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Design of Precast System of Construction for Multi Storied Buildings

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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.

I. OBJECTIVES.

1.To design a system for jointing of structural elements to facilitate PreCastbuilding construction.

The joint between various structural elements such as beam to beam andcolumn to column should notaffect the ductility of the beam- column jointas the aim is to ensure that the performance of the Pre-Caststructure is notdifferentfromthatofacast insitu,monolithic structure.

2.Toensure thatthejointtransfersshearandmoments efficientlysothattheperformance of the assembly is as close as possible to that ofa cast insitu,monolithic structure.
Conventionaldetailingofstructuralmemberswillnotbesuitabletoachieve the above objective. Therefore, innovative detailing is requiredtoensure that stresses generated from the action of the connection pinsareadequatelyresisted.

3.To select a test frame with worst loads and spans normally occuring in aG+8structure.
As we want to ensure that this precast system is suitable for buildings uptoG+8, it is necessary to identify worst conditions of loads and spans that willoccur realistically.The test frame dimensions and loads should be chosen aspertheaboveobjective.Itshouldbeacceptabletoadopttheresultsforstructureswithlesser floorsandspansbyscalingdowntheresults.

4 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 are compared for concrete and also the reinforcement critical locations is compared for the two models.

5To 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 PotentiometersA model of the monolithic frame is made and analyzed using Ansys software. Comparison of resultsfrom simulationtothoseofthe static load test is made.

II. LITERATURE SURVEY

1.StudyonPropertiesofPrefabricatedConcreteStructuresunderSeismicLoad(1)
InternationalJournalofAdvancedResearchinScience, CommunicationandTechnology(IJARSCT) Volume11,Issue1, November2020 .

•T.Subramani,2018 Hestatesthatprecastconcreteisa notableinnovation in which, for rapid growth, some standardised units produced inproductionfacilitiesare used.

AlsoareferenceventureintheChennairegionistakenandshownintheETABS.programmingtobreakdownthe structureandplan.

•Majid Divan, 2011 Postulates,for public multi-storey buildings, the use ofprefabricatedconcreteframeswithprefabricatedconcreteshearwallsduetoimproved performance quality and reduced production time may be a goodoption

•W.C. Stone, 1998 The analytically simulated performance under seismicloads of moment-resistant precast concrete frames with hybrid connectionsisevaluated.Toreflecttheinelasticbehaviourofthehybridprecastlinkarea,animprovedand flexiblehystereticmodelwasdeveloped.

2. **ComparativeStudybetweenRCCStructuresandPrefabricated Structures.**(7)
SiddhantSinghal,BilalSiddiqui
InternationalResearchJournalofEngineeringandTechnology(IRJET)2.
Thispapertalksaboutthemeritsanddemeritsofprefabricated Construction,it alsostates why there is a need ofPrefabricated System(PS).Along withdifferenttypesof PS.

3.Prefabricationinhouseconstructions InternationalJournalofLow-Carbon Technologies XudongZhao,SaffaRiffatThis paper examines the development and status of prefabricationtechniquesandtheirapplicationinbuildingconstruction.Anoverviewofthecurrent UK house building market and its status in terms of the utilization ofprefabrication techniques has been made. Investigation of past engineeringpractices and existing knowledge of prefabrication has allowed several lowcost techniques to be summarised. These would minimise the initialinvestment required to adopt prefabrication and so increase its marketpotential forUKhouseconstruction.

4.PrefabricationinDevelopingCountries:acasestudyofIndia. RyanE. SmithAssistantProfessor,ShilpaNarayanamurthyGraduateResearcher
Prefabrication in India began with the emergence of the Hindustan HousingFactory(HHF). The HHF pioneered the production of pre-stressed concreterailwaysleeperstoreplacedilapidatedwoodensleepersonIndianRailways.
Prefabricationtechnologyisaproductiontechnologyorknowledgebasedand notaconsumption technology or productbased.

5 Use of Prefabrication to Minimize Construction Waste (8)- A Case StudyApproach
C.M. Tam ,Vivian W.Y.Tam,JohnK.W. Chan&WilliamC.Y.Ng
This paper uses four private building projects as case studies to demonstratethe effectiveness in the use of prefabrication to minimize construction wastein Hong Kong. The wastage levels of the four projects are compared withconventional cast in-situ methods under similar project natures andconditions.ThehindrancesandthefuturetrendofadoptingprefabricationinHongKong arealsoexamined.

6 Prefabricationtechniquesforresidentialbuilding(9)
MSPalanichamy,MepcoSchlenkEngineeringCollege,
This paper deals with the prefabrication techniques for residential buildingusingasystemofPrecastunitsforcolumns,beams,roofandwalls.Precast R.C.C. planks and partially Precast R.C. joists are considered for flooringIroofingsysteminthispaperandspecialtypesofPrecastwallpanelsarerecommended. Prefabricated columns with a specific configuration, beamsand staircase units are considered in this paper. Special emphasis has beenmade with respect to the various joints and connections and the details ofthesearediscussed.AcomparisonofthecostofconstructionofPrecastsystem with that of a conventional construction unit has also been made.Finally, identified that large scale adopting of such a

Precast systems willeventually result in considerable cost reduction with the added advantages ofexecutionspeed.

7 REVIEWOF TESTINGPROTOCOLSFORPRECASTCONCRETE STRUCTURALCOMPONENTS ….IITK (1)

From the different literatures, reports and state of the art articles around the globe, itcanbeseenthatresearchinprecastconcretestructuresisstillneedvigorousresearch work to overcomevarious difficulties and uncertainties in implementation.As per the need of time, proper guidelines and codes considering precast concretedesign and detailing are required for bench marking the construction process usingprecast concrete. Also, proper numerical and experimental investigations are neededto developstandard codes fordifferenttype of loading scenarios fordifferent typesof structures. In Indian context, there are no proper guidelines to implement precastconcretetestinganddesignforconstructionpurpose.Hence,itisnecessarytocollaborate with government research laboratories and research institutions to comeforwardwithajointventuretodevelopthehandbookforprecastconstructionpractices.

III. INTRODUCTION

THE NEED FOR THE PROJECT. Multi-storey RCC framed buildings in India havebeen constructed by using the same techniques, material anddesign over the last 75 years.There have been upgrades in thematerial used,the construction methodologies in practice andtheapproach to designing suchstructures.ForExample,therehavebeenattempts atmechanizationinproduction and pouring of concrete.(RMC) The strength of concrete used hasalmost doubled. We have seen progress in design from working stress method tothe limit state method of design. Mode of design has moved on from laboriousmanual calculations to F.E.A. based computer programs which can incorporatecomplexloadssuch asseismic andwindloads.

However, the end results are not very different from what they used to be. Westillhavestructureswhichsufferfromdefectswhich areintrinsictoconstruction which is carried out on site. We still have structures which sufferfrom improper concreting, poor curing and poorly monitored lines and levels.Moreover,suchstructuresstilltaketoolongtoconstruct.Theaveragecompletion time of a 5 to 8storey structure is anywhere between one andhalfto four years, depending on size and design. In today’s fast paced world, thisneeds to be brought down to 6 to 8months. Saving in time is the only way for abuildertosaveonconstructioncostandatthesametime achieveamuchsuperiorproduct.

THE THOUGHT PROCESS.

Itwasthereforedecidedtotakeupawholenewapproach tothewayinwhichmulti-storeybuildingsareconstructed.Thethoughtprocessis listedbelow.

Prefabricated–Pre-engineeredTechnologytobedeveloped.

1.Theentireconstructionprocesswillshifttoafactory. Thecomponents will be manufactured in a factory and will be transported tothesitewhere theywillonlybe assembledmechanically.

2.Designofcomponentswillbestandardized.Betterdesigntechniquescanbeused.Amodularapproachtodesignwillbeused.

3.Muchstrongermaterialcanbeusedastheirqualitycanbemonitored inafactoryenvironment.

4.Betterplanningfortheconstructionprocessispossibleasproduction schedules are more reliable when undertaken in a factory duetolessornumberof variables.

5. Better testing is possible.

6. 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 theNationalHighwaynetwork.Therefore shipping of the product to the national and internationalmarkets 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 focushasnowshiftedfromtheluxurysegmenttomasshousingforthelowandmiddleincomegroup.Thistechnologyshallhavegreatrelevanceinthescenario.Also,slumredevelopmentprojectswheretransitcampsarerequiredis a potential market as the structures build by using this technology will bedesigned in such a way that they could be dismantled and erected at any otherlocation.

InnovationandproposedTechnology:

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Although there exists many technologies for prefabricated homes, most of themcater to "mass housing" where a large number of houses/homes or units havingthesameplan,elevationandfinishesareconstructed.Thesearemainlyconstructed by making walls in a factory which is normally located on the samesiteas thatof theproject.Thewallsarethen erectedin place byusingheavylifting machinery and tower cranes. Part of the construction is still necessarilydoneon the site.
Thetechnologyunderdevelopmentshallnotimposeanylimitationsonarchitectural design. This technology will be used to construct structures whichare presently constructed using conventional methods, thus yielding volumeswhicharerequiredtosustaintheoperations.Thistechnologyissuitedtoconstructmulti-storeybuildingsinareaswhicharealreadydenselybuilt without disturbing the other dwellings in the vicinity as there is no sound, anddust pollution.It is intended to use pre-engineered concrete components which **will have specially designed efficient joints to transfer forcesand moments** to form the structural elements of the frame. The design of thecomponents shall be standardized thereby resulting in savings in terms oftime(for design) and reduced wastages. A full-fledged work floor with state oftheartcad cam machinerywithEOTcranesetc. isenvisaged.

Corephilosophy:

Move the joints way from the beam-column junction: The biggestchallenge to be overcome for such structures is the efficiency of the joints ofdifferentelementslikebeamsandcolumns.Thefirststepwastoensurethatonlytwomembersareconnectedateveryjointinsteadofattemptingtojoinmore thantwomembers. It was therefore decided to move the joint location from the usual beamcolumn junction to a distance as close as possible to the point of contraflectureintheBendingmomentdiagramwhich usuallyoccursbetween0.15to0.2L.

Joint Detailing: The maximum shear on the central pins is calculatedand the governs the dia. The pin is threaded with suitable gauge. The insideof the cone has a matching female thread. The cone is made out of mild steel.Theconcreteiscastwithaconicalcavitytomatchtheconeshape.Circularreinforcement,meshsuitablytransfersthestressfromtheconcretesurfacetothespeciallydesigned "infinity shaped" rod passing around the two pins to resist theshearing forces.

Splice joints: It was decided to adopt splice joints as it is desirable forthe joint to be aesthetically pleasing. The two sides of the splice will be heldtogether by means of conical pins which will be secured by suitable threadson a central pin. External loads on the elements will generate reactions onthese cones/pins which are opposite in direction and will form a couple. Thereinforcement inside the concrete is designed to resist the shearing forcesthat aregenerated.

Standardization: In order to achieve economy in form work design, itis necessary to adopt standard sizes of sections for various elements likebeams,columnsandslabs.

Formworkdesign:Thedemandontheformworkisasfollows.

•**Dimensional accuracy**: Since the structure is going to be assembled unit byunit, dimensional accuracy of units especially in the joint section is of utmostimportance. Without dimensional accuracy, the joints will not be possible ifglobalpositionsoftheholesarenotachieved.

•**Form finish:** As the intention is to avoid plastering completely, it is necessaryto achieve surfaces which are absolutely true in a plane have a surface which isreadytoreceivepaintfinish directly.

•**De-shuttering and Reassembly of Forms:** Since time is of essence, the formwork is designed to minimize the time taken to assemble the formwork, placereinforcement and pour concrete. Sides are also designed by keeping in mindtheeaseofDe-shuttering.

•**Number of uses**: The design of the formwork is also done by keeping in mindthat the same is going to be used for as many uses as possible. Therefore, thereis provision to replace the material which comes in contact with concrete asandwhenrequired easily.

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 outin 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 modelsimulated in ansys software is made with defections of the actually constructed frameand measured with LVDT instrumentation. (Part II)

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Part I (F E M ANALYSIS)

●Atestbeamrepresentativeofmaximumspansandloads(Span:6m)(load=48kn/m) generally occurring in commercial and residential structures isselectedforthestudy.Andthesameismodelledandanalysedin AnsysSoftware.

●The assembled beam having two cantilevers and a central beam is joint togetherattwolocations usingpins,the endsarefixed andstaticloadisapplied.

●Arepresentativemodelwasdonebymakingsuitablechangestothereinforcement detailing of the constructed model so as to arrive at an efficientmeshaspertheansys requirement.

●TotalDeformation,StressandStrainvaluesoftheabovemodelweregenerated.

●Similarlyalltheabovestepswererepeatedforamonolithicbeamforthecomparisonwith eachotherandalsothepermissiblevaluesasperIS456.

Part II (EXPERIMENTAL VERIFICATION)

●The actually constructed frame was loaded up to design load and the deflectionsweremeasuredusingLinearPotentiometers(LVDT).SimilarframewasmodelledinAnsys andthedeflectionswerecompared.

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. Figures 01 and 02 show the frame and typical loading of the selected frame.

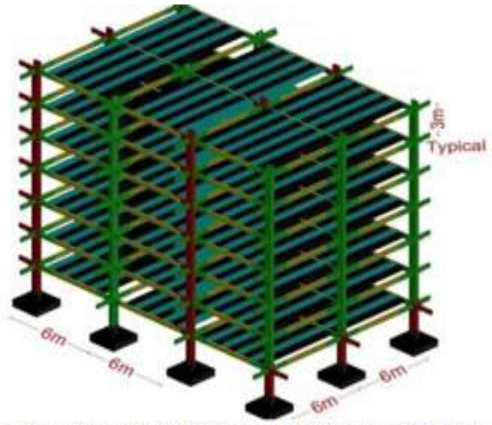
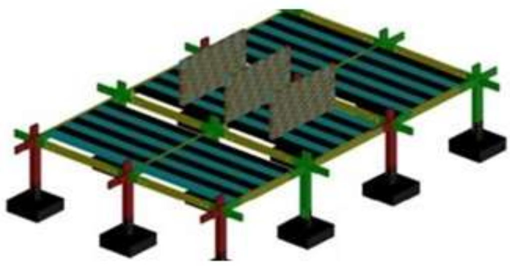


FIGURE 01: MULTI STORIED PREFABRICATED BUILDING.



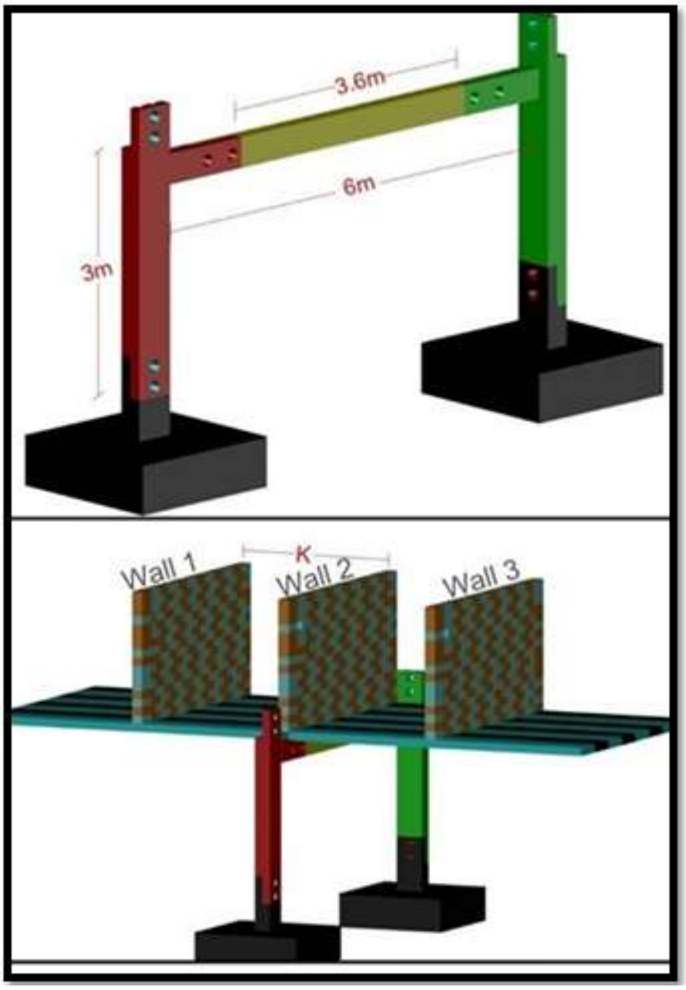


FIGURE 03: TEST FRAME WITH ALL THE LOADING ELEMENTS

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.

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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.

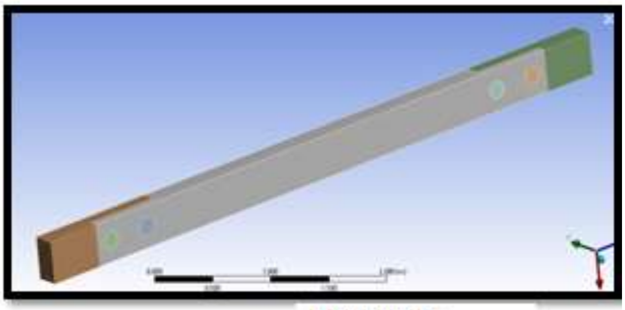

FIGURE 04:

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 figure 10.

A. 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.

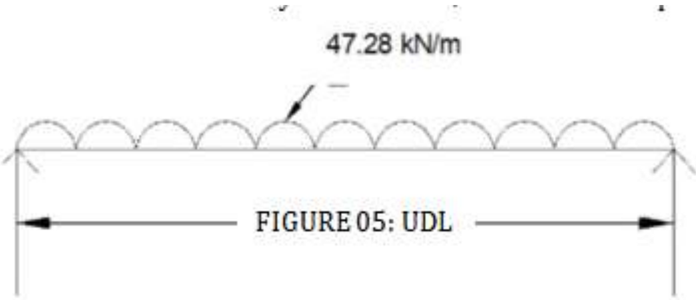

FIGURE 05: UDL

B. Calculation of shear generated due to fixity .

Udl (w) = 47.28kN/m, Reaction = 47.28 x 4.3 /2 = 101.65

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Since this will be shared by two pins Reaction per pin = 50.826 Kn.
Fixed End Moment due to UDL

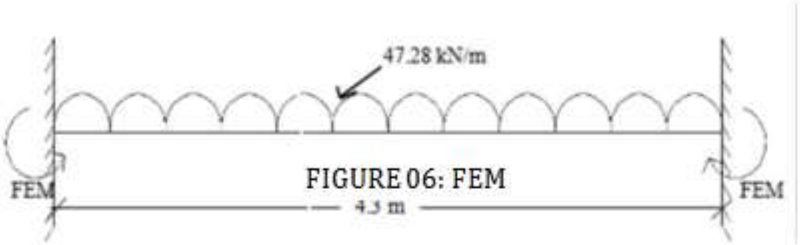

FIGURE 06: FEM

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

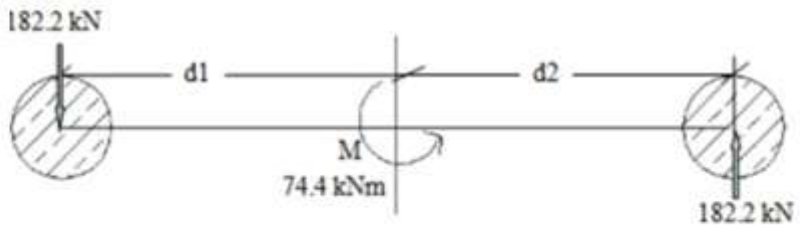

FIGURE 07: COUPLE

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.

<u>Calculation of Diameter of the Pin (bolt) required to resist the calculated shear.</u>

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

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

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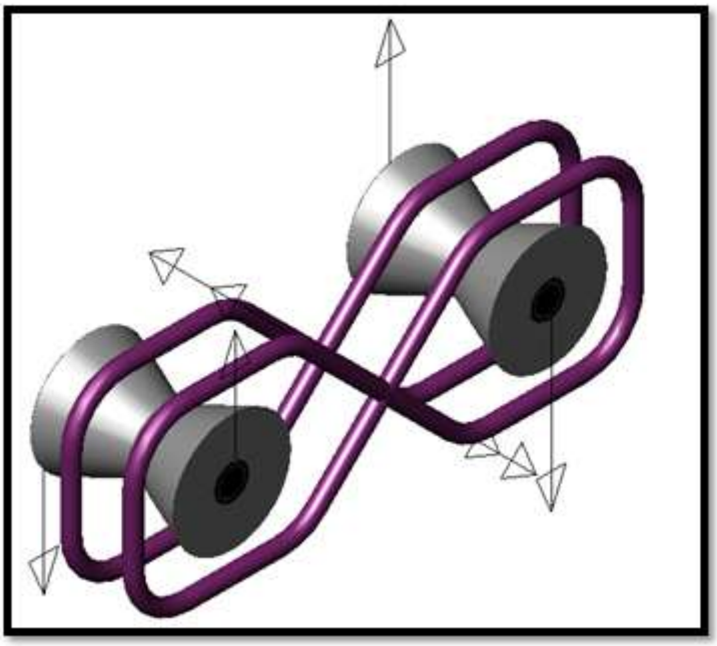

FIGURE 08: 3D DEPICTION OF FORCES

Tensile Force Calculations in "Infinity Shaped" rings.
As seen in Figure 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.

GEOMETRY.

The Geometry of the Test frame is shown in Figure 09 as an exploded view. This frame is taken from the ground floor of the multi storied frame which is shown in Figure01. 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.

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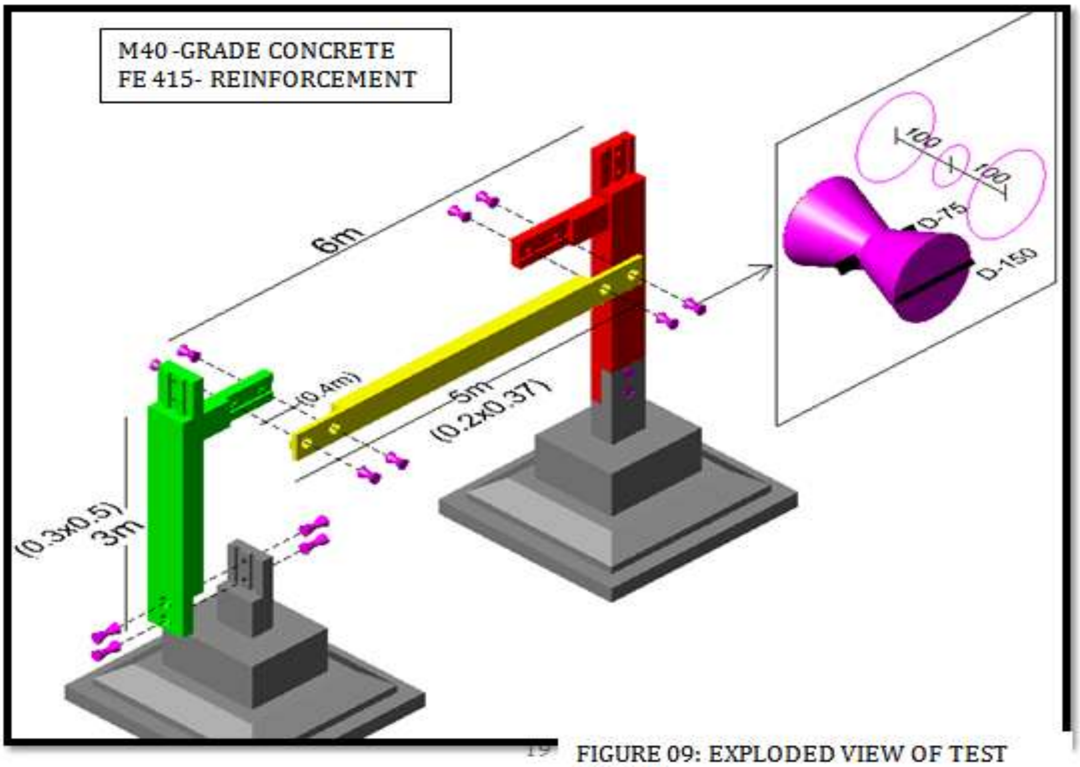

FIGURE 09: EXPLODED VIEW OF TEST FRAME

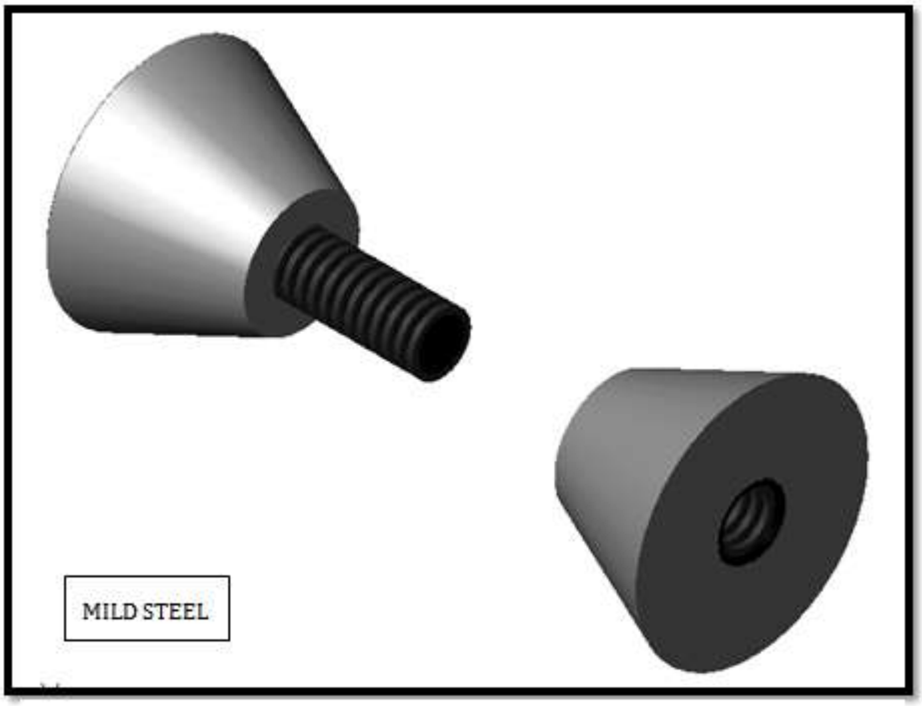

FIGURE 10:3D VIEW OF PIN (THREADED BOLT) & CONICAL NUTS

Fig 10 shows the assembled arrangement of the central threaded pin. The conical nuts have a female thread and

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fit perfectly over the threaded bolt. The entire assembly fills the cavities in the concrete thus providing a complete locking arrangement.

FIGURE 11: PICTURE OF ASSEMBLEDPIN (THREADED BOLT) &CONICAL NUTS

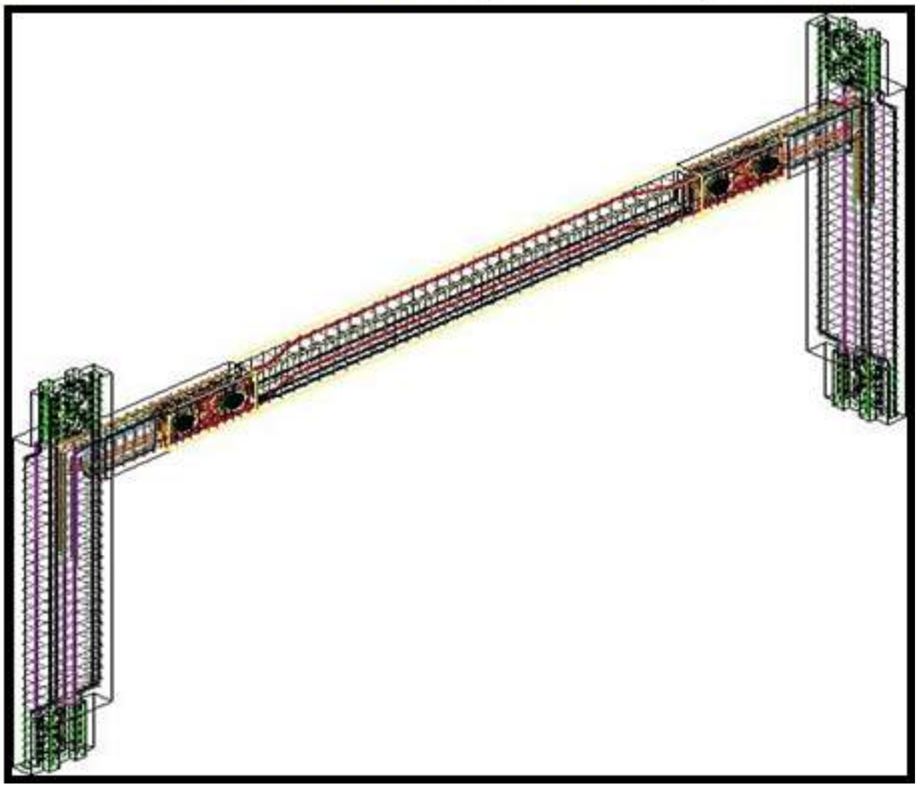

Figure12shows the arrangement of the reinforcement in the entire frame. Note that ductile detailing as per IS: 13920-2016 in the beam-cantilever joint is not disturbed. Reinforcement details are as follows.

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

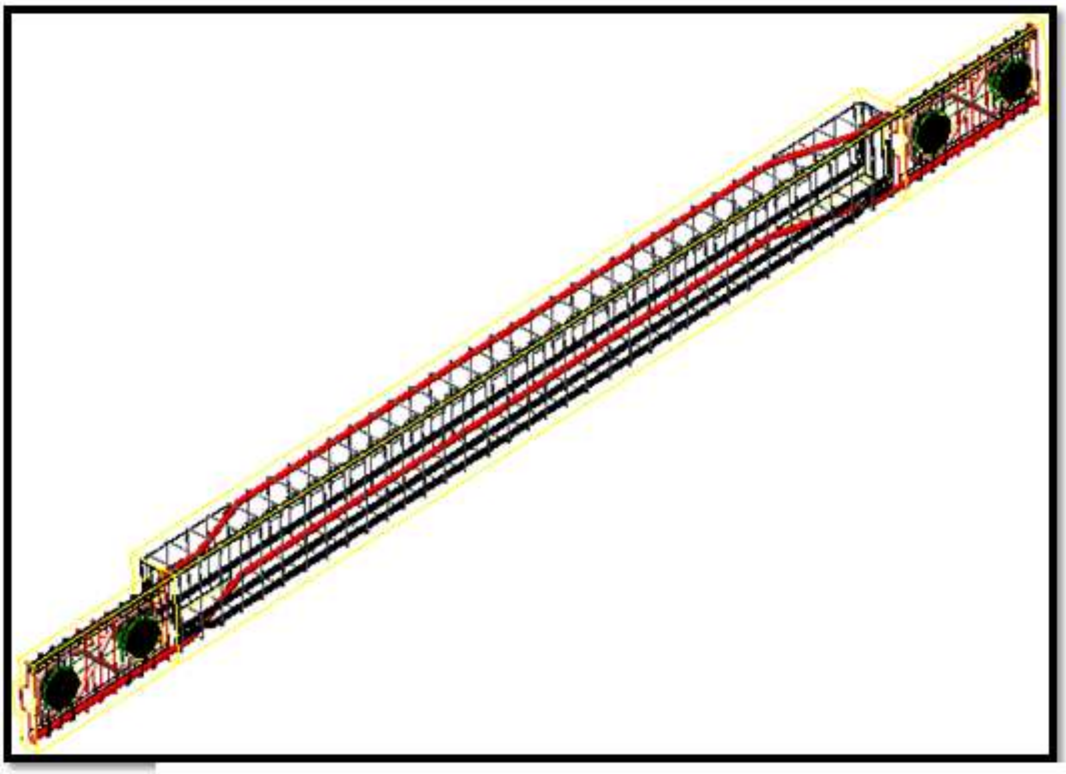

FIGURE 13: REINFORCEMENTDETAILSOFTHEBEAM

Figure 13 shows the arrangement of reinforcement in the central beam

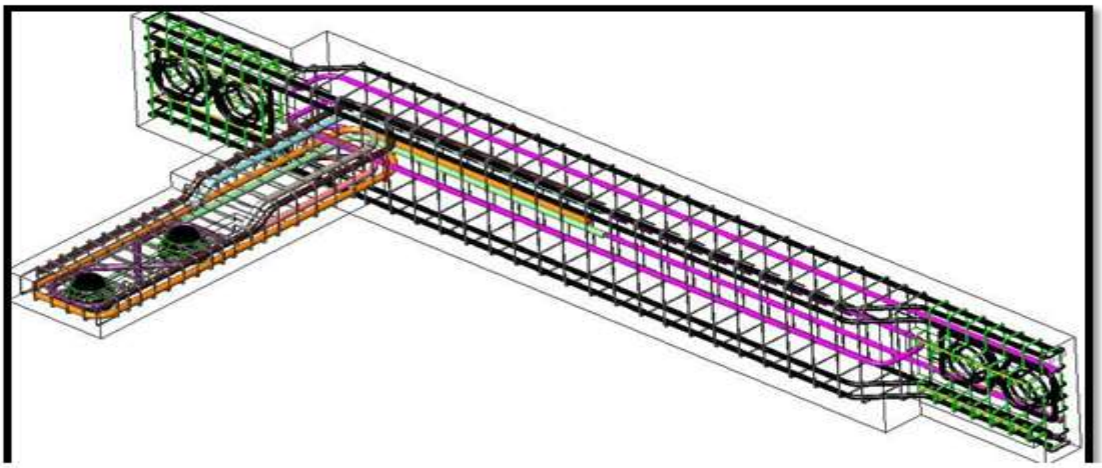

FIGURE 14: REIN FORCEMENT DETAILS OF COLUMN

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Figure 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 figure 10 and figure 11.

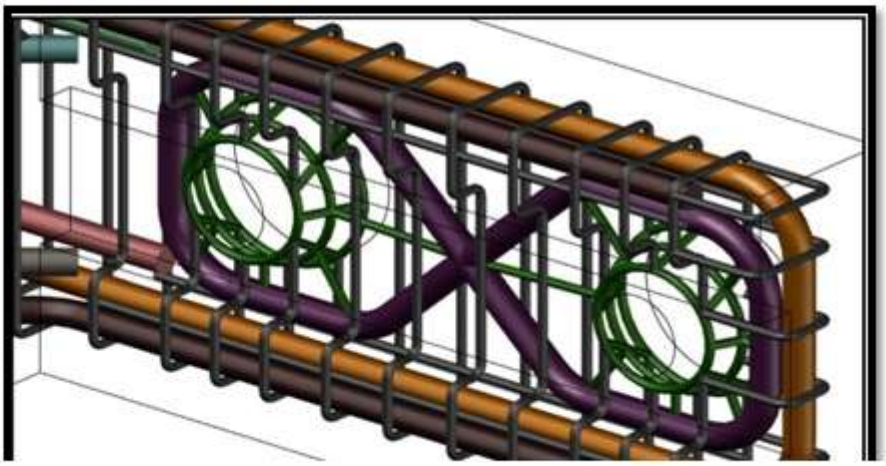

FIGURE 15: REINFORCEMENT DETAILS OF THE CONNECTIONCAGE.

Figure 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.

FIGURE16: CONNECTION CAGE WITH INFINITYRING.

Figure 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.

FIGURE 17: COLUMN CONNECTION CAGE

Figure 17 shows the cones which are part of the form work to create the conical cavities in the concreteto 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.

FIGURE 18:
REINFORCEMENTINWOODENSHUTTERING.

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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 over all project time.

FIGURE 19: STEELSHUTTERING.

Figure 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.

FIGURE 20: COLUMN AFTERDE-SHUTTERING.

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In Figure 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.

FIGURE 21: COLUMNSAFTERDE-SHUTTERING ,READY FOR TRANSPORTATION

FIGURE 22: ASSEMBLYOFFRAMEINPROGRESS.

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.

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FIGURE 23: ASSEMBLEDFRAME

FIGURE 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 Figure24, Figure 25 & Figure 26.

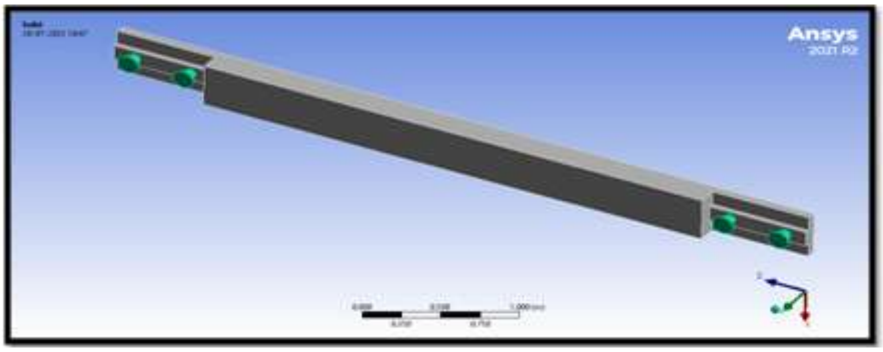

FIGURE 24: BEAM MODEL WITH PINS (ANSYS)

Figure 25 shows the end of the beam in close up and Figure 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.

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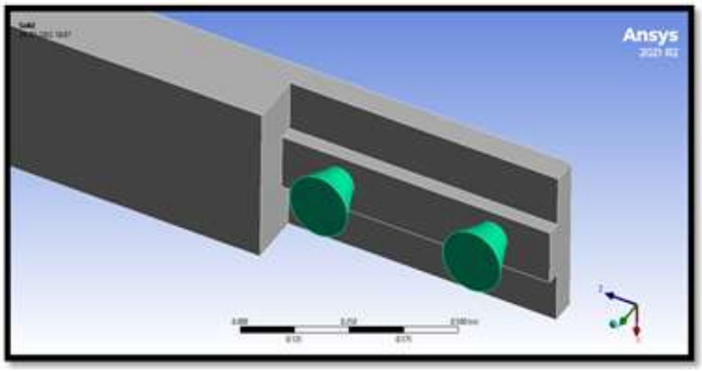

FIGURE25: BEAM WITH PINS (CLOSE UP)

FIGURE 26: CANTILEVER WITH PINS

The Figure 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.

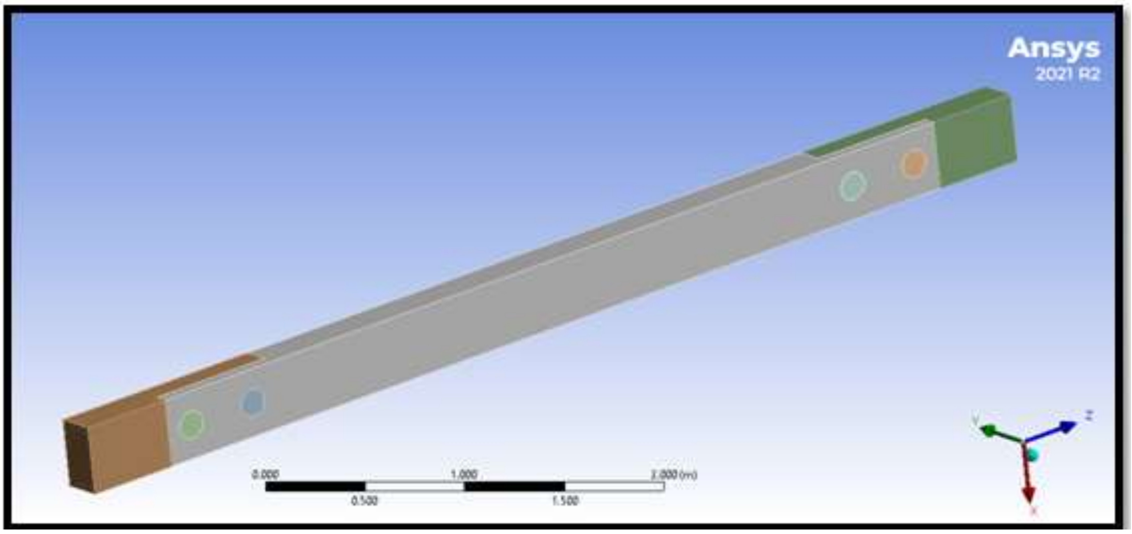

FIGURE27: ASSEMBLED BEAM MODEL (CLOSE UP)

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 a yield strength of 415 N/mm2. Properties of mild steel, i.e. yield strength of 250 n/mm2 is 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.

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MESHING

Figure 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.

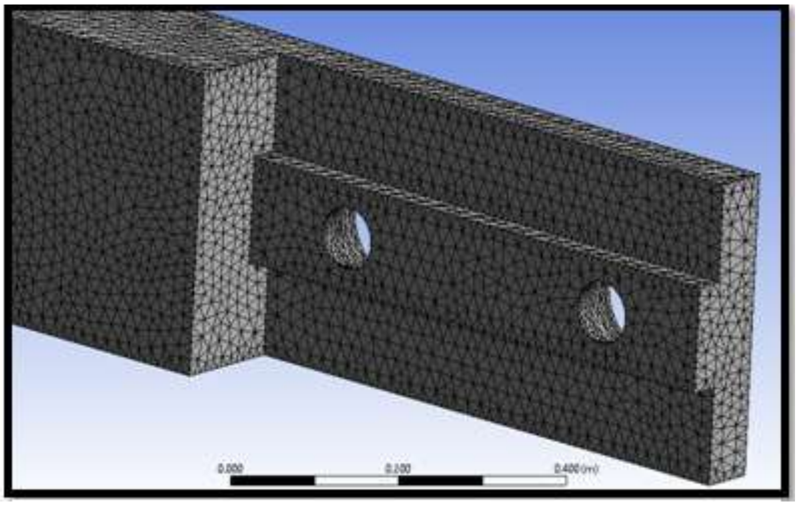

FIGURE28: MESHED BEAMBEAM (CLOSE UP)

Figure 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.

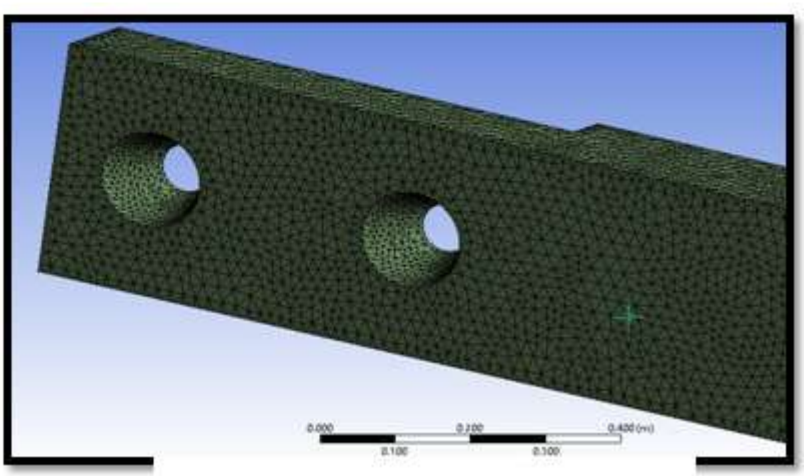

FIGURE 29: MESHED CANTILEVER

Figure 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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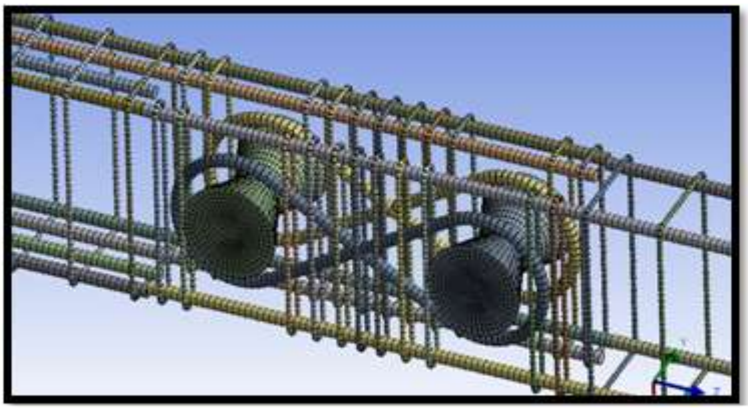

FIGURE30: MESHED CONNECTION REINFORCEMENT

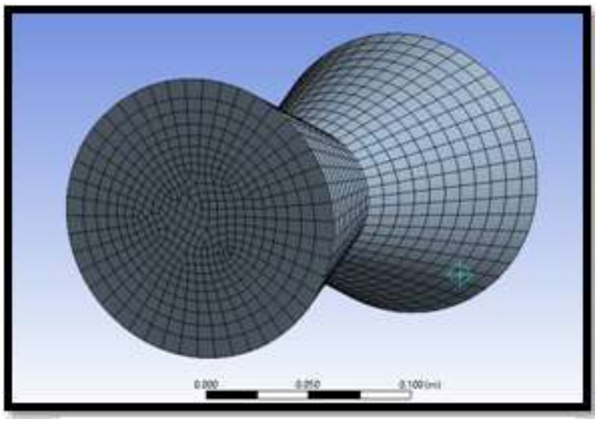

FIGURE 31: MESHED PIN

Figure 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.

LOADING AND BOUNDARY CONDITIONS'

Figure 32 shows the loading & Boundary conditionsThe 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.

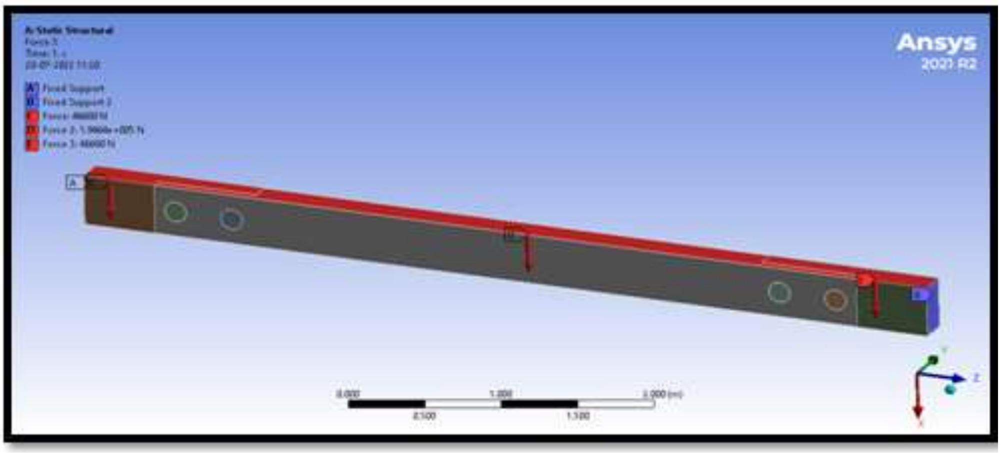

FIGURE 32: LOADS AND BOUNDARY CONDITIONS.

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RESULTS- I

FEM ANALYSIS

It can be seen from figure33 and figure34 that the contours follow a similar patter for the assembled and the monolithic model. The deflection for the assembled model is 1mm more than monolithic model

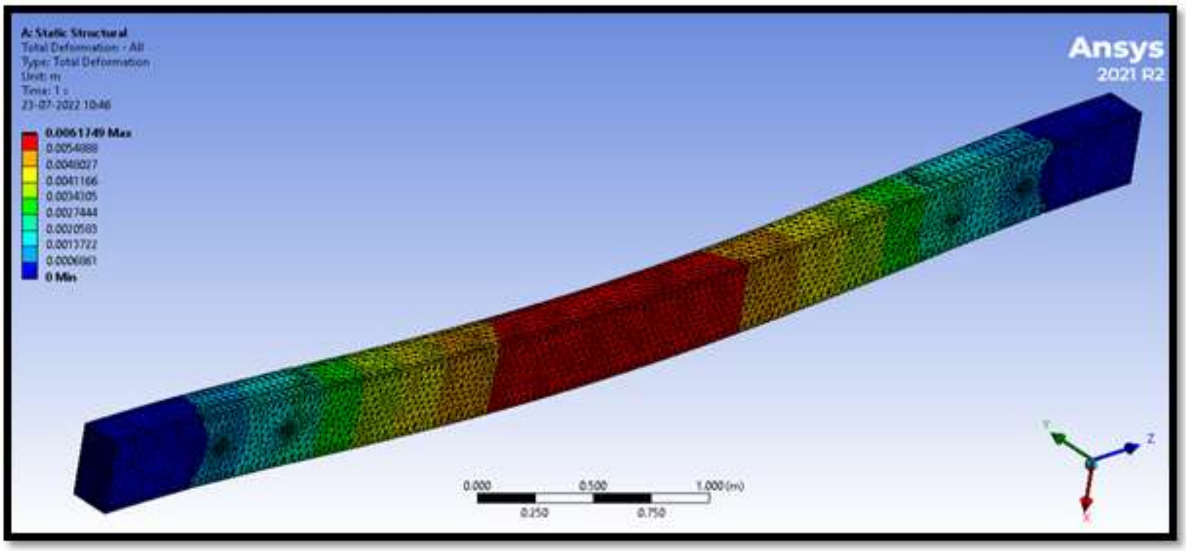

FIGURE 33: DEFLECTION IN ASSEMBLED BEAM.

DEFLECTION

Figure 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 (figure 34) is different from that of the assembled model.

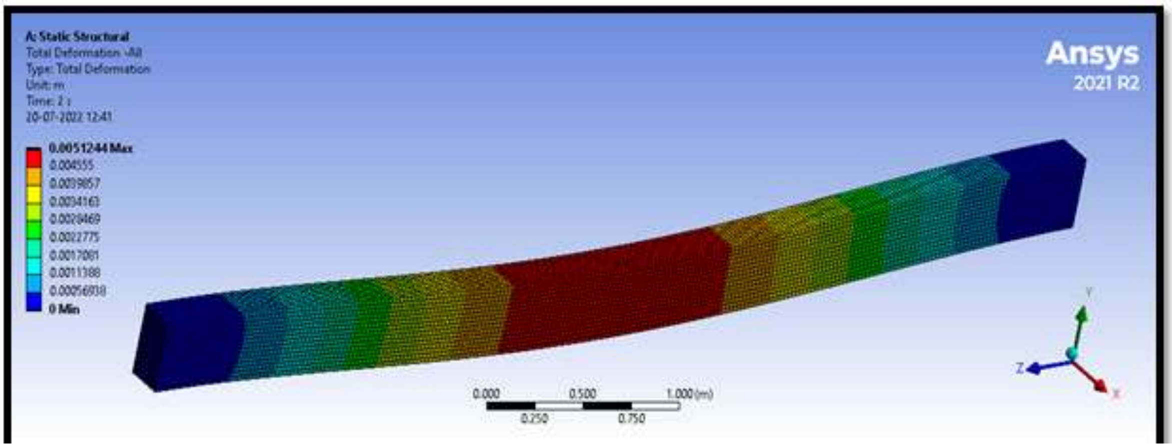

FIGURE 34: DEFLECTION IN MONOLITHIC BEAM.

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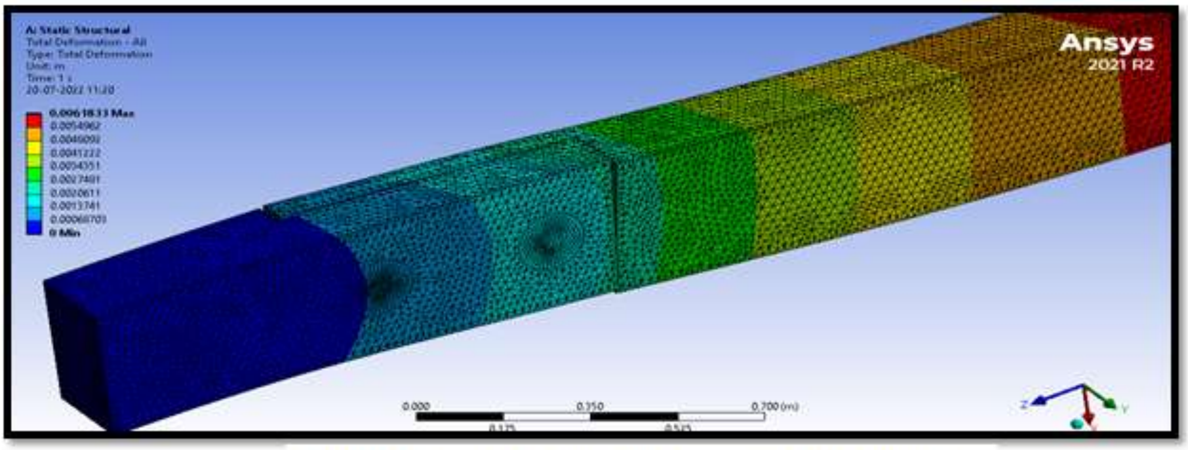

FIGURE 35: DEFLECTION IN CONNECTION REGION.

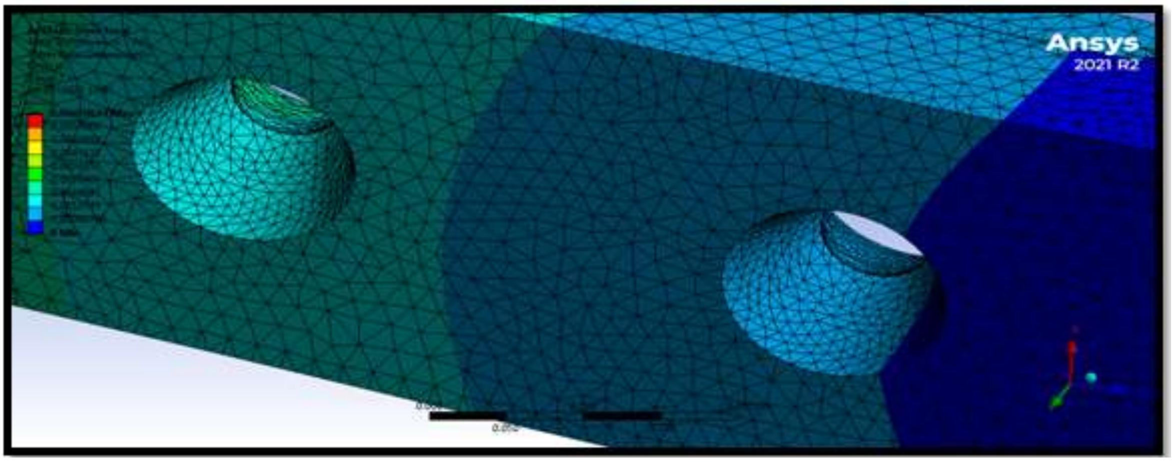

FIGURE 36: DEFLECTION IN CONICAL CAVITIES IN CONNECTION REGION.

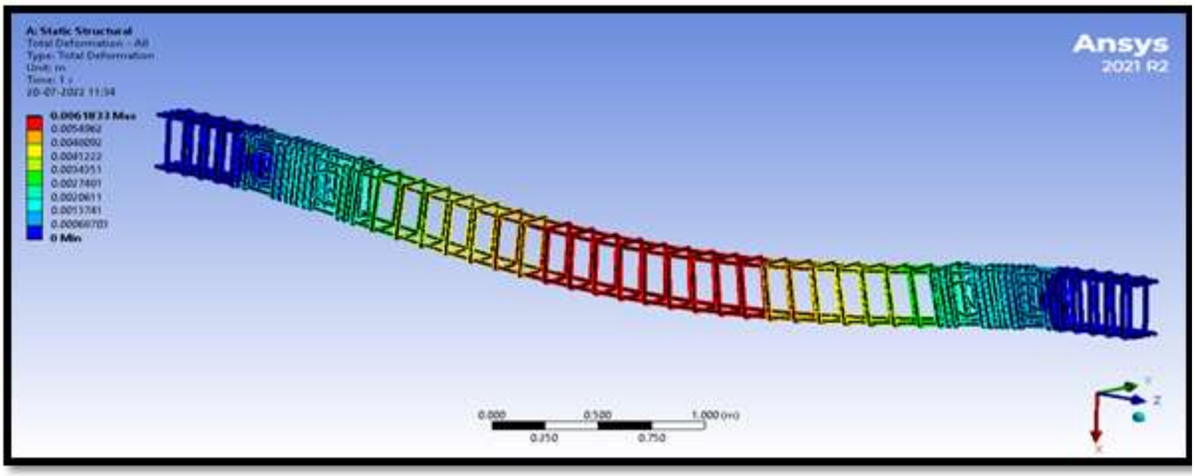

FIGURE 37: DEFLECTION IN REINFORCEMENT

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DEFLECTION

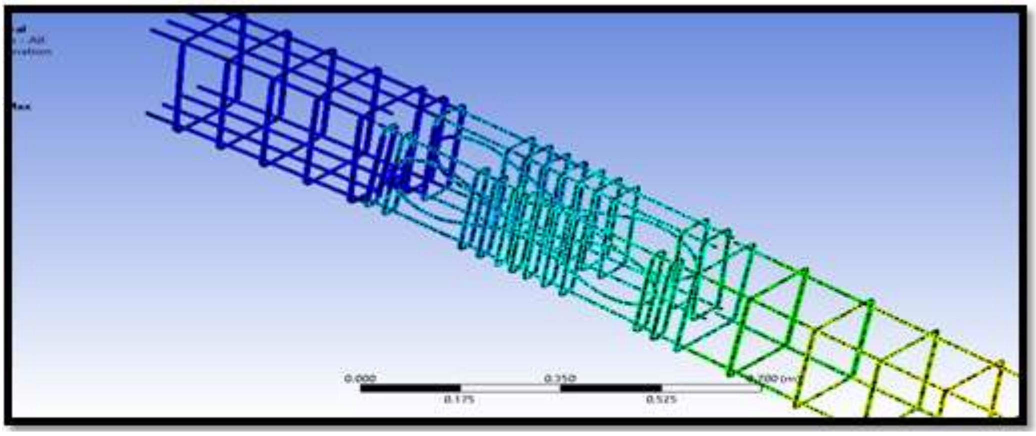

FIGURE 38: DEFLECTION IN ASSEMBLED REINFORCEMENT CLOSE UP

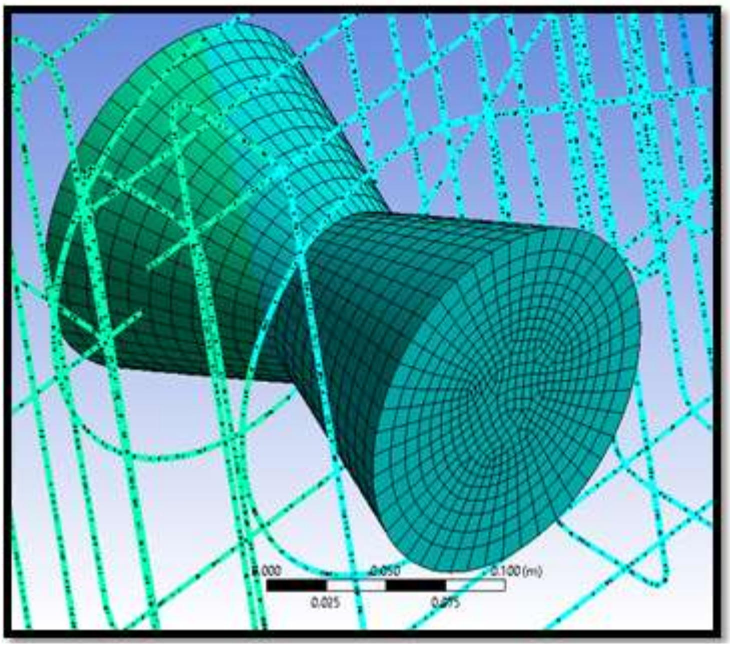

FIGURE 39: DEFLECTION IN CONICAL PINS

STRESS.

From Figure 40&Figure41 it can be seen that stress distribution in assembled model and the monolithic model is similar. Stress in both the models are 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.

STRESS.

Figure 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.

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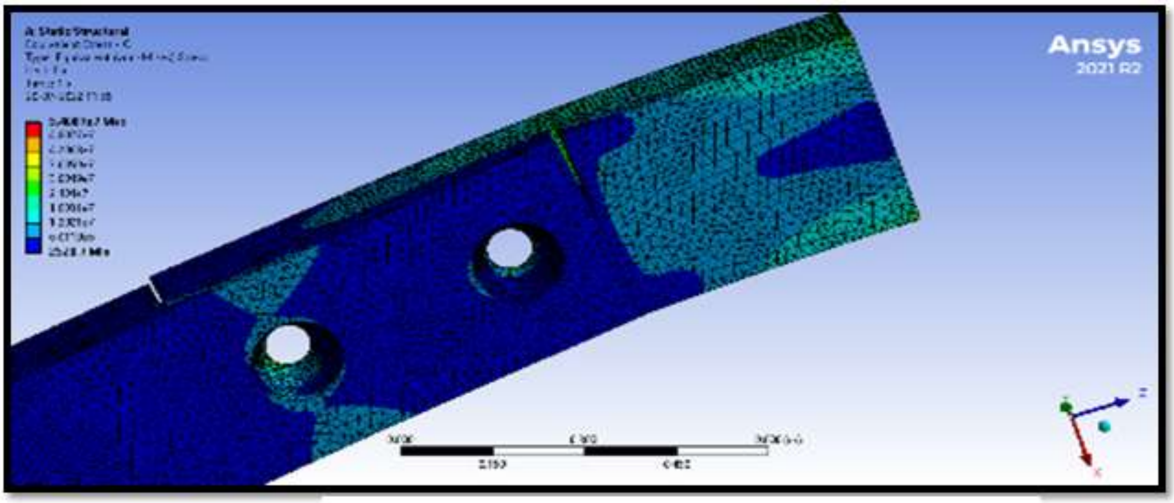

FIGURE42: STRESS IN CONNECTION REGION

Figure 43: shows the stress distribution in the reinforcement in the connection region. It can be seen that the stress in the infinity ring is more than the reinforcement in the surrounding region, thereby validating the calculations and design.

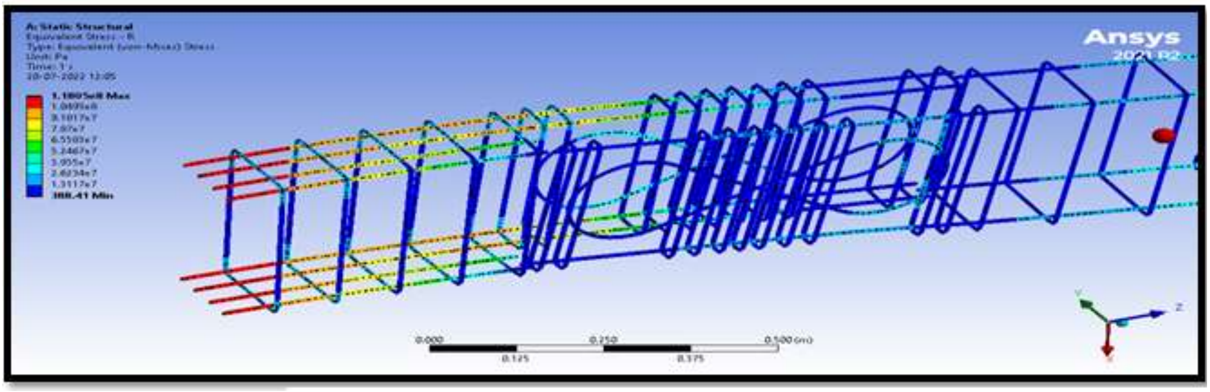

FIGURE43: STRESS IN CONNECTION REINFORCEMENT.

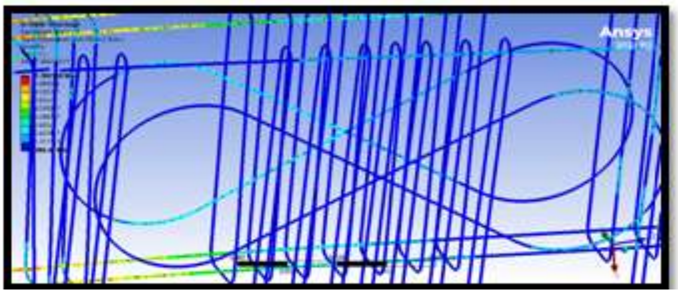

FIGURE 44: STRESS IN INFINITY

STRESS.

Figure 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.

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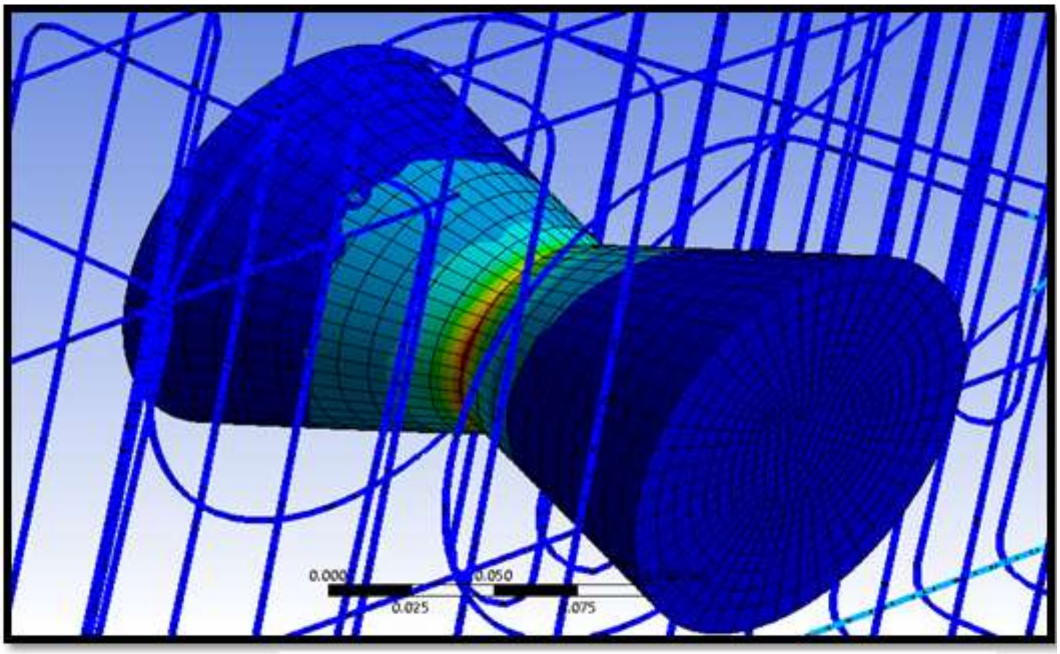

FIGURE 45: STRESS IN CONICAL PIN

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

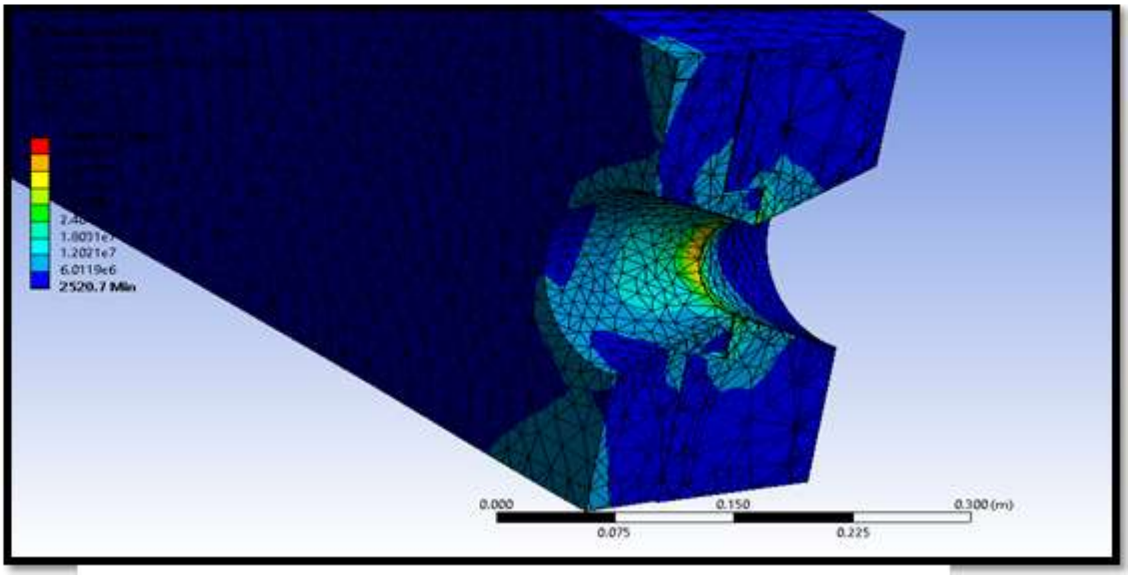

FIGURE46: STRESS IN CONICAL CAVITY – SECTIONAL VIEW

STRAIN.

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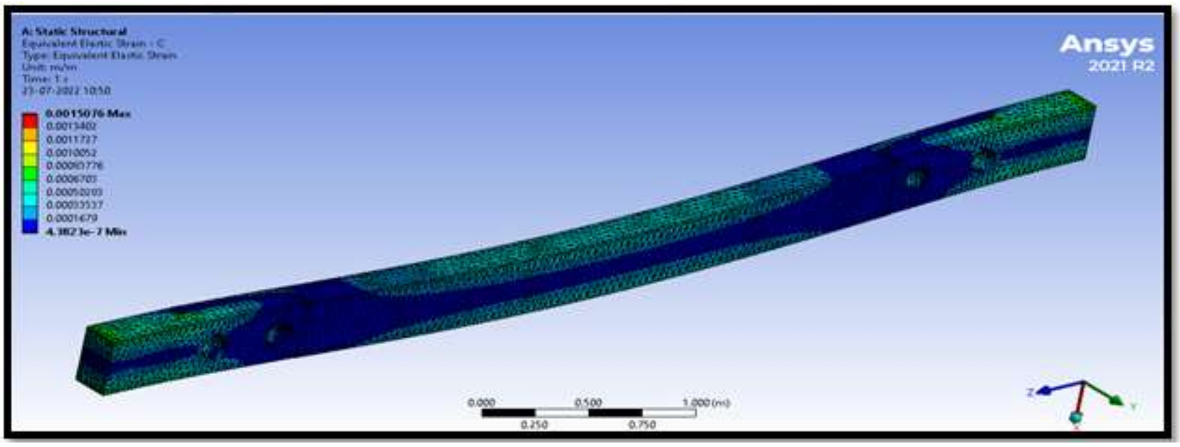

FIGURE 47: STRAIN IN ASSEMBLED BEAM.

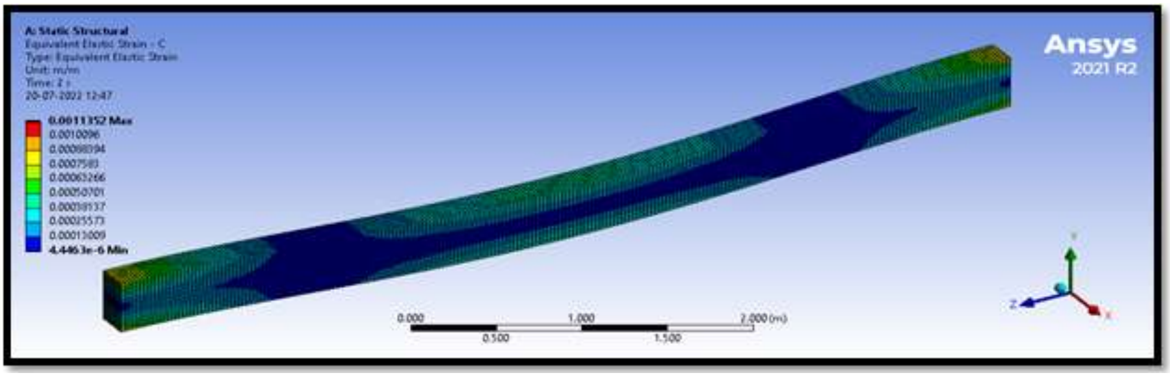

FIGURE 48: STRAIN IN MONOLITHIC BEAM.

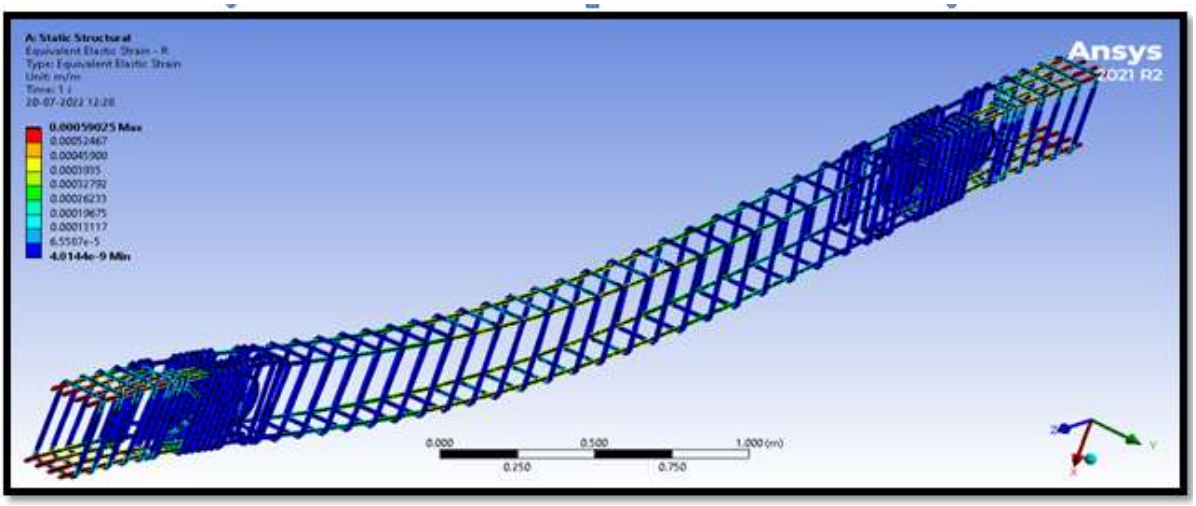

FIGURE 49: STRAIN IN REINFORCEMENT OF ASSEMBLED BEAM

STRAIN.

Figure 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.

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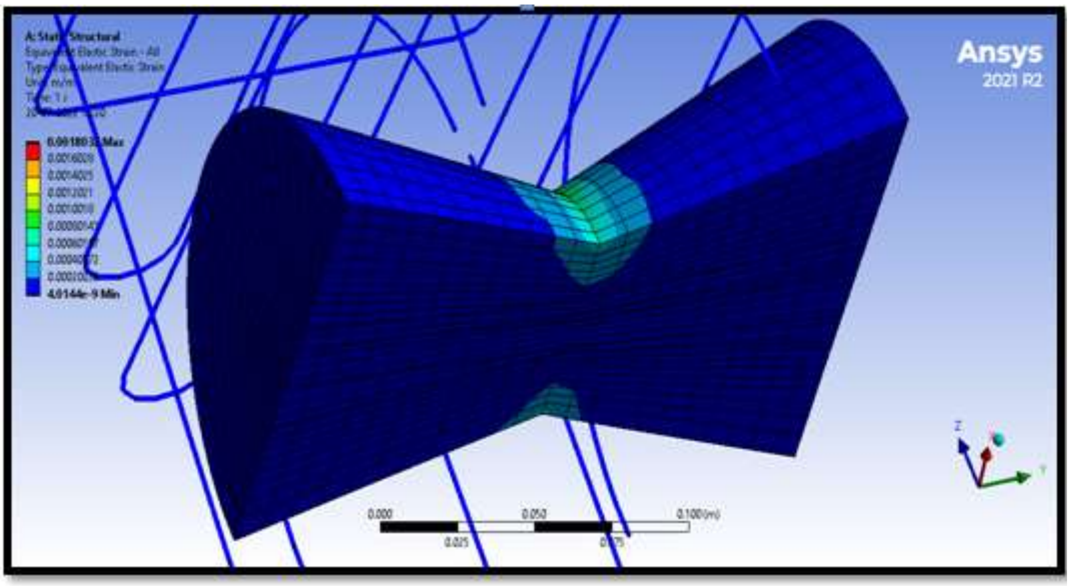

FIGURE50: STRAIN IN PIN – SECTIONAL VIEW

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

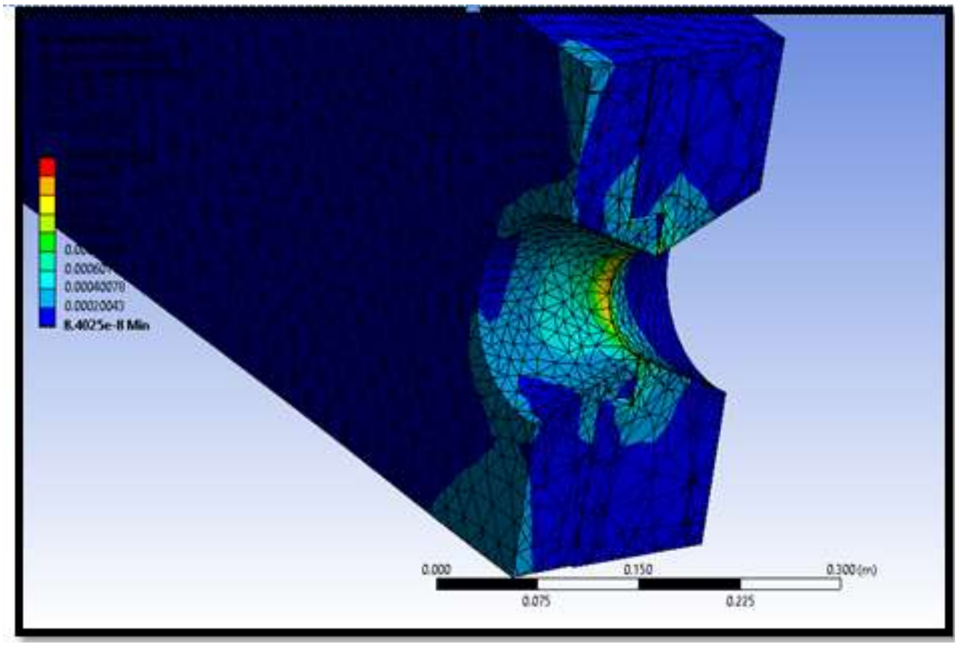

FIGURE 51: STRAIN IN CONICAL CAVITY -SECTIONAL VIEW

RESULTS- IIEXPERIMENTAL VERIFICATION

TESTING PROCEDURE FOR DEFLECTION

I. Deflection Measurement Using Linear PotentiometerThe measurement of the deflection for the beam is carried out with the help of linearpotentiometers.

II. Working Principle

Linear Potentiometer changes mechanical linear movement of the piston stroke intoelectrical 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 ispositioned along the length of the resistor. The magnitude of this output voltage isdirectly proportional to its relative position along the length of the resistor. Thepotentiometer is appropriately connected to a

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moving system then any movement inthat 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 lineardisplacement.

II. Methodology used for measurement of deflection

Step 1 – Marking of deflection measurement locations

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

Step 2 – Surface preparation

Surface preparation is done to enable proper contact between tip of LinearPotentiometer and the surface of the beam. The surface should be flat where the tip ofLinear 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 theStructural member vibrations and other movements. The platform to support the linearpotentiometer is made to be even and the same is checked with the spirit level. Thelinear potentiometer is clamped on its stand which is fixed and secured with theplatform. The contact with the bottom of the slab is established with the head of theplunger such that the initial reading is recorded with the corresponding change in thevoltage.

III. – Data collection and analysisThe data collection and analysis is done with the help of DGC cDAQ 9178 softwarefor deflection. Three locations as shown is figure 55 are marked as P1, P2 & P3 where the deflections are measured by installation of the Potentiometers as described above.

FIGURE 52: LINEAR POTENTIOMETER

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FIGURE 53: POTENTIOMETER INSTALLED.

FIGURE 54: DATA ACQUISITION – SET UP

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FIGURE 55: FULLY LOADED SUSPENDED PLATFORM.

FIGURE 56: FULLY LOADED SUSPENDED PLATFORM. – SIDE VIEW

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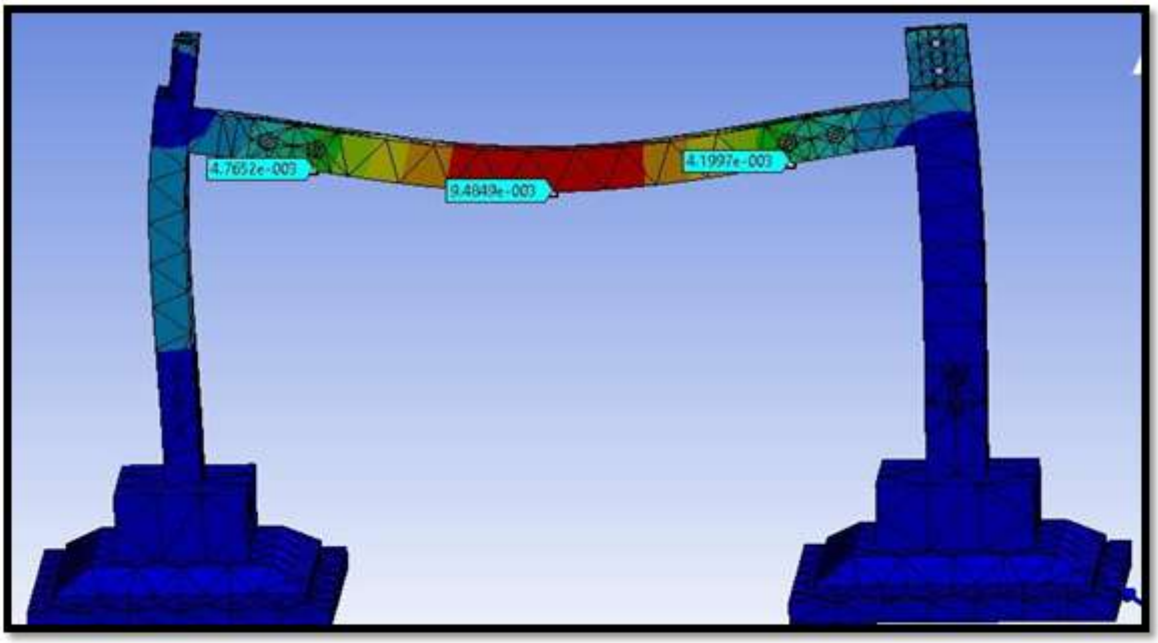

FIGURE 57: FRAME WITH DEFLECTION VALUES-ANSYS

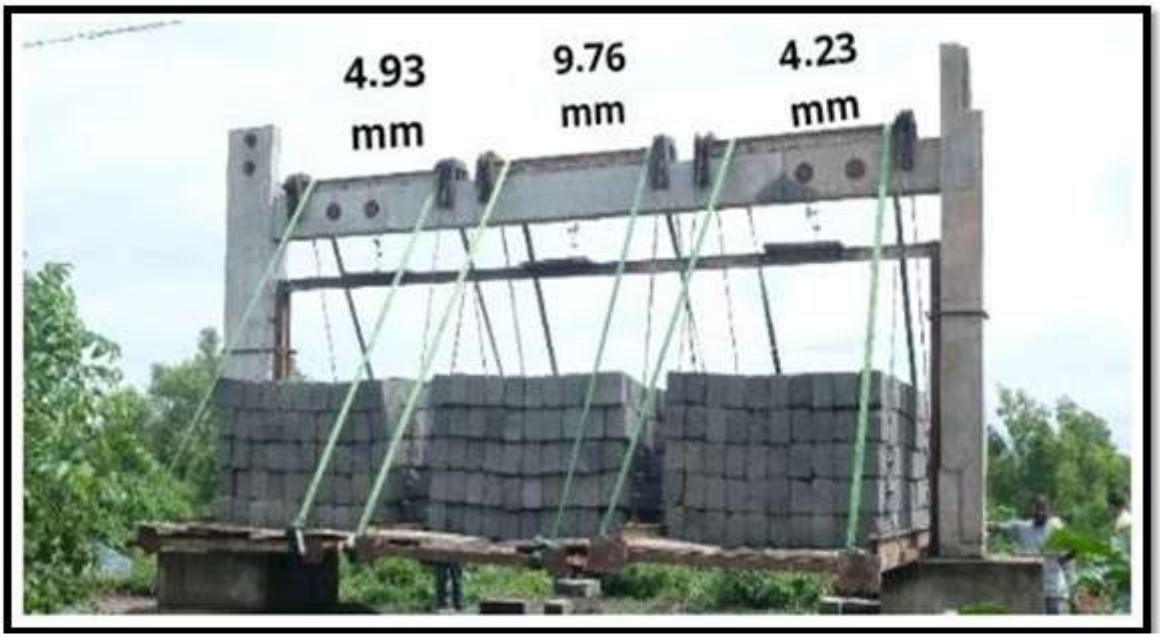

FIGURE 58: ACTUAL FRAME WITH MEASURED VALUES OF DEFLECTION

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.

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

ASSEMBLED BEAM	MONOLITHIC BEAM

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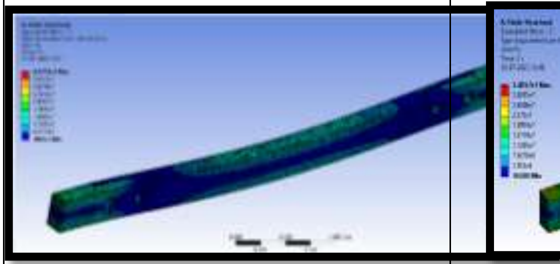
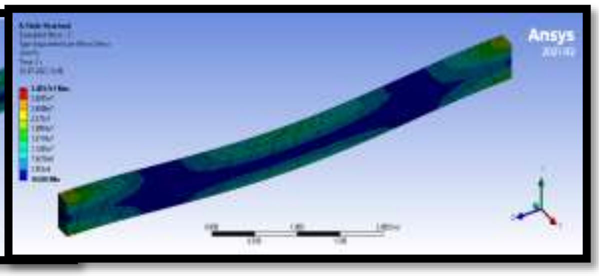
	
Max stress = 6.1 N/mm2	Max stress = 3.85 N/mm2

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

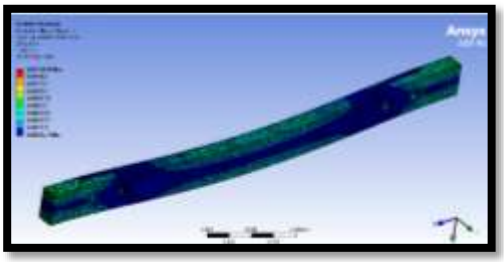
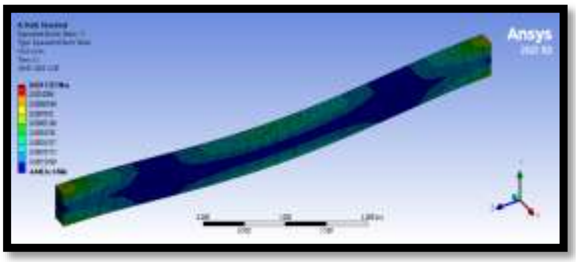
ASSEMBLED BEAM	MONOLITHIC BEAM
	
Max stress = 0.0002	Max stress =0.0001

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

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.

TABLE 4: DEFLECTION READINGS CAPTURED ON SITE.

		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 5: COMPARISON OF DEFLECTION BETWEEN ASSEMBLED PRECAST TEST FRAME AND MONOLITHIC FEM MODEL			
	P1	P2	P3
CONSTRUCTED TEST FRAME	4.93	9.76	4.23
FEM MONOLITHIC MODEL	4.76	9.48	4.2

CONCLUSIONII: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.
ThedesignedPre-Castmethodcanbeadoptedforconstructionofmultistoriedstructures. However, following points need consideration.

•There is need for further testing of Space frames with static loadsandseismic (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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