mm
Grade of concrete	M-25
Grade of steel	Fe-415
Wall thickness	100 mm for external wall
Post tensioning wire	600 mm diameter cable
X Bracing	I.S.A. 150 × 150 × 15 mm

RESULTS & DISCUSSIONS

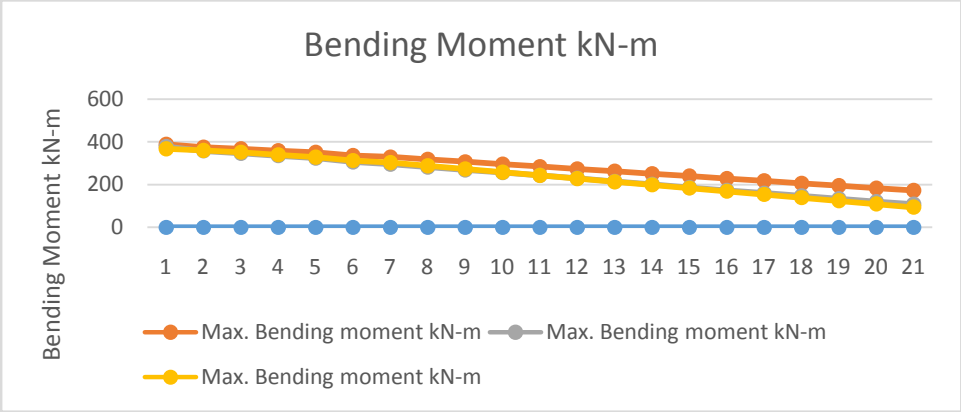


Fig 5: Max. Bending Moment kN-m

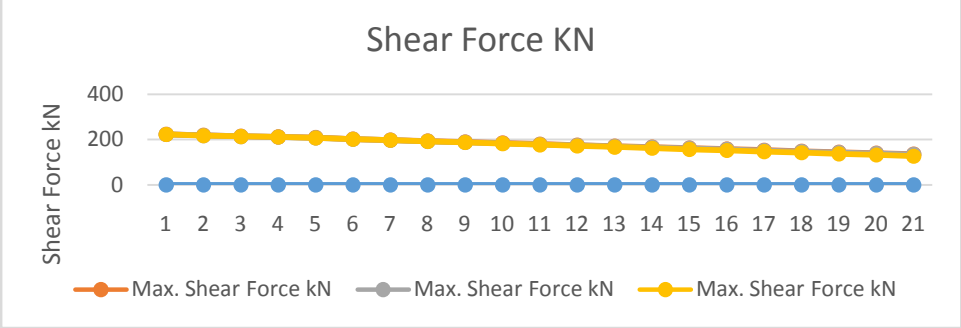


Fig 6: Max. Shear Force kN

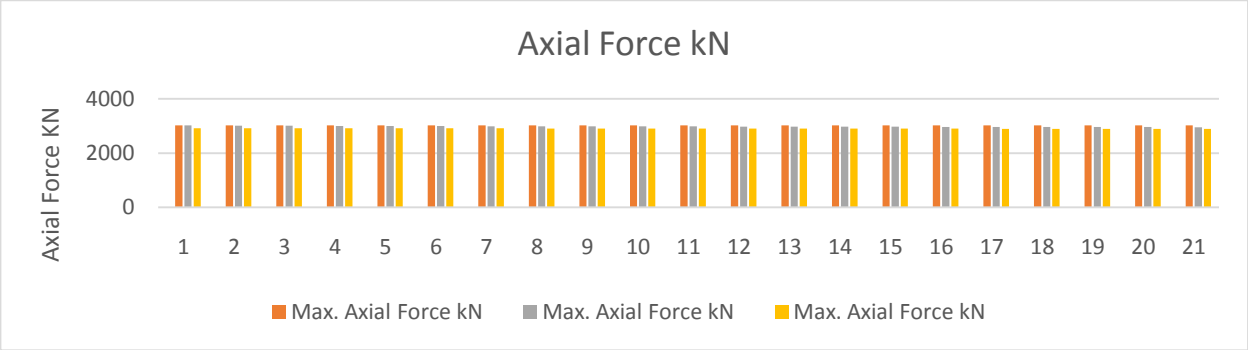


Fig 7: Axial Force

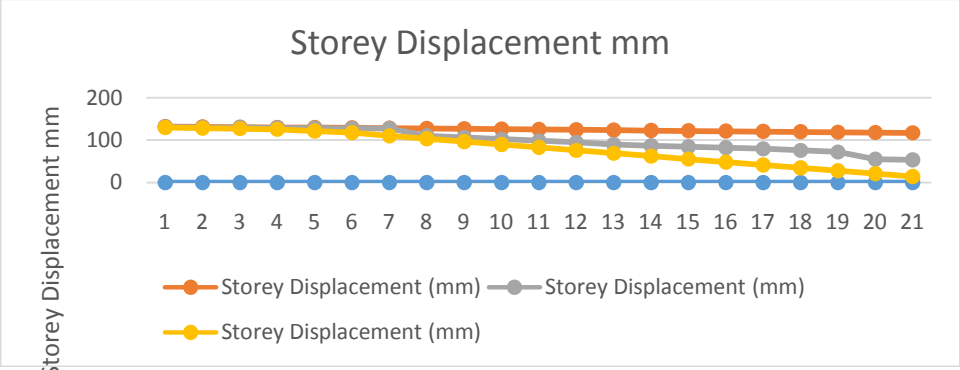


Fig 8: Max. Displacement

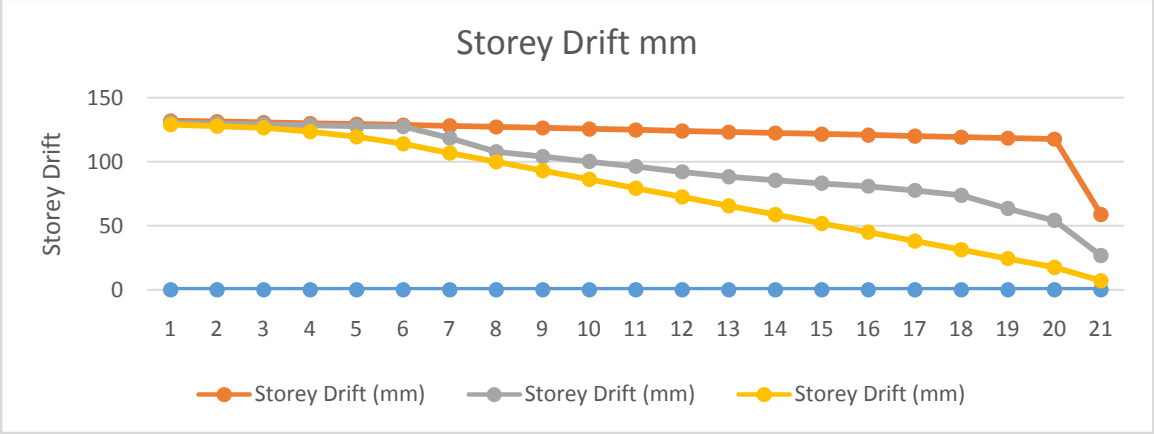


Fig 9: Storey Drift

Table 2: Cost Analysis

S . No.	Type of frame	Qty. of concrete	Qty. of reinforcement Kg	Concrete rate/cu.m	Reinforcement rate/kg	Cost of concrete	Cost of reinforcement
1	Post tensioning	1700.45	15900	3500	48	59,51,575	763200
2	Bare Frame	1830.44	17850	3500	48	64,06,540	856800
3	Bracing Frame	1783.54	16370	3500	48	62,42,390	785760

CONCLUSIONS

The Study presents the consequences of executing the post-tensioning cables in the structure. It has been observed in above chapter that post tensioning cable is resisting lateral displacement within the permissible limit as per I.S. 1893:I:2016.

In this examination following outcomes are seen:

1. The bending moment in every story decreases because of the post tensioning cables, these members helps in resisting overall failure of the structure. In this investigation, it was observed that 47.7% decrement in tall structure is seen with post tensioning cable, which will cause economical sections since bending moment is directly proportional to area of steel.
2. Storey shear is said as the forces generating on individual level of structure due to lateral forces.It is seen in the above chapter that storey shear is observed 13.87% less in post-tensioning structure, which results in rigid structure.
3. Storey displacement is said as the displacement in X & Z direction due to lateral forces which may cause structure failure. In the above chapter it is observed that with the effect of P-delta analysis which cause non linear static effect on edges and outer part of the structure, post tensioning cales are capable of resisting load 18.09% more than general structure which is important as per safety and stability of a high rise structure.
4. Axial forces are the distribution of load in vertical members this is the distribution of live load through beam to column, this is considered as the pressure to be consider while designing column for bearing such load, due to lateral forces the distribution of vertical pressure disturbs which causes uneven distribution which may cause buckling of columns. In the above chapter it is observed that due to more staility in post tebsion cable structure distribution of vertical pressure is even and 12.20 % les due to stability.
5. Storey Drift is the average displacement of two consecutive floors which is require to justify the displacement graph at individual level. In the above chapter it is seen that post-tensioning structure brings about a 19% decrease in relative displacement of the structure in comparison to general structure.
6. In terms of Cost, It is observed that post tensioning structure is 17% more economical in terms of material (concrete and rebar) due to less bending moment.

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SUSTAINABILITY OF LOW CARBON CEMENT CONCRETE

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ABSTRACT

The durability features of concrete mixtures composed with the, Portland Slag Cement (PSC), Portland Pozzolana Cement (PPC) and Composite Cement (CC) of Low Carbon Group of Cements is being investigated in the current paper. The concrete permeability was tested by Rapid chloride penetration test (RCPT), and accelerated mortar bar method was used for potential Alkali Reactivity test, both tests were used to evaluate concrete of ratio of water to cementitious material (w/c) 0.35 and 0.45 for each type of cement. The compressive strength was tested at 1, 3, 7 and 28 days. The results shows that trend of strength recovery for CC and PPC types of cements from early to later age are similar and the recovery of strength for PSC based concrete from 28 days to 56 days are highest amongst all. For CC based concrete the RCPT values lowest and very low permeable for all groups and PPC and PSC based concrete indicates low permeable category. Both CC and PSC based concrete have much greater resistance against ASR attack than PPC. PPC have also good resistance against ASR. Thus the durability features of concrete composed with CC is relatively robust in comparison with PPC and PSC.

KEYWORDS: Low Carbon Cement, Micro structure, Micro mortar, Durability, Permeability

INTRODUCTION

The Portland Pozzolana Cement (PPC), Portland Slag Cement (PSC), and Composite Cement (CC) are emerging composites of low carbon cement family used worldwide for construction. Low Carbon Cement is getting acceptability as general purpose cement for constructions in India. Since more than five decade now, the PPC & PSC are used in India whereas CC is the new entrant. The low carbon cement is widely used in construction due to its potential in improving durability characteristic of concretestructure and its influence in improving geo-environmental sustenance.

Composite cement is generally composed of Portland cement and one or more inorganic materials that take part in the hydration reactions, This definition excludes the admixtures even though they influence the hydration process significantly (1). The inorganic materials are usually called minerals additions and most important of such materials are Fly ash, Ground granulated blast furnace slag, limestone, natural and artificial Pozzolana, micro silica etc. All these materials are fundamentally cementitious material. The subtle difference between the blended and composite is this; if only one cementious materials is present in cement it is called blended else it is called composite cement.

However, hydration process of both cement and composite cement is extremely complex (2). The development of hardened properties of concrete at a given age principally depends on the hydration characteristics of the cementitious materials of the mixture. This, in turn, is a function of characteristics and quantity of the cement, mineral admixtures such as fly ash, granulated slag and the quantity of particle having a size less than 125µ (0.492 x 10-2 in.) in fine aggregate (3). The improvement in the properties of concrete is directly related to the grain and pore refinement within transition zone in the microstructure (4).The hydration process greatly depends on the characteristics of cementitious materials, that is, OPC plus

supplementary cementitious materials (SCM) and some external factors like ambient temperature andhumidity.

Durability greatly depends on the four characteristics. (i) Permeation (ii) Migration (iii) Diffusion (iv) Sorptivity, of Concrete. Permeability refers to flow through a porous medium. The fluid transport characteristics of concrete depends primarily on the structure of the hydrated cement paste, more responsible expression shall be formation of microstructure of matrix. Inadequate durability manifests itself by the deterioration which can be due to either or collective internal and external factors or to internal cause within the concrete itself. Mega defects such as honeycomb, cold joints, high porosity etc. in concrete structure are largely responsible for the early aging problem or loss of durability. Nevertheless the inadequate durability of mega defect free concrete structure is primarily due to different degree ofpermeability.

The permeability is reduced with partial cement replacement with fly ash regardless of type and percentage substitution level (6). Progressive increase in cement replacement level with fly ash of slag, the mortar or concrete permeability reduces sharply. This phenomenon is due to formation of gradually robust microstructure with increased dosage of fly ash addition in concrete.

Continuous and discreet phases are the two components of concrete micro structure and the permeability of concrete mainly depends on its microstructure formations. The continuous phase refers to bulk of the hardened cement paste or micro mortar whereas the coarse aggregate encapsulated by interfacial transition zone is known as discrete phase. The discrete phase considerably contributes to the permeability of concrete due to its porous framework of microstructure. Despite the higher porosity of discrete phase, the permeability of concrete is controlled by the bulk of the hardened paste which is the only continuous phase in concrete. The permeability of concrete system is characterised by the nature of the pore system and thickness of the continuous phase between two aggregates. Many literature reports the relation of ratio of solid to space of paste and permeability of the concrete. The permeability decreases exponentially with the increasing trend of solid spaceratio (7).

The insight gained from the works of many authors discussed here that many important properties related to performance of concrete both in plastic and hardened state are largely dependent on the microstructure and the volume of paste and mortar. The permeability and durability, formation of robust microstructure and thicker continuous phase and workability of concrete are governed by paste and mortar volume.

The primary objective of this paper was to study the durability features of Concrete composed with PPC, PSC and CC with w/c 0.45 and 0.35. The durability features, such as the permeability and probable resistence of potential ACR reactivity of Concrete was determined. Additionally compressive strength of concrete at 1, 3, 7 & 28 days were also studied.

EXPERIMENTAL PROGRAMME

Six concrete mixtures were used for this experimental programme. Two grades of concrete mixture were proportioned with each of the three types of cement; PPC, PSC & CC. The water cement ratio was 0.35 and 0.45 and same course and fine aggregate, chemical admixture and water were used for the preparation of concrete mixtures. The mixing protocol, curing and environmental control of specimen preparation and testing was identical throughout the tenure of the experiment in total compliance with IS 4031:1996. The concrete cube compressive strength was performed at 1, 3, 7, and 28 days in accordance to IS1199:19988, IS 516:19939 and IS 4031:199610, the RCPT was conducted at 28 days following ASTM C120211. The potential Alkali reactivity test by accelerated mortar bar method was determined in accordance to IS 2386 Part VII-1963 amendment 312.

Table-1 Physical and Chemical properties of Cements

Parameter	Binder		
	PPC	PSC	CC
Chemical Parameters			
IR %	29.14	1.12	18.72
Magnesia %	1.2	4.2	1.8
Sulphuric anhydrite %	1.48	1.98	2.12
Sulphide Sulphate %	NA	0.42	0.38
LOI %	2.1	1.0	1.2
Chloride %	0.02	0.02	0.02
Physical Parameters			
Fineness (m^2/kg)	322	354	352
Normal consistency %	33	33	36
Specific gravity	2.92	2.95	2.91
Setting time (minutes)			
a) Initial	130	145	150
b) Final	245	260	280
Soundness			
a) Le-chat(mm)	0.5	0.5	0.5
b) Auto Clave%	0.034	0.022	0.027
Compressive Strengths (MPa)			
a) 3 Days	19	21	19
b) 7 Days	27	38	28
c) 28days	46	52	45
Clinker Replacement level %	35	58	58

MATERIALS AND CONCRETEMIXTURE

Table 1 shows the physical and chemical properties of cement. The cement used for these experimental programs were Composite Cement (CC), conforming to IS 16415:201513, Portland Pozzolana Cement (PPC) conforming to IS 1489 (Part-1):199114, and Portland Slag Cement (PSC) conforming to IS 455:198915.

Natural river sand grading zone II as fine aggregate and aggregate were used for the study. The physical characteristic of the aggregates is listed in Table 2. Crushed stone Coarse aggregate of MSA 20mm and 10mm both having identical shape and origin were taken for the studies. The grading characteristics of both the coarse and fine aggregates fell within the stipulations of IS 383:201616.

Table 2 Physical properties of aggregate

Properties	Mode	Fine aggregate	Coarse aggregate	
			CA 10 (10mm)	CA 20 (20mm)
Fineness modulus	Ratio	2.63	6.0	7.09
Water absorption	%	0.08		
Specific gravity	Ratio	2.63	2.84	2.84
Mater passing 75	%	1.2	NA	NA

A total of 6 mixtures were tested with the cement content maintained around 350 kg/m^3& 440 kg/m^3. The water to cement (w/c) ratios varied for two grades; 0.45 and 0.35 for PPC, PSC & CC. A laboratory pan mixture of 90 liters capacity was employed for mixing. The composition of mixtures is listed in Table 3. The compressive strength of the concrete was

determined by crushing 150 mm x 150 mm x 150 mm cubes in a universal testing machine as per IS 4031:198810. The cubes were demoulded 1 day after casting and then cured in waterat 27± 20 C for 28 days before testing.

Table 3 Composition of Concrete Mixture (Kg/m^3)

Concrete Parameter		Cement	Water	W/C	Fine Aggregate	Coarse Aggregate	Chemical Add mixture	
Group	No.						Type	Quantity
PPC	1	351	158	0.45	689	1266	PCE	1.23
	2	444	156	0.35	622	1247	PCE	2.685
PSC	3	347	156	0.45	700	1287	PCE	1.57
	4	437	153	0.35	647	1297	PCE	1.416
CC	5	351	158	0.45	691	1269	PCE	1.55
	6	443	155	0.35	636	1275	PCE	1.91

RESULTS&DISCUSSIONS

The clinker for manufacturing of PPC; PSC & CC was taken from single source for this study. The equivalent Alkali level expressed in terms of Na_2O of the clinker was 0.52%. The hydration kinetics is greatly influenced by the amount of soluble alkali available in the paste and due to this strength recovery pattern of PPC, PSC & CC are different when clinker component is selected from multiple sources of varying alkali content. The clinker component of PPC, PSC & CC from single source would act as a equalizing factor to regulate the speed of hydration. The clinker replacement level with fly ash, slag and a combination of fly ash & slag for PPC, SPC and CC are 35, 58 and 58 % respectively. The specific gravity of all these three types of cement is different due to varying replacement level with cementitious materials having different specificgravity (Table 1).

The physical properties of aggregate to a limited extent are given in Table 2. The fine aggregate size is lower than 150µ is 2.8% and no such powdery material is present in coarse aggregate. Six concrete mixtures were proportioned using PPC, PSC & CC for a target strength of 28 days of 25 MPa and 40 MPa. PCE based chemical admixture was used for making the concrete mixture just workable to improve the easiness of filling the mould and compaction for specimen preparation.

The design variable of concrete mixture was fixed for 25 MPa grade I) (Cement content 350 kg/m^3 II) w/c 0.45 and for 40 Mpa grade I) Cement content 440 kg /m^3 II w/c 0.35. Outcome of the concrete mixture proportioning are listed in Table 3 and computational value of cement content, the variation was within ±1% from the design criterion. The w/c and cement content are two independent parameters of concrete mixture fundamentally responsible for controlling hydration kinetics and development of denser micro structure and pore structure refinement of the hydrated cement paste. These two parameters, intern solely dictate the strength development, permeability, durability and host of the other properties of harden state concrete.

Table 4 Compressive strength and Split tensile strength of different mixture

Group	Mixture No.	Density (kg/m^3)	Compressive Strength (MPa)				Split tensile strength (MPa)
			1 day	3 days	7 days	28 days	
PPC	1	2457	6.8	15.3	25	33.4	3.3
	2	2557	14.8	25	42.1	53.8	5.1
PSC	3	2526	7.7	20.9	24.6	34.7	4.3

	4	2526	12.3	30.5	41.1	51.2	5.4
CC	5	2483	7.3	16	23.5	33	3.3
	6	2541	6.6	25	37.4	45.7	4.6

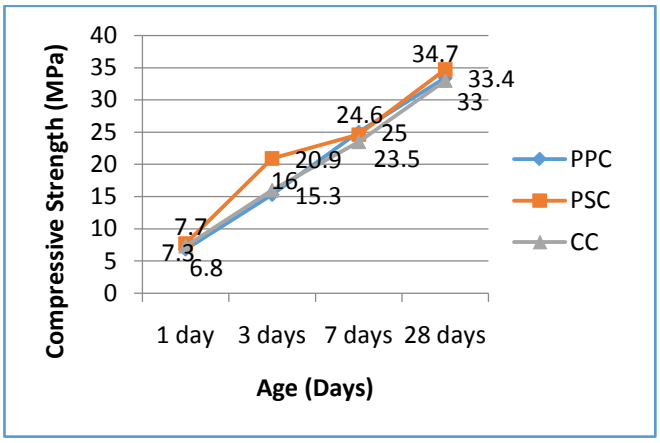


Figure-1 Compressive strength of concrete (w/c0.45)

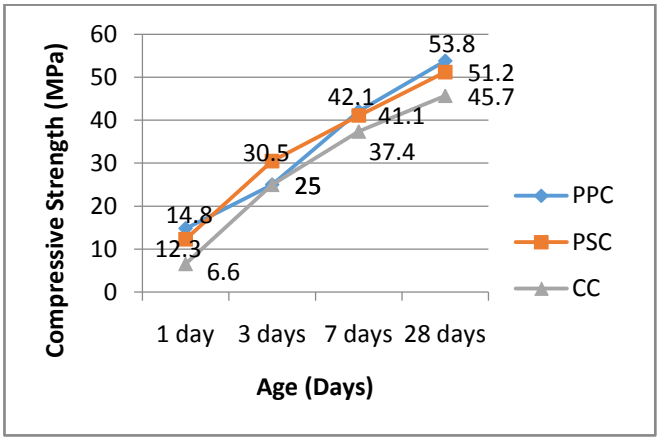


Figure-2 Compressive strength of concrete (w/c 0.35)

The compressive strength and spit tensile strength of all mixtures are given in Table 4, Figure 1 & Figure 2. The strengths recovery profile of all mixes is falling in a close band. The paste comprises the collective volume of Cement, Water and Chemical admixture. The 2.8% of fine aggregate shall be one of the components of micro mortar in addition to paste volume. The paste micro mortar, mortar volume and RCPT results of all mixes are show in Table 5.
The paste, micro mortar and mortar volume of CC mixes are relatively higher than the mixes with PPC and PSC. The RCPT values of CC mixes are the lowest amongst all. The paste micro mortar and mortar volume of PPC mixes, again, relatively higher than the PSC mixes and RCPT values of PPC mixes is lower than PSC mixes. The RCPT values of mixes is influenced by paste volume and decreased with increase paste, micro mortar and mortar volume regardless of w/c of the mixes.

Table 5 Paste and Micro motor volume, RCPT values of Different mixture

Group	Mixture No.	Paste Volume (Lts)	Micro mortar Volume (Lts)	Mortar Volume (Lts)	RCPT (coulombs)
PPC	1	279	283	541	1448
	2	311	313	547	480
PSC	3	275	278	541	1587

	4	302	305	548	624
CC	5	280	283	543	1239
	6	309	312	551	268

The RCPT values are reducing sharply with increased paste, micro mortar and mortar volume. The lower RCPT value is a good indicator of lower permeability of concrete. The continuous phase of micro structure gets thicker with increased paste volume. The inter particle separation of aggregate, mortar intercept and continuous phase gets widen with increased paste, micro mortar and mortar volume in a mix.

The permeability of concrete is greatly influenced by the pore structure or system within the bulk of the hardened cement paste and transition zone. The permeability of concrete is reduced significantly by lowering w/c up to certain level and it could further be minimized through proper design of the composition and volume of thepaste.

Increase in the alkalinity of the pore water requires that hydration of certain amount of cement of the cementitious material has taken place. Cement of different characteristics takes different span of time to reach the required pH level of the concrete/mortar system. The particle size, mineralogical compositions and the chemical characteristics of fly ah as well as proportion in quantity of fly ash with cement also play important role in pozzolanic reaction. One more important aspect of fly ash is that it slows down the hydration process.

Inadequate quantity of paste volume or mortar brings the coarse aggregate closer to each other, thus the inter-particle separation of aggregates is reduced. In absence of adequate mortar intercept, the coarse aggregate, more precisely discrete phase is in contact, resulting formation of uninterrupted transportation corridor of fluid throughout concrete. The mortar intercept is a barrier between two discrete phases and prevent towards formation of transportation of fluid from one discrete phase to other. This is due to the fact that, the permeability of continuous phase is significantly lower than the discrete phase and thinner the mortar intercept, the concrete becomes more susceptible of permeability with gradual mortar to mortar intercept.

Table 6 Results on potential Alkali Silica Reactivity

Group	Mortar Bar expansion %	Fine Aggregate		Coarse Aggregate	
		Dissolved Silica millimole/ ltr.	Reduction of Alkali millimole/ ltr.	Dissolved Silica millimole/ ltr.	Reduction of Alkali millimole/ ltr.
PPC	0.027	25	160	31	145
PSC	0.012	25	160	31	145
CC	0.017	25	160	31	145

The potential alkali silica reactivity test result is listed in Table 6. The results reveal that the potential reactivity of both fine and coarse aggregates are innocuous. The potential alkali silica reactivity test determined by, accelerated mortar bar method IS 2386, Part VII shows that the expansion is low for all the mixes and no possibility of alkali silica reaction is expected. The results of expansion of PSC and CC mortar bar are much lower than PPC mortar bar. The PSC and CC cement have relatively higher resistance against alkali silica reactivity than PPC. The sole source of alkali in concrete, responsible for ASR attack, is from cement and clinker component in particular for PPC, PSC and CC. Many literature reports that threshold value of alkalis for initiation of ASR is 0.6% of Na_2O_2. Due to different level of clinker replacement with fly ash, slag in PPC & PSC and a combination of fly ash & slag in CC, brings down the active alkalis in the paste. The active alkali present in PPC, PSC &CC are 0.31%, 0.19% and 0.19% respectively. This could possibly be a cause for much

lower expansion of mortar bar of CC and PSC than PPC. The alkalis available in fly ash is in a glassy phase with low solubility and does not take part in alkali silica reactions. The beneficial effect of high clinker replaced cement with fly ash, slag and a combination of both such as CC and PSC, develops denser micro structure of hydrated cement paste resulting to low penetrability of concrete which reduces the mobility of alkali generated2. The reasons is due to this alkali silica reactor is controlled to a greatextent.

CONCLUSIONS

The experimental programme designed to evaluate the compressive strength recovery at 1, 3, 7, 28 days, RCPT values and potential alkai reactivity of concrete composed with PPC, PSC and CC with W/C of 0.35 and 0.45 for each type of cement. Base on the test results of the study, following conclusion could be drawn.

The ratio of water to cementitious materials has a critical bearing on regulating the concrete compressive at all ages and response to initial strength. The strength recovery pattern of all concrete are similar and falling in a narrow band. RCPT values are influenced by both, ratio of water to cementitious material and paste volume. The RCPT values of CC based concrete is the lowest and fall under the category of very low permeability level. The RCPT values of PPC and PSC based concrete indicate low permeable features.

Potential Alkali Silica or Carbonate reaction is influenced by type and volume of supplementary cementitious (SCM) material and w/c of the mix. Bothe CC and PSC based concrete have much greater potential to resist ASR or ACR attacks than the concrete composed withPPC.

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EFFECT OF JUTE FIBRE ON WORKABILITY AND DENSITY OF CONCRETE

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ABSTRACT
In this study, jute fiber was used as fibrous material in concrete to improve the overall strength. Jute fiber of different lengths and percentages were selected. Four various percentages of jute fibers were implemented here, such as 0%, 0.25%, 0.50%, 0.75% and 1.0% of the volume of plain concrete. Again, the lengths of jute fiber were selected such as 10 mm to 40 mm for each percentage to find out the best potential percentage along with length. In this study various mix is prepared with variable jute fiber content. Workability and density were compared with plain concrete. Total 30 numbers of cubes were prepared. Testing results were discussed and analysed to find optimum use of jute fibre for this particular research program.

KEYWORDS: Jute fiber, JFRC, plain concrete, Mechanical strength, compressive strength.

INTRODUCTION
In general both developing and developed countries it has been a technological challenge in the field of the design of low cost and durable fiber reinforced cement concrete. The type of fiber currently been used include steel carbon polymer glass and natural fibers. Cost effective consideration have limited usage of carbon fibers in cementitious composites on a marketable level for their non ecological performance. Natural fibers have the potential to be used as reinforcement to skeptical the innate scarcities in cementitious materials.
Today, we need to control the rapid used of admixture and chemical used in concrete and use of natural fiber like jute and all other fiber reinforcement. Use of admixture they get polluted the environment and they are more costly so this type of problem to overcome by natural fiber reinforced concrete they are eco-friendly and low cost composite.

EARLIER INVESTGATION & SCOPE OF THE STUDY
The scope of natural fiber can be effective material to reinforce concrete strength which will not only explore a way to improve the properties of concrete. it will also explore the use of jute and restrict the utilization of polymer which is environmentally detrimental. The currently scope and research of jute fiber reinforced concrete is limited only in experiment study but these can be introduce jute fiber varying ratio of the ingredients in concrete, water cement ratio, and length and volume of fiber to know the effect of parameter as mentioned.
Application of jute fibre in mortar and concrete is recent trend and many studies have been conducted to evaluate the performance characteristic of concrete reinforced with jute fibre. The brief literature reviews of the latest studies are as follows.
Baruah and Talukdar (2007) Investigated coir fibre reinforced concrete with the volume fraction 0%, 0.5%, 1.0%, 1.5%, 2.0% by the volume fraction of concrete shows compressive strength, split tensile strength, modulus of rupture, shear strength and toughness continuously increases up to 2% volume fraction of concrete.

Kim et al. (2012) described the effect of volume fraction on the compressive strength of jute fiber concrete author concluded Jute fibers do affect the workability and the compressive strength of the jute fiber concrete improved by 40% when compared to the one without fiber.
Prof. Shriram H. Mahure (2014) had studied about the fresh and hardened properties of jute fiber reinforced concrete using Fly ash as partial replacement of cement in different percentages in addition to filler. The fresh properties have been determined by computing the Slump value, V-funnel value and L-box value and the hardened properties are determined by computing the Compressive strength, Flexural strength and Split tensile strength of the specimens. It is observed that the fresh properties of concrete shows an acceptable value upto 30% increases.
Vipul kumar (Aug 2015) observed on adding natural jute fibre to the concrete mix workability decreases because of the water absorbing property of jute fibre. And fiber cement ratio is taken as 0.5%, 1%, 1.5% and fiber length was taken as 5mm-10mm. Author found that the compressive strength of experimental cubes comes out to be higher than that of plain concrete. Author observed that the difference in Initial setting and final setting time of mix increased with increased ratio of fiber – cement.

NEED OF THE STUDY

Today, we need to control the rapid used of admixture and chemical used in concrete and use of natural fiber like jute and all other fiber reinforcement. Use of admixture they get polluted the environment and they are more costly so this type of problem to overcome by natural fiber reinforced concrete they are eco-friendly and low cost composite.

IV AIM OF THE STUDY

The aim of the study is “An experimental study of jute fiber reinforcement concrete" and Compare the all property to plan concrete and determine the optimum content of jute fiber.

OBJECTIVE OF THE STUDY

Objective of the current research are
1. To produce jute fibre reinforced concrete in laboratory using jute fibre 0%, 0.25%, 0.50%, 0.75% and 1.0% for M25 grade of concrete.
2. Test the cube specimen in the laboratory.
3. Determine the optimum content of jute fiber to achieved desired property of concrete.

MATERIAL USED AND THEIR PROPERTIES

Cement: Locally available Ordinary Portland Cement (OPC) 53 grade was used.
Sand: Locally procured natural sand was used as fine aggregate in concrete. Locally available Narmada sand (zone-II) was used.
Aggregate: A combination of 20mm nominal size aggregate and 10mm nominal size aggregate is used as coarse aggregate in this experimental program. Both types of coarse aggregate were locally procured.
Water: The water used in the mortar was ordinary tap water from the Bhopal city.
Jute fiber: It helps to make best quality industrial yarn, fabric, net, and sacks. It is one of the most versatile natural fibres that have been used in raw materials for packaging, textiles, non-textile, construction, and agricultural sectors. Jute fibres were prepared with locally available jute bags.

EXPERIMENTAL PROGRAM

To conduct experimental program following trials were prepared in the laboratory.
1. M25JF0 (0%jute fiber content)

2. M25JF0.25 (0.25% jute fiber content)
3. M25JF0.50 (0.50% jute fiber content)
4 M25JF0.75 (0.75% Jute fiber content)
5 M25JF1.0 (1.0% jute fiber content)

RESULTS AND DISCUSSION

Workability: The effect of workability with the mixing content of jute fiber is clearly show the table 1.

Table 1: Workability of M25 concrete of different jute content by slump cone test

Mix	Jute Fiber (in %)	Slump value in mm
M25JF0	0%	55
M25JF0.25	0.25%	15
M25JF0.50	0.50%	20
M25JF0.75	0.75%	04
M25JF1.0	1%	00

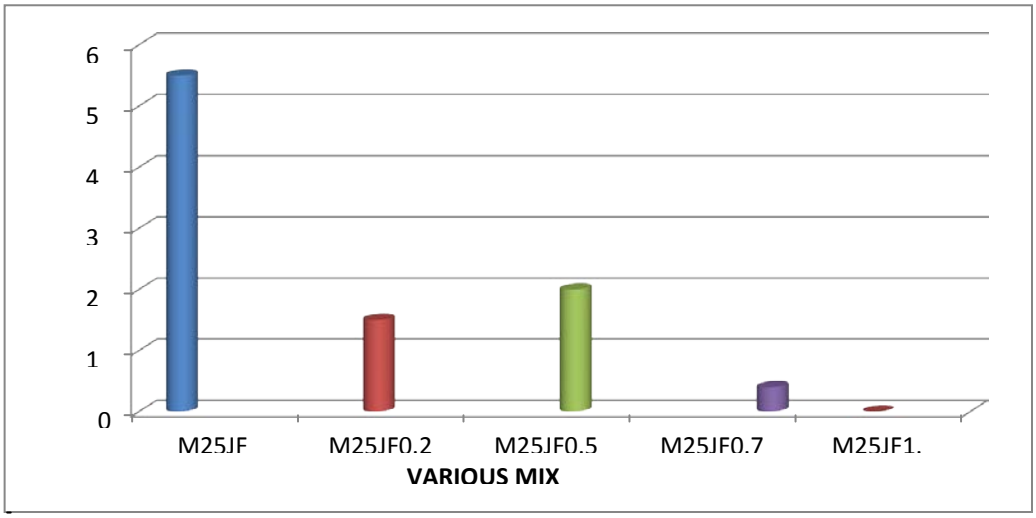


Fig. 1 show the variation of workability of various mix of concrete

Bar chart 1 generally the workability slump value is 55 mm this is the range of good workability of concrete but if we increase the jute fiber the value of slump is decreases at jute fiber is 0.50% content and concrete designation is M25JF0.50 the poorest workability show is 1% of jute content at concrete designation isM25JF1.0 So as we can clearly say that jute fiber reduce the workability of concrete in significant amount. When we increase the content of jute fiber the workability decreases this result in handling and placing problem. We can use a work like lower grade of concrete and if use the same content we increase the w/c ratio to maintain the workability. Also we can use the admixture to attain a desire degree of workability.

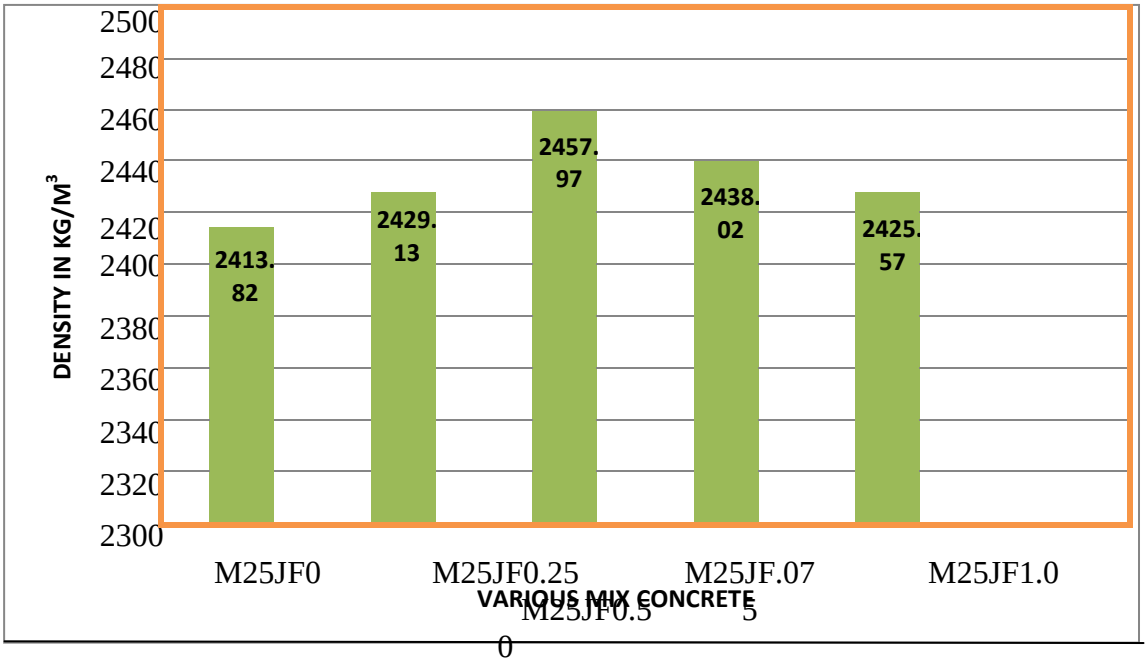
DENSITY OF CONCRETE

The density of concrete also determine in these test program density of concrete is another parameter to evaluation of concrete mix in the test program based on the 1-day weight of cubes. Generally weight taken cube size as per IS Standard is 150mm×150mm×150mm at the time of demoulding of cubes and after 24 hours of

casting, was calculated and observations of density of concrete with mixing of jute fiber variation table shows the values.

Table 2 show the test result of density of various mixes

Mix designation	Average wt of cubes in gram	Density of cubes kg/m^3
M25JF0	8146.66	2413.82
M25JF0.25	8198.33	2429.13
M25JF0.50	**8228.33**	**2457.97**
M25JF0.75	8295.66	2438.02
M25JF1.0	8186.33	2425.57



Graph 2 show the variation of density of various mix concrete

In the table and bar chat clearly shows that the variation of density of concrete is not in the wide range but changes are significant manner. It is clear that the density of concrete at 0% fiber content is 2413.82 kg/m^3 and we can see from fig that addition of jute fibre results in increase of density. JFRC best result is obtained at M25JF0.50 (0.50% content of jute fiber).

CONCLUSIONS

Present investigation involves casting and testing of concrete using jute fiber as reinforcing material. This study mainly aims to analyse workability and density of JFRC. On the basis of results obtained in the laboratory it can be concluded that use of jute fibre fiber reduce the workability of concrete in significant amount. When we increase the content of jute fiber the workability decreases this result in handling and placing problem. We can use a work for lower grade of concrete and if use the same content we increase the w/c ratio to maintain the workability. Also we can use the admixture to attain a desire degree of workability. Jute fibre results in increased density of concrete Variation of density of concrete is not in the wide range but changes are in significant manner.

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RECENT INNOVATIONS IN CEMENT CONCRETE USING GROUND GRANULATED BLAST SLAG: REVIEW

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ABSTRACT
The ground granulated blast furnace is a waste product from the iron manufacturing industry, which may be used as partial replacement of cement in concrete due to inherent cementing properties, which can be added in cement, without compromising on its strength and durability, which will result in green house gases, in addition to sustainable management waste. GGBS to the concrete modifies various properties of concrete like tensile strength, compressive strength, binding properties and find desired strength.

KEYWORDS: GGBS, plain concrete, Mechanical strength, compressive strength.

INTRODUCTION
Concrete is the most important material used for construction all over the World. The components of concrete are cement, fine aggregate, coarse aggregate and water. Concrete is the first choice for construction in many countries today. This has led to the fast vanishing of natural resources. The emission of CO2 into the atmosphere results from the production of cement, deterioration, poor performance, and inadequate resistance to hostile environment of many concrete structures. Hence, there is an urgent need for construction industry to look for alternative pozzolanic materials like fly ash, GGBS, silica fumes, red mud etc.
Generation of industrial by-products has increased significantly with industrialization. One such by-product from iron smelting industry is iron slag, which is generated from blast-furnaces while extracting iron. This blast furnace slag is used to make a cementitious material by grinding it into fine powder, known as Ground Granulated Blast Furnace Slag (GGBS)[1]. Ground granulated blast furnace slag is the industrial by-product which provide excellent binding properties to concrete and serve as a replacement of cement. This alternative is generally termed as Supplementary cementious materials (SCMs)[2-5].
GGBS cement can be added to concrete in the concrete manufacturer's batching plant, along with Portland cement, aggregates and water. The normal ratios of aggregates and water to cementitious material in the mix remain unchanged. GGBS is used as a direct replacement for Portland cement, on a one-to-one basis by weight. Replacement levels for GGBS vary from 30% to up to 85%. Typically 40 to 50% is used in most instances. The use of GGBS in addition to Portland cement in concrete in Europe is covered in the concrete standard EN 206:2013. This standard establishes two categories of additions to concrete along with ordinary Portland cement: nearly inert additions (Type I) and pozzolanic or latent hydraulic additions (Type II). GGBS cement falls in the latter category. As GGBS cement is slightly less expensive than Portland cement, concrete made with GGBS cement will be similarly priced to that made with ordinary Portland cement.
It is used partially as per mix ratio.

EARLIER INVESTGATION & SCOPE OF THE STUDY
Ground granulated blast-furnace slag (GGBFS) was used as a partial replacement of cement concrete. The performance characteristics of GGBFS is more beneficial give more strength,

more stability, more durability and more future sustainable comparison than the cement. GGBS able to resists designed compressive force and it is also fire resistance. In this paper, we analysis and review the paper whose already done in the field.

SIGNIFICANT CONTRIBUTIONS IN THE FIELD OF CONCRTE USING GGBS

Application of GGBS is done by partially replacement of cement concrete is recent trend and many studies have been conducted to evaluate the performance characteristic of GGBS. The brief literature reviews of the latest studies are as follows.

B.Kaviya R et al. [1] conducted study on ground granulated blast furnace slag partial replacement of OPC (43-grade) by GGBS up to20% by total weight of OPC and Fine aggregates up to40% by Marble slurry. This study investigates the performance of concrete mixture in terms of compressive strength of cube, flexural strength of beam and split tensile strength of cylinder for 7 days and 28 days respectively. Author reported, the most optimized mix of GGBS based concrete to be 30% from both compressive and strength of concrete. However, beyond 40%of replacement, the strength decreases.

Chaithra H L et al.[2] Investigated, the cement has been replaced by GGBS in the range of 30%, 40% and 50% by weight of cement, quarry sand in the range of40%, 50% and 60% by weight of cement for M40 grade mix. concrete. The cube, beams and cylinders are tested for compressive, Flexural and tensile strengths. It is found that by the partial replacement of cement with GGBS and sand with Quarry sand helped in improving the strength of the concrete substantially compared to normal mix concrete. These tests are carried out to determine the mechanical properties of concrete up to 7, 28, 56, 90 days for compressive strength, 28 days for split tensile strength and flexural strength. Study shows, It is observed that GGBS based concretes have achieved an increase in strength for 40% replacement of cement at the age of 28 days. As the percentage of QS increased, the Compressive strength also increases.

S.K. Sirajuddin et al.[3] Investigated the effect of the replacement percentages of cement with GGBS are 20%, 40% and 60% by weight and fine aggregate with QD are 25%, 50% and 75% by weight. These combinations of materials were used to study compressive strength, spilt tensile strength and flexural strength tests and the results obtained were compared with the control concrete. It is observed that the optimum replacement percentages of GGBS and QD are 40% and 50% respectively. Result shows by author, replacement of cement with 40% GGBS and fine aggregate with 50% QD is suitable for getting higher strengths compared with control mix Sonali K. Gadpalliwar et al.[4] Experiment conducted of Partial replacement of natural sand (NS) with Quarry sand and partial replacement of cement with GGBS and Rice husk Ash can be an economic alternative. This research is carried out in three phase, in first phase mix of M40 grade concrete with replacement of 0%,15%,30%,45%,60%,75%,90% and 100% of quarry sand with natural sand is carried out to determine the optimum percentage of replacement at which maximum compressive strength is achieved. Author found, Compressive strength increases with increase of percent of quarry sand upto certain limit. Concrete acquires maximum increase in compressive strength at 60% quarry sand replaced by natural sand for M40 grade of concrete. By adopting same critical mix and replacing cement by GGBS, it is found that by increasing the percentage of GGBS; workability increases but strength decreases.

Yogendra O. Patil et al [5] Conducted study in Ordinary Portland Cement, partially replaced by ground granulated blast furnace slag in different proportions varying from 0% to 40%. It is observed from the investigation thatthe strength of concrete is inversely proportional to the

% of replacement of cement with ground granulated blast furnace slag. It is concluded that the 20% replacement of cement is possible without compromising the strength with 90 days curing. Result shows by author is the compressive strength and flexural strength of cement concrete containing various % of GGBS at the age of 7, 28 and 90days are given in Table 4 and 5 respectively.

Suresh Singh Kushwah et al. [6] Conducted study in experimental work investigates feasibility of using GBFS as replacement of natural sand and GGBS as replacement of cement in concrete respectively. Concrete cubes have been prepared and their compressive strength is checked for M30 grade of concrete. Thus, it can be concluded that GGBS and GBFS can be used to partially replace 55% and 50% in concrete respectively without affecting their compressive strength. Thereby reducing carbon dioxide emission and curtailing cost of concrete by 20.25%. Results showed that in order to make cost effective and eco-friendly concrete, fifty-five percent of GGBS and fifty percent of GBFS can be used in concrete as partial replacement to cement and river sand respectively.

D. Suresh et al. [7] Investigated, the movement of moisture of GGBS mixes, probably due to the dense and strong microstructure of the interfacial aggregate/binder transition zone are probably responsible for the high resistance of GGBS mixes to attack in aggressive environments such as silage pits. The mineral composition of GGBS cement paste (with less aluminate and than Portland cement) probably contributes to this resistance.

Gopireddy Madan et al. [8] Investigated, the partial replacement of cement, sand and coarse aggregate were done in different trail mix for M40 grade of concrete separately. The mix trail with different percentage of GGBS, SSW and QD by 0%, 10%, 20%, 30%, 40%, 50%, 60% and 70% in order to find the optimum percentage use in concrete. From these trial mixes, percentage optimization of GGBS, SSW and QD used in concrete for the further investigation.It was found that 40%, 50% and 60% replacement of GGBS, QD and SSW respectively gives maximum strength to concrete.

Sai Indrasena Reddy et al. [9]investigated, the workability of the blend diminishes independent of level of GGBS replacing concrete. At steady rate substitution of River Sand with Robo sand, the functionality of the solid doesn't get affected as rate GGBS substitution the concrete is differed. Robosand can supplant River Sand 100% without affecting Compressive Strength.

REMARKABLE OBSERVATIONS ON PERFORMANCE CHARACTERISTICS OF CONCRETE USING GGBS.

B.Kaviya R et al. [1] It is observed that GGBS based concrete have achieved an increase in strength for 30% replacement of cement at the age of 7 and 28 days.

Chaithra H L et al.[2]It is observed that GGBS based concretes have achieved an increase in strength for 40% replacement of cement at the age of 28 days. And the optimum split tensile strength of concrete was found to be at 40% replacement of cement by GGBS and 50%replacement of sand by QS. S.K. Sirajuddin et al. [3] reported that replacement of cement with 40% GGBS and fine aggregate with 50% QD is suitable for getting higher strengths compared with control mix. Sonali K. Gadpalliwar et al. [4] reported concrete acquires maximum increase in compressive strength at 60% quarry sand replaced by natural sand for M40 grade of concrete. Gopireddy Madan et al. [8] reported, It was found that 40%, 50% and 60% replacement of GGBS, QD and SSW respectively gives maximum strength to concrete.

CONCLUSIONS

On the basis of current study reviewed here and significant observation by various researchers in the field of GGBS. It was observed that addition of GGBS to the concrete modifies various properties of concrete like tensile strength, compressive strength, binding properties and find desired strength.

STRENGTH PROPERTIES OF CONCRETE USING GROUND GRANULATED BLAST SLAG

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ABSTRACT
The ground granulated blast furnace is a waste product from the iron manufacturing industry, which may be used as partial replacement of cement in concrete due to inherent cementing properties, which can be added in cement, without compromising on its strength and durability, which will result in green house gases, in addition to sustainable management waste. GGBS to the concrete modifies various properties of concrete like tensile strength, compressive strength, binding properties and find desired strength. In this study concrete were produced in laboratory using GGBS in combination of fly ash in 0%, 5%, 10%, 15% and 20%. Compressive strength at the age of 7 days and 28 days were found for cube specimen in the laboratory. Results were analysed and compared with the control mix concrete.

KEYWORDS- GGBS, plain concrete, mechanical strength, compressive strength.

INTRODUCTION
Concrete is the most important material used for construction all over the World. The components of concrete are cement, fine aggregate, coarse aggregate and water. Concrete is the first choice for construction in many countries today. This has led to the fast vanishing of natural resources. The emission of CO2 into the atmosphere results from the production of cement, deterioration, poor performance, and inadequate resistance to hostile environment of many concrete structures. Hence, there is an urgent need for construction industry to look for alternative pozzolanic materials like fly ash, GGBS, silica fumes, red mud etc. Generation of industrial by-products has increased significantly with industrialization. One such by-product from iron smelting industry is iron slag, which is generated from blast-furnaces while extracting iron. This blast furnace slag is used to make a cementitious material by grinding it into fine powder, known as Ground Granulated Blast Furnace Slag (GGBS) [1]. Ground granulated blast furnace slag is the industrial by-product which provides excellent binding properties to concrete and serve as a replacement of cement. This alternative is generally termed as Supplementary cementious materials (SCMs) [2-5].

GGBS cement can be added to concrete in the concrete manufacturer's batching plant, along with Portland cement, aggregates and water. The normal ratios of aggregates and water to cementitious material in the mix remain unchanged. GGBS is used as a direct replacement for Portland cement, on a one-to-one basis by weight. Replacement levels for GGBS vary from 30% to up to 85%. Typically 40 to 50% is used in most instances. The use of GGBS in addition to Portland cement in concrete in Europe is covered in the concrete standard EN 206:2013. This standard establishes two categories of additions to concrete along with ordinary Portland cement: nearly inert additions (Type I) and pozzolanic or latent hydraulic additions (Type II). GGBS cement falls in the latter category. As GGBS cement is slightly less expensive than Portland cement, concrete made with GGBS cement will be similarly priced to that made with ordinary Portland cement.
It is used partially as per mix ratio.

EARLIER INVESTGATION & SCOPE OF THE STUDY

Ground granulated blast-furnace slag (GGBFS) was used as a partial replacement of cement concrete. The performance characteristics of GGBFS is more beneficial give more strength, more stability, more durability and more future sustainable comparison than the cement. GGBS able to resists designed compressive force and it is also fire resistance.

B.Kaviya R et al. [1] conducted study on ground granulated blast furnace slag partial replacement of OPC (43-grade) by GGBS up to20% by total weight of OPC and Fine aggregates up to40% by Marble slurry. This study investigates the performance of concrete mixture in terms of compressive strength of cube, flexural strength of beam and split tensile strength of cylinder for 7 days and 28 days respectively

S.K. Sirajuddin et al.[3] Investigated the effect of the replacement percentages of cement with GGBS are 20%, 40% and 60% by weight and fine aggregate with QD are 25%, 50% and 75% by weight. These combinations of materials were used to study compressive strength, spilt tensile strength and flexural strength tests and the results obtained were compared with the control concrete. It is observed that the optimum replacement percentages of GGBS and QD are 40% and 50% respectively.

Yogendra O. patil et. al [5] Conducted study in Ordinary Portland Cement, partially replaced by ground granulated blast furnace slag in different proportions varying from 0% to 40%. It is observed from the investigation thatthe strength of concrete is inversely proportional to the % of replacement of cement with ground granulated blast furnace slag.

D. Suresh et al. [7] Investigated, the movement of moisture of GGBS mixes, probably due to the dense and strong microstructure of the interfacial aggregate/binder transition zone are probably responsible for the high resistance of GGBS mixes to attack in aggressive environments such as silage pits. The mineral composition of GGBS cement paste (with less aluminate and than Portland cement) probably contributes to this resistance.

REMARKABLE OBSERVATIONS ON PERFORMANCE CHARACTERSTICS OF CONCRTE USING GGBS.

B.Kaviya R et al. [1] it is observed that GGBS based concrete have achieved an increase in strength for 30% replacement of cement at the age of 7 and 28 days.

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S.K. Sirajuddin et al. [3] reported that replacement of cement with 40% GGBS and fine aggregate with 50% QD is suitable for getting higher strengths compared with control mix.

Sonali K. Gadpalliwar et al. [4] reported concrete acquires maximum increase in compressive strength at 60% quarry sand replaced by natural sand for M40 grade of concrete.

Gopireddy Madan et al. [8] reported, it was found that 40%, 50% and 60% replacement of GGBS, QD and SSW respectively gives maximum strength to concrete.

OBJECTIVE OF THE STUDY

Objective of the current research are
1. To produce concrete in laboratory using GGBS in combination of fly ash in 0%, 5%, 10%, 15% and 20% for M40 grade of concrete.
2. Test the cube specimen in the laboratory.
3. Determine the optimum content of GGBS to achieve desired property of concrete.

VI. MATERIAL USED AND THEIR PROPERTIES

Cement: Locally available Ordinary Portland Cement (OPC) 53 grade was used.
Sand: Locally procured natural sand was used as fine aggregate in concrete. Locally available Narmada sand (zone-II) was used.
Aggregate: A combination of 20mm nominal size aggregate and 10mm nominal size aggregate is used as coarse aggregate in this experimental program. Both types of coarse aggregate were locally procured.
Water: The water used was ordinary tap water from the Bhopal city.
Fly ash: Fly ash used in this study was collected from Sarni thermal power plant.
GGBS: Commercially available Bags of GGBS were used in various proportions in this study.

EXPERIMENTAL PROGRAM

To conduct experimental program following trials were prepared in the laboratory. Total 6 trial mix was prepared and each trial has 6 cube specimens, so total 36 cube specimen were casted and tested in the laboratory.
M00 = OPC concrete **(C+S+A+W)**
MF0 = OPC 50 % + Fly ash 50% **(C+FA+S+A+W)**
MF5 = OPC 45 % + Fly ash 50% + 5 % GGBS **(C+FA+GGBS+S+A+W)**
MF10 = OPC 40 % + Fly ash 50% + 10 % GGBS **(C+FA+GGBS+S+A+W)**
MF15 = OPC 35 % + Fly ash 50% + 15 % GGBS **(C+FA+GGBS+S+A+W)**
MF20 = OPC 30 % + Fly ash 50% + 20 % GGBS **(C+FA+GGBS+S+A+W)**

RESULTS AND DISCUSSION

Compressive Strength
The average compressive strength is calculated in this test program. Compressive strength of cube of size 150 × 150 × 150 mm is tested for 7 days and 28 days.
Compressive strength at the age of 7 days of cube specimen
Table 1 shows compressive strength of GGBS concrete at the age of 7days. From table we can see that compressive strength were found 33.40 N/mm^2 for mix M00, 25.50N/mm^2 for MF0, 28.50 N/mm^2 for MF5, 31.53N/mm^2 for MF10, 26.00 N/mm^2 for MF15 and 18 N/mm^2 for MF20. It was observed that effect of use of GGBS has great influence on strength. It was observed that effect of using GGBS results in increase compressive strength. Maximum increase in strength was found at 10 % after that compressive strength decreases.

Compressive strength at the age of 28 days of cube specimen
Table 1 shows compressive strength of GGBS concrete at the age of 28days. From table we can see that compressive strength were found 49.00 N/mm^2 for mix M00, 35.02N/mm^2 for MF0, 39.00 N/mm^2 for MF5, 44.00 N/mm^2 for MF10, 36.05 N/mm^2 for MF15 and 33 N/mm^2 for MF20. It was observed that effect of addition of GGBS has great influence on 28 days strength. It was observed that effect of using GGBS results in increase compressive strength. Maximum increase in strength was found at 10 % after that compressive strength decreases.

Mix	7 days compressive strength N/mm^2	28 days compressive strength N/mm^2
M00	33.40	49.00
MF0	25.50	35.02
MF5	28.50	39.00
MF10	31.53	44.00
MF15	26.00	36.05
MF20	18.00	33.00

CONCLUSIONS

On the basis of experimental study conducted in the laboratory it can be concluded that

1. Use of GGBS influence compressive strength of concrete significantly.
2. GGBS has great influence of 28 days compressive strength.
3. Maximum increase in compressive strength were found at 10 % GGBS.

STUDY OF MODIFIED CONCRETE BY USING GLASS POWDER

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ABSTRACT
The production of cement used in concrete is one of the main problems targeted by the industry and academic research institutes, first to protect the environment from CO2 released in the atmosphere by the intensive production of cement to satisfy the increasing demand of this binder over time, on second hand to improve its properties to better meet the new obligations of quality and performance. On another stage the glass waste began imposing wide discussions on the feasibility and the efficiency of its recycling because this material is non-degradable and possesses very good physical, mechanical, thermal and mineralogical properties which make it possible to be used in large field of production. In this study various mixeswere prepared with variable (5% 10% 15% and 20%) glass powdercontent. Workability and density were compared with plain concrete. Testing results were discussed and analyzed to find optimum use of glass powder content for this particular research program.

KEYWORDS- Glass powder, plain concrete, mechanical strength, compressive strength, flexural strength and split tensile strength.

INTRODUCTION
Theenvironmental and economic concern is the greatest challenge the concrete industry is confronting.Cement manufacturing industry is one of the carbon dioxide emitting sources besides deforestationand the combustion of fossil fuels. The production of Portland cement, an essential ingredient ofconcrete, leads to the release of significant amounts of CO2, a greenhouse gas; one ton of Portlandcement clinker production is supposed to produce approximately one ton of CO2 and othergreenhouse gases. Decomposition of industrial waste is among big problems every country is facingthese days. Use of recycled materials in construction is the most convenient option because of thelarge quantity and due to widespread sites of construction.

Today, world faces a serious problem of disposal of large quantities of industrial waste. The disposal of these wastes without proper attention creates hazardous impact on environmental healthas the landfills are increasing due to tonnes of industrialwaste disposal. So Glass powder is used in this study as the addition with cement in concrete ithelps to reduce the quantity of industrial waste.

NOTEWORTHY CONTRIBUTIONS IN THE FIELD OF CONCRTE USING GLASS POWDER
The brief literature reviews of the latest studies are as follows.
Sasikumar&Tamilvanam (2016) performed an experiment investigation on properties of glasspowder as a partial replacement of cement. Main parameter investigated in this study is M30 gradeconcrete with partial replacement of cement by glass powder 0%, 5%, 10%, 20%,

25%.the optimum7 and 28 days compressive strength has been obtained in the 25% glass powder replacement level.

Kumar & Dhaka (2016) write a review paper on partial replacement of cement with glass powderand its effect on concrete properties the main parameter investigated in this study M35 concretewith partial replacement of glass powder with varying 0,5,10,15,20,25% by weight of cement. Thepaper presents a detailed study on compressive strength, flexural strength and split tensile strengthfor 7 days and 28 days respectively.

Jain &pawade (2015) studied the characteristics of glass powder concrete. The physical properties of
high strength glass powder concrete and their sensitivity of curing procedures were referencePortland cement concrete ,having their the same concrete content as the glass powder concrete.

Bhat and Bhavanishankar Rao (2014) concluded that an increase of 27% compressive strengthcould be achieved when 20% cement was replaced by glass powder in concrete if W/C ratio wasn't maintained, but the results showed a decrease of compressive strength when W/C ratio wasmaintained.

Shekhawat and Aggarwal (2014), Gunalaan and KanapathyPillary (2013), found that theworkability is reduced with the rate of cement replacement, and explained this reduction by theincrease in the surface area of the glass powder and also the angular shape of the glass particles.

Sharma & Seema (2012) examined the effect of partial replacement of cement with glass powder oncompressive strength of concrete with w/c ratio as 0.5 and percentage replacement was 0%,10%,15%,20%,25%. The optimum compressive strength is obtained at 20% cement replacement by aglass powder at all levels.

Rivard, &Labrecque, 2009 Concluded thatthe pozzolanic activity is usually related to fine particles where the silica is released after thedestruction of the silica network by the hydroxide ions, combines with calcium from Portlandite toform C-(N,K)-S-H which improve concrete properties also Pereira de Oliveira, Castro Gomes, andSantos (2012) reported that the glass powders fineness is a fundamental parameter in the glasspozzolan's production to increase their potential reactivity.

OBJECTIVE OF THE STUDY

Objective of the current research are
1. To produce cement concrete in laboratory using glass powder0%, 5%, 10%, 15% and 20% for M35 grade of concrete.
2. Test the cube specimen in the laboratory.
3. Determine the optimum content of glass powder to achieve desired property of concrete.

MATERIAL USED AND THEIR PROPERTIES

Cement: Locally available Ordinary Portland Cement (OPC) 43 grade was used.
Sand: Locally procured natural sand was used as fine aggregate in concrete. Locally available Narmada sand (zone-II) was used.

Aggregate: A combination of 20mm nominal size aggregate and 10mm nominal size aggregate is used as coarse aggregate in this experimental program. Both types of coarse aggregate were locally procured.
Water: The water used in the mortar was ordinary tap water from the Bhopal city.
Glass powder: Locally availablewaste glass powder was collected Bhopal. Before addingglass powder in the concrete it is powdered to desired size.

EXPERIMENTAL PROGRAM

To conduct experimental program following trials were prepared in the laboratory.
M0 = 100 % OPC
M1 = 5% Glass powder (GP) + 40% Fly ash + 60% Ordinary Portland cement (OPC)
M2 = 10% Glass powder (GP) + 40% Fly ash + 60% Ordinary Portland cement (OPC)
M3 = 15% Glass powder (GP) + 40% Fly ash + 60% Ordinary Portland cement (OPC)
M4 = 20% Glass powder (GP) + 40% Fly ash + 60% Ordinary Portland cement (OPC)

RESULTS AND DISCUSSION

Workability

In this experimental program various mix of concrete are preparedwith the varying content of Glass powder like 5% 10% 15% and 20%. Slump cone test were conducted in the laboratory.

Table 1: workability of M35 concrete of different glass powder content by slump cone test

Mix	Jute Fiber (in %)	Slump value in mm
M0	0%	55
M1	5%	55
M2	10%	45
M3	15%	38
M5	20%	20

Table 1 shows the effect of workability with the addition of glass powder with fly ash in concrete. Workability of mix M0 was found same as M1. This is the range of goodworkability of concrete but if we increase the glass powder the value of slump is decreases and thepoorest workability was found at 20%glass powder content at mix M4. Theresults mark clearly a considerable decrease of consistency where increasing the glass powder in the mixture results in a significant decrease of concrete workability; this loss of consistency depends onthe rate of glass powder. The decrease of workability could be interpreted due to the high surfacetension of glass powder, while the quantity of cement plays a reverse role, due to the amount ofpaste in the batches, which increases the constancy of concrete by its gelatinous property.

Density of concrete

The density of concrete wasalso determined in the test program. Density of concrete is anotherparameter to evaluation of concrete mix in the test program based on the 1-day weight of cubes. Generally weight taken cube size as per IS Standard is 150mm×150mm×150mm at the time ofdemoulding of cubes and after 24 hours of casting, was calculated and observations of density ofconcrete with mixing of glass powder variation table shows the values.

Table 2 show the test result of density of various mixes

Mix designation	Average wt of cubes in gram	Density of cubes kg/m^3
M0	8630	2557.03
M1	8630	2557.03
M2	8540	2530.37
M3	8450	2503.37
M5	8610	2551.11

The result clearly shows (table 1 and table 2) that the variation of density of concrete is not in thewide range but changes in significant manner. It is clear that the density of concrete at 5% additionof glass powder with cement content was found 2557.03kg/m3 and the lowest density of concrete at 15%addition of glass powder with cement in concrete. There is no major decrease in value of density ofconcrete with content of glass powder.

CONCLUSIONS

In this study concrete mixes of variousproportions with w/c ratio 0.4 to 0.5 were prepared in the aboratory to determine workability and density. Present investigation involves casting and testing of concrete using glass powder. This study mainly aims to analyse workability and density of concrete using waste glass powder form industry. On the basis of results obtained in the laboratory it can be concluded that use of glass fiber reduce the workability of concrete in significant amount. When we increase the content of glass powder the workability decreases this result in handling and placing problem. We can use a work for lower grade of concrete and if use the same content we increase the w/c ratio to maintain the workability. Also we can use the admixture to attain a desire degree of workability. Addition of glass powder results in increased density of concrete. Variation of density of concrete is not in the wide range but changes are in significant manner.

Slump decreases with increasing glass powder in concrete with respect to reference mix. Maximum slump was recorded for mix M1 having 5% glass powder. There is continuous decrement in the value of slump by the addition of glass powder. Finally it may suggest that the glass powder in concrete if used in correct proportion and manner it is likely a viable option for structural use as it has advantages that construction is greener and environment friendly.Glass powder is an economical and eco-friendly easily available industrial waste to enhance the concrete property.

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NON-DESTRUCTIVE TESTING OF CONCRETE USING EGG SHELL POWDER AND CERAMIC WASTE POWDER

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ABSTRACT
Concrete is in abundance used material in construction around the world and cement, a major material of concrete is being costly and only moderately available, researches or experiments are conducted to study the variations in the strength characteristics of concrete by replacement of cement partially or fully by cheaper or locally available materials .In this study Eggshell powder (ESP), Fly ash and (CWP) Ceramic waste powder were used as alternative material for cement replacement. ESP and CWP are used as a partial replacement of cement and various non destructive properties (UPV test and rebound hammer test) were determined and analysed. Egg shell powder and ceramic waste powder was 0%, 5%, 10% and 15% while the percentage of fly ash was 40%.

KEYWORDS- Egg shell powder, ceramic waste powder, fly ash, concrete, compressive strength, destructive testing.

INTRODUCTION
Development of a nation not only depends upon the technology but also depends upon the infrastructure. Without concrete infrastructure is not possible. Because throughout the world, concrete is being widely used for the construction of most of the buildings, bridges, road etc. The main element of concrete is cement. Since cement price is volatile and demand is so high, an alternate material can be used for replacement of cement. Ceramic waste powder is settled by alleviation and then dumped away which results in environmental pollution, in addition to forming dust in summer and menacing both agriculture and public health. Therefore, utilization of the ceramic waste powder in various industrial sectors especially the construction.
Malaysian is one of the largest eggs eaters in the world consumed 20 millions egg daily. Most of the eggshell waste is commonly disposed in the landfills without pre-treatment because it was traditionally useless. Although eggshell is known as natural solid waste which is non-hazardous, Therefore, ESP powder can be utilized in the production of concrete. In this study Eggshell powder (ESP), Fly ash and (CWP) Ceramic waste powder were used as alternative material for cement replacement.

OBJECTIVE OF THE STUDY
The objective of this research work is to evaluate performance of various concrete mixes produced by partial replacement of cement by fly ash along with other two alternative material ESP and CWP.

EARLIER INVESTGATION OF THE STUDY
Brief literature review of the current research studied is summarized here.
Amarnath Yerramala discussed influence of amount of ESP powder on concrete properties. It was found that after 10 % strength performance was lower. **Dhanalakshmi M** varied egg shell powder upto 12.5% and fly ash is added to optimum egg shell powder content 7.5%.

Mtallib and Rabiu observed that higher the contents of ESA in the cement, the faster the setting of cement. The decreased setting time of OPC was due to addition of ESA portrays ESA as an accelerator. **Chin Siew Choo** reported that water cement ratio of 0.4 produces medium workability, ESP as filler in concrete had improved the compressive strength of concrete and maximum strength was obtained at 10% replacement.
Jayasankar et al. Conducted a experimental study on properties of concrete by substituting rice husk ash, fly ash and egg shell powder to cement in concrete. Author reported that RHA, ESP and fly ash mixed cubes when added with grades above M25 may results in the decreased strength level.
Sathanantham et al (2014): carried out a study on concrete produced by replacing fine aggregate partially by rice husk ash and egg shell powder. The maximum strength was observed at 20% for compressive, split tensile and flexural strength.

MATERIAL PROPERTIES

Cement: Locally available Ordinary Portland Cement (OPC) 43 grade was used.
Sand: Locally procured natural sand was used as fine aggregate in concrete. Locally available Narmada sand (zone-II) was used.
Aggregate: A combination of 20mm nominal size aggregate and 10mm nominal size aggregate is used as coarse aggregate in this experimental program. Both types of coarse aggregate were locally procured.
Water: The water used in the mortar was ordinary tap water from the Bhopal city.
Fly ash: Class F fly ash was used in this study and it was collected from Bina Power Corporation Limited, District, Sagar Madhya Pradesh.
Ceramic waste powder (CWP): The tile dust is obtained from home-made ceramics. The specific gravity of tile dust is found to be 2.62 and the fineness is found to be 7.5%.
Egg shell powder: Homemade egg shell powder was used in various proportions in this study.

EXPERIMENTAL PROGRAM

To conduct experimental program following trials were prepared in the laboratory.
T- A-I = Plain concrete **(C+S+A+W)**
T- FA-I = Fly ash-40% Replaced by cement **(C+FA+S+A+W)**
T-A–II =ESP-5% & Fly ash-40% Replaced by cement **(C+ESP+FA+S+A+W)**
T-A-III = ESP-10% & Fly ash-40% Replaced by cement **(C+ESP+FA+S+A+W)**
T-A–IV = ESP-15% & Fly ash-40% Replaced by cement **(C+ESP+FA+S+A+W)**
T-B–I = CWP-5% & Fly ash-40% Replaced by cement **(C+CWP+FA+S+A+W)**
T-B-II = CWP -10% & Fly ash-40% Replaced by cement **(C+CWP+FA+S+A+W)**
T-B–III = CWP -15% & Fly ash-40% Replaced by cement **(C+CWP+FA+S+A+W)**
T-C–I = ESP-5% + CWP-5% & Fly ash-40% Replaced by cement **(C+CWP+ESP+FA+S+A+W)**
T-C-II = ESP-10% + CWP -10% & Fly ash-40% Replaced by cement**(C+CWP+ESP+FA+S+A+W)**
T-C–III= ESP-15% + CWP -15% & Fly ash-40% Replaced by cement **(C+CWP+ESP+FA+S+A+W)**
Note:-**C**-Cement, **S**-Sand, **A**-Aggregate, **W**-Water, **ESP**-Egg shell powder **CWP**-Ceramic waste powder, **FA**-Fly ash

CASTING OF SPECIMENS

Various concrete mixes were produced in the lab using 0-15% ESP, 0-15% CWP and 40% FA by Volume of cement and these mixes were compared with control mix.

RESULTS AND DISCUSSION

Figures (from 1 to 3) show result of non destructive testing of concrete casted in the laboratory.

ESP

1. As per Ultrasonic Pulse Velocity Test ESP concrete shows excellent grading.
2. Rebound Hammer shows ESP concrete of poor uniformity.

CWP

1. As per Ultrasonic Pulse Velocity Test CWP concrete shows good grading.
2. Rebound Hammer shows CWP concrete of good uniformity.
3. It can be used in high-rise building, tall structure, road pavement, and soil stabilization.

Combination of ESP and CWP

1. As per Ultrasonic Pulse Velocity Test ESP+CWP concrete also shows excellent grading.
2. Rebound Hammer shows ESP +CWP concrete of good uniformity.

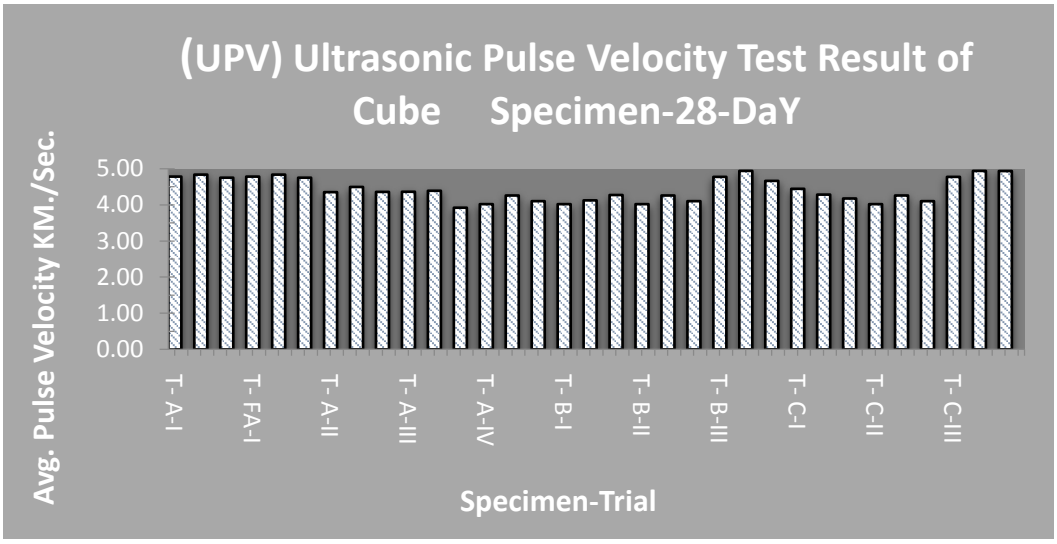


Figure 1. Ultrasonic Test Results of Cubes (Age of Specimen 28Days)`

REBOUND HAMMER TEST

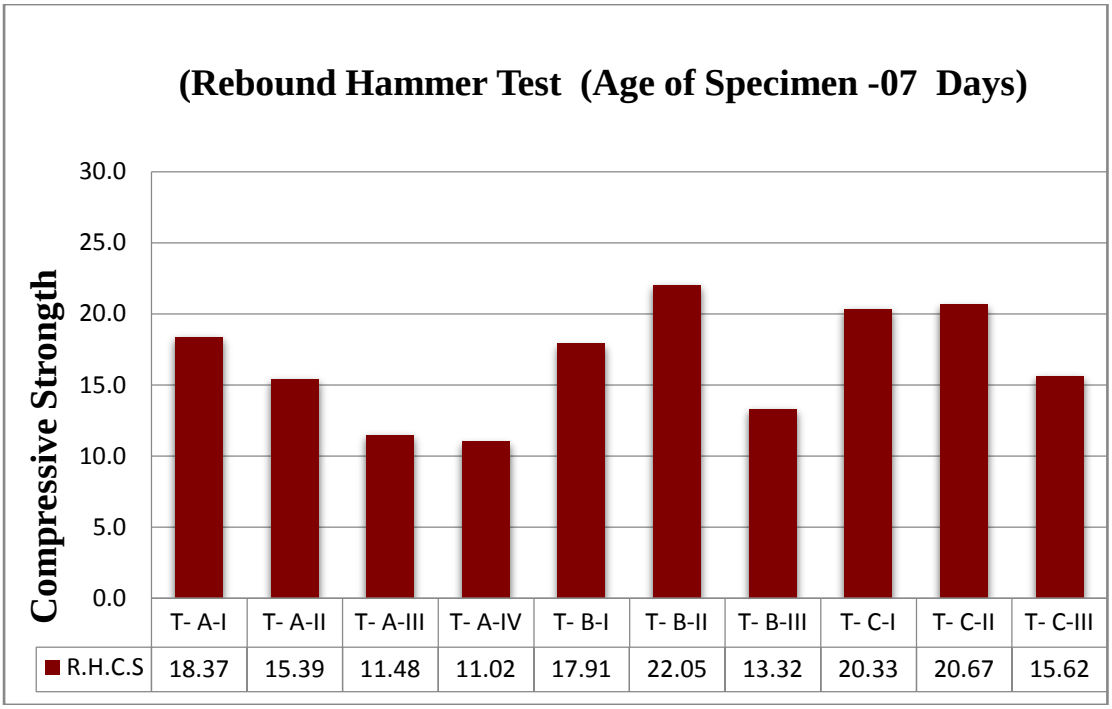


Figure2. Rebound Hammer Test Results of Cubes (Age of Specimen 7 Days)

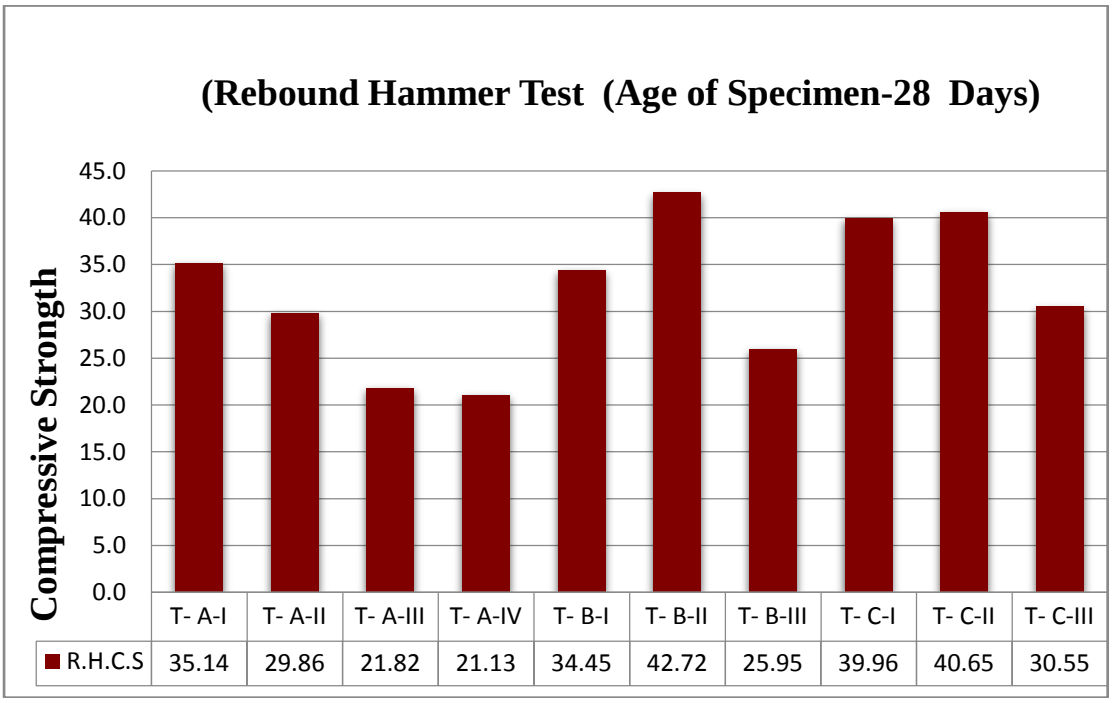


Figure3. Rebound Hammer Test Results of Cubes (Age of Specimen 28 Days)

CONCLUSIONS

On the basis of experimental investigation carried out in the laboratory it can be concluded that ESP concrete can be used in different parts of building, beam column flooring and filling material, municipal city beautification work paver block, rural road pavement, and soil stabilization. CWP concrete can be used in high-rise building, tall structure, road pavement, and soil stabilization and combination of ESP and CWP can be used in different civil construction building Beam column flooring and filling material, r.c.c. road pavement, in irrigation projects canal lining and soil stabilization.

STRENGTH PROPERTIES OF JUTE FIBRE REINFORCED CONCRTE

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ABSTRACT
In this study, jute fiber was used as fibrous material in concrete to improve the overall strength. Jute fiber of different lengths and percentages were selected. Four various percentages of jute fibers were implemented here, such as 0%, 0.25%, 0.50%, 0.75% and 1.0% of the volume of plain concrete. Again, the lengths of jute fiber were selected such as 10 mm to 40 mm for each percentage to find out the best potential percentage along with length. In this study various mix is prepared of variation of jute fiber and plain concrete and compare strength properties like compressive strength and split tensile strength of jute fiber reinforce concrete (JFRC). Total 30 numbers of cubes were prepared for compressive strength tests and 15 cylinders were prepared to split tensile strength tests. Testing results were discussed and analysed to find optimum use of jute fibre for this particular research program.

Keywords- jute fiber, JFRC, plain concrete, Mechanical strength, compressive strength.

INTRODUCTION
In general both developing and developed countries it has been a technological challenge in the field of the design of low cost and durable fiber reinforced cement concrete. The type of fiber currently been used include steel carbon polymer glass and natural fibers. Cost effective consideration have limited usage of carbon fibers in cementitious composites on a marketable level for their non ecological performance. Natural fibers have the potential to be used as reinforcement to skeptical the innate scarcities in cementitious materials.
Today, we need to control the rapid used of admixture and chemical used in concrete and use of natural fiber like jute and all other fiber reinforcement. Use of admixture they get polluted the environment and they are more costly so this type of problem to overcome by natural fiber reinforced concrete they are eco-friendly and low cost composite.

EARLIER INVESTGATION & SCOPE OF THE STUDY
The scope of natural fiber can be effective material to reinforce concrete strength which will not only explore a way to improve the properties of concrete. it will also explore the use of jute and restrict the utilization of polymer which is environmentally detrimental. The currently scope and research of jute fiber reinforced concrete is limited only in experiment study but these can be introduce jute fiber varying ratio of the ingredients in concrete, water cement ratio, and length and volume of fiber to know the effect of parameter as mentioned.
Application of jute fibre in mortar and concrete is recent trend and many studies have been conducted to evaluate the performance characteristic of concrete reinforced with jute fibre. The brief literature reviews of the latest studies are as follows.
Baruah and Talukdar (2007) Investigated coir fibre reinforced concrete with the volume fraction 0%, 0.5%, 1.0%, 1.5%, 2.0% by the volume fraction of concrete shows compressive strength, split tensile strength, modulus of rupture, shear strength and toughness continuously increases up to 2% volume fraction of concrete.

Kim et al. (2012) described the effect of volume fraction on the compressive strength of jute fiber concrete author concluded Jute fibers do affect the workability and the compressive strength of the jute fiber concrete improved by 40% when compared to the one without fiber.
Prof. Shriram H. Mahure (2014) had studied about the fresh and hardened properties of jute fiber reinforced concrete using Fly ash as partial replacement of cement in different percentages in addition to filler. The fresh properties have been determined by computing the Slump value, V-funnel value and L-box value and the hardened properties are determined by computing the Compressive strength, Flexural strength and Split tensile strength of the specimens. It is observed that the fresh properties of concrete shows an acceptable value upto 30% increases and also the hardened properties of concrete is significantly improved when compared to the conventional mix.
Vipul kumar (2015) brought out a solution for waste management problem. On adding natural jute fibre to the concrete mix the outcome is workability of fibre reinforced concrete decreases because of the water absorbing property of jute fibre. And fiber cement ratio is taken as 0.5%, 1%, 1.5% and fiber length was taken as 5mm-10mm Observed decrement in slump values from 0.5% to 1.5% due to the water absorption capacity of fiber. Author found that the compressive strength of experimental cubes comes out to be higher than that of plain concrete with an average value of 46N/mm2. Author observed that the difference in Initial setting and final setting time of mix increased with increased ratio of fiber – cement .The standard consistency or water cement ratio increases with an increase fiber cement ratio.

NEED OF THE STUDY

Today, we need to control the rapid used of admixture and chemical used in concrete and use of natural fiber like jute and all other fiber reinforcement. Use of admixture they get polluted the environment and they are more costly so this type of problem to overcome by natural fiber reinforced concrete they are eco-friendly and low cost composite.

AIM OF THE STUDY

The aim of the study is “An experimental study of jute fiber reinforcement concrete" and Compare the all property to plan concrete and determine the optimum content of jute fiber.

OBJECTIVE OF THE STUDY

Objective of the current research are
1. To produce jute fibre reinforced concrete in laboratory using jute fibre 0%, 0.25%, 0.50%, 0.75% and 1.0% for M25 grade of concrete.
2. Test the cube specimen in the laboratory.
3. Determine the optimum content of jute fiber to achieved desired property of concrete.

MATERIAL USED AND THEIR PROPERTIES

Cement

Locally available Ordinary Portland Cement (OPC) 53 grade was used.

Sand

Locally procured natural sand was used as fine aggregate in concrete. Locally available Narmada sand (zone-II) was used.

Aggregate

A combination of 20mm nominal size aggregate and 10mm nominal size aggregate is used as coarse aggregate in this experimental program. Both types of coarse aggregate were locally procured.

Water

The water used in the mortar was ordinary tap water from the Bhopal city.

Jute fiber-

It helps to make best quality industrial yarn, fabric, net, and sacks. It is one of the most versatile natural fibres that have been used in raw materials for packaging, textiles, non-textile, construction, and agricultural sectors. Jute fibres were prepared with locally available jute bags.

EXPERIMENTAL PROGRAM

To conduct experimental program following trials were prepared in the laboratory.

1. M25JF0 (0%jute fiber content)
2. M25JF0.25 (0.25% jute fiber content)
3. M25JF0.50 (0.50% jute fiber content)
4 M25JF0.75 (0.75% Jute fiber content)
5 M25JF1.0 (1.0% jute fiber content)

RESULTS AND DISCUSSION

Compressive Strength

Fig 1 shows compressive strength of JFRC at 7days. Fig clearly shows increase in strength at 0.50 % when compared to control mix. At higher content of jute fibre there is reduction in compressive strength. Fig 2 shows compressive strength of JFRC at 28 days. It was observed that effect of addition of jute fibre has great influence on 28 days strength. Increase in strength was found at 0.25 %, 0.50% and 0.75%. Maximum increase in strength was found at 0.75 % after that compressive strength decreases. Overall 14% increment in strength was found when compared with the control mix.

Splitting Tensile Strength

Fig. 3 shows the split tensile strength obtained by testing the cylindrical specimen for M25 grade of concrete four all the mixes. Fig. shows split tensile strength was found 2.6 mpa, 2.7mpa, 2.75 mpa, 3.1 mpa and 2.1 mpa at jute fibre content of 0%, 0.25 %, 0.50%, 0.75% and 1.0 %. Maximum strength found at 0.75 % after that split tensile strength decreases. Overall 20 % increment in strength was found when compared with the control mix.

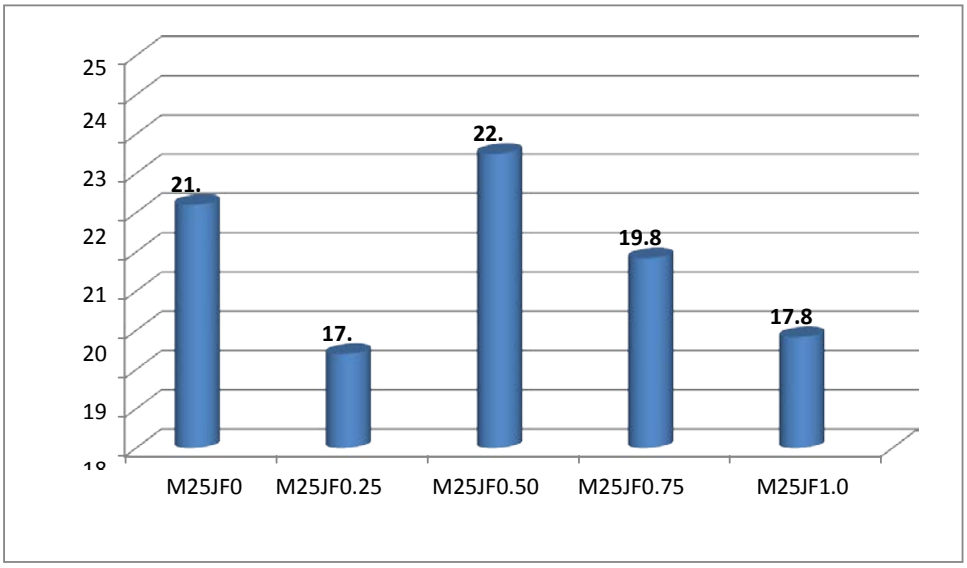


Fig.1. compressive strength at the age of 7 days

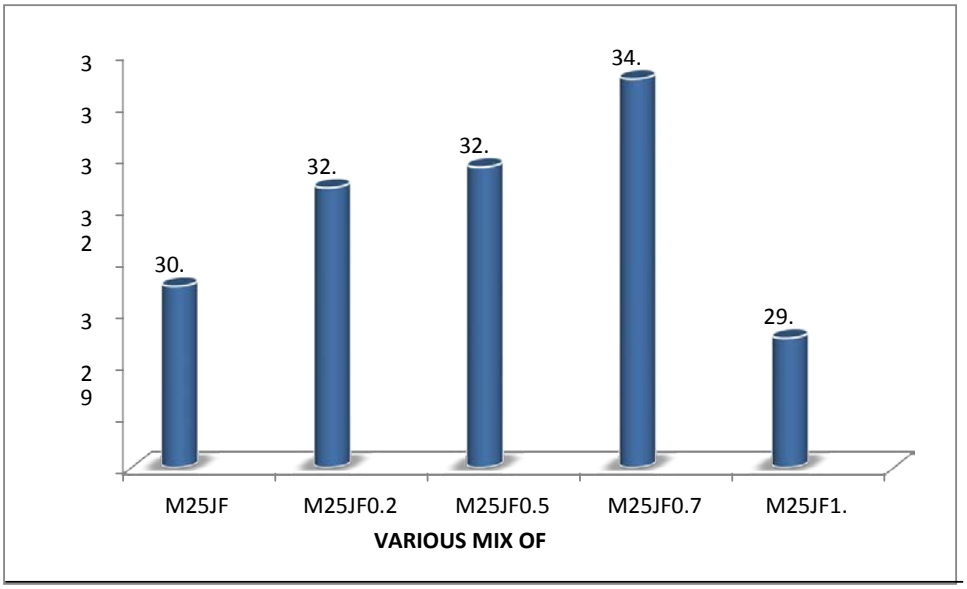


Fig.2. compressive strength at the age of 28 days

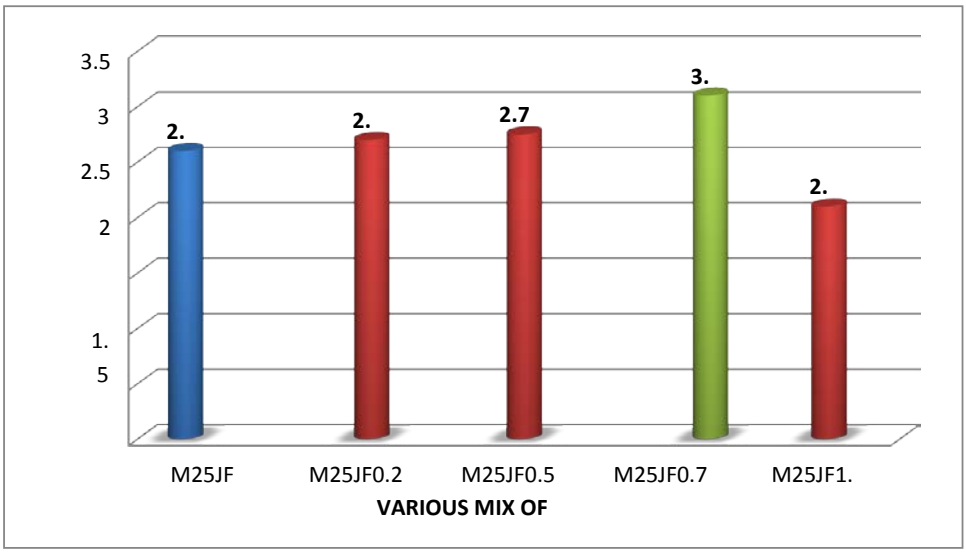


Fig.3. Split tensile strength at the age of 28 days

CONCLUSIONS

Present investigation involves casting and testing of concrete using jute fiber as reinforcing material. This study mainly aims to determine the strength parameter and optimum content of jute fiber to satisfy all parameters of concrete. On the basis of results obtained in the laboratory following conclusion can be drawn.

1. Use of jute fibre increases both the strength compressive as well as split tensile strength of concrete.
2. Use of jute fibre in concrete has great influence on 28 days compressive strength rather than 7 days strength.
3. Maximum increase in compressive strength was found at 0.75 % for this particular research. Overall 14% increment in strength was found when compared with the control mix.
4. Maximum increase I split tensile strength found at 0.75 % for this particular research. Overall 20 % increment in strength was found when compared with the control mix.

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DESTRUCTIVE TESTING OF CONCRETE USING EGG SHELL POWDER AND CERAMIC WASTE POWDER

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ABSTRACT
Due to the rapid development of industrial, technical and increase in the human population the world is facing the problems of waste disposal today. The increasing variety of environment has become a very challenging task for human's environment. In the olden times, the wastewater generated was generally used to be of organic nature and they used to be easily disintegrated in the land, but with the modern development, the proportion of hazardous substances in the waste has increased manifold, due to which it is necessary to restore the effluents there are a lot of problems to face.
In this study Eggshell powder (ESP), Fly ash and (CWP) Ceramic waste powder were used as alternative material for cement replacement. ESP and CWP are used as a partial replacement of cement and various properties were determined. Egg shell powder and ceramic waste powder was 0%, 5%, 10% and 15% while the percentage of fly ash was 40%.

KEYWORDS- Egg shell powder, ceramic waste powder, fly ash, concrete, compressive strength, destructive testing.

INTRODUCTION
Concrete is in abundance used material in construction around the world and cement, a major material of concrete is being costly and only moderately available, researches are conducted to study the variations in the strength characteristics of concrete by replacement of cement partially or fully by cheaper or locally available materials . Ceramic waste powder is settled by alleviation and then dumped away which results in environmental pollution, in addition to forming dust in summer and menacing both agriculture and public health. Therefore, utilization of the ceramic waste powder in various industrial sectors especially the construction.
Malaysian is one of the largest eggs eaters in the world consumed 20 millions egg daily. Most of the eggshell waste is commonly disposed in the landfills without pre-treatment because it was traditionally useless. Although eggshell is known as natural solid waste which is non-hazardous, Therefore, ESP powder can be utilized in the production of concrete. In this study Eggshell powder (ESP), Fly ash and (CWP) Ceramic waste powder were used as alternative material for cement replacement.

OBJECTIVE OF THE STUDY
The objective of this research work is to evaluate performance of various concrete mixes produced by partial replacement of cement by fly ash along with other two alternative material ESP (egg shell powder) and CWP (ceramic waste powder).

EARLIER INVESTGATION OF THE STUDY
Brief literature review of the current research studied is summarized here.
Amarnath Yerramala discussed influence of amount of ESP powder on concrete properties. It was found that after 10 % strength performance was lower. **Dhanalakshmi M**

varied egg shell powder upto 12.5% and fly ash is added to optimum egg shell powder content 7.5%.

Mtallib and Rabiu observed that higher the contents of ESA in the cement, the faster the setting of cement. The decreased setting time of OPC was due to addition of ESA portrays ESA as an accelerator. **Chin Siew Choo** reported that water cement ratio of 0.4 produces medium workability, ESP as filler in concrete had improved the compressive strength of concrete and maximum strength was obtained at 10% replacement.

Jayasankar et al. Conducted a experimental study on properties of concrete by substituting rice husk ash, fly ash and egg shell powder to cement in concrete. Author reported that RHA, ESP and fly ash mixed cubes when added with grades above M25 may results in the decreased strength level.

Sathanantham et al (2014): carried out a study on concrete produced by replacing fine aggregate partially by rice husk ash and egg shell powder. The maximum strength was observed at 20% for compressive, split tensile and flexural strength.

MATERIAL PROPERTIES

Cement: Locally available Ordinary Portland Cement (OPC) 43 grade was used.

Sand: Locally procured natural sand was used as fine aggregate in concrete. Locally available Narmada sand (zone-II) was used.

Aggregate: A combination of 20mm nominal size aggregate and 10mm nominal size aggregate is used as coarse aggregate in this experimental program. Both types of coarse aggregate were locally procured.

Water: The water used in the mortar was ordinary tap water from the Bhopal city.

Fly ash: Class F fly ash was used in this study and it was collected from Bina Power Corporation Limited, District, Sagar Madhya Pradesh.

Ceramic waste powder (CWP): The tile dust is obtained from home-made ceramics. The specific gravity of tile dust is found to be 2.62 and the fineness is found to be 7.5%.

Egg shell powder: Homemade egg shell powder was used in various proportions in this study.

EXPERIMENTAL PROGRAM

To conduct experimental program following trials were prepared in the laboratory.

T- A-I = Plain concrete (C+S+A+W)
T- FA-I = Fly ash-40% Replaced by cement (C+FA+S+A+W)
T-A–II =ESP-5% & Fly ash-40% Replaced by cement (C+ESP+FA+S+A+W)
T-A-III = ESP-10% & Fly ash-40% Replaced by cement (C+ESP+FA+S+A+W)
T-A–IV = ESP-15% & Fly ash-40% Replaced by cement (C+ESP+FA+S+A+W)
T-B–I = CWP-5% & Fly ash-40% Replaced by cement (C+CWP+FA+S+A+W)
T-B-II = CWP -10% & Fly ash-40% Replaced by cement (C+CWP+FA+S+A+W)
T-B–III = CWP -15% & Fly ash-40% Replaced by cement (C+CWP+FA+S+A+W)

T-C–I = ESP-5% + CWP-5% & Fly ash-40% Replaced by cement (C+CWP+ESP+FA+S+A+W)
T-C-II = ESP-10% + CWP -10% & Fly ash-40% Replaced by cement(C+CWP+ESP+FA+S+A+W)
T-C–III= ESP-15% + CWP -15% & Fly ash-40% Replaced by cement (C+CWP+ESP+FA+S+A+W)

Note:-C-Cement, S-Sand, A-Aggregate, W-Water, ESP-Egg shell powder CWP-Ceramic waste powder, FA-Fly ash

CASTING OF SPECIMENS

Various concrete mixes were produced in the lab using 0-15% ESP, 0-15% CWP and 40% FA by Volume of cement and these mixes were compared with control mix.

RESULTS AND DISCUSSION

Compressive strength at the Age of 07 Days and 28 days

It can be seen from table 1 and fig. 1 that the replacement of cement with fly ash reduces the compressive strength at the age of 7 days. Addition of egg shell powder also results in decreased compressive strength. Addition of ceramic waste powder results in increased compressive strength at 5, 10 and 15 %. CWP concrete shows better performance than ESP concrete and highest compressive strength was found at 10 % CWP. Combination of ESP and CWP shows significance improvement in strength but strength obtained in lab is less than the control mix. Similar pattern of compressive strength was found at the age of 28 days (table 1 and fig. 2)

Table 1 Compressive test report all Cubes (Age of Specimen 07 and 28 Days)

Cube trial	Concrete Type	Avg Comp. strength (N/mm^2)	
		7days	28days
T-A-I	Plain concrete	28.02	40.03
T-FA-I	Fly ash-40% Replace by cement in concrete	22.46	33.03
T-A-II	ESP-5% & Fly ash-40% Replace by cement in concrete	19.48	28.65
T-A-III	ESP-10% & Fly ash-40% Replace by cement in concrete	16.13	24.82
T-A-IV	ESP-15% & Fly ash-40% Replace by cement in concrete	15.78	22.87
T-B-I	CWP-5% & Fly ash-40% Replace by cement in concrete	26.84	36.27
T-B-II	CWP-10% & Fly ash-40% Replace by cement in concrete	30.04	41.72
T-B-III	CWP-15% & Fly ash-40% Replace by cement in concrete	26.38	39.37
T-C-I	ESP-5% + CWP -5% & Fly ash-40% Replace by cement in concrete	23.86	34.08
T-C-II	ESP-10% + CWP -10% & Fly ash-40% Replace by cement in concrete	25.30	36.66
T-C-III	ESP-15% + CWP -15% & Fly ash-40% Replace by cement in concrete	26.95	37.43

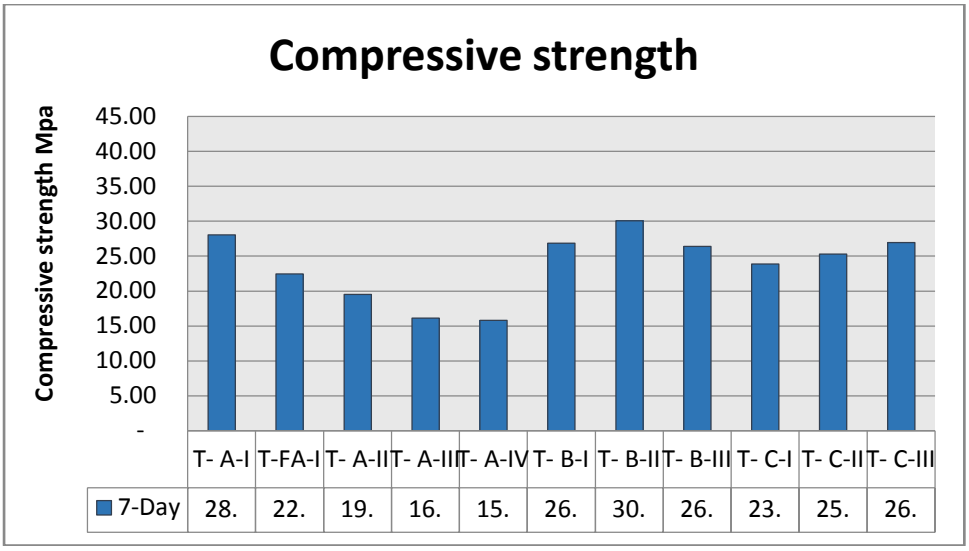


Figure 1 Compressive Strength of all trials (Age of Specimen 7 Days)

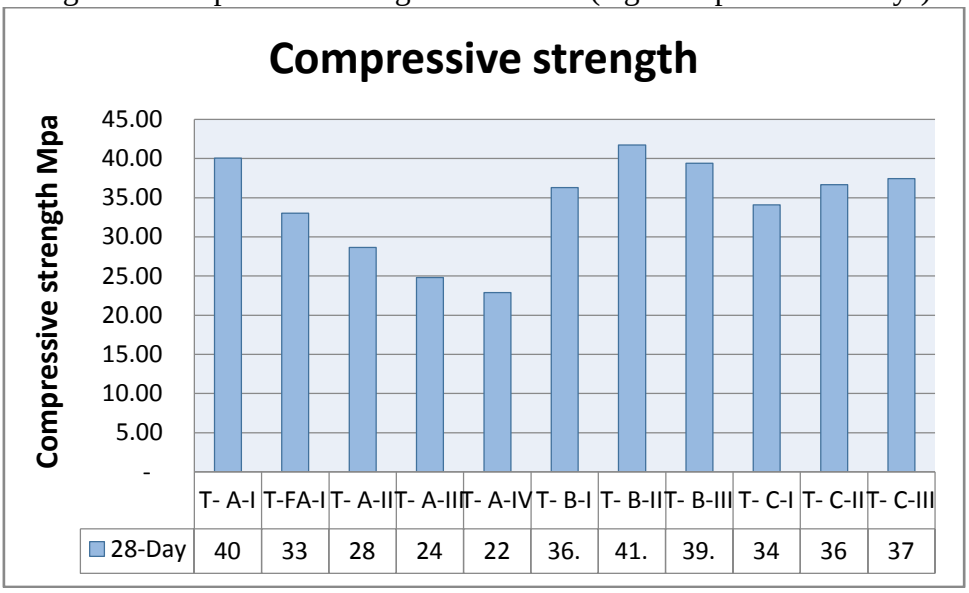


Figure 2 Compressive Strength test result Graph for all Trial (Age of Specimen 28 Days)

CONCLUSIONS

On the basis of experimental investigation carried out in the laboratory following conclusion can be drawn in regard of concrete produced with egg shell powder ESP and ceramic waste powder (CWP).

1. The replacement of cement with fly ash reduces the compressive strength at the age of 7 and 28 days. Addition of egg shell powder also results in decreased compressive strength.
2. Addition of ceramic waste powder results in increased compressive strength at 5, 10 and 15 %. CWP concrete shows better performance than ESP concrete and highest compressive strength was found at 10 % CWP.
3. Combination of ESP and CWP shows significance improvement in strength but strength obtained in lab is less than the control mix.
4. CWP concrete can be use in high-rise building, tall structure, road pavement, and soil stabilization etc.

5. Concrete produced with combination of ESP and CWP can be used in different civil construction building, flooring and filling material, r.c.c. road pavement, in irrigation projects canal lining and soil stabilization.

A REVIEW STUDY ON THE TRUSSLESS SHEETING WITH TRUSS SHEETING FOR DIFFERENT SPAN OF INDUSTRIAL SHEDS

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ABSTRACT
in this present study, comparing the cost of roofing on steel (angular) truss, hollow tube truss with self-supporting roof for the opening of 6 meter, 12 meter, 18 meter and 24 meter. This paper presents a study on behaviour and economical of roof trusses and purlins by truss provided along large span due to material saving as compare roof truss provided along width. in this present study, comparing the cost of roofing on steel (angular) truss, hollow tube truss with self-supporting roof for the opening of 6 meter, 12 meter, 18 meter and 24 meter. hollow tube truss is 40.35% and 44.15% cheaper than Steel (Angular) Truss and Self-Supporting Roof for the opening of 6 meter. Similarly, 24.37% and 21.97% cheaper for the opening of 12 meter. as the length of opening increase from 12 meter self-supporting roof becomes cheaper. self-supporting roof is 36.26% and 4.41% cheaper than steel (angular) truss and hollow tube truss for the opening of 18 meter. self-supporting roof is 28.50% and 11.42% cheaper than steel (angular) truss and hollow tube truss for the opening of 24 meter.

KEYWORDS: Truss, Hollow Pipe, Tubular Pipe, GI trussless Sheet, Steel (Angular) Truss STAAD.Pro V8i,

INTRODUCTION
The roofs are not only to protect the structure and its internal element but they also give an aesthetic view to the structure. As the roof occupies a very huge area and consumes large amounts of material for its construction, optimal design of such a structure is of utmost importance. The earlier research on the roof structures was mainly concentrated on the geometric design of the roof structure, giving less or no significance to the structural engineering aspect. But once the financial implications associated with roof system is confronted, the need for economical structural system is fully realized. With the invention of self-supporting structural elements as roof covering, it is now possible to develop innovative shapes of roofs especially for industrial, sports & other service buildings and public areas where long spans are desirable.Steel is a material which has high strength per unit mass. Steel as a construction material is one of the very important materials used in the industry, the reason is because of its characteristics and properties that it has. Steel is strong, hard, tough, ductile, fire resistant and has also got a very high melting point. The designing of industrial Steel Structure includes designing of the structural elements including principal rafter or roof truss, column and column base, purlins, sag rods, tie rods, gantry girder, bracings, etc. India has the second fastest growing economy in the world and a lot of it, is attributed to its construction industry which figures just next to agriculture in its economic contribution to the nation. So, in regard of the same Steel industry is growing rapidly. The use of steel structures is not only economical but also ecofriendly at the time when there is a threat of global warming. Here, “economical” word is stated considering time and cost.

OBJECTIVES

1. Analysis of roofing with truss and trussless.
2. Effect of changes in section of roof truss.
3. Optimization of roofing system for industrial sheds of varying type of span.
4. Design and estimation of the all cases.

PURPOSE OF STUDY

Self-supporting Roofing is a constructive solution in which there isn't always any support roofing structure. The metallic profile is supported at the load beams. It is a metallic ribbed profile which it is curved by means of inlaying. This profile is fixed with screws to the beams. Way to this fact, this solution is the lightest one in the market.

For this fact, in this present study comparison of roofing is done between the Steel (Angular) Truss, Hollow Tube Truss and Self-supporting Roofing for different openings. From this study conclusion will be made that which roofing is cheaper as compared to others.

LITERATURE REVIEW

1)Chavan& et al. (2014)The study aims to evaluate the economic significance of the Hollow Structural Sections (HSS) in contrast with open sections. This study was carried out to determine the percentage economy achieved using Hollow Structural Sections (HSS) so as to understand the importance of cost effectiveness. The technique used in order to achieve the objective included the comparison of different profiles for various combinations of height and material cross-section for given span and loading conditions. The analysis and design phase of the project was performed using STAAD PRO V8i. The sample results of STAAD analysis were validated with the results of Manual analysis

2)Kalyanshetti, &Mirajkar (2012) Most of the steel structures are builted-up with conventional sections of steels which are designed and constructed by conventional methods. This leads to heavy or uneconomical structures. Tubular steel sections are the best replacements to the conventional ones with their useful and comparatively better properties. It is obvious that due to the profile of the tube section, dead weight is likely to be reduced for many structural members .which derives overall economy. This study is regarding the economy, load carrying capacity of all structural members and their corresponding safety measures. Economy is the main objective of this study involving comparison of conventional sectioned structures with tubular sectioned structure for given requirements. For study purpose superstructure-part of an industrial building is considered and comparison is made. Study reveals that, upto 40 to 50% saving in cost is achieved by using tubular sections.

3)Nora & et al. (2015) The main purpose of this study is to regarding the economy, load carrying capacity of structural member. This study involving comparison between sectioned structural members for given requirement of superstructure part of an industrial building. Study reveals that, upto30% to 40% saving in cost is achieved by using tubular sections.

4)Shujat& Desai (2018) This study provides the comparative study of Conventional steel building (CSB), Pre Engineered Building (PEB) and Tubular Structure. The design is made as per IS 800-2007. Dead load, Live load and wind load calculation is made IS 875 part I, II and III respectively. The concept includes the technique of providing the best possible section according to the optimum requirement. This concept has many advantages over the

Conventional Steel Building (CSB) concept of buildings with roof truss. Design and analysis is done with the help of STAAD Pro V8i Software.

5)Kureshi1 & Desai (2017)In this study the foot over bridges are made up of different steel sections and these steel sections are either conventional steel sections or closed hollow section. For instance, angle section, circular hollow steel section (CHS), rectangular hollow section (RHS) and square hollow section (SHS). An exertion has been done here to calculate discrepancy in weight between conventional steel section and closed hollow section with different truss configurations like Pratt truss, Howe truss and Warren truss. Even an attempt has been done here to analyze and design of foot over bridge with multiple truss configuration and all the data's are checked for both steel sections using draft code IS 800:2007,IRC:SP:56-200 and IS 806 for limit state design. Manual calculation is verified by software STAAD PRO V8i.The outcome received by both steel sections is elevated in form of consumption of steel with different heights, spans and configurations.

6)Bokade & Vairagade (2017) Industrial building is the design and construction of buildings serving industry. Such buildings rose in importance with the industrial revolution, and were some of the pioneering structures of modern architecture. Paper covered two types of industrial building such as conventional and pre-engineered building. Pre- Engineered Building (PEB) concept is a new conception of single storey industrial building construction. This methodology is versatile not only due to its quality pre-designing and prefabrication, but also due to its light weight and economical construction. The concept includes the technique of providing the best possible section according to the optimum requirement. This concept has many advantages over the Conventional Steel Building (CSB) concept of buildings with roof truss.

7)Jayaraman & et.al. (2014)This study involves in examination of theoretical investigations of specimens in series. Overall two methods were designed and comparison of all the internal force, economical, and hence, to evaluate the co-existing moments and shear forces at the critical cross-section with same configuration area by keeping all other parameters constant. The theoretical data are calculated using Indian Standard code IS 875-1975 (part III), IS 800 – 2007 using limit state method, IS 800- 1984 using working stress method and the section properties of the specimens are obtained using steel table. The specimens are designed under uniformly distributed loading with simply supported condition. The studies reveal that the theoretical investigations limit state method design is high bending strength, high load caring capacity, minimum deflection and minimum local buckling& distortional buckling compare to the working stress method. But working stress method is most economical compare to the limit state method design.

8)Sharma & Singh (2018) Maximum of the metal systems is built up with conventional sections of steels that are designed and built through conventional methods. This ends in heavy or uneconomical structures. Tubular metal sections are the great replacements to the traditional ones with their beneficial and comparatively better residences. Its miles apparent that due to the profile of the tube section, dead weight is likely to be decreased for lots structural members .which derives ordinary economy. This study is concerning the financial system, load wearing ability of all structural participants and their corresponding protection

measures. Economy is the main goal of this look at regarding assessment of conventional sectioned systems with tubular sectioned structure for given necessities. For look at purpose superstructure-part of a business building is taken into consideration and evaluation is made. Examine famous that, up to 40 to 50% saving in value is performed by means of using tubular sections.

9)Meena and et. al. (2015)This paper viably passes on that Pre-Engineered Steeel building can be effectively structured by basic plan strategies in understanding Low weight adaptable casings of Pre-Engineered Steeel building offer higher protection from tremor loads. In the wake of breaking down, coming up next are the finishes of Pre-Engineered Steeel Building when contrasted and Conventional Steeel Buildings.

10) Bhadoria and Pathak (2017) Comparative investigation made on different models of Pre-Engineered Building and Conventional steel structure demonstrates that PEB is a practical choice and it tends to be inferred that up to a specific estimation of clear range Pre-built structure are most conservative alternative and after a particular range steel amount in PEB is practically same as that of ordinary steel structure. Arrangement of decreased segment in PEB makes it affordable and decreasing of segment is done according to the twisting minute graph. From all the examination caused it to can be reasoned that steel utilization in PEB is on a normal 30% lesser than ordinary steel structure. PEB casings are light and more adaptable than ordinary steel outlines and give higher protection from seismic powers.

11) Katkar and Phadtare (2018) as of late, the presentation of Pre-Engineered Building (PEB) idea in the plan of structures has helped in advancing structure. Long range, Column free structures are the most fundamental in a mechanical structures and Pre Engineered Buildings (PEB) satisfy this necessity alongside diminished time and cost when contrasted with customary structures. This philosophy is adaptable not just because of its quality predesigning and construction, yet in addition because of its light weight and prudent development. The present work introduces the near investigation and plan of customary steel outlines with solid sections and steel segments and Pre Engineered Buildings (PEB). In this work, a modern structure of length 44m and width 20m with material framework as customary steel support and pre-built steel bracket is investigated and planned by utilizing STAAD Pro V8i.

12) Wankhade and Pajgade (2014)This paper adequately passes on that PEB structures can be effectively planned by straightforward plan strategies as per nation gauges. In light of the examination, it very well may be presumed that PEB structures are more worthwhile than CSB structures as far as cost viability, quality control speed in development and straightforwardness in erection. The paper likewise confers straightforward and affordable thoughts on starter structure ideas of PEBs. The idea portrayed is useful in understanding the plan system of PEB idea.

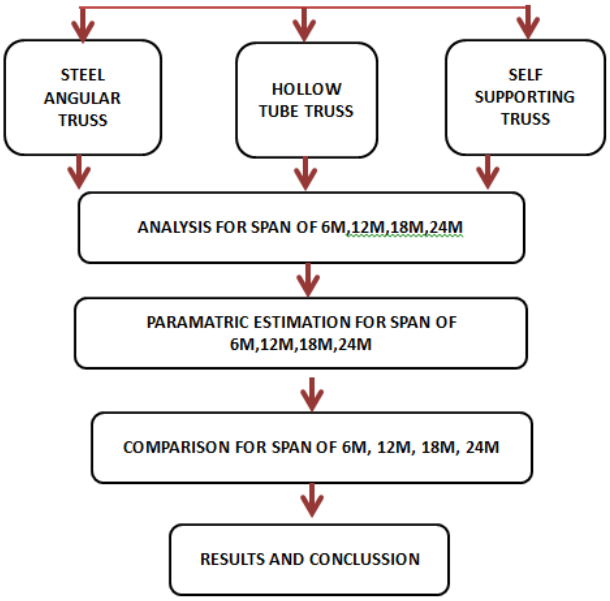
13) Subashini and Valentina (2015)This paper adequately passes on that PEB structures can be effectively planned by basic plan methods as per nation benchmarks. Low weight adaptable edges of PEB offer higher protection from quake loads18,19. PEB rooftop

structure is practically 26% lighter than customary Steel Building. In auxiliary individuals, lightweight Z purlins are utilized for PEB structure, while heavier hot-moved segments are utilized for CSB. Bolster responses for PEB are lesser than CSB according to examination. Lightweight establishment can be received for PEB which prompts straightforwardness in structure and decrease in expense of development of establishment. Substantial establishment will be required for CSB structure. PEB Building cost is 30% lesser than the expense of CSB structure17. PEB offers minimal effort, quality, solidness, structure adaptability, flexibility and recyclability. To finish up Pre-Engineered Building development gives end clients a substantially more efficient and better answer for long range structures where enormous section free regions are required.

14) Bhagatkar and et. al. (2015) From the past headway, the utilization of PEB is executed and consistently expanding, however its use isn't all through the development business. It is investigated that PEB structures can be effectively planned by straightforward plan systems as per nation norms, it is vitality productive, expedient in development, spares cost, practical and most significant its dependable when contrasted with ordinary structures. In this way PEB technique must be executed and investigated for more yields.

15) Thakre and Vairagade (2016) In this paper the pre-designed steel Building framework development has extraordinary points of interest to the single story structures, pragmatic and proficient option in contrast to regular structures, the System speaking to one focal model inside various controls. . The adoptability of PEB in the spot of Conventional Steeel Building (CSB) structure idea brought about numerous preferences, including economy and simpler manufacture. In this examination, a mechanical structure (Ware House) is dissected and planned by the Indian models, IS 800-1984, IS 800-2007 and furthermore by alluding MBMA-96 and AISC-89. The economy of the structure is talked about as far as its weight examination, between Indian codes (IS800-1984, IS800-2007) and American code (MBMA-96), and between Indian codes (IS800-1984, IS800-2007).

METHODOLOGY



DESIGN AND CALCULATIONS

In this work includesdesigning and calculations of roofing on Steel (angular) truss, hollow tube truss and self-supporting roofing is done for the opening of 6 meter, 12 meter, 18 meter and 24 meter and sheeting up to 10 meter. Once the design is completed using STAAD.pro v8i, estimating and costing is done for all the roofing. Then comparison of rate is done, through which conclusion.

Table 1 Design and estimating of steel (angular) truss

for 6 m opening			for 12 m opening			for 18 m opening			for 24 m opening		
Profile	Length (Mms)	Weight (Newt)	Profile	Length (Mms)	Weight (Newt)	Profile	Length (Mms)	Weight (Newt)	Profile	Length (Mms)	Weight (Newt)
ISA25X25X5	2100003	362.971	ISA50X50X8	40500	2305.38	ISA80X80X10	600002	6959.811	ISA80X80X10	79500.02	9221.75
ISA55X55X8	189737	1192.27	ISA90X90X10	37107.95	4874.525	ISA110X110X12	553172	10666.051	ISA120X120X10	73545.91	13163.871
ISA50X50X8	1248686	710.789	ISA65X65X10	34785.72	3206.649	ISA75X75X10	853802	9182.372	ISA75X75X10	109856.9	11814.757
ISA65X65X8	8000002	5998.036	ISA70X70X10	120000	11983.781	ISA90X90X10	240000	31526.563	ISA90X90X10	240000	31526.563
	Total =	8264.065		Total =	22370.335		Total =	58334.797		Total =	65726.941
Unit conversion: - 8264.065 Newton = 8.43 quintal			Unit conversion:- 22370.335 Newton = 22.81 quintal			Unit conversion:- 58334.797 Newton = 59.48 quintal			Unit conversion:- 65726.939 Newton = 67.02 quintal		

Table .2 Design and estimating of steel (Hollow) truss

for 6 m opening			for 12 m opening			for 18 m opening			for 24 m opening		
Profile	Length (Mms)	Weight (Newt)	Profile	Length (Mms)	Weight (Newt)	Profile	Length (Mms)	Weight (Newt)	Profile	Length (Mms)	Weight (Newt)
35 mm Dia Pipe	2100003	348.427	45 mm Dia Pipe	40500	2199.159	60 mm Dia Pipe	600002	7330.53	60 mm Dia Pipe	79500.02	9712.952
30 mm Dia Pipe	189737	801.326	40 mm Dia Pipe	37107.95	3134.398	45 mm Dia Pipe	553172	8260.288	50 mm Dia Pipe	73545.91	11647.879
15 mm	12	244.	30 mm Dia	34785.	2098.	40 mm	8538	4636.	45 mm	10985	7290.87

Dia Pipe	486.86	848	Pipe	72	746	Dia Pipe	0.29	169	Dia Pipe	6.9	
30 mm Dia Pipe	8000 0.02	1810 .007	35 mm Dia Pipe	12000 0	5792. 025	45 mm Dia Pipe	2400 00	1448 0.054	45 mm Dia Pipe	24000 0	22806.0 94
	Total =	3204 .608		Total =	1322 4.328		Total =	3470 7.041		Total =	51457.7 95
Unit conversion:- 3204.607 Newton = 3.20 quintal			Unit conversion:- 13224.328 Newton = 13.22 quintal			Unit conversion:- 34707.041Newton = 35.40 quintal			Unit conversion:- 51457.795Newton = 52.57 quintal		

SECTION-3 DESIGN OF TRUSS-LESS SHEETING

Table 3
Design of self-supporting sheet for 6 meter

Abstract of Estimated Cost

Item No.	Particulars of Items	Quantity	Unit	Rate Rs.	Rate P.	Per	Amount Rs.	Amount P.
1	Galvalume roofing	60	Sq.m	1090		Sq.m	65,400	
Add 3% for Contingencies							1,962	
Add 2% for Workcharged Establishment							1,308	
Grand total							68,670	

Table 4
Design of self-supporting sheet for 12 meter

Abstract of Estimated Cost

Item No.	Particulars of Items	Quantity	Unit	Rate Rs.	Rate P.	Per	Amount Rs.	Amount P.
1	Galvalume roofing	120	Sq.m	1090		Sq.m	1,30,800	
Add 3% for Contingencies							3,924	
Add 2% for Workcharged Establishment							2,616	
Grand total							1,37,340	

Table 5
Design of self-supporting sheet for 18 meter

Abstract of Estimated Cost

Item No.	Particulars of Items	Quantity	Unit	Rate Rs.	Rate P.	Per	Amount Rs.	Amount P.
1	Galvalume roofing	180	Sq.m	1090		Sq.m	1,96,200	
Add 3% for Contingencies							5,886	
Add 2% for Workcharged Establishment							3,924	
Grand total							2,06,010	

Table 6
Design of self-supporting sheet for 24 m

Abstract of Estimated Cost

Item No.	Particulars of Items	Quantity	Unit	Rate Rs.	Rate P.	Per	Amount Rs.	Amount P.
1	Galvalume roofing	240	Sq.m	1090		Sq.m	26,61,600	
Add 3% for Contingencies							7,848	
Add 2% for Workcharged Establishment							5,232	
Grand total							2,74,680	

RESULTS AND DISCUSSIONS

In this work includesResults And Discussion of roofing on Steel (angular) truss, hollow tube truss and self-supporting roofing is done for the opening of 6 meter, 12 meter, 18 meter and 24 meter and sheeting up to 10 meter. Once the design is completed using STAAD.pro v8i, estimating and costing is done for all the roofing. Then comparison of rate is done, through which conclusion will be made.

Table 7:- Data of trusses for different openings

Truss Opening (in meter)	Parameters	Steel Truss	Hollow Truss	Structure less truss
6 M	Cost	58,683	35,000	68,670
	Weight (quintal)	8.43	3.20	Self wt.of sheet
12 M	Cost	1,41,701	1,07,156	1,37, 340
	Weight (quintal)	22.81	13.224	Self wt.of sheet
18 M	Cost	3,23,220	2,15,517	2,06,010
	Weight (quintal)	59.48	35.41	Self wt.of sheet
24 M	Cost	3,84,140	3,10,104	2,74,680
	Weight (quintal)	67.02	52.57	Self wt.of sheet

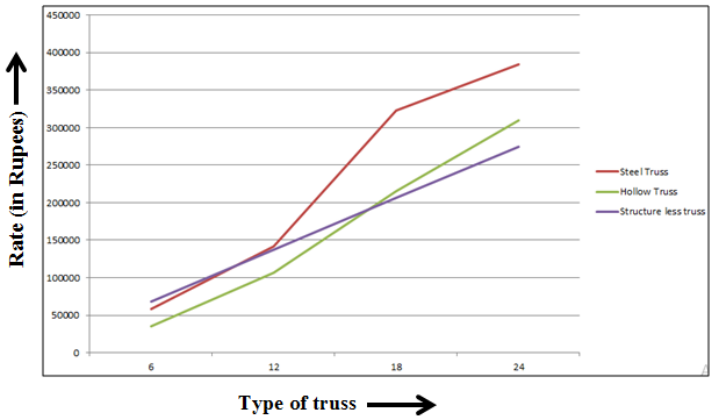


Figure 1:- Increase in rate of trusses for different openings

1. For 6 m opening of Hollow tube is 40.35% cheaper than Steel (angular) truss and 44.15% cheaper than Truss-less sheeting.
2. For 6 m opening of Hollow tube truss is 62.04% saving material than Steel (angular) truss.
3. For12 meter opening of truss sheeting on Hollow Tube Truss is 24.37% cheaper than Steel (Angular) truss and 21.97% cheaper than Truss-less Sheeting.
4. For 12 m opening of Hollow tube truss is 42.02% saving material than Steel (angular) truss.
5. For 18 meter opening of truss Truss-less Sheeting is 36.26% cheaper than Steel (Angular) truss and 4.41% cheaper than Hollow Tube Truss.
6. For 18m opening of Hollow tube truss is 40.46% saving material than Steel (angular) truss.
7. For 24 meter opening of truss Truss-less Sheeting is 28.50% cheaper than Steel (Angular) truss and 11.42% cheaper than Hollow Tube Truss
8. For 24 m opening of Hollow tube truss is 21.56 % saving material than Steel (angular) truss.
9. Steel angular truss is too heavy and costly

10. Hollow tube truss is good for long span trusses
11. Truss-less sheeting structure is more economical for more than 18 m.
12. Truss-less sheeting structure is more economical than Hollow Tube Truss and Steel (angular) truss the saving depends on various parameters.

CONCLUSIONS

After the research work some conclusions are made. The conclusions are given below:

1. For 6 m opening of Hollow tube is 40.35% cheaper than Steel (angular) truss and 44.15% cheaper than Truss-less sheeting.
2. For12 meter opening of truss sheeting on Hollow Tube Truss is 24.37% cheaper than Steel (Angular) truss and 21.97% cheaper than Truss-less Sheeting.
3. For 18 meter opening of truss Truss-less Sheeting is 36.26% cheaper than Steel (Angular) truss and 4.41% cheaper than Hollow Tube Truss.
4. For 24 meter opening of truss Truss-less Sheeting is 28.50% cheaper than Steel (Angular) truss and 11.42% cheaper than Hollow Tube Truss
5. Truss-less sheeting structure is more economical than Hollow Tube Truss and Steel (angular) truss the saving depends on various parameters.

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