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Design of Precast System of Construction for Multi Storied Buildings

PROJECT REVIEW REPORT

A report submitted in partial fulfillment of the requirements for the Award of Degree of

MASTER OF ENGINEERING
In
STRUCTURAL ENGINEERING
By

Mr. ABHIJEET SALKAR
Roll No 202126017
Under Supervision of

DR. GANESH HEGDE.
Professor Civil Engineering

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DEPARTMENT OF CIVIL ENGINEERING
GOA COLLEGE OF ENGINEERING – FARMAGUDI PONDA- GOA- 403401
Approved by AICTE, Permanently affiliated to Goa University

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ABSTRACT

Despite improvements in conventional construction techniques and material, building construction industry suffers from several issues like time delay, quality issues, higher costs, wastages, and pollution. In the immediate future, the industry is likely to shift to Pre- Cast methods of construction from conventional in-situ methods to overcome the above issues. Pre-Cast methods of construction are already in use in the industry but are restricted mainly to mass housing (as against custom architecture housing).

Therefore, a novel Pre-Cast (Form finished) method of construction is sought to be developed which will be suitable for individual structures with custom architecture advantages. The method developed is to Pre-Cast the components in a factory. This will result in better quality control and testing. The method is also suitable for standardization and modular approach in construction which will result into cost saving. It also allows us to reduce project time.

In the Pre-Cast construction, the design and construction of the joint between structural members is critical to the structural behavior of the overall assembly. In the present method the location of this joint is chosen to be in proximity to the point of contra flexure. Thus, the ductility of the beam-column joint (IS 13920, 2016 for ductile detailing) is also not compromised. The Pre-Cast assembled beam is tested for static loading. The performance of Pre-Cast assembled beam is also simulated using Finite Element Method. The Experimental results are compared with FEM simulation. It is concluded that the performance of Pre-Cast assembled beam is meeting all required criteria and good agreement with all the codal stipulations.

Keywords:- *Pre-Cast Construction, FEM, Joint.*

OBJECTIVES

- **To design a system for jointing of structural elements to facilitate Pre Cast building construction.**

The joint between various structural elements such as beam to beam and column to column should not affect the ductility of the beam- column joint as the aim is to ensure that the performance of the Pre-Cast structure is not different from that of a cast in situ, monolithic structure.

- **To ensure that the joint transfers shear and moments efficiently so that the performance of the assembly is as close as possible to that of a cast in situ, monolithic structure.**

Conventional detailing of structural members will not be suitable to achieve the above objective. Therefore, innovative detailing is required to ensure that stresses generated from the action of the connection pins are adequately resisted.

- **To select a test frame with worst loads and spans normally occurring in a G+8 structure.**

As we want to ensure that this precast system is suitable for buildings upto G+8, it is necessary to identify worst conditions of loads and spans that will occur realistically. The test frame dimensions and loads should be chosen as per the above objective. It should be acceptable to adopt the results forstructures with lesser floors and spans by scaling down the results.

- **To model and simulate the performance of only the beam in structure by using FEM software and compare results with that of a cast in situ monolithic structure**

The model of two beams , one having two cantilevers and a central beam joint together at two locations and the other is monolithic are prepared and analysed. Deflection, Stress, Strain is compared for concrete and also the reinforcement critical location is compared for the two models.

- **To carry out static load test and compare the results with simulation of a similar monolithic frame with similar load conditions.**

The frame is loaded by constructing a suitable loading platform. Static load in the form of cement concrete blocks are loaded up to the design load. Deflection of the beam is measured at critical locations by using Linear Potentiometers A model of the monolithic frame is made and analyzed using Ansys software. Comparison of results from simulation to those of the static load test is made.

LITERATURE SURVEY

➢ **Study on Properties of Prefabricated Concrete Structures underSeismic Load (1)**

International Journal of Advanced Research in Science, Communication and Technology (IJARSCT) Volume 11, Issue 1, November 2020.

- **T. Subramani, 2018** He states that precast concrete is a notable innovation in which, for rapid growth, some standardised units produced in production facilities are used.

Also a reference venture in the Chennai region is taken and shown in theETABS. programming to break down the structure and plan.

- **Majid Divan, 2011** Postulates, for public multi-storey buildings, the use of prefabricated concrete frames with prefabricated concrete shear walls due to improved performance quality and reduced production time may be a good option
- **W.C. Stone, 1998** The analytically simulated performance under seismic loads of moment-resistant precast concrete frames with hybrid connections is evaluated. To reflect the inelastic behaviour of the hybrid precast link area, an improved and flexible hysteretic model was developed.

➢ **Comparative Study between RCC Structures and Prefabricated Structures.** (7)

Siddhant Singhal, Bilal Siddiqui
International Research Journal of Engineering and Technology (IRJET)2.
This paper talks about the merits and demerits of prefabricated
Construction, it also states why there is a need of Prefabricated
System (PS). Along with different types of PS.

➢ **Prefabrication in house constructions International Journal of Low-Carbon**

Technologies Xudong Zhao, Saffa Riffat This paper examines the development and status of prefabrication techniques and their application in building construction. An overview of the current UK house building market and its status in terms of the utilization of prefabrication techniques has been made. Investigation of past engineering practices and existing knowledge of prefabrication has allowed several low cost techniques to be summarized. These would minimize the initial investment required to adopt prefabrication and so increase its market potential for UK house construction.

➢ **Prefabrication in Developing Countries: a case study of India.**

Ryan E Smith Assistant Professor, Shilpa Narayanamurthy Graduate Researcher
Prefabrication in India began with the emergence of the Hindustan Housing Factory(HHF). The HHF pioneered the production of pre-stressed concrete railway sleepers to replace dilapidated wooden sleepers on Indian Railways.
Prefabrication technology is a production technology or knowledge based and not a consumption technology or product based.

➢ **Use of Prefabrication to Minimize Construction Waste (8)** - A Case StudyApproach

C. M. Tam , Vivian W. Y. Tam , John K. W. Chan & William C. Y. Ng
This paper uses four private building projects as case studies to demonstrate the effectiveness in the use of prefabrication to minimize construction waste in Hong Kong. The wastage levels of the four projects are compared with conventional cast in-situ methods under similar project natures and conditions. The hindrances and the future trend of adopting prefabrication in Hong Kong are also examined.

- **Prefabrication techniques for residential building (9)**

M S Palanichamy, Mepco Schlenk Engineering College, This paper deals with the prefabrication techniques for residential building using a system of Precast units for columns, beams, roof and walls. Precast

R.C.C. planks and partially Precast R.C. joists are considered for flooring I roofing system in this paper and special types of Precast wall panels are recommended. Prefabricated columns with a specific configuration, beams and staircase units are considered in this paper. Special emphasis has been made with respect to the various joints and connections and the details of these are discussed. A comparison of the cost of construction of Precast system with that of a conventional construction unit has also been made. Finally, identified that large scale adopting of such a Precast systems will eventually result in considerable cost reduction with the added advantages of execution speed.

- **Review of Testing Protocols for Precast Concrete Structural Components …. Iitk (1)**

From the different literatures, reports and state of the art articles around the globe, it can be seen that research in precast concrete structures is still need vigorous research work to overcome various difficulties and uncertainties in implementation. As per the need of time, proper guidelines and codes considering precast concrete design and detailing are required for bench marking the construction process using precast concrete. Also, proper numerical and experimental investigations are neededto develop standard codes for different type of loading scenarios for different typesof structures. In Indian context, there are no proper guidelines to implement precast concrete testing and design for construction purpose. Hence, it is necessary to collaborate with government research laboratories and research institutions to come forward with a joint venture to develop the handbook for precast construction practices.

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CHAPTER ONE
INTRODUCTION

THE NEED FOR THE PROJECT. Multi-storey RCC framed buildings in India havebeen constructed by using the same techniques, material and design over the last 75 years. There have been upgrades in the material used, the construction methodologies in practice and the approach to designing such structures. For Example, there have been attempts at mechanization in production and pouring of concrete. (RMC) The strength of concrete used has almost doubled. We have seen progress in design from working stress method to the limit state method of design. Mode of design has moved on from laborious manual calculations to F.E.A. based computer programs which can incorporate complex loads such as seismic and wind loads.

However, the end results are not very different from what they used to be. We still have structures which suffer from defects which are intrinsic to construction which is carried out on site. We still have structures which suffer from improper concreting, poor curing and poorly monitored lines and levels. Moreover, such structures still take too long to construct. The average completion time of a 5 to 8 storey structure is anywhere between one and half to four years, depending on size and design. In today's fast paced world, this needs to be brought down to 6 to 8 months. Saving in time is the only way for a builder to save on construction cost and at the same time achieve a much superior product.

➢ *The Thought Process.*

It was therefore decided to take up a whole new approach to the way in whichmulti-storey buildings are constructed. The thought process is listed below.

Prefabricated – Pre-engineered Technology to be developed.

- The entire construction process will shift to a factory. The components will be manufactured in a factory and will be transported to the site where they will only be assembled mechanically.
- Design of components will be standardized. Better designtechniques can be used. A modular approach to design will be used.
- Much stronger material can be used as their quality can bemonitored in a factory environment.
- Better planning for the construction process is possible as production schedules are more reliable when undertaken in a factory dueto lessor number of variables.
- Better testing is possible.
- The total time required to erect a structure can be shrunk to 6 to 8 months.

The location of such a factory should ideally be near a port and have access to the National Highway network. Therefore shipping of the product to the national and international markets will be easy. The weight of each component is to be kept below 1.5 tonnes for ease of handling, transportation and erection.

Currently the housing industry has entered a recessionary trend and the focus has now shifted from the luxury segment to mass housing for the low and middle income group. This technology shall have great relevance in the scenario. Also, slum redevelopment projects where transit camps are requiredis a potential market as the structures build by using this technology will be designed in such a way that they could be dismantled and erected at any other location.

➢ *Innovation and proposed Technology:*

Although there exists many technologies for prefabricated homes, most of them cater to "mass housing" where a large number of houses/homes or units having the same plan, elevation and finishes are constructed. These are mainlyconstructed by making walls in a factory which is normally located on the same site as that of the project. The walls are then erected in place by using heavy lifting machinery and tower cranes. Part of the construction is still necessarily done on the site.

The technology under development shall not impose any limitations on architectural design. This technology will be used to construct structures which are presently constructed using conventional methods, thus yielding volumes which are required to sustain the operations. This technology is suited to construct multi-storey buildings in areas which are already densely built without disturbing the other dwellings in the vicinity as there is no sound, and dust pollution. It is intended to use pre-engineered concrete components which **will have specially designed efficient joints to transfer forces and moments** to form the structural elements of the frame. The design of the components shall be standardized thereby resulting in savings in terms of time (for design) and reduced wastages. A full-fledged work floor with state of the art cad cam machinery with EOT cranes etc. is envisaged.

➢ *Core philosophy:*

Move the joints way from the beam-column junction: The biggest challenge to be overcome for such structures is the efficiency of the joints of different elements like beams and columns. The first step was to ensure that only two members are connected at every joint instead of attempting to join more than two members. It was therefore decided to move the joint location from the usual beam column junction to a distance as close as possible to the point of contra lecture in the Bending moment diagram which usually occurs between 0.15 to 0.2 L.

Joint Detailing: The maximum shear on the central pins is calculated and the governs the dia. The pin is threaded with suitable gauge. The inside of the cone has a matching female thread. The cone is made out of mild steel. The concrete is cast with a conical cavity to match the cone shape. Circular reinforcement, mesh suitably transfers the stress from the concrete surface to the specially designed "infinity shaped" rod passing around the two pins to resist the shearing forces.

Splice joints: It was decided to adopt splice joints as it is desirable for the joint to be aesthetically pleasing. The two sides of the splice will be held together by means of conical pins which will be secured by suitable threads on a central pin. External loads on the elements will generate reactions on these cones/pins which are opposite in direction and will form a couple. The reinforcement inside the concrete is designed to resist the shearing forces that are generated.

Standardization: In order to achieve economy in form work design, it is necessary to adopt standard sizes of sections for various elements like beams, columns and slabs.

Form work design: The demand on the form work is as follows.

- **Dimensional accuracy**: Since the structure is going to be assembled unit by unit, dimensional accuracy of units especially in the joint section is of utmost importance. Without dimensional accuracy, the joints will not be possible if global positions of the holes are not achieved.
- **Form finish:** As the intention is to avoid plastering completely, it is necessary to achieve surfaces which are absolutely true in a plane have a surface which is ready to receive paint finish directly.
- **De-shuttering and Reassembly of Forms:** Since time is of essence, the form work is designed to minimize the time taken to assemble the formwork, place reinforcement and pour concrete. Sides are also designed by keeping in mind the ease of De-shuttering.
- **Number of uses**: The design of the formwork is also done by keeping in mind that the same is going to be used for as many uses as possible. Therefore, there is provision to replace the material which comes in contact with concrete as and when required easily.

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CHAPTER TWO
METHODOLOGY

Since the performance of the assembled frame depends upon the efficiency of the joint, the scope of the study is limited to the behavior of the beam. Therefore simulation of the assembled beam and also that of a monolithic beam is carried out in FEM Software and then the results of the two models are compared. (Part I)

However, since the deflection of the column in a frame also contributes to the total Deflection of the beam, Comparative study of deflections of the entire frame model simulated in ansys software is made with defections of the actually constructed frame and measured with LVDT instrumentation. (Part II)

➢ *Part I (F E M ANALYSIS)*

- A test beam representative of maximum spans and loads (Span: 6m) (load=48kn/m) generally occurring in commercial and residential structures is selected for the study. And the same is modelled and analysed in Ansys Software.
- The assembled beam having two cantilevers and a central beam is joint togetherat two locations using pins, the ends are fixed and static load is applied.
- A representative model was done by making suitable changes to the reinforcement detailing of the constructed model so as to arrive at an efficient mesh as per the ansys requirement.
- Total Deformation, Stress and Strain values of the above model were generated.
- Similarly all the above steps were repeated for a monolithic beam for the comparison with each other and also the permissible values as per IS 456.

➢ *Part II (EXPERIMENTAL VERIFICATION)*

- The actually constructed frame was loaded up to design load and the deflections were measured using Linear Potentiometers (LVDT). Similar frame was modelledin Ansys and the deflections were compared.

➢ *SELECTION OF TEST FRAME.*

Since the technology is meant for multi storied structures (G+8), the test frame with largest span and worst loads is selected. Figs 01 and 02 show the frame and typical loading of the selected frame.

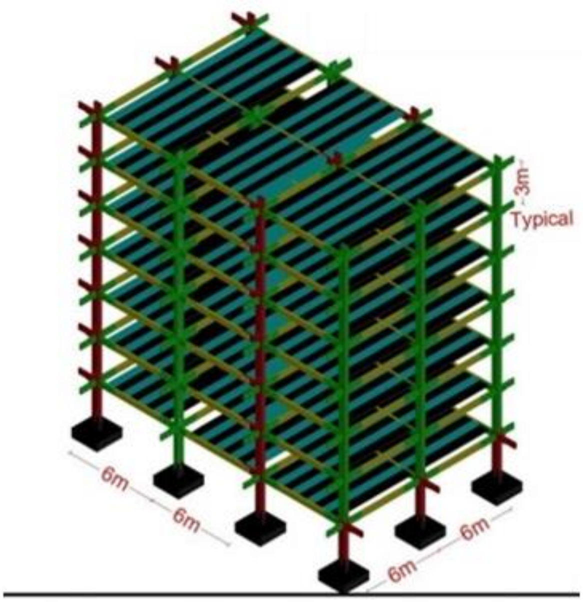

FIG 01: MULTI STORIED PREFABRICATED BUILDING.

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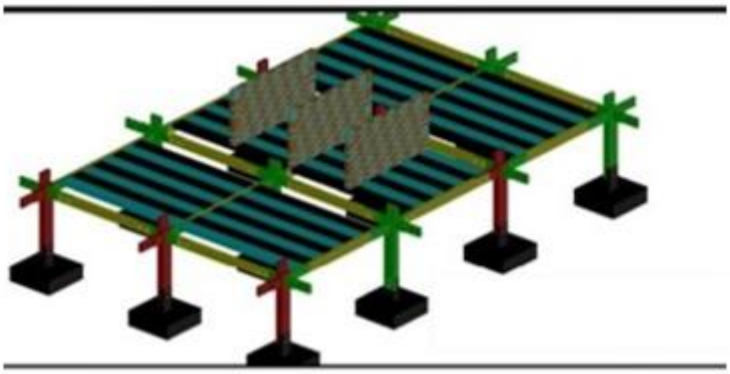

FIG 02: ARRANGEMENT OF BEAMS, SLABS AND WALLS FOR DESIGN LOAD CALCULATIONS

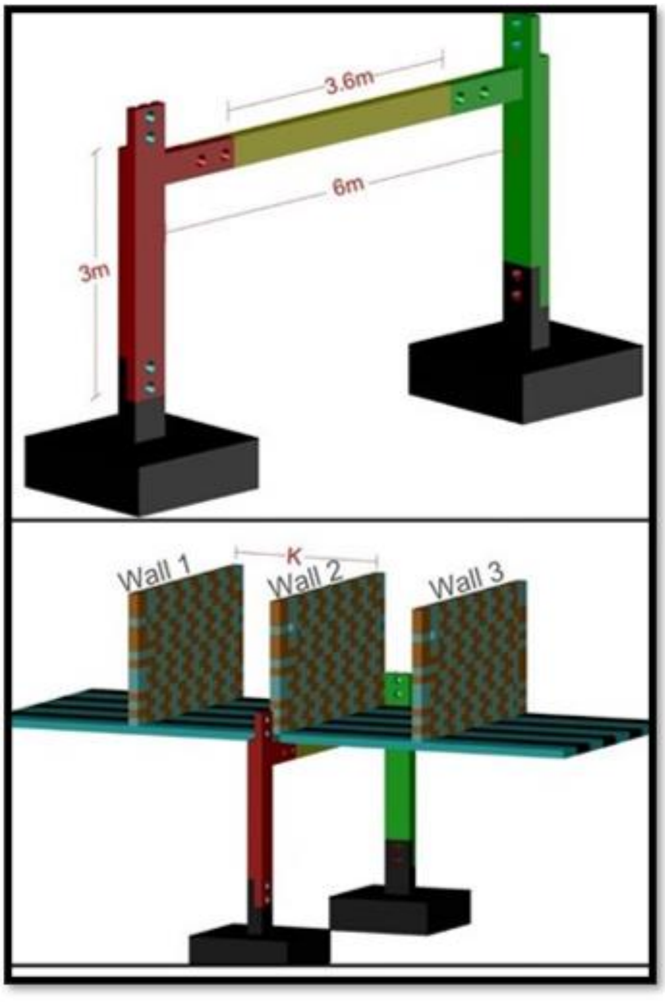

FIG 03: TEST FRAME WITH ALL THE LOADING ELEMENTS

Maximum loading on a beam is observed when wall transfer load directly to slab shown above. Wall1 and Wall 3 are transferring load to the slab directly. Wall 2 is transferring load the beam under consideration. Also load from machinery which will be used during erection is accounted for in the load calculations.

➢ *LOADS ANALYSIS AND DESIGN*

Note: Standard procedure to design the RCC sections of the beam and columns have been used as per codal provisions of IS 456 2000 and IS 13920 2016. Detailed calculations of the same are not included in this study

Loads under Consideration.
1. Beam Self weight
Width 0.2m, depth 0.5m, Self wt per/m = 0.2 x 0.5 x 25 = 2.5Kn/m
2. Wall loads (2 nos)
Width = 0.2m, Height = 2.5m, wt / m = 0.2 x 2.5 x 20 = 10 kn/m
3. Slab dead load.
Length = 6 m, depth = 0.15m, wt/ m = 6 x 0.15 x 25 = 18.75 kn
4. Slab live load.
Live load = 2 kN/m2, total load / m = 2 x 5 = 10 kN/m
5. Walls supported on slab (assuming the slabs are at 2m from the beam)
Height = 3m, width = 0.1m self wt = 0.2 x 3.0 x 20 = 6Kn.
Load on beam = 4kN/m.
6. Machinery load. It is intended to make use of especially designed machinery to erect members in place. The maximum weigh of such elements along with the machinery will be restricted to 4 tonnes. Hence load on each wheel will be 10 kN.
1. Beam Self weight = 2.5kN/m
2. Wall load = 10 kN/m
3. Slab dead load = 18.75 kn/m
4. Live load = 4 kN/m
5. Wall load = 4 Kn/m
6 . Machinery load = 2.03 /kn.
Total UDL ON BEAM = 47.28 kN/m.

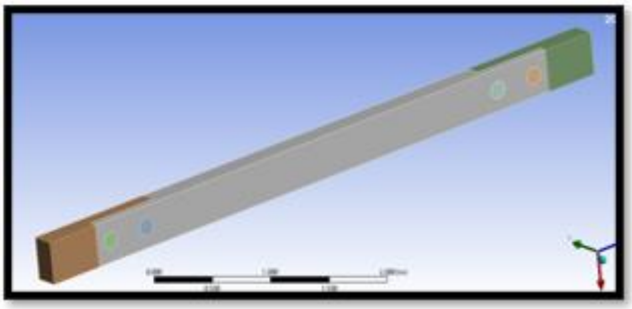

FIG 04: BEAM

➢ *PIN (BOLT) DIAMETER CALCULATIONS*

The connection between the cantilever (which is monolithic with the column) and the beam is achieved by using two pins (central bolt with conical pins) at the distance of 400mm from each other. The view of the bolt and conical bolts (the entire assembly referred to as pins) is shown in Fig 10.

- Calculation of direct shear / reaction on the pin.

Udl (w) = 47.28 kN/m
Let x be the distance between the CG's of the two connection
Therefore x = 4.3 meters(beam span)
Reaction at x = w/2 = 101.652 kN.
This direct reaction is shared by the two rods, Hence reaction per rod = 50.826 kN.

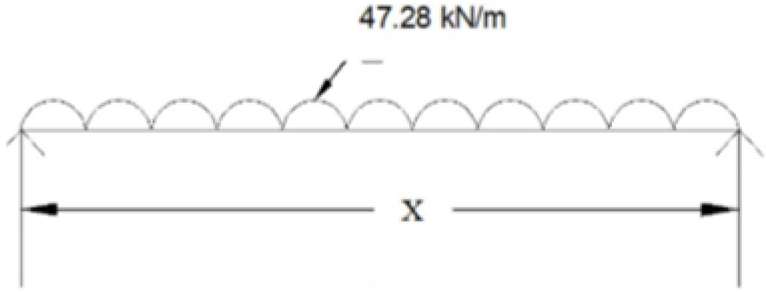

FIG 05: UDL

- Calculation of shear generated due to fixity.

Udl (w) = 47.28kN/m, Reaction = 47.28 x 4.3 /2 = 101.65
Since this will be shared by two pins Reaction per pin = 50.826 Kn.
Fixed End Moment due to UDL

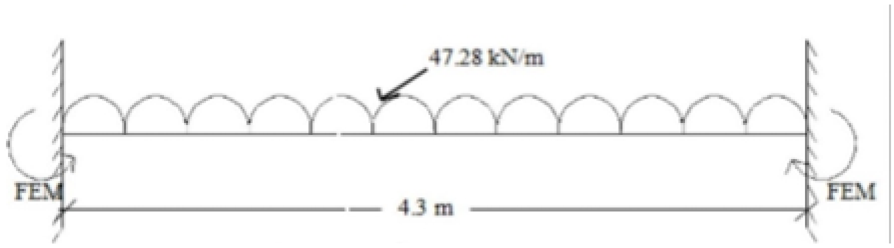

FIG 06: FEM

FEM = (wx2/)12, w = 47.28 kN/m , x = 4.3 . FEM = 72.852 kN-m

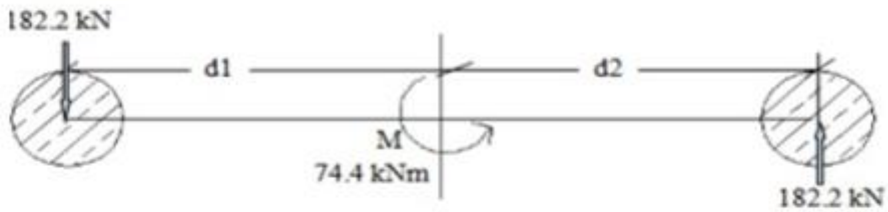

FIG 07: COUPLE

This moment is by a "Couple". Hence the force on the two pins is given by F = M x d / sum (d2), d is the distane between the pins.

D1 = d 2 = d = 0.2m
M = 74.4 kN-m,
Therefore, F = 182.1265 kN.
Total maximum Reaction = 182.1265 + 50.826 = 232.95 kN.
Total Factored maximum Reaction = 349.34 kN.

- Calculation of Diameter of the Pin (bolt) required to resist the calculated shear.

Reaction/Capacity (R) = Fy (Yield strength) x Cross-section Area (A) / Sqrt(3) x Safety factor(S.F)
Cross-section Area = {(R) x Sqrt (3) x (S.F)} / {Fy} R = 349.34 kN
Π x D (Diameter)^2 / 4 = {(R) x Sqrt (3) x (S.F)} / {Fy} S.F = 1.15
Fy = 250 N/mm2

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D (Diameter) = Sqrt ([{(R) x Sqrt (3) x (S.F)} / {Fy}] x 4 /π)
= Sqrt ((349.34 x 1000 x Sqrt (3) x 1.15 x 4) / (250 x π))
= 59.53 mm
= Dia. Of the Pin (Bolt)
that takes shear

Hence 60 mm Dia. Pin (Bolt) is used

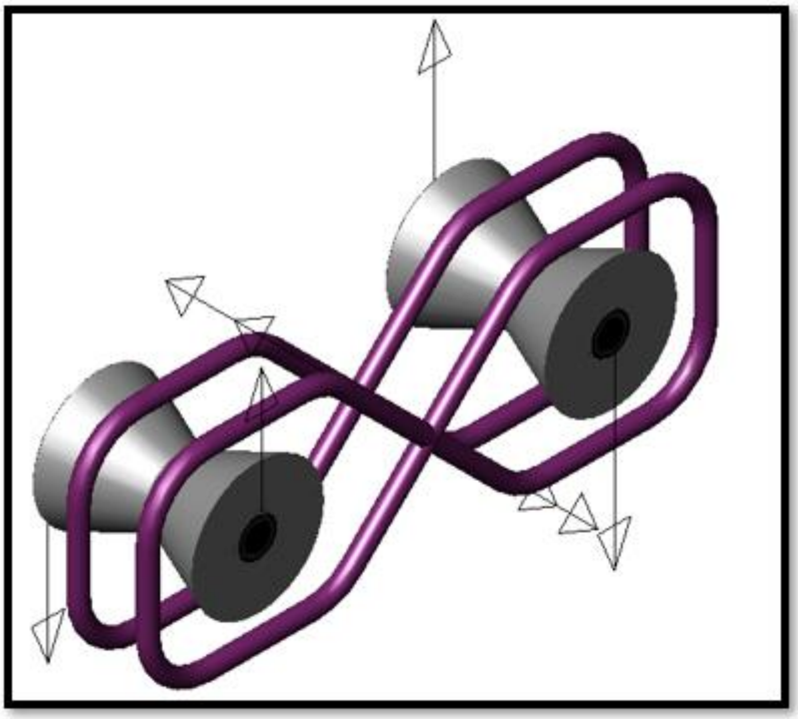

FIG 08: 3D DEPICTION OF FORCES

Tensile Force Calculations in "Infinity Shaped" rings.
As seen in Fig 08, the forces in the pins will generate tensile forces in the "infinity shaped "rings
Factored force = 350 kn.
Resolved force = 350/Sin (45) = 411.32 kN.
Since two rods are resisting this force (one in the beam and other in the cantilever),
Capacity of each 25mm dia bar = fy x Ast.
= 500 x 490 x 2 = 490 kN.
Shear in Connection Region
Vs = 0.87 Fy x Sin(theta) , Fy = 500, Asv = 490.87 Sin (theta) = 0.707
Vs. = 302 kN.
Therefore Nominal Shear Reinforcement is required.

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CHAPTER THREE
GEOMETRY

The Geometry of the Test frame is shown in Fig 09 as an exploded view. This frame is taken from the ground floor of the multi storied frame which is shown in Fig 01. This frame has the worst loading conditions normally encountered in multi storied buildings.

The frame consists of two foundations with stub columns, two columns and a central beam as shown above. The columns have a cross section of 300mm x 500 mm and 3000mm long. The cross section of the beam is 200 mm x 370mm and is 5000 mm long. The splice joint is 700m long in the beam section and is 500 mm long in the column section.

In the connection region, conical cavities are cast to accommodate the conical nuts. These have diameters of 150 mm on the outside and 75mm on the inside as shown above.

M40 -GRADE CONCRETE
FE 415- REINFORCEMENT

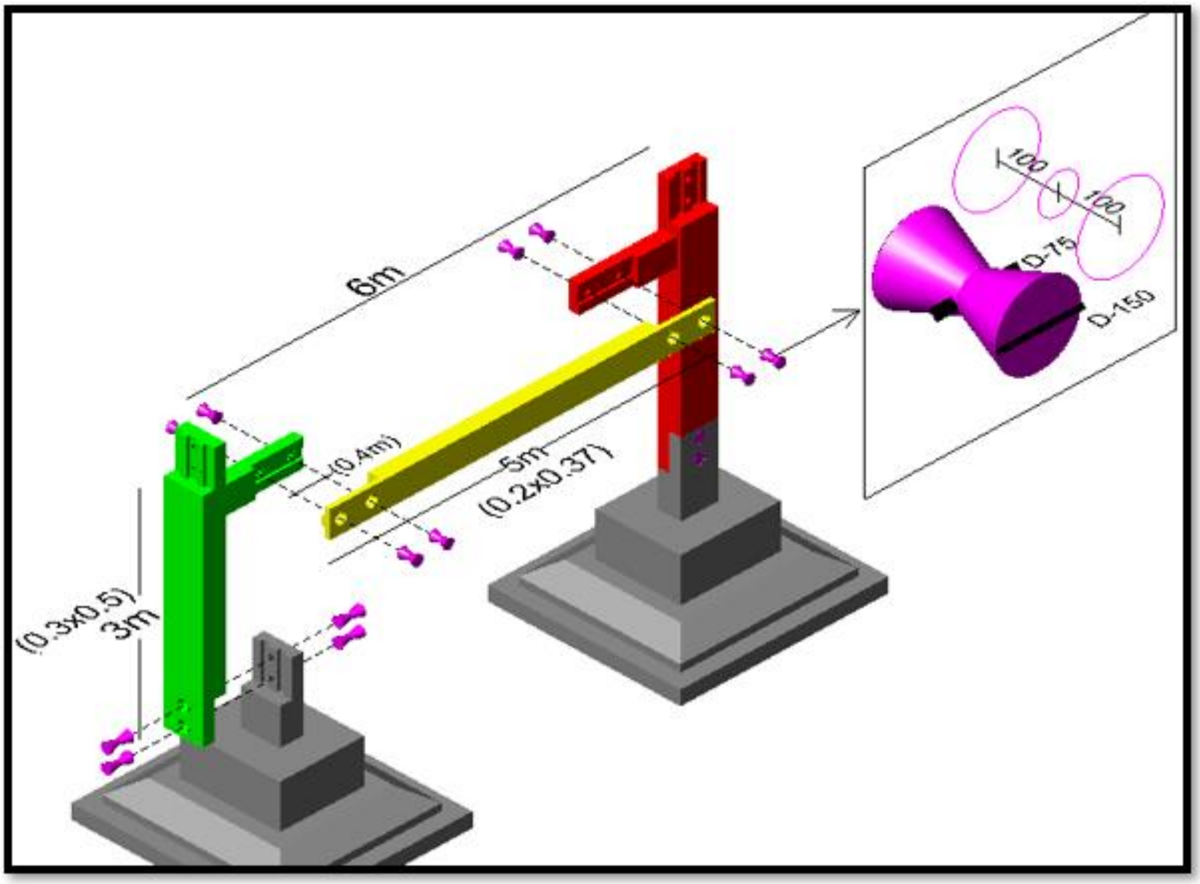

FIG 09: EXPLODED VIEW OF TEST FRAME

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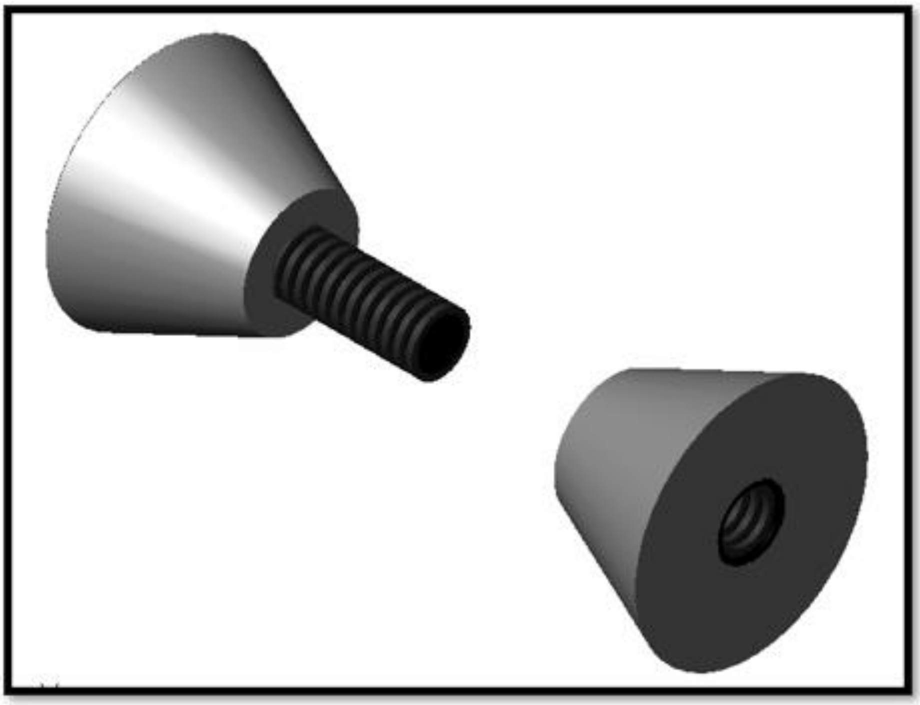

MILD STEEL
FIG 10: 3D VIEW OF PIN (THREADED BOLT) & CONICAL NUTS

Fig 10 shows the assembled arrangement of the central threaded pin. The conical nuts have a female thread and fit perfectly over the threaded bolt. The entire assembly fills the cavities in the concrete thus providing a complete locking arrangement.

FIG 11: PICTURE OF ASSEMBLED PIN (THREADED BOLT) & CONICAL NUTS

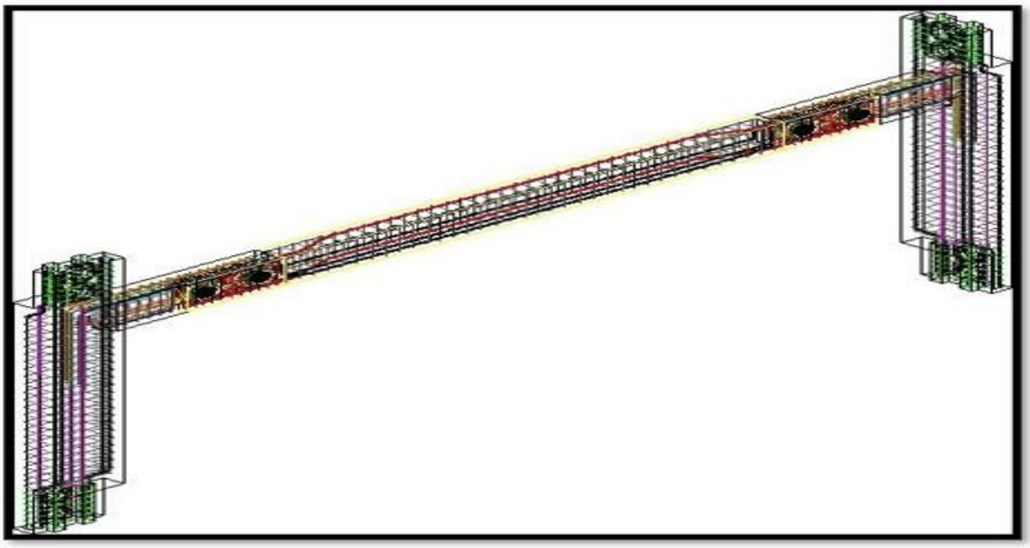

FIG 12: General Arrangement of Reinforcement in the Frame

Fig 12 shows the arrangement of the reinforcement in the entire frame. Note that ductile detailing as per IS: 13920-2016 in the beam-cantilever joint is not disturbed.
Reinforcement details are as follows.

Central Beam	Top Reinforcement	4 nos 20 dia
	Bottom Reinforcement	4 nos 20 dia
	stirrups	8 dia @ 150 c/c
Cantilever	Top Reinforcement	4 nos 20 dia
	Bottom Reinforcement	4 nos 20 dia
	stirrups	8 dia @ 150 c/c
Columns	Longitudinal reinforcement	10 nos 20 dia
	Ties	8 dia @ 150 c/c

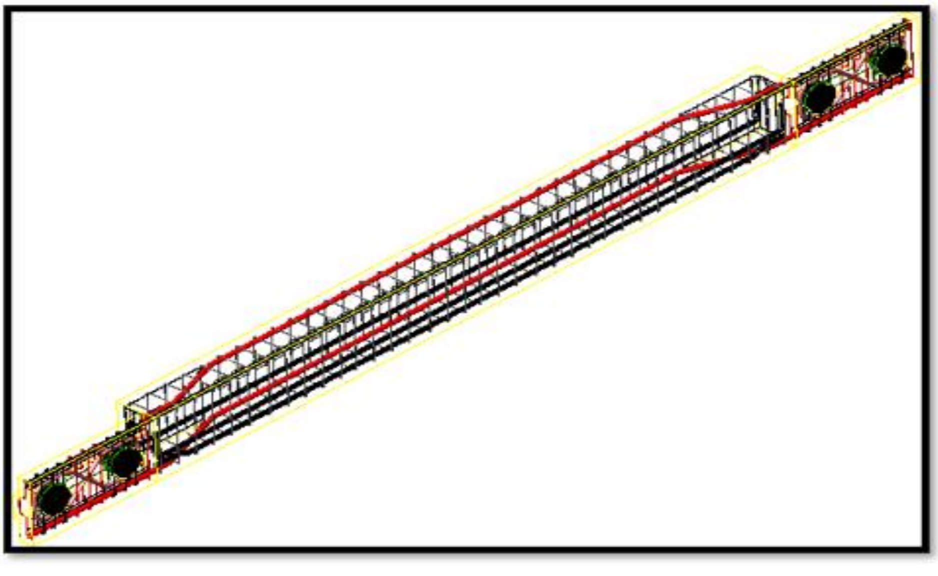

FIG 13: REINFORCEMENT DETAILS OF THE BEAM

Fig 13 shows the arrangement of reinforcement in the central beam

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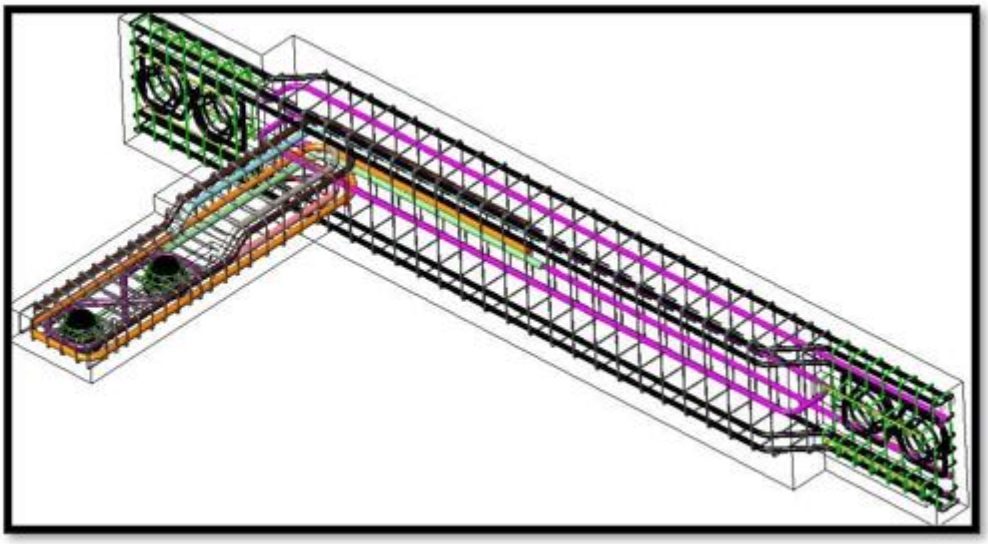

FIG 14: REINFORCEMENT DETAILS OF COLUMN

Fig 14 shows the reinforcement details of the column. The connection regions of the beam and that of the cantilever attached to the column overlap in Male-female fashion and are held together mechanically by the conical pin arrangement as shown in Fig 10 and Fig 11.

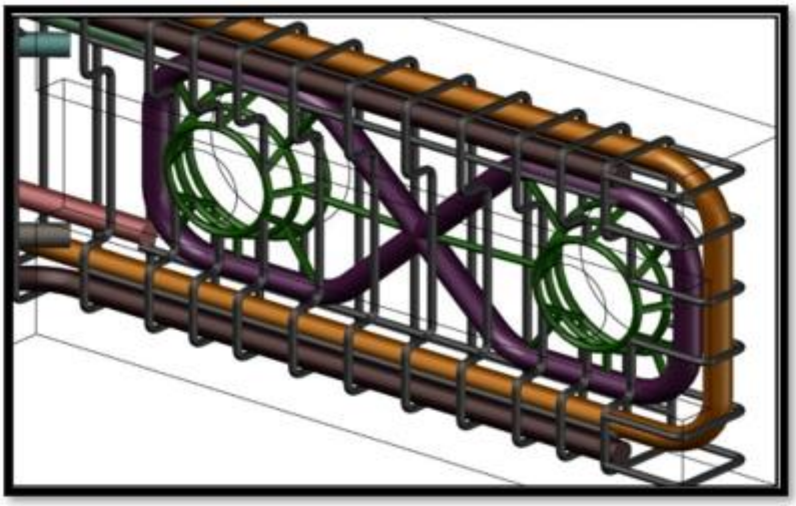

FIG 15: REINFORCEMENT DETAILS OF THE CONNECTION CAGE.

Fig 15 shows the 3D drawing of the connection cage. The “infinity ring” is passed around the cavities so that the two pins passed through these cavities are able to Generate a “couple” in reaction to the externally applied load. generate a “couple” in reaction to the externally applied load.

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FIG 16: CONNECTION CAGE WITH INFINITY RING.

Fig 16 shows the fabricated connection cage. Infinity ring is placed between the top and bottom reinforcement of the cantilever. The same is held in place by a mesh of reinforcement which connects the infinity ring to the concentric reinforcement around the conical caviity.

FIG 17: COLUMN CONNECTION CAGE

Fig 17 shows the cones which are part of the form work to create the conical cavities in the concrete to accommodate the conical pin assembly. Reinforcement is bent at a slope of more than 1:6 so that there is continuity of the same from the body of the column into the connection region. Additonal caging reinforcement is provided wherever required.

- *About Shuttering material:*

Due to the demands of dimensional accuracy, two types of shuttering were experimented with. Wooden shuttering and steel shuttering. Wooden shuttering was found to be easy to work with but accuracy was found to be better with steel shuttering. However, steel shuttering was found to be more costly, difficult to work with.

It has therefore been concluded that a combination of the two materials will be adopted to take advantage of the better properties of steel as well as wood.

FIG 18: REINFORCEMENT IN WOODEN SHUTTERING.

It was decided to keep the shuttering vertical to enable a form finish as the aim is to have a structure with form finish. There will be no plaster used. This will contribute towards sustainability as less sand will be used and also reduce cost and overall project time.

FIG 19: STEEL SHUTTERING.

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Fig 20 and 21 show the columns after de-shuttering. The columns are free bodies and can be transported to the location at which they are to be assembled. The weight of each component, ie. The columns and the beams are designed to be less than 1.5 tonnes so that there is ease of handling during transportation and erection.

FIG 20: COLUMN AFTER DE-SHUTTERING.

In Fig 21 columns are seen upside down. This is done keeping in mind that the CG of the component should be low. This helps during transportation and handling.

FIG 21: COLUMNS AFTER DE-SHUTTERING, READY FOR TRANSPORTATION

FIG 22: ASSEMBLY OF FRAME IN PROGRESS.

The stub columns were grouted into the footings which were cast on site. The columns were then kept in position and fastened by using pins as shown in the picture. The central beam was then fastened. A mobile crane was used to assemble the entire frame.

FIG 23: ASSEMBLED FRAME

FIG 23 shows the final assembled frame. The joints in the beam and columns are clearly seen. The frame is now ready for testing. A suspended loading platform is fabricated on this frame to enable loading upto the design load of 288 kN.

- *GEOMETRY ANSYS MODEL*

Ansys workbench was chosen as the software allows the modelling of reinforcement. The geometry model was done using the modeler in the software. Concrete was modelled as solid bodies and reinforcement was modelled as line bodies. Only the beam was considered as the focus of this study is on the performance of the joint. Therefore, only the cantilever which is monolithic with the column in the actual frame is considered for analysis as is shown below in Fig24, Fig 25 & Fig 26.

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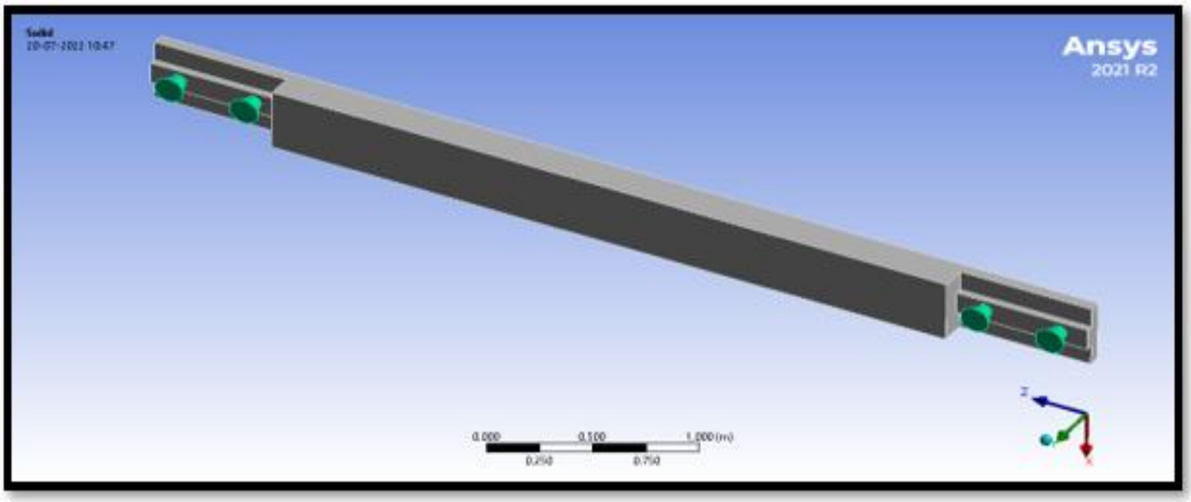

FIG 24: BEAM MODEL WITH PINS (ANSYS)

Fig 25 shows the end of the beam in close up and Fig 26 shows the close up of the cantilever part. The connection regions of the beam and that of the cantilever overlap and the cavities in the concrete are co centric. The two also have matching male-female projections.

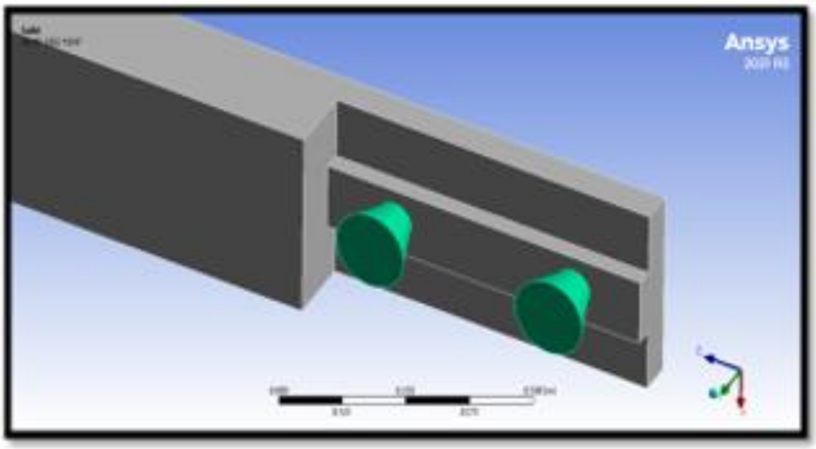

FIG 25: BEAM WITH PINS (CLOSE UP)

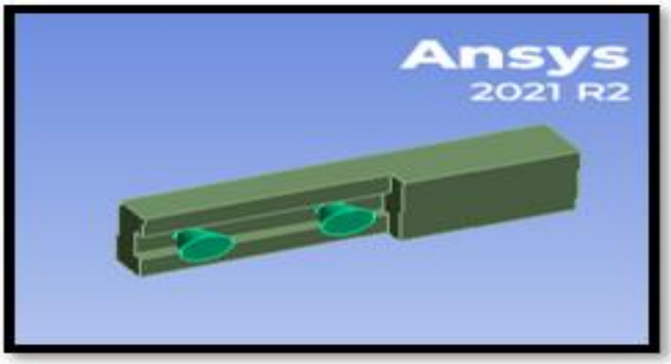

FIG 26: CANTILEVER WITH PINS

The Fig 27 shows the complete assembled model prepared in the ansys modeler. A similar model which is completely monolithic is also prepared so that the results can be compared.

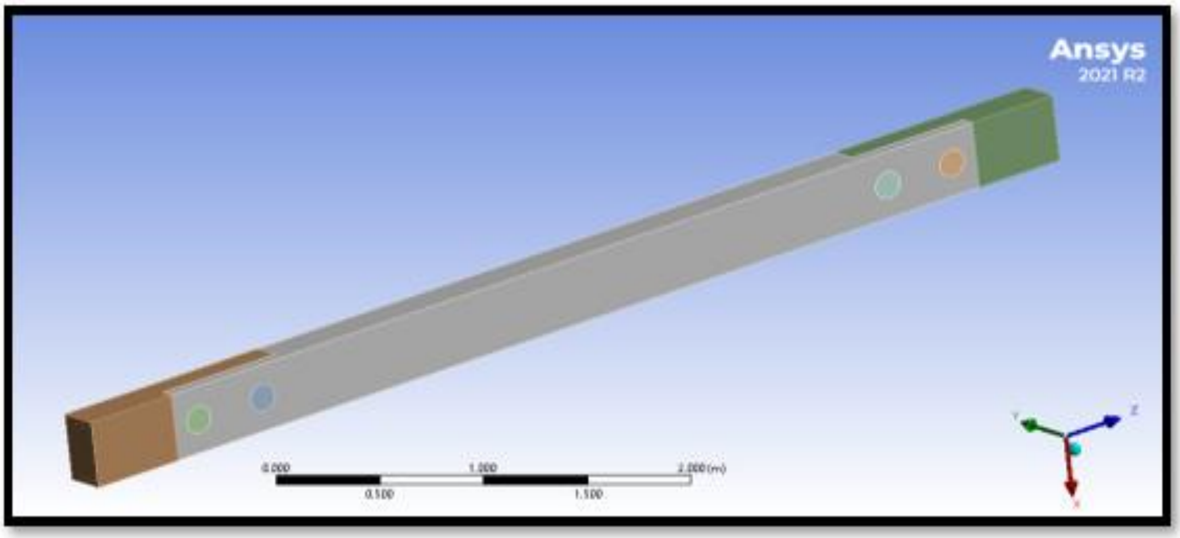

FIG 27: ASSEMBLED BEAM MODEL

Further steps in Ansys are as follows.

- **Assignment of material properties:** Concrete is assigned 40 N/mm2 compressive strength. Reinforcement is chosen as a non linear material with yield strength of 415 N/mm2. Properties of mild steel, i.e. yield strength of 250 n/mm2 are assigned to the Pins and Cones.
- **Deciding the type of contact between the components**: Bonded contact is chosen as the most suitable (default).
- **Meshing**: Default meshing size is opted for. The meshing is done on each of components including the reinforcement. Meshing quality was checked and found to be satisfactory. The same is seen in the results as variation in properties such as deformation, stress and strain are clearly seen. Mesh type and Mesh size control was governed by the software. It is seen that Hexahedral and Tetrahedral elements of average size 2cms was chosen by the software.
- **Boundary Conditions and loading**: The movements in all three directions of the ends of the cantilevers are restricted (Fixed Support, zero degree of freedom). A Uniformly distributed total load of 288 kN is applied on the top surface of the entire beam in the downward direction.
- **Solution:** Final step is to proceed to solve. After the solution, results can viewed for various chosen parameters of Deflection, Stress and Strain.

➢ *MESHING*

Fig 28, shows the meshing in the connection region of the beam. It is seen that the mesh is denser in the conical cavities and also at the intersection of sharp edges. That the density in the conical cavity is much more than that in the other areas is clearly seen.

Mesh type is tetrahedral and Meshing size is 2cms.

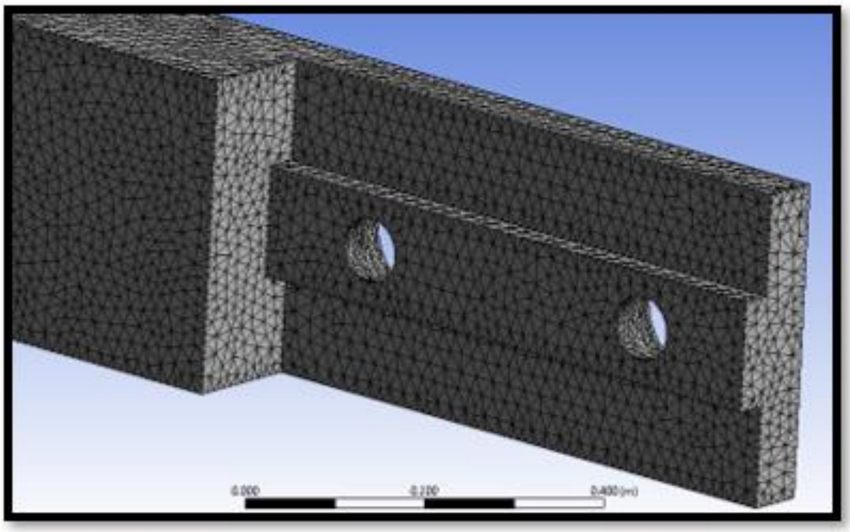

FIG 28: MESHED BEAM

Fig 29, shows the meshing in the connection region of the cantilever. It is seen that the mesh is denser in the conical cavities and also at the intersection of sharp edges. That the density in the conical cavity is much more than that in the other areas is clearly seen. Mesh type is tetrahedral and Meshing size is 2cms.

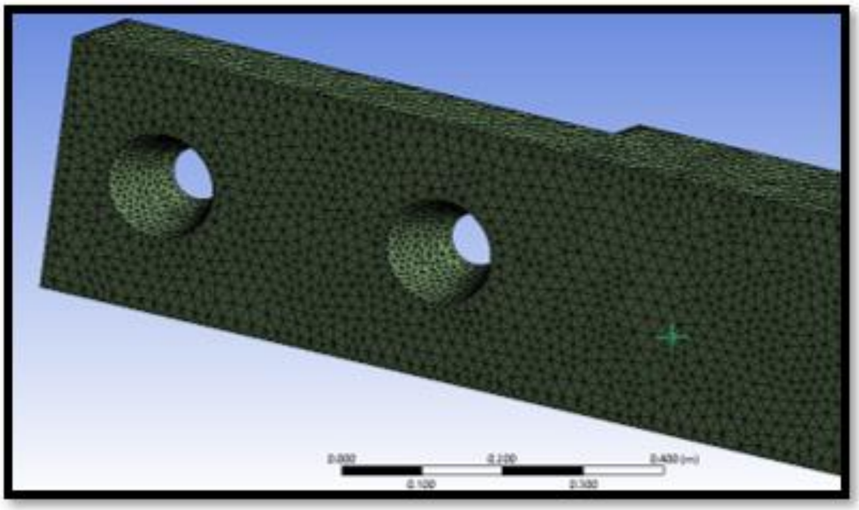

FIG 29: MESHED CANTILEVER

Fig 31, shows the meshing in the Pin. It is seen that the mesh is denser in the central part of the Pin. It is also seen that the meshing adopts an irregular shape in the central part of the pin and is also much denser than the other areas. The meshing on the conical surface is Hexahedral. It is seen that the nodes in the central pin match the inside surface of the pin.

Fig 30 shows the meshing in every reinforcement bar. The software treats the reinforcement as line bodies. The meshing seen on line bodies has a circular pattern.

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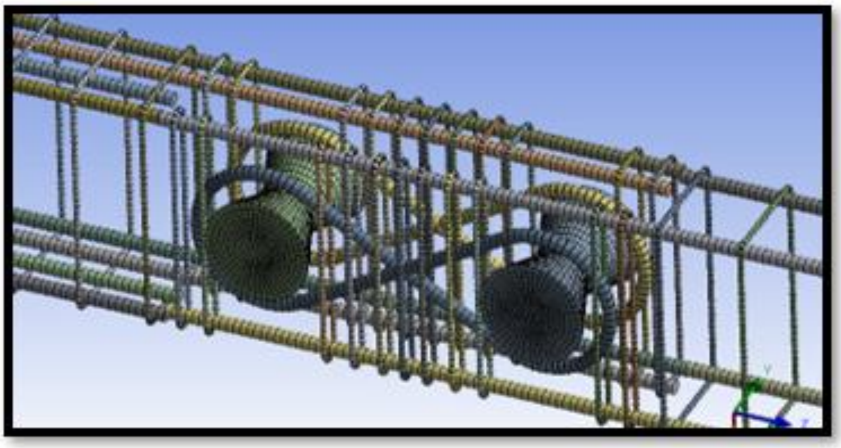

FIG 30: MESHED CONNECTION REINFORCEMENT

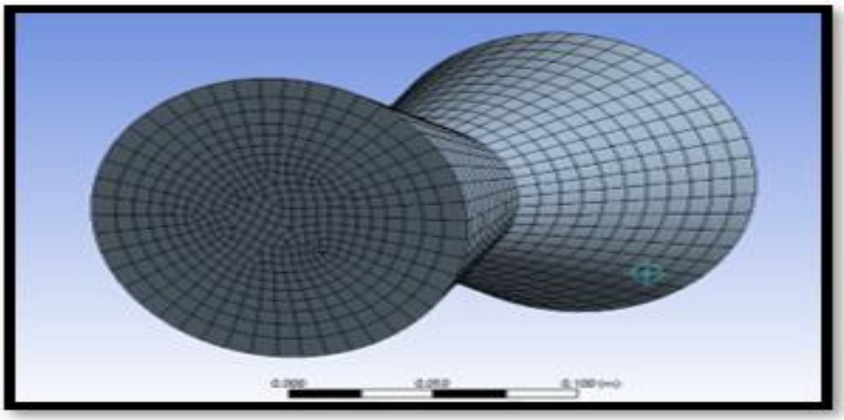

FIG 31: MESHED PIN

➢ *LOADING AND BOUNDARY CONDITIONS*

Fig 32 shows the loading & Boundary conditions The Faces A & B of the cantilever are marked as fixed. A total load of 288kn is distributed over the length of the beam. both the ends of the beam are fixed.

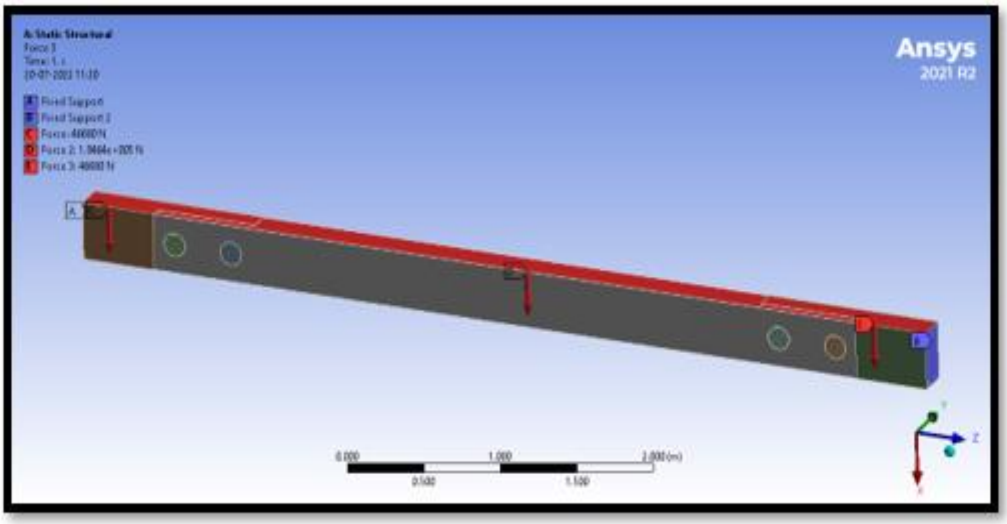

FIG 32: LOADS AND BOUNDARY CONDITIONS.

CHAPTER FOUR
RESULTS

❖ *RESULTS- I*

➢ *FEM ANALYSIS*

It can be seen from Fig 33 and Fig 34 that the contours follow a similar pattern for the assembled and the monolithic model. The deflection for the assembled model is 1mm more than monolithic model

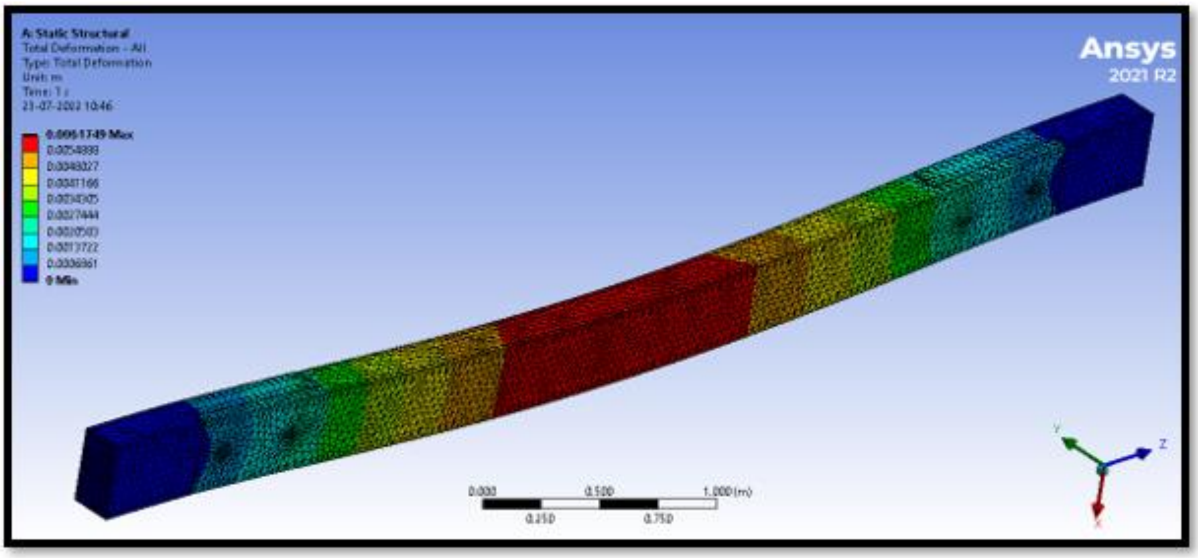

FIG 33: DEFLECTION IN ASSEMBLED BEAM.

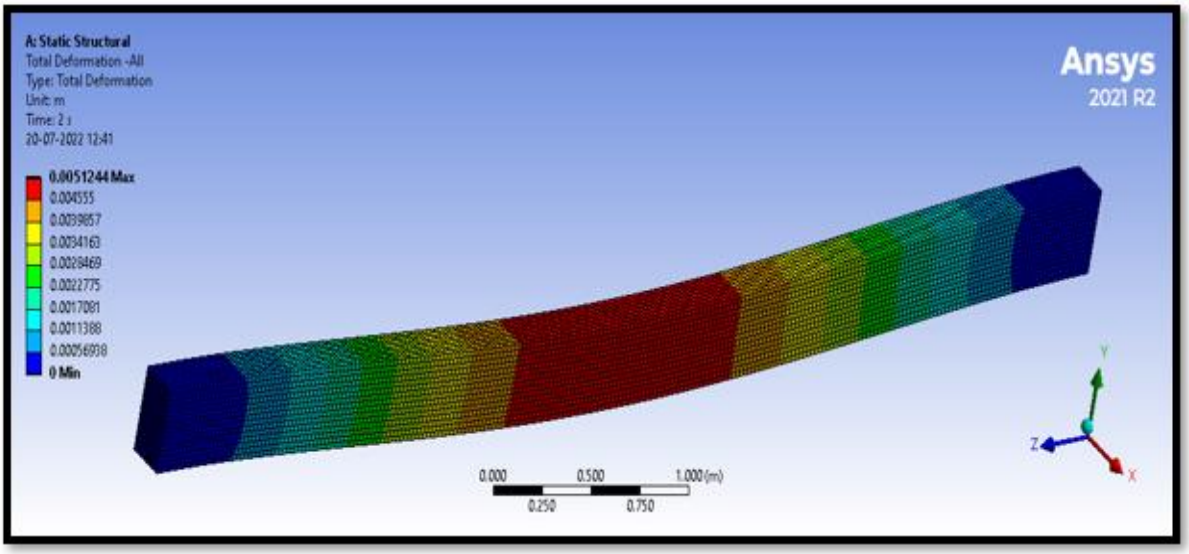

FIG 34: DEFLECTION IN MONOLITHIC BEAM.

➢ *DEFLECTION*

Fig 35 shows that the meshing pattern at intersecting edges is denser than plane surfaces. The mesh around the pins is also denser. Since the scale of the deflection in the vertical plane is distorted, the shift in the alignment at the joints is noticeable. It is also noticed that the type of mesh in the monolithic model (Fig 34) is different from that of the assembled model.

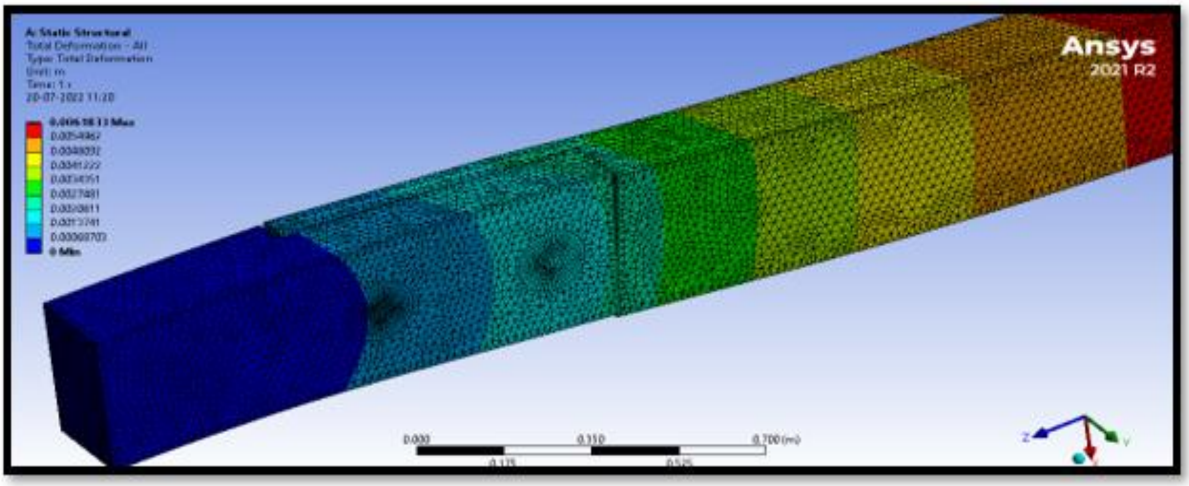

FIG 35: DEFLECTION IN CONNECTION REGION

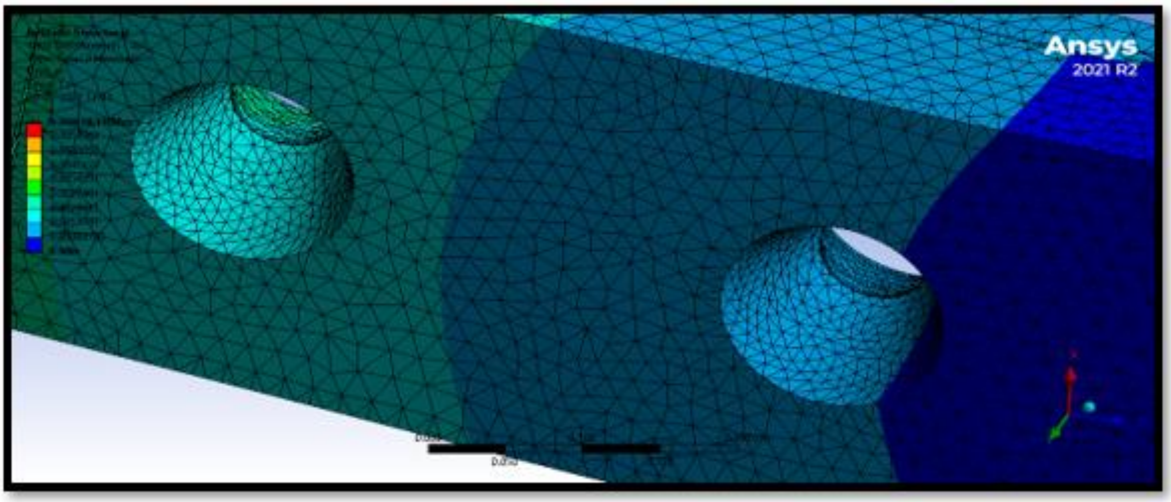

FIG 36: DEFLECTION IN CONICAL CAVITIES IN CONNECTION REGION.

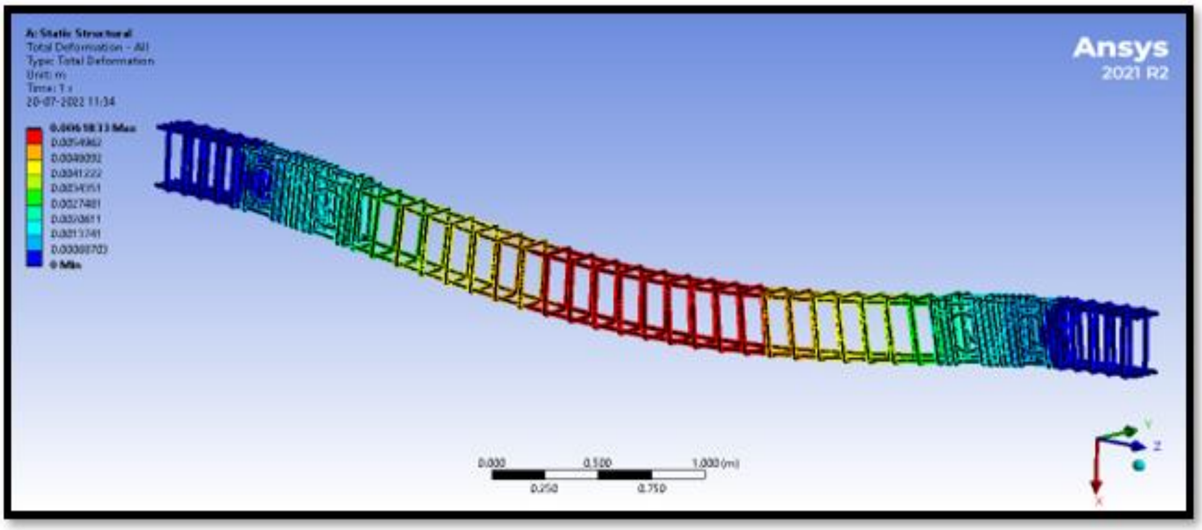

FIG 37: DEFLECTION IN REINFORCEMENT

➢ *DEFLECTION*

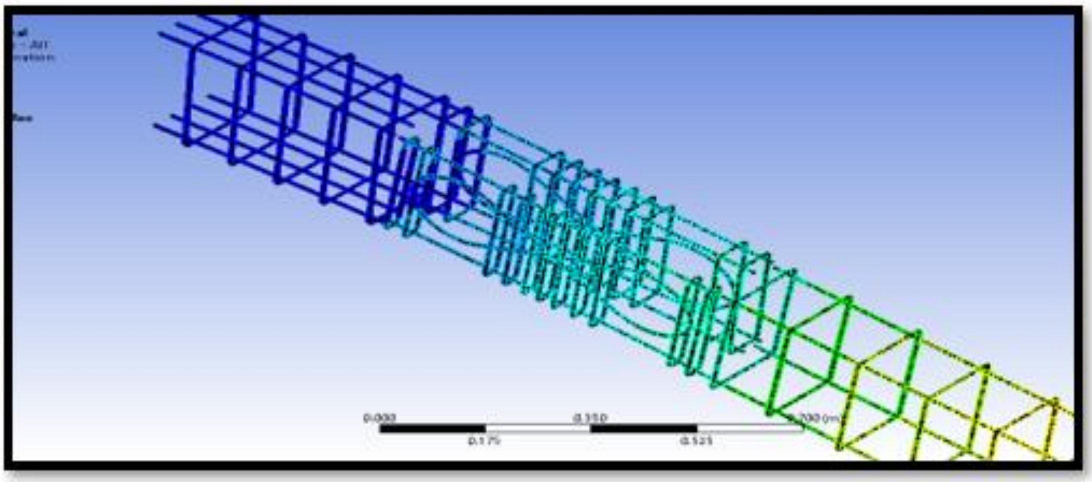

FIG 38: DEFLECTION IN ASSEMBLED REINFORCEMENT CLOSE UP

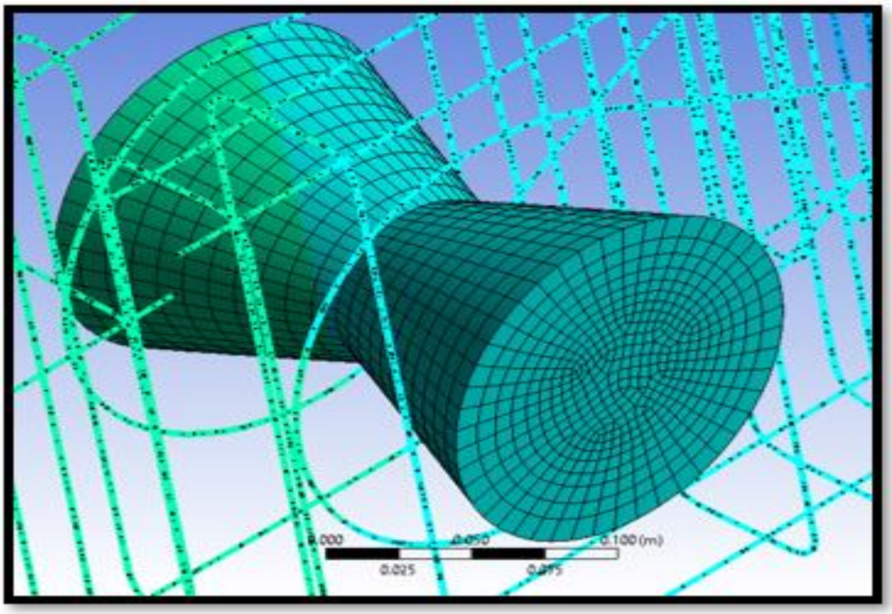

FIG 39: DEFLECTION IN CONICAL PINS

➢ *STRESS.*

From Fig 40 & Fig 41 it can be seen that stress distribution in assembled model and the monolithic model is similar. Stress in both the models is within max permissible limits. It can be seen that the stress in the central band in the neutral axis region is the least and increases in the bands nearing the top and bottom fibres of the beam section. It is also noted that the stress in the contraflexure region is minimum in the entire section of the beam. Thus the decision to locate the joint in the proximity to the points of contraflexure is validated. The top and bottom fibres near the fixed ends are seen to have higher stresses.

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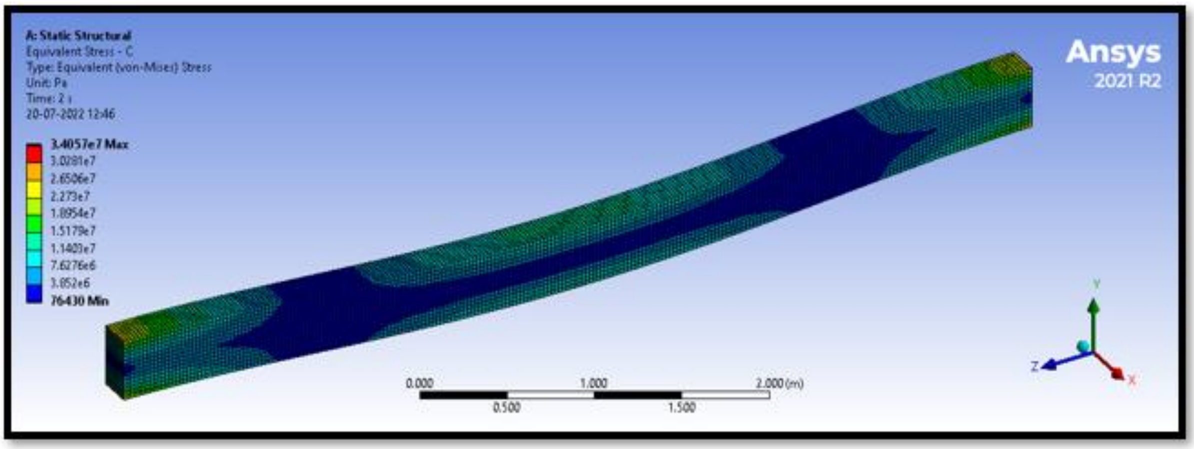

FIG 40: STRESS IN MONOLITHIC BEAM.

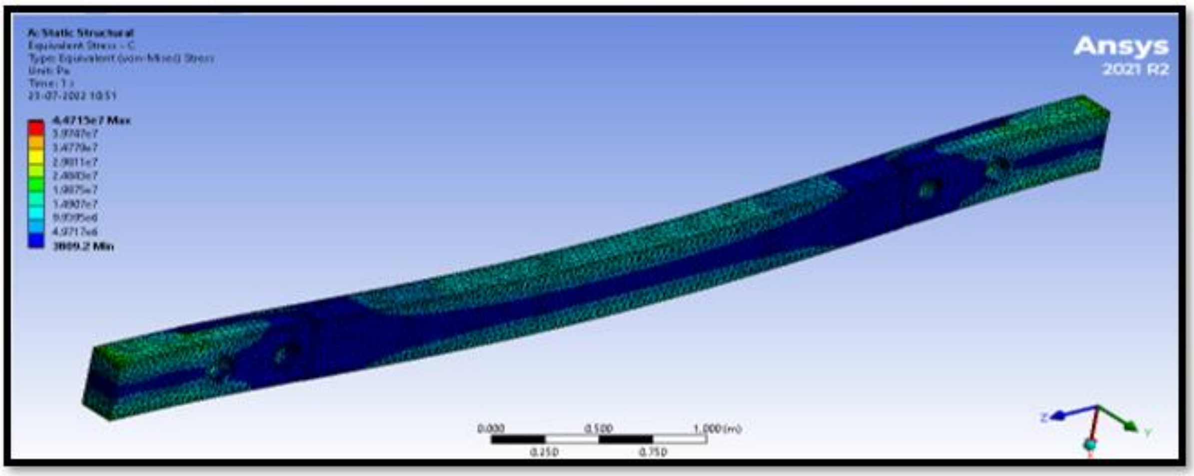

FIG 41: STRESS IN ASSEMBLED BEAM.

➢ *STRESS.*

Fig 42 shows that the stress concentration in the first concrete cavity is higher than the surrounding region but is with permissible limits. It is also seen that the section around the first pin of the beam has higher stress levels than the section around the second pin.

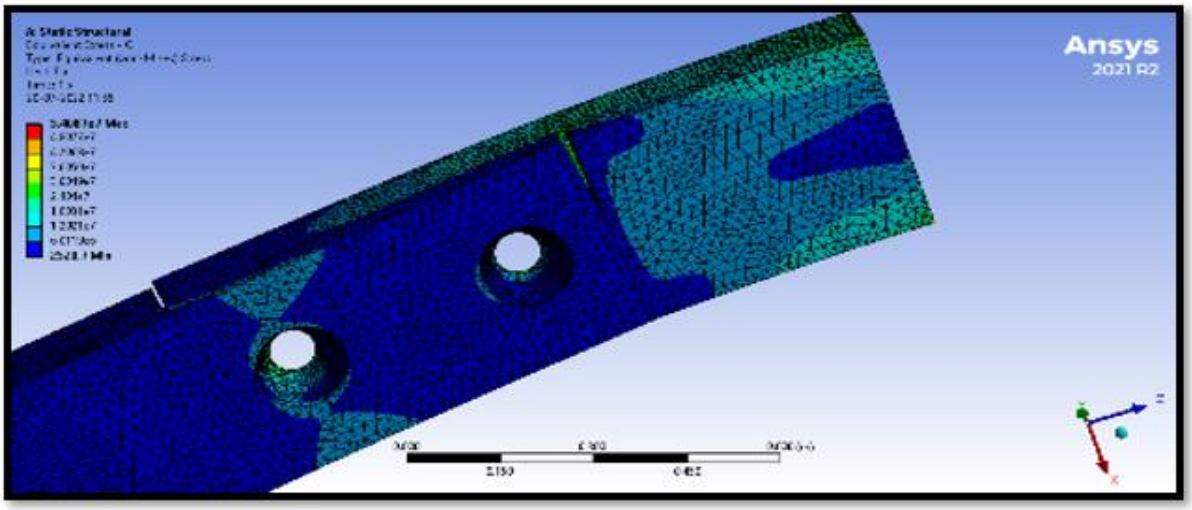

FIG 42: STRESS IN CONNECTION REGION

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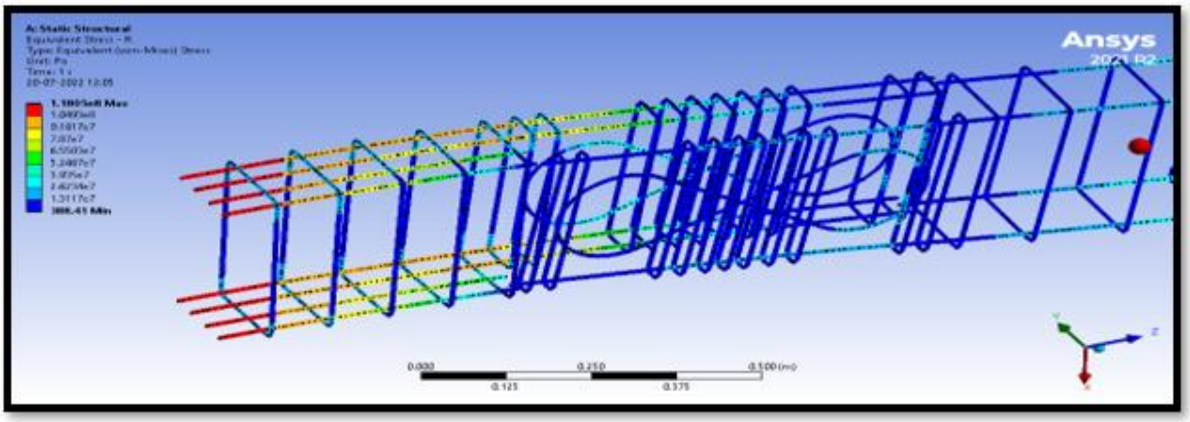

FIG 43: STRESS IN CONNECTION REINFORCEMENT.

FIG 44: STRESS IN INFINITY

Fig 44 is a zoomed in picture showing the higher stress in the infinity rings. It also shows the diametrically opposite sides of the infinity rod are stressed more.

- *STRESS.*

Fig 45 shows the stress distribution pattern across the conical pins. As the diameter is least in the centre, the stress is seen to be highest. However, the stress is within permissible limits.

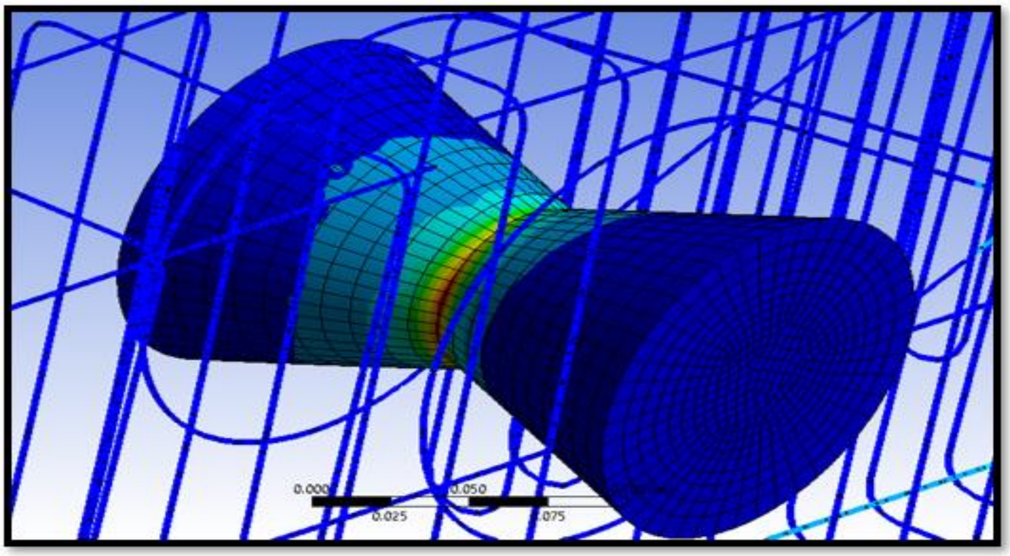

FIG 45: STRESS IN CONICAL PIN

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Fig 46 Shows stress in concrete is also maximum at the edge (where the dia is least) but is within permissible limits

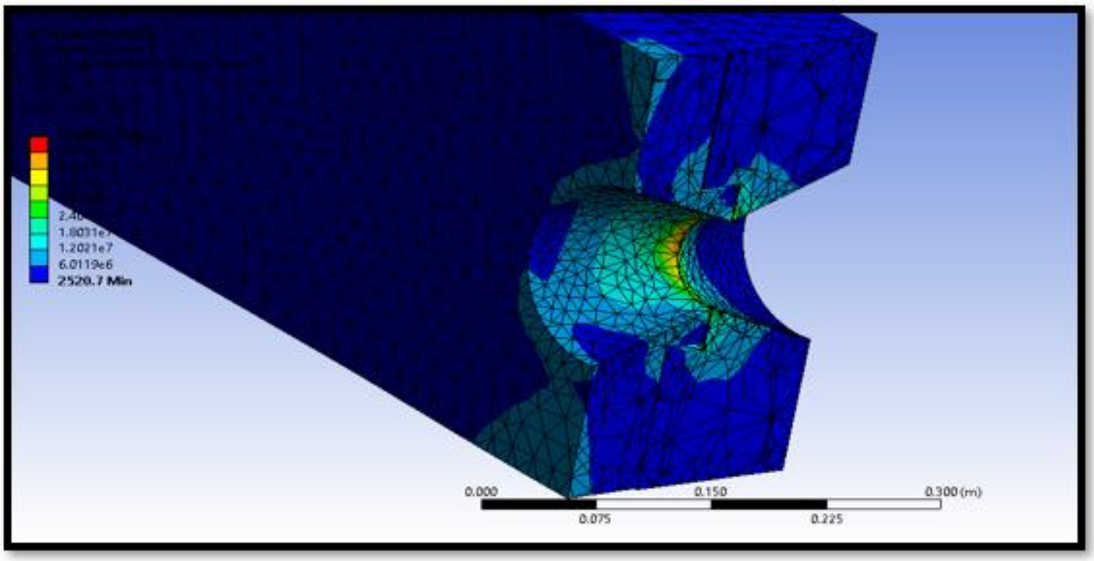

FIG 46: STRESS IN CONICAL CAVITY – SECTIONAL VIEW

➢ *STRAIN*

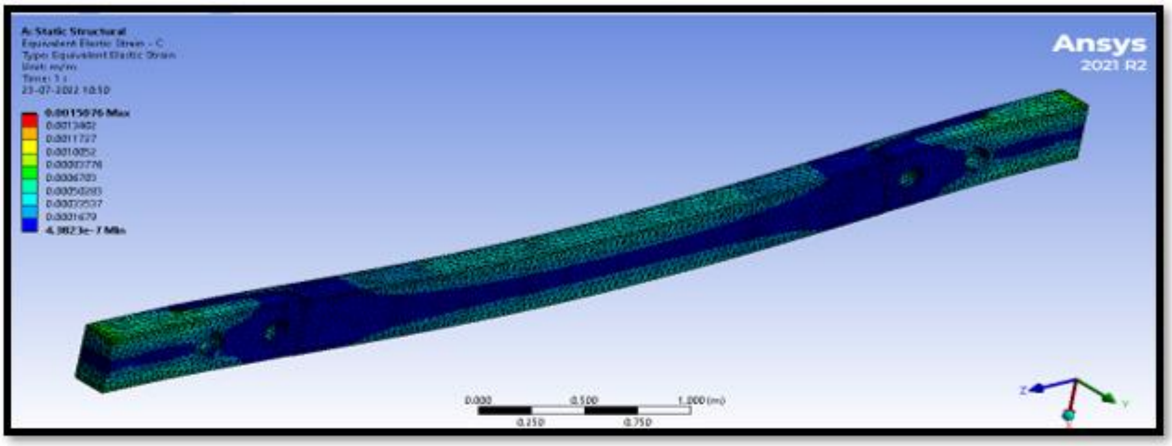

FIG 47: STRAIN IN ASSEMBLED BEAM.

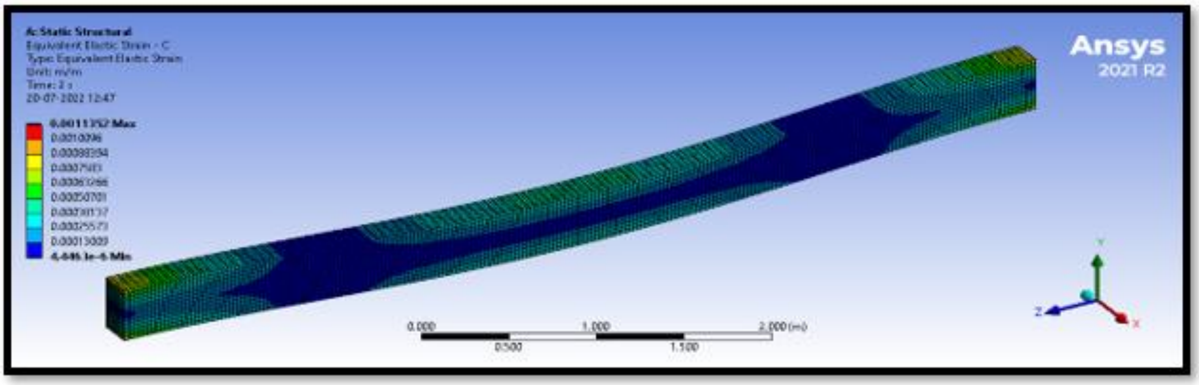

FIG 48: STRAIN IN MONOLITHIC BEAM.

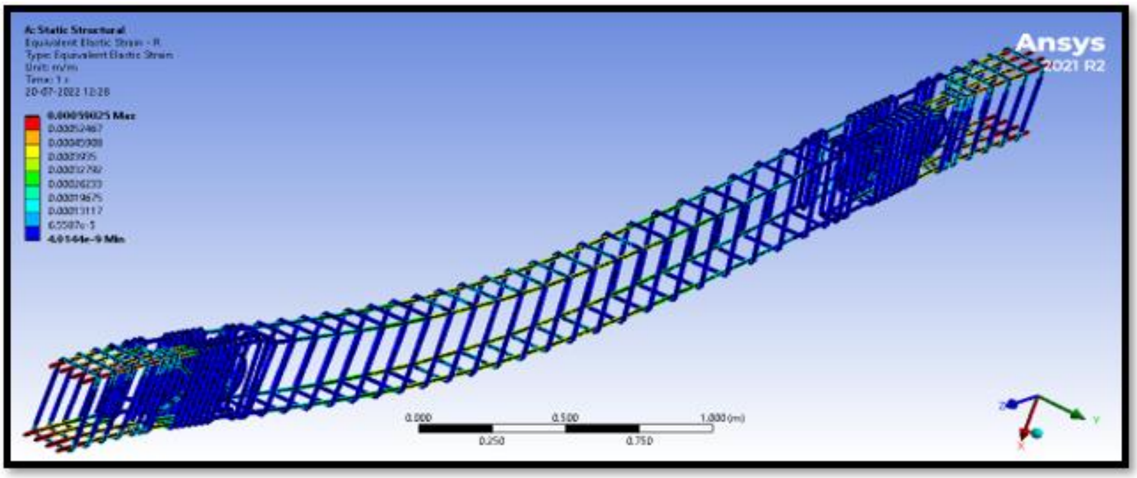

FIG 49: STRAIN IN REINFORCEMENT OF ASSEMBLED BEAM

- *STRAIN.*

Fig 50 Shows that the strain is higher in the concrete cavity as well as the in the Pin where the diameter is the least. Also, there is similarity between the stress and strain contours in the region.

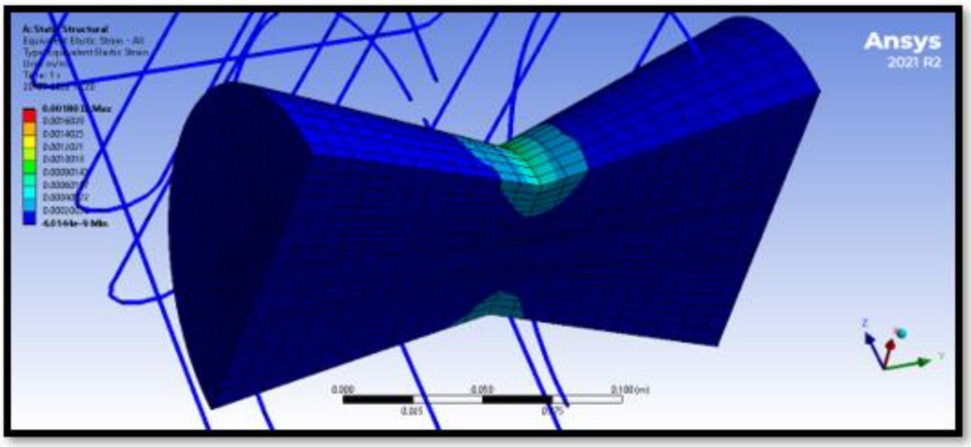

FIG 50: STRAIN IN PIN – SECTIONAL VIEW

From the Fig 51 it is seen the strain is maximum in the center and decreases as we move outward

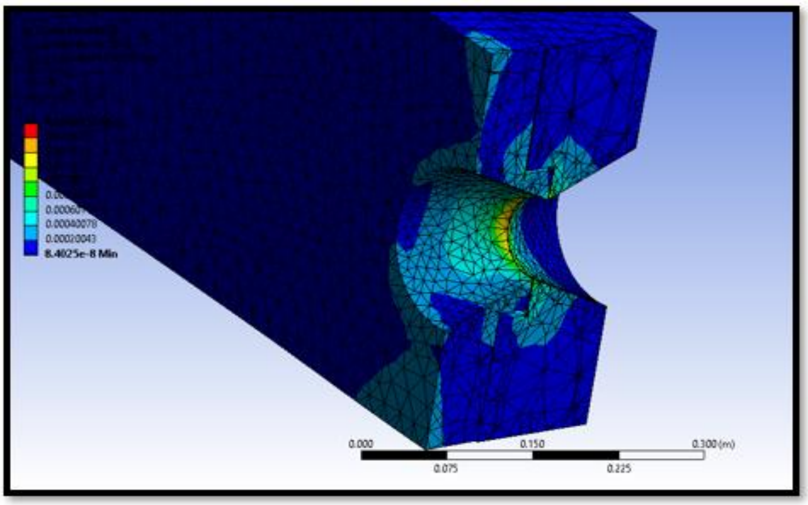

FIG 51: STRAIN IN CONICAL CAVITY -SECTIONAL VIEW

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❖ *RESULTS- II EXPERIMENTAL VERIFICATION*

FIG 52: LINEAR POTENTIOMETER

➢ *TESTING PROCEDURE FOR DEFLECTION*

- Deflection Measurement Using Linear Potentiometer The measurement of the deflection for the beam is carried out with the help of linear potentiometers.

- Working Principle

Linear Potentiometer changes mechanical linear movement of the piston stroke into electrical signals. The Linear potentiometer is an electrical device comprising a resistor with a sliding piston, which allows the voltage to be varied depending upon where it is positioned along the length of the resistor. The magnitude of this output voltage is directly proportional to its relative position along the length of the resistor. The potentiometer is appropriately connected to a moving system then any movement in that system will cause the piston to move resulting in change in the output voltage. This signal provides a direct measurement of position or changes in position. Hence, although still a potentiometer, it is of use as a sensor for measuring linear displacement.

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- Methodology used for measurement of deflection

Step 1 – Marking of deflection measurement locations

Three different locations were marked on the beam (shown in the Fig)

Step 2 – Surface preparation

Surface preparation is done to enable proper contact between tip of Linear Potentiometer and the surface of the beam. The surface should be flat where the tip of Linear Potentiometers is to be placed.

Step 3 – Fixing of Linear Potentiometer

The linear potentiometer is fixed in such a way that it is independent of the Structural member vibrations and other movements. The platform to support the linear potentiometer is made to be even and the same is checked with the spirit level. The linear potentiometer is clamped on its stand which is fixed and secured with the platform. The contact with the bottom of the slab is established with the head of the plunger such that the initial reading is recorded with the corresponding change in the voltage.

- Data collection and analysis The data collection and analysis is done with the help of DGC cDAQ 9178 software for deflection. Three locations as shown is Fig 55 are marked as P1, P2 & P3 where the deflections are measured by installation of the Potentiometers as described above.

FIG 53: POTENTIOMETER INSTALLED.

FIG 54: DATA ACQUISITION – SET UP

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FIG 55: FULLY LOADED SUSPENDED PLATFORM.- SIDE VIEW

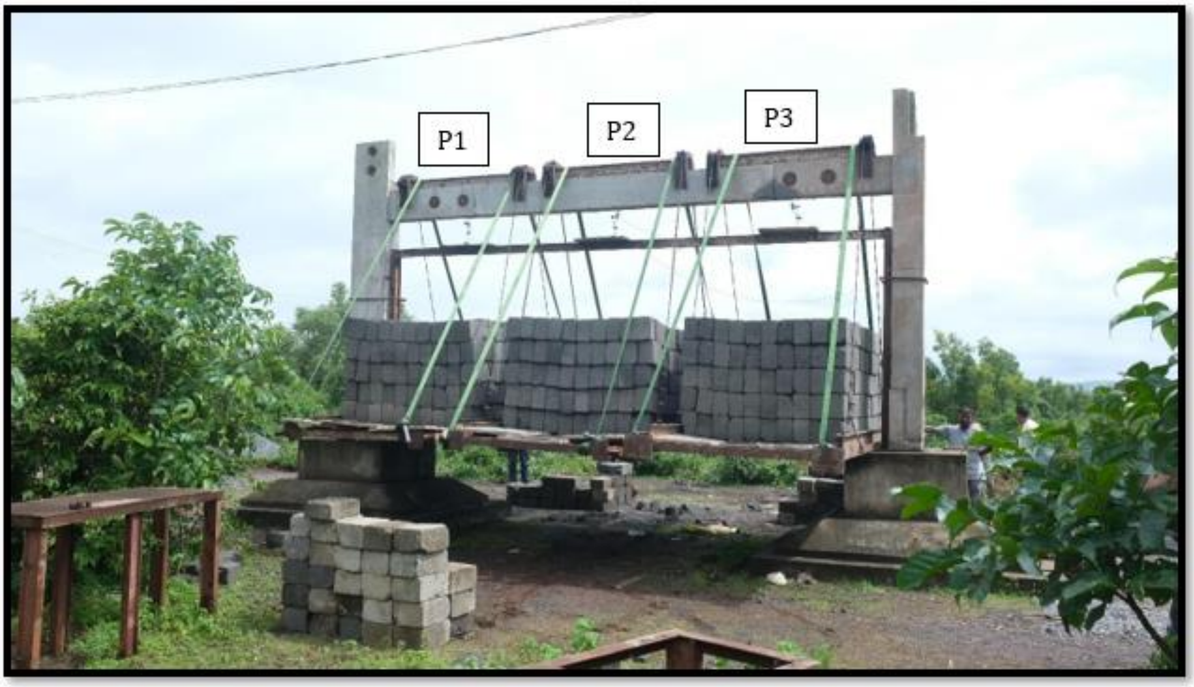

FIG 56: FULLY LOADED SUSPENDED PLATFORM

FEM Model of the entire frame is made on Ansys software. Probes are inserted to record the deflections at the locations marked as P1, P2 & P3. The deflections at the locations are as seen in the Fig 57 below.

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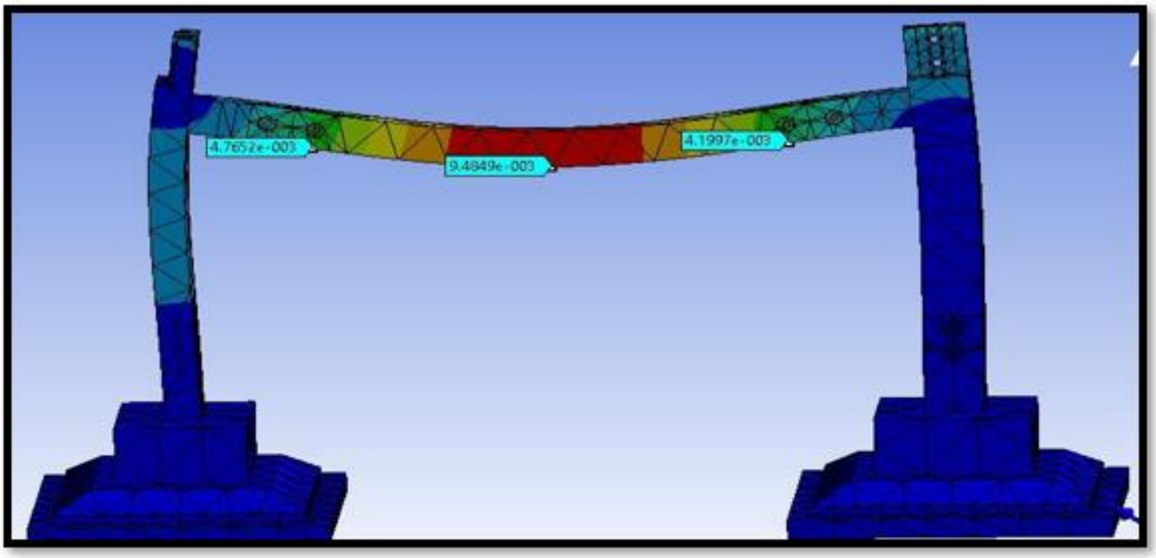

FIG 57: FRAME WITH DEFLECTION VALUES – ANSYS

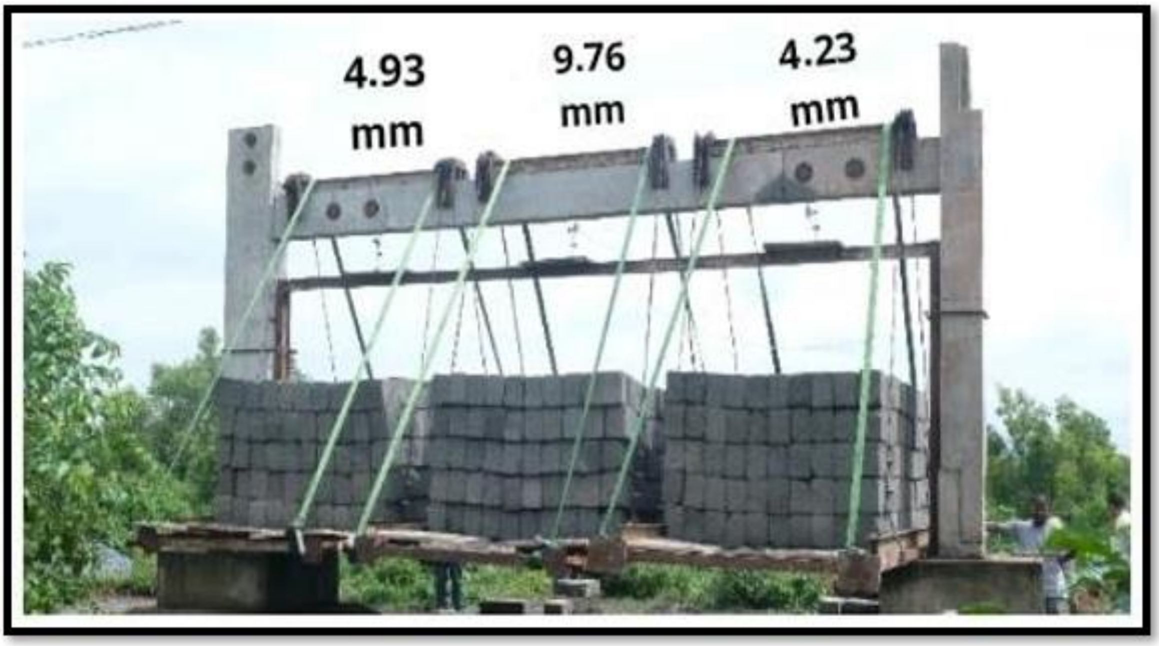

FIG 58: ACTUAL FRAME WITH MEASURED VALUES OF DEFLECTION

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CHAPTER FIVE
CONCLUSION

❖ CONCLUSION I –

(Comparison of results between Ansys models of Assembled and Monolithic beams.)

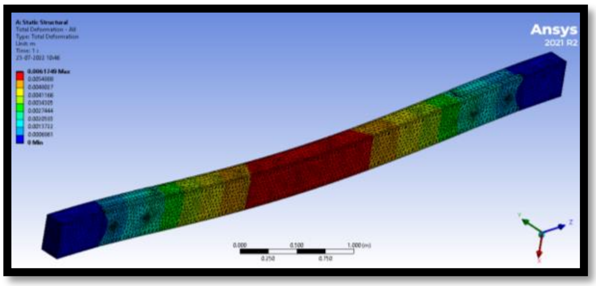
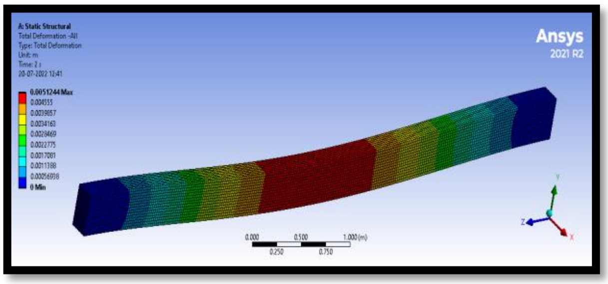
ASSEMBLED BEAM	MONOLITHIC BEAM
	
Max deflection = 6.175 mm	Max deflection = 5.124mm

Table 1:- Comparative deflection of Assembled and Monolithic beams in FEM analysis

Permissible deflection for a span of 6m is 17.14mm (span / 350)as per cl 23.2 of IS 456 (2000)

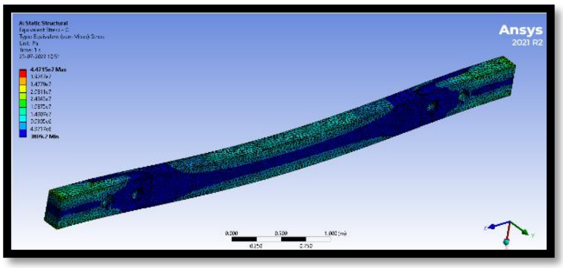
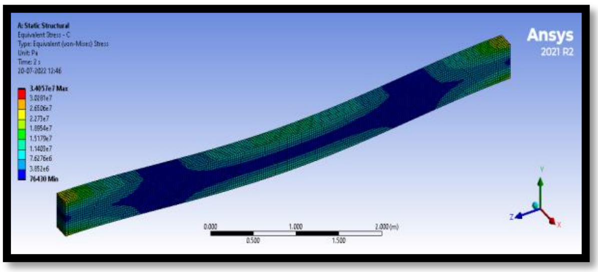
ASSEMBLED BEAM	MONOLITHIC BEAM
	
Max stress = 6.1 N/mm2	Max stress = 3.85 N/mm2

Table 2:- Comparative stress of Assembled and Monolithic beams in FEM analysis

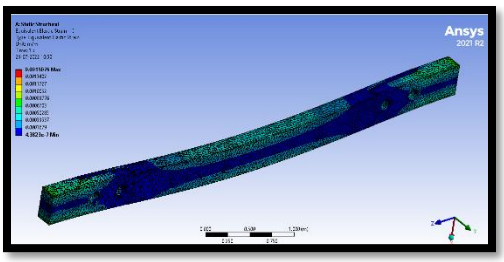
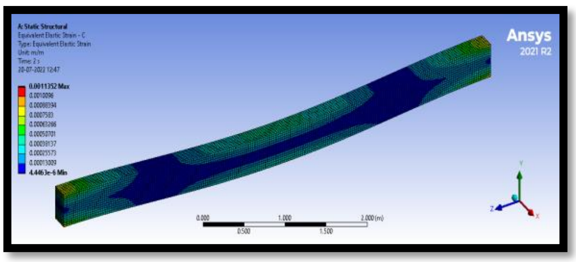
ASSEMBLED BEAM	MONOLITHIC BEAM
	
Max stress = 0.0002	Max stress =0 .0001

Table 3:- Comparative strain of Assembled and Monolithic beams in FEM analysis
(Permissible = 0.0035 as per IS 456)

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Conclusion I: From Table 1 & Table 2, it can be seen that deflection of the assembled beam is more that of the monolithic beam by 1.06 mm. However, it is well within the permissible limits of Span / 350 = 17.14mm**.** Parameters of Stain and Stress are also within permissible limits. **Therefore the efficiency of the joint is successfully established Therefore the efficiency of the joint is successfully established**.

		P1		P2		P3
initial reading		14.48		13.54		18.77
layer 1		12.58		11.64		17.93
layer2		11.62		9.75		17.09
layer3		10.67		7.85		16.25
layer4		10.02		5.96		15.41
final reading.		9.72		4.06		14.57
Difference		4.76		9.48		4.20

TABLE 4:- DEFLECTION READINGS CAPTURED ON SITE.

	P1	P2	P3
CONSTRUCTED TEST FRAME	4.93	9.76	4.23
FEM MONOLITHIC MODEL	4.76	9.48	4.2

TABLE 5:- COMPARISON OF DEFLECTION BETWEEN ASSEMBLED PRECAST TEST FRAME AND MONOLITHIC FEM MODEL

❖ **CONCLUSION II** :

The total deflection (as measured on field) compare well with those from the ansys simulation as shown in Table 7. **It can therefore be concluded that the jointing system designed performs efficiently and can be adopted for construction of multi storied buildings.**

➢ *INFERENCE*

The designed Pre-Cast method can be adopted for construction of multi storied structures. However, following points need consideration.

- There is need for further testing of Space frames with static loads and seismic (lateral) loads.
- Other components like slabs and walls and their connections with beams and columns need to be designed and tested before adoption.
- Refinement in Shuttering design to enable faster assembly and dis-assembly is also required.
- There is need for standardization in section sizes to achieve speed and efficiency.
- Need to develop erection and handling machinery.

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