Dynamic Analysis of Composite Column Using Time History Analysis

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Abstract- *In the present paper nonlinear stress–strain relations are assumed for concrete, reinforcing steel and structural steel materials. The proposed procedure was compared with test results of 3 I-shape,3 cicular-shape,3 square-shape reinforced concrete columns subjected to short-term axial load and biaxial bending, and also some experimental results available in the literature for composite columns compared with the theoretical results obtained by the proposed procedure& ANSYS , good degree of accuracy was obtained. An experimental investigation on the structural behavior of steel tubular columns in-filled with plain and steel reinforced concrete is presented in this study. A total of 9 concrete-filled steel tubular columns were constructed and tested subjected to axial load. The main variables considered in the test study were the cross section, slenderness, concrete compressive strength and the load conventionally. In the presented study, a theoretical method for the prediction of ultimate strength capacity and load-deflection curves of concrete filled steel tube columns is proposed. In the analysis procedure, the nonlinear behavior of the materials is considered and the slenderness effect has been taken into account. The experimental ultimate strength capacities and load-deflection curves of steel concrete-filled tube columns have been compared with the analysis results and discussed in the paper. The results indicate that the addition of steel section in core concrete has considerable effect on the behavior of concrete-filled steel tube columns.*

An encased composite column is a column composed of a steel shape core encased in concrete with additional longitudinal reinforcing steel and lateral ties is analysed for specified ground motion

Keywords- ANSYS, Composite Column,Time History Analysis, Finite element, Response Spectrum Method

I. INTRODUCTION

CONCRETE ENCASED COMPOSITE COLUMN:-

The cross-sectional area of the steel core must comprise at least 1% of the total composite cross section.

1. The nominal strength of the section is determined using the plastic stress distribution method or the more general strain compatibility method. These methods are similar to those used in reinforced concrete column design.
2. Slenderness effects are accounted for the same as in axially loaded steel columns.
3. One simple approach to design of doubly symmetric composite beam-columns is to use the straight line interaction equations defined in Chapter H. This approach parallels that used for design of wide-flange or HSS steel columns but yields a significantly more conservative estimate of the beam-column capacity for composite columns than it does for steel beam columns.

1.2.3 General Concept of Composite Columns

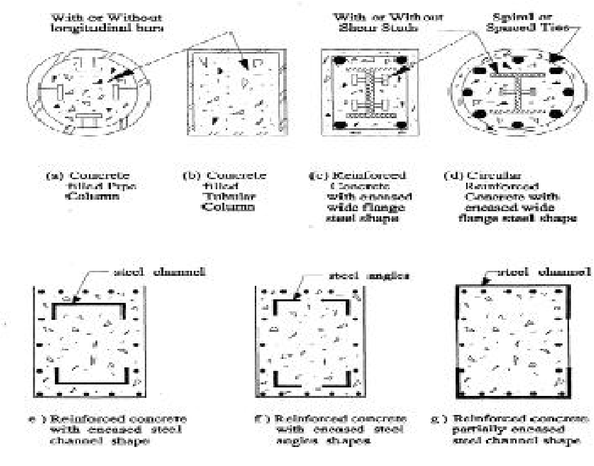


Fig 4:- Different types of encased column

Fig. 4 (a) to (d). Fig.(a) and (b) shows a composite column in which the steel pipe and tube Composite columns may be of two kinds:(1) Concrete-filled pipe and tubular steel columns, and (2) concrete-encased steel columns, as shown in serve both as form and reinforcement. The column may be reinforced with longitudinal bars and have shear studs welded to the structural steel shape in order to maintain the elements together as a unit. Fig.4(c) shows a wide flange structural steel shape encased by concrete and reinforced with longitudinal bars, in some cases shear studs are welded to the steel to improve shear transfer between steel and surrounding concrete. In early 1950's, structural steel shapes were encased by plain concrete without longitudinal bars, such encasement was used mainly for fireproofing purposes.

II. LITRATURE REVIEW

Furlong RW. (1979) ,The design of composite columns in Great Britain is covered in the 1979 edition of the British Standard BS 449, which allows some compressive stresses in the concrete encasement based on a modular ratio of 30, and the allowable stress in the steel using a larger radius of gyration. The British Standard limits the total axial load of the composite column to twice the axial load on the steel section alone, and the bending moments are to be carried only by the steel section. A straight line type interaction formula is specified for combined axial load and bending moment with only the axial compressive load to be increased due to the encasement. In mid-1960's, Stevens in 1965 performed an experimental work on composite columns under eccentric loads, followed by theoretical work done by Bondable in 1966 and Basu in 1967 and 1969 respectively (01).

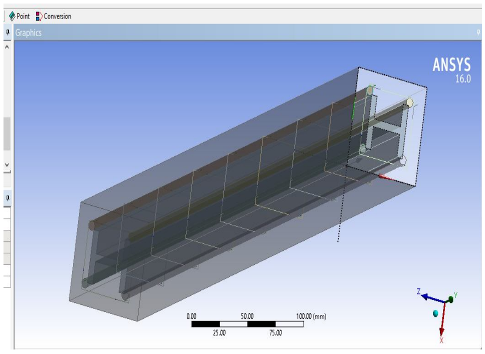
Brondum Nielsen (1986;83) attempted to predict the actual strength of eccentrically loaded composite columns. Based on the results of extensive tests on axially and eccentrically loaded encased steel columns conducted at the Building Research Station in London in 1965, where the primary variable was the slenderness radio, Stevens in 1965 compared the column curve generated by these tests with those of reinforced concrete columns and concluded that due to the similarity of the behavior of the reinforced and encased columns, similar principles of design should govern both forms of columns. In 1963 the ACI Building Code specified the use of an allowable axial load equation for structural steel encased in concrete based on the allowable stress in concrete with a corresponding equivalent modular ratio of 100.(02)

Stevens (1965) ,The bending moments were to be resisted by the structural steel section alone. Stevens in 1965 recommended an ultimate strength equation for short columns and a straight line type interaction formula for eccentrically loaded columns and suggested the use of the same reduction coefficients for long composite columns as those being used for long reinforced concrete columns to account for the slenderness effects. Later in 1967 Basu presented a theoretical approach to compute the failure load of eccentrically loaded composite columns. Their method is based on the assumption that the deflected shape of the column is part of a cosine curve. From the previous theoretical approach, Basu and Sommerville in 1969 presented an empirical method for predicting the failure loads of composite columns under axial load and different end eccentricities. This empirical method was derived from numerical results obtained from a generalized computer program for a large number of encased and filled rectangular tubular columns. Basu and Hill also reported analytical results obtained by the use of a new computer program that took into account the actual equilibrium shape of the deflected column rather than the previously assumed cosine deflected shape. The British approach of the 1970's for the design of composite columns was based on the work of Basu and Sommerville and Virdi and Dowling respectively.(03)

III. NUMERICAL MODELING

For testing the column 9 columns are casted which are as follows:

3 no of RCC reinforced column with I-section.
3 no of RCC reinforced column with Square-section.
3 no of RCC reinforced column with Circular-section.



2.1 Finite Elements

The definition of the proposed numerical model was made by using finite elements available in the ANSYS code default library. The three-dimensional elements SOLID 186 were adopted to discretize the concrete slab, which are also able to simulate cracking behavior of the concrete under tension (in three orthogonal directions) and crushing in compression, to evaluate the material Non-linearity and also to enable the inclusion of reinforcement (reinforcement bars scattered in the concrete region). The representation of the steel section was made by the SHELL 43 elements, which allow for the consideration of Non-linearity of the material and show linear deformation on the plane in which it is present. The modeling of the shear connectors was done by the BEAM 189 elements, which allow for the configuration of the cross section, enable consideration of the Non-linearity of the material and include bending stresses. The TARGE 170 and CONTA 173 elements were used to represent the contact slab-beam interface. These elements are able to simulate the existence of pressure between them when there is contact, and separation between them when there is not. The two material contacts also take into account friction and cohesion be-tween the parties.

2.2 Materials properties

The characteristics of the MODEL and the real properties of materials are presented in Table 1. It is noteworthy mentioning that this study also considered other configurations for the connectors, as number, height and diameter.

2.3 Constitutive relations

It was considered that the steel section has a multilinear elastic plastic constitutive relationship with an isotropic hardening consideration, associated with the von Mises' plasticity criterion. The stress-strain curve followed the constitutive model presented in and it was used in and, as shown in Figure 6. The adopted model for the steel connectors is a bi-linear isotropic hardening, also associated with von Mises' plasticity criterion. Figure 7 shows the stress-strain diagram for the steel connectors. The constitutive relationship for the steel reinforcement follows a perfect elastoplastic model and it is also associated with von Mises' plasticity criterion, based on the relationship between uniaxial tensions and their respective plastic deformations, as shown in the stress-strain diagram in Figure 4. For the concrete slab, the constitutive tension relationship followed the CONCRETE model, provided by ANSYS, which is based on the Willam-Warnke solution and allows for the material cracking. This model was also used in and. For the concrete in compression, on the other hand, von Mises' laminating criterion was adopted. The model represents the behavior of a multilinear isotropic concrete hardening, given by the stress-strain diagram in Figure 5. The solution for the contact between the concrete slab, the steel section and the connectors made use of the Pure Lagrange Multiplier method, also provided by ANSYS. This method assumes that there is No interpenetration between the two materials when the contact is closed and also that the slip is null, as long as it does Not reach the shear stress limit .The parameters that define if the contact is open or closed are set by FTOLN, which refers to a minimum value of penetration as to presume that the contact is closed and TNOP, which refers to a minimum value of Normal tension to the contact surface, so that the status changes to open. The absolute value adopted for FTOLN was -0.01 cm. For the TNOP the value adopted was 0.18 kN/cm^2. The established value of the friction coefficient between steel and concrete was 0.4 and, for cohesion, an estimated number of 0.18 kN/cm^2 was taken from average values of adhesion tension related to the initial slip of the interface

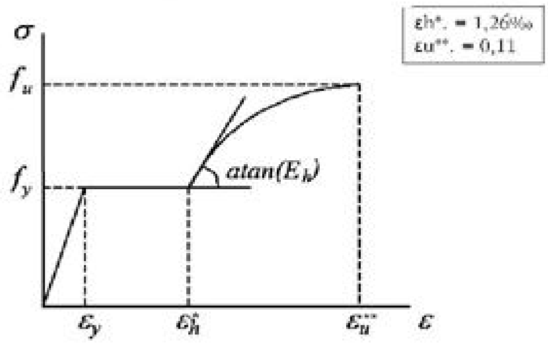


Fig No. 06 Constitutive relation for steel profile (8)

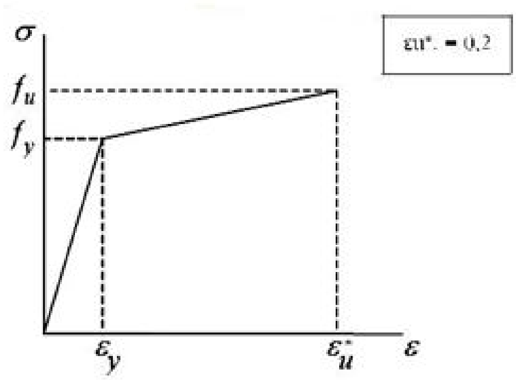


Fig No. 07 Constitutive relation for the shear connectors (10)

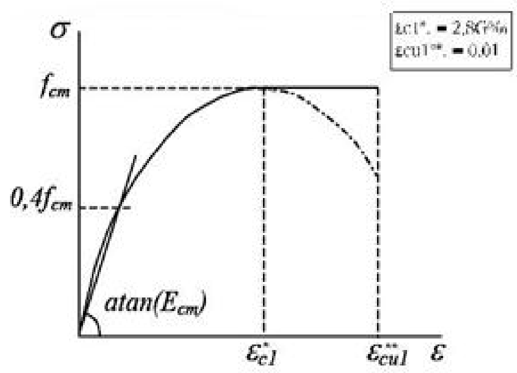


Fig.No.05 Constitutive relation for the concrete

2.4 Finite elements mesh

The model designed for the numerical analysis was defined by four types of elements that form the concrete slab with added reinforcements, such as steel beam, shear connectors and the pair of contact at the slab-beam interface.

The elements were established separately, but the Nodes were one by one coupled on the interface between them. The finite element mesh developed for all elements followed the same methodology and degree of refinement presented in Figure 10 and Figure 11 shows the finite element mesh for the components cited, where (a) corresponds to the concrete slab, (b) to the steel beam, (c) to the shear connectors and (d) to the pair.

2.5 Couplings and linkages

The couplings connecting the elements consider the Nodes superposition, with the degrees of freedom adapted, as illustrated in Figure 8. The contact between the slab and the beam was established by the CONTA 173 elements, attached to the section web, and TARGE 170, attached to the inferior surface of the slab. The beam-connector link was considered as a clamped metal pin in the steel section, with rotations and translations made compatible. On the slab-connector interface, translational referring to the X and Z axis were also made compatible and, at the Node below the pin head, there was a consideration of coupling in the Y direction to represent the mechanical anchoring between the head of the connector and the concrete slab. Attempting to reproduce a movable type support, the degrees of freedom related to the translation in X and the rotation in Z were Not restricted at referred Nodes of the composite beam support. At the Nodes of the central section of the composite beam, a symmetry condition was applied, also provided by ANSYS and, consequently, a restriction of degrees of freedom. Figure 8 shows the symmetry condition, the binding of the composite support beam in detail, and also the coupling between the materials. When applying mixed beams loading without shoring, it was assumed that the steel section would support its dead weight and that the recently set concrete on the table would Not have joint between the two materials. The behavior as a composite beam would only occur after the concrete curing, when it would be possible to apply an external load, because the composite beam would have reached the expected resistance as set in the project. Thus, by the time it would start acting as a composite beam, the structure would already be deformed.
In this context, to simulate the loading application in beam A3, the Birth & Death's technique, available in ANSYS , was adopted.

This technique, which allows for elements activation and inactivation of a discretized mesh, consists of the multiplication of the value of the inactivated entity in the stiffness matrix and a reduction factor, which practically blocks the effects of the results of such entity. In this paper the adopted reduction factor was 10^{-6}. Firstly, the concrete slab and the shear connectors were inactivated and the structure dead weight was applied to the steel section. Secondly, the concrete slab was activated and the applied load was used in regard of the solidarity slab-beam work. The structure dead weight was inputted into the modeling according to the unit weight of the materials, which were: 24 kN/m³ for the concrete and 77 kN/m³ for the steel girder, connectors and reinforcements. The applied load was incrementally and monotonically included immediately after the action of the dead weight of the composite beam. Although concentrated in the middle of the span, the load was considered as spread throughout a small area, applied at the Nodes of the upper surface of the concrete slab, centered on the axis of the beam, according to the experimental model presented in. Both the structure dead load and the applied load were included incrementally in the model to take into account the nonlinear behavior of the materials that form the composite beam. Figure 9 shows the composite beam with an applied load concentrated on the mid-span.

TIME HISTORY ANALYSIS

Dynamic analysis using the time history analysis calculates the building responses at discrete time steps using discredited record of synthetic time history as base motion. If three or more time history analyses are performed, only the maximum responses of the parameter of interest are selected..
In linear dynamic method, the structure is modeled as a multi degree of freedom (MDOF) system with a linear elastic stiffness matrix and an equivalent viscous damping matrix. The seismic input is modeled utilizing time history analysis, the displacements and internal forces are found using linear elastic analysis. The playing point of linear dynamic procedure as for linear static procedure is that higher modes could be taken into account.

In order to study the seismic behavior of structures subjected to low, intermediate, and high-frequency content ground motions, dynamic analysis is required. The STAAD Pro [1] software is used to perform linear time history analysis.

3.1.1 Ground Motion Records

Buildings are subjected to ground motions. The ground motion has dynamic characteristics, which are peak ground acceleration (PGA), peak ground velocity (PGV), peak ground displacement (PGD), frequency content, and duration. These dynamic characteristics play predominant rule in studying the behavior of RC buildings under seismic loads. The structure stability depends on the structure slenderness, as well as the ground motion amplitude, frequency and duration. Based on the frequency content, which is the ratio of

PGA/PGV the ground motion records are classified into three categories:

1) High-frequency content PGA/PGV > 1.2
2) Intermediate-frequency content 0.8< PGA/PGV< 1.2
3) Low-frequency content PGA/PGV < 0.8

The ratio of peak ground acceleration in terms of acceleration of gravity (g) to peak ground velocity in unit of (m/s) is defined as the frequency content of the ground

.Figure 3.1 c shows the variation of 1979 Imperial Valley-06 (Holtville Post Office) H-HVP225 component ground acceleration versus time with -0.253 g PGA. The second curve is the ground velocity, obtained by integrating the acceleration-time function. The PGV is -0.488 m/s. Integration of ground velocity gives the ground displacement, displayed as the lowest trace. The peak ground displacement is 0.316 m. In the same manner, Figure 4.5-4.6 shows the variation of ground acceleration versus time with PGA, ground velocity versus time with PGV, and ground displacement versus time with PGD for corresponding ground motions. Then from the acceleration and velocity curves of the ground motion, frequency content, which is the ratio of PGA/PGV, can be obtained. and (2) the procedure chosen to introduce the missing baseline in the record. [35]

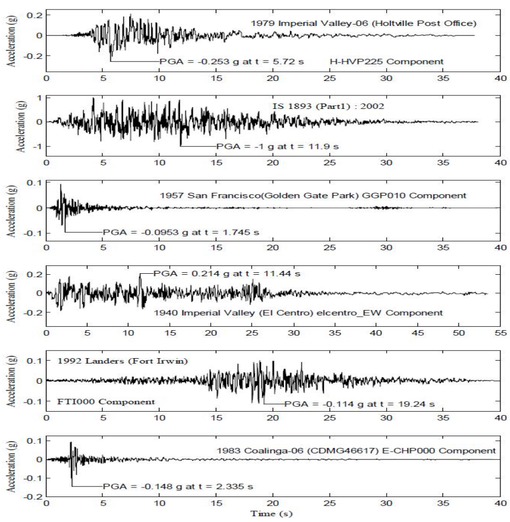


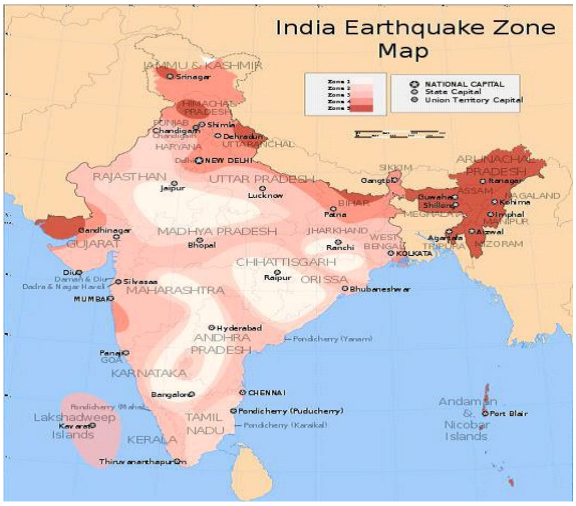
Figure 3.1.1: Ground motion acceleration versus time with PGA value of 1979 Imperial Valley-06 (Holtville Post Office) H-HVP225 component, IS 1893 (Part1) : 2002, 1957 San Francisco (Golden Gate Park) GGP010 component, 1940 Imperial Valley (El Centro) elcentro_EW component, 1992 Landers (Fort Irwin) FTI000 component, and 1983 Coalinga-06 (CDMG46617) E-CHP000 component

3.2 Design Criteria

Following are the major steps in determining the seismic forces:

3.2.1 Determination of base shear:

For the determination of seismic forces, the country is classified in four seismic zones:



The total design lateral force or design base shear along any principal direction shall be determined by this expression

$V_b = A_{h*}W$…………………………….………….... (B)

Where,

Ah = design horizontal seismic coefficient for a structure
W= seismic weight of building
The design horizontal seismic coefficient for a structure Ah is given by

Z is the zone factor given in Table 2 of IS 1893:2002 (part 1) for the maximum considered earthquake (MCE) and service life of a structure in a zone. The factor 2 is to reduce the MCE to the factor for design base earthquake (DBE)

It is the importance factor, depending upon the functional use of the structure, characterized by hazardous consequences of its failure, post-earthquake functional needs, historical or economic importance. The minimum values of importance factor are given in table 6 of IS 1893:2002 R is the response reduction factor, depending on the perceived seismic damage performance of the structure, characterized by ductile or brittle deformations. The need for introducing R in base shear formula

Sa/g is the average response acceleration coefficient for rock and soil sites as given in IS 1893:2002 (part 1). The values are given for 5 % of damping of the structure.

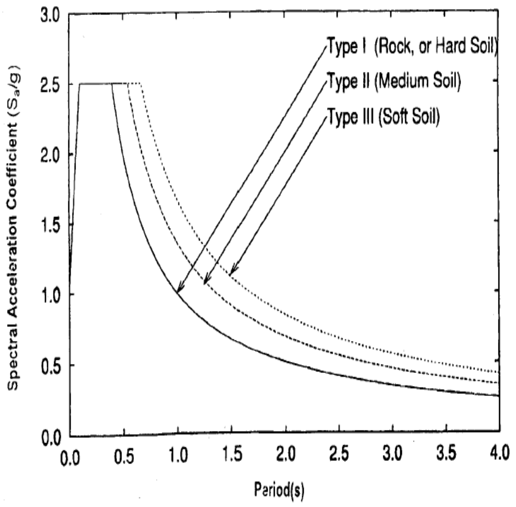


Figure no.3.2.2 IS code spectra from IS 1893:2002 (Part I)

Response Spectrum Method

Response spectrum analysis is a procedure for computing the statistical maximum response of a structure to a base excitation. Each of the vibration modes that are considered may be assumed to respond independently as a single-degree-of-freedom system. spectra which determine the base acceleration applied to each mode according to its period (the number of seconds required for a cycle of vibration).
Response spectrum analysis produces a set of results for each earthquake load case which is really in the nature of an envelope. It is apparent from the calculation, that all results will be absolute values - they are all positive. Each value represents the maximum absolute value of displacement, moment, shear, etc. that is likely to occur during the event which corresponds to the input response spectrum.

Material modeling

The definition of the proposed numerical model was made by using finite elements available in the ANSYS code default library. SOLID186 is a higher order 3-D 20-node solid element that exhibits quadratic displacement behavior. The element is defined by 20 nodes having three degrees of freedom per node: translations in the nodal x, y, and z directions. The element supports plasticity, hyper elasticity, creep, stress stiffening, large deflection, and large strain capabilities. It also has mixed formulation capability for simulating deformations of nearly incompressible elastoplastic materials, and fully incompressible hyper elastic materials. The geometrical representation of is show in SOLID186 fig 22.

This SOLID186 3-D 20-node homogenous/layered structural solid were adopted to discrete the concrete slab, which are also able to simulate cracking behavior of the concrete under tension (in three orthogonal directions) and crushing in compression, to evaluate the material non-linearity and also to enable the inclusion of reinforcement (reinforcement bars scattered in the concrete region).The element SHELL43 is defined by four nodes having six degrees of freedom at each node. The deformation shapes are linear in both in-plane directions. The element allows for plasticity, creep, stress stiffening, large deflections, and large strain capabilities The representation of the steel section was made by the SHELL 43 elements, which allow for the consideration of non-linearity of the material and show linear deformation on the plane in which it is present. The modeling of the shear connectors was done by the BEAM 189 elements, which allow for the configuration of the cross section, enable consideration of the non-linearity of the material and include bending stresses as shown in fig 3.5. CONTA174 is used to represent contact and sliding between 3-D "target" surfaces (TARGE170) and a deformable surface, defined by this element. The element is applicable to 3-D structural and coupled field contact analyses. The geometrical representation of CONTA174 is show in fig 3.2. Contact pairs couple general ax symmetric elements with standard 3-D elements. A node-to-surface contact element represents contact between two surfaces by specifying one surface as a group of nodes. The geometrical representation of is show in TARGET 170 fig 3.

The TARGET 170 and C0NTA 174 elements were used to represent the contact slab-beam interface. These elements are able to simulate the existence of pressure between them when there is contact, and separation between them when there is not. The two material contacts also take into account friction and cohesion between the parties.

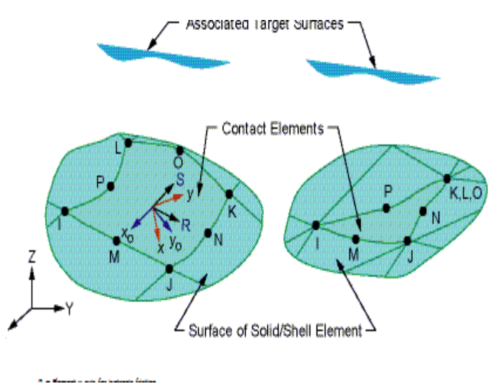


Fig.no.3.2. CONTA 174

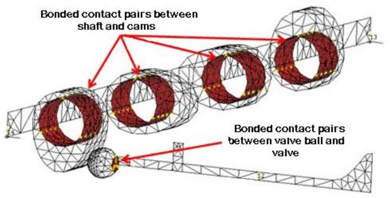


Fig.no.3.3 TARGET 170

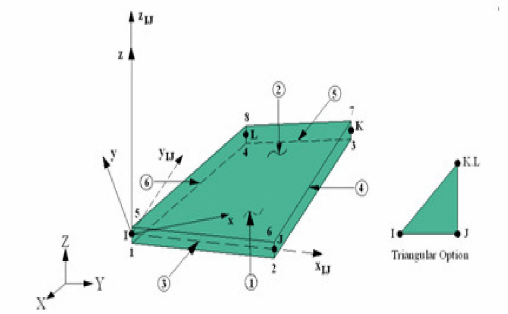


Fig.no.3.4 Shell

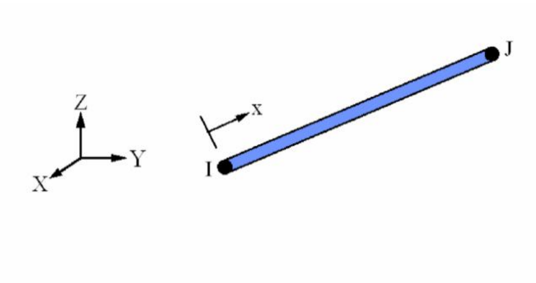


Fig.no.3.5 Beam

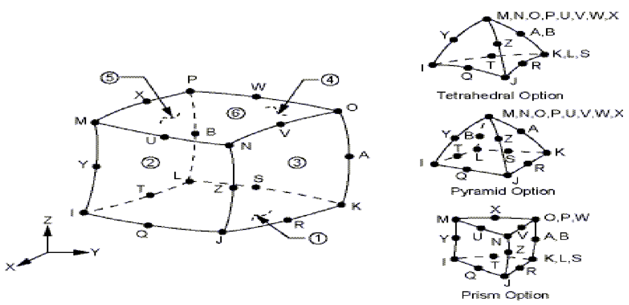


Fig.3.6 Solid 186

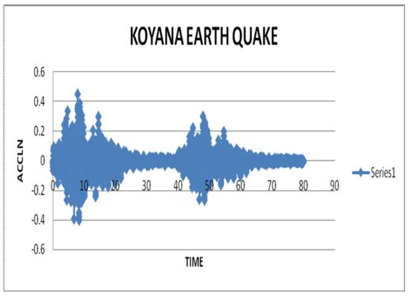
Material properties

TABLE No.1

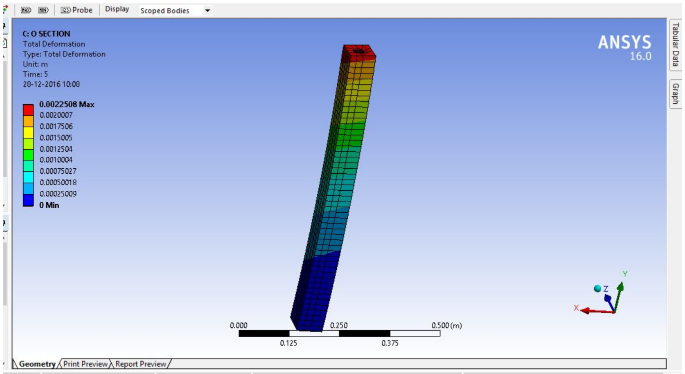
Sr.No.	Material	Property	Value
1	Structural steel	Yield stress f_{ry}(MPa)	265
		Ultimate strength f_{ru}(MPa)	410
		Young's modulus Es(MPa)	205×10^3
		Poisson's ratio µ	0.3
		Ultimate tensile strain et	0.25
2	Reinforcing bar	Yield stress f_{ry}(MPa)	250
		Ultimate strengthf_{ru} (MPa)	350
		Young's modulus Es(MPa)	200×10^3
		Poisson's ratio µ	0.3
		Ultimate tensile strain et	0.25
3	Concrete	Compressive strengthf_{rc}(MPa)	42.5
		Tensile strengthf_{ry}(MPa)	3.553
		Young's modulus E_c(MPa)	32920
		Poisson's ratio µ	0.15
		Ultimate compressive strain e_1	0.045

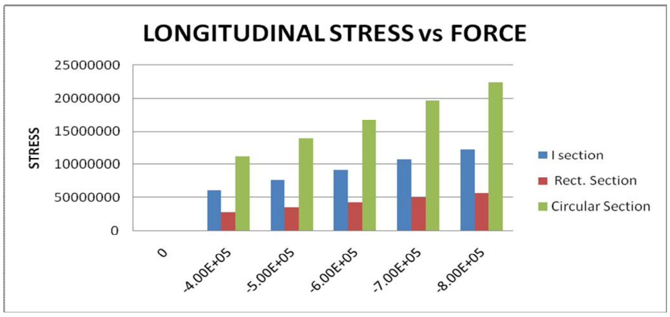
III. RESULTS AND DISCUSSIONS

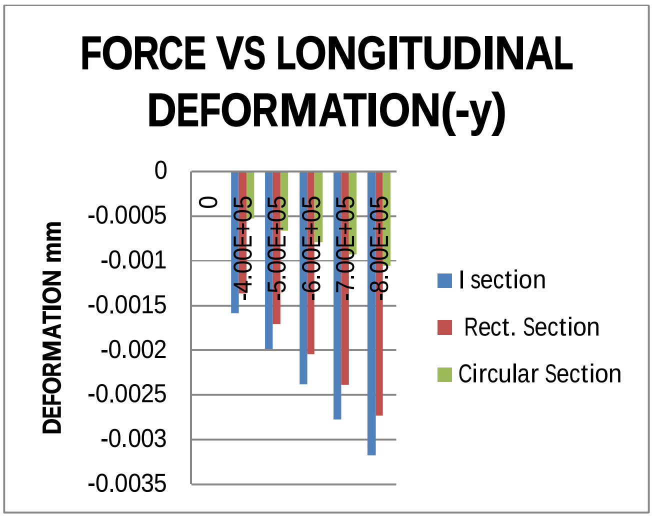
DATA COLLECTED

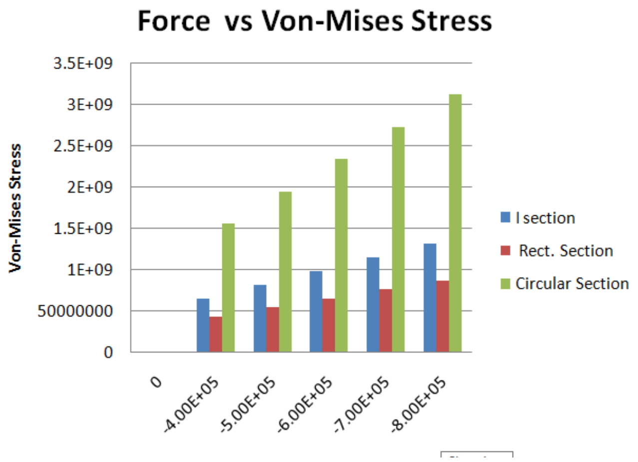


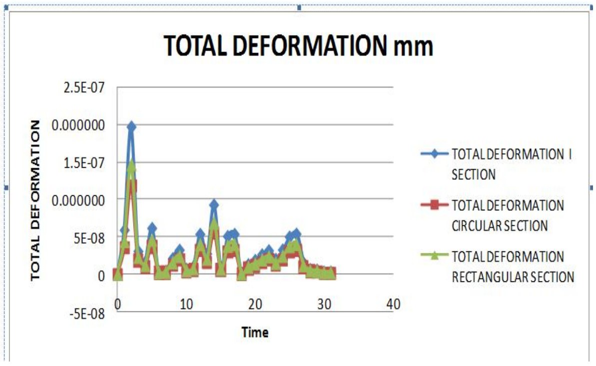
Graph 01 shows comparative results of vertical displacements at mid-span with the increment of the applied load. These results refer to the first stage of the simulation and compare well with values experimentally obtained and numerically presented in and in this work. It is Note-worthy that the computational model developed in took into consideration shored composite beams, while this work and the experimental tests shown in deal with Non-shored composite beams Figure 11 shows that, under the elastic range, results for the composite beams are similar for both the experimental and numerical models.

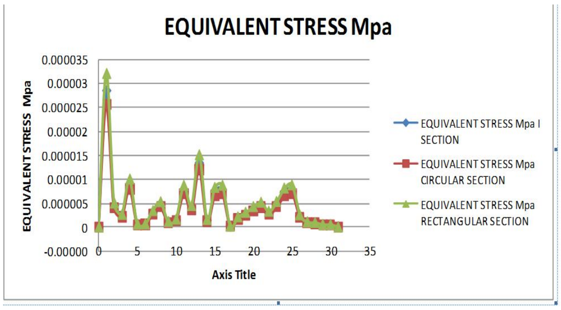












IV. CONCLUSIONS

1. Maximum load carring capacity is found in I-section @ 10-15% more than Circular section and Rectangular section of 800 mm length and 65 mm X 65 mm in cross section
2. In later stage of study validation of specimen is carried out using FEA tool ANSYS.16, normal stress, strain and loading capacity of model is validated and error occurs @ 5% which is quite acceptable.From ANSYS models total deformation ,vonmises stress and normal stress and following results are obtained:
3. Total deformation and deformation in longitudinal direction is 15-20% less in I section as compared to Circular section and Rectangular section

4. Von mises stress and normal stress found maximum in Circular section therefore it should be avoided for heavier loads but due to reduction in concrete it can be used as floating column
5. For dynamic analysis el centro data is used and it is observed that the total deformation, equivalent stress, normal stress and shear stress is observed less circular section as compared to other sections it is due to less weight as compared to other sections.
6. Therefore it can be concluded that the circular section found more stable and economical for dynamic loading due to reduction i seismeic weightbut normal stress and equivalent stress due to direct axial load is more than other section due to reduction in concrete area

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