Dynamic Analysis of Plan Asymmetric Moment Resisting Frame Considering Soil Structure Interaction

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Abstract— This paper aims towards the linear dynamic analysis of asymmetric reinforced concrete building considering the effect of soil–structure interaction. Asymmetry in structure can produce a torsional response coupled with translational response during earthquakes. Torsional behavior of structure includes using the design eccentricities. In this paper G+ 8 R.C. moment resisting frame with 'L' shape are analyze by Response Spectrum Analysis (with and without considering design eccentricity) and Linear Time History Analysis (with IS code compatible time histories). The analysis and design is carried out as per IS 456:2000, IS 1893(Part-1): 2002 and IS 13920:1993. The supports are modeled as fixed, hinged and effect of SSI is accounted by Winkler's approach computed in accordance with the soil sub-grade modulus (Ks) and equivalent spring stiffness according to FEMA-356. Cohesion-less (Ø) soil with medium type is considered. Soil bearing capacity is considered 300kN/m^2. Whole analysis is carried out using STAAD.ProV8i SS5. This study reveals that, consideration of design eccentricity and soil structure interaction in seismic analysis has great impact on response of structure.

Key words: Dynamic Analysis, Response Spectrum Analysis, Linear Time History Analysis, Soil Structure Interaction, Design Eccentricity, Torsion

I. Introduction

Earthquake is the most disastrous due to its unpredictability and huge power of devastation. It is most important to realize that an earthquake can have widely differing effects on different types of buildings depending upon their qualities of symmetry and regularity. Asymmetric buildings are more vulnerable to earthquake hazards compared to the buildings with symmetric configuration. Seismic analysis is essential in asymmetric structure for achieving batter distribution of lateral force producing during earthquake. Buildings with an asymmetric plan undergo coupled lateral and torsional motions during earthquakes. Torsional behavior of structure includes using the design eccentricities, which take into account both natural and accidental sources of torsion. The design eccentricity in building codes should be modified in order to achieve the desirable goal of similar ductility demands on asymmetric-plan and symmetric-plan systems.

Seismic behavior of buildings simultaneously affect by supporting soil medium. Generally in the analysis of structures, the base of structures is assumed to be fixed but in reality the supporting soil medium allows movement to some extent due to its property to deform. Hence, the interactive dynamic response of a structure during an earthquake significantly depends on the characteristics of the ground motion, the surrounding soil medium, its properties and the structure itself. Seismic analysis of a structure strongly recommends the usage of a whole structural system considering the superstructure, foundation and ground giving rise to an area called Soil Structure Interaction (SSI).

It was traditionally been considered that SSI can conveniently be neglected for conservative design. In addition, neglecting SSI tremendously reduces the complication in the analysis of the structures which has tempted designers to neglect the effect of SSI in the analysis. This conservative simplification is valid for certain class of structures and soil conditions. Unfortunately, the assumption does not always hold true. IS 1893(Part I):2002 suggests that SSI may not be considered in the seismic analysis of structure supported on rock or rock like material. But, the code does not provide a standard procedure for considering SSI in the seismic analysis. Developing counties like India, such investigation is the need of the hour.

The main objective of earthquake engineering is to design and build a structure in such a way that the damage to the structure and its structural component during an earthquake is minimized. For achieving such kind of goal, 'seismic analysis of structure considering soil structure interaction' should be essential.

A. *Seismic Analysis*

IS 1893-2002 provides both static and dynamic procedures for the determination of design seismic forces for buildings. Seismic analysis methods as per IS 1893:2002 (Pt-1) are as follows.

1) *Static Analysis*
   - Equivalent Static Analysis
2) *Dynamic Analysis*
   - Response Spectrum Analysis
   - Time History Analysis

B. *Response Spectrum Analysis*

This approach permits the multiple modes of response of a building to be taken into account. The structural response can be defined as a combination of many modes. Computer analysis can be used to determine these modes for a structure. For each mode, a response is obtained from the design spectrum, corresponding to the modal frequency and the modal mass, and then they are combined to estimate the total response of the structure. The RSA utilizes the response spectra to give the structural designer a set of possible forces and deformations a real structure would experience under earthquake loads.

C. *Time History Analysis*

Time-history analysis is a step-by-step analysis of the dynamic response of a structure to a specified loading that may vary with time. The analysis may be linear or nonlinear. Time history analysis is used to determine the dynamic response of a structure to arbitrary loading. Here, Linear time history analysis is carried out with IS code compatible time histories.

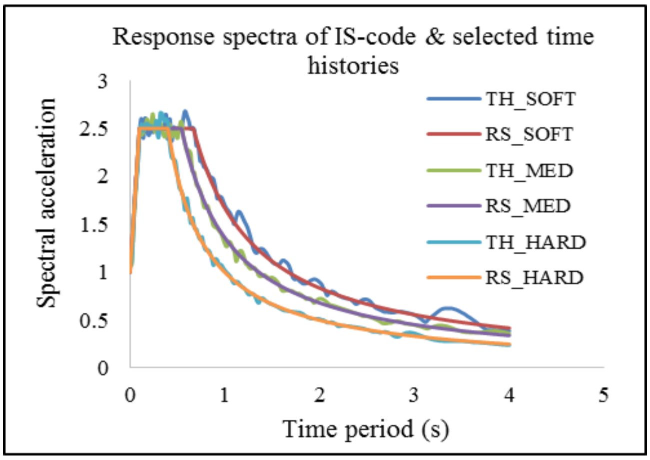


Fig. 1: Response spectra of IS-code compatible time history

D. Soil Structure Interaction

The process in which the response of the soil influences the motion of the structure and the motion of the structure influences the response of the soil is termed as soil-structure interaction. Considering soil-structure interaction makes a structure more flexible and thus, increasing the natural period of the structure compared to the corresponding rigidly supported structure. Approaches for SSI are,

- Winkler approach
- Equivalent spring stiffness

II. METHODOLOGY

In this paper G+ 8 R.C. moment resisting frame of 'L' shape is done according to IS 456:2000 such that it also fulfills the proportioning requirement of IS 13920:1993. The proportioning is done with fix support condition. Cohesion-less (Ø) soil with medium type of soil is considered & soil bearing capacity is considered 300kN/m2.

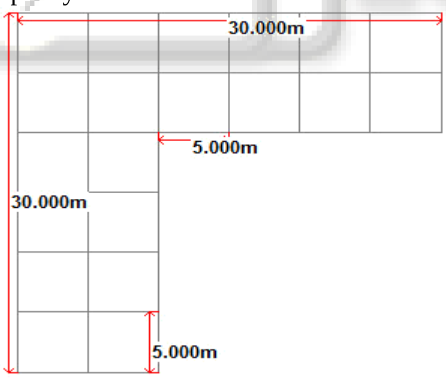


Fig. 2: Plan of asymmetric frame

No. of floors	G+8
Total height of building (h)	30m
Story height	3m
Bay width	5m × 5m
Live Load (L.L)	3 kN/m^2 at typical floor and 1.5 kN/m^2 on roof
Slab thickness	140 mm
Floor finish	1 kN/m^2
Wall	230mm thick masonry wall at periphery
Zone	4
Importance factor (I)	1
Response reduction factor (R)	5 (S.M.R.F)
Concrete grade in beam	M-25
Concrete grade in column	M-30
Steel grade	Fe415
Unit weight of soil (γ)	18 kN/m^2
Poisson's ratio (μ)	0.3

Table 1: General data of building

Level	**Size of beam (mm)**
Roof beam	230x350
5 to 8 outer	300x500
5 to 8 inner	250x450
1 to 4 outer	300x550
1 to 4 inner	250x500
Plinth beam	230x450
Level	**Size of column (mm)**
6 to roof	400x400
3 to 6	450x450
Base to 3	500x500

Table 2: Element Specification

A. Torsion

Torsional behaviour of structure includes using the design eccentricity as per IS1893:2002,

$$edi = 1.5\ est + 0.05\ bi = est - 0.05\ bi$$

No.	**Load combination**
1	1.5 (DL + IL)
2	1.2 (DL + IL ± EXTP)
3	1.2 (DL + IL ± EXTN)
4	1.2 (DL + IL ± EZTP)
5	1.2 (DL + IL ± EZTN)
6	1.5 (DL ± EXTP)
7	1.5 (DL ± EXTN)
8	1.5 (DL ± EZTP)
9	1.5 (DL ± EZTN)
10	0.9 DL ± 1.5 EXTP
11	0.9 DL ± 1.5 EXTN
12	0.9 DL ± 1.5 EZTP
13	0.9 DL ± 1.5 EZTN
Note	EXTP: EQ load in X direction with torsion +ve EXTN: EQ load in X direction with torsion -ve EZTP: EQ load in Z direction with torsion +ve EZTN: EQ load in Z direction with torsion -ve

Table 3: Load Combination generate due to torsion

B. Winkler's Approach

In Winkler's approach the soil medium is represented by a number of identical but mutually independent, closely spaced, discrete, linearly elastic springs. According to this idealization, deformation of foundation due to applied load is confined to the loaded region only.

In this approach it is assumed that the sub grade consists of an infinite array of individual elastic springs each of which is not affected by others. The spring constant is equal to the modulus of sub grade reaction (K_s).

The soil sub-grade modulus is calculated from soil bearing capacity (S.B.C) and allowable settlement (δ).

$$K_s = {}^{P}/_{\delta}$$

$$= {}^{300}/_{0.04} = 7500 \text{ kN/m}^3$$

Where,

P = Pressure against footing (kN/ m^2),
K_s = Modulus of sub-grade reactions (kN/ m^3)
δ = Allowable settlement (m)

The footing is modelled in the software as 3D plate element beneath the column and sub-grade is modelled as springs. Finite Element Analysis (FEA) is used for analysing this problem of plate resting on springs. FEA is computer based numerical technique for calculating the strength and behaviour of engineering structures, where in a whole structure is discrete into number of elements and each element is analysed individually. While modelling foundation thickness of the plate beneath the column has given almost half to the floor height as for making it as rigid element. All the other plates have the thickness equal to the thickness of footing. Meshing is applied in the plate with aspect ratio in between 1 to 2 and vertical spring is applied at each node beneath the meshed plate and the stiffness value of spring is given equal to soil sub-grade modulus (K_s).

C. Equivalent Soil Spring Stiffness by FEMA 356

The movement of the foundation is generally considered in two perpendicular horizontal directions and in vertical direction. The rotations of the same about these three directions should also be considered when equivalent soil spring stiffness is generated. So in this approach effect of SSI is considered by equivalent springs with six degrees of freedom (DOF).

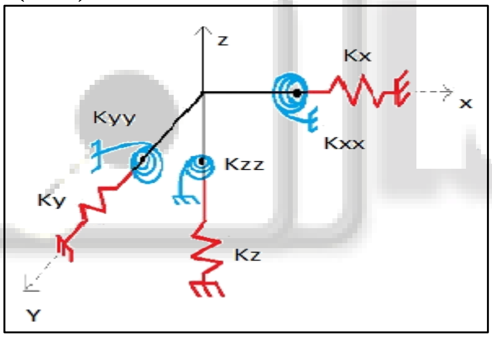


Fig. 3. Equivalent Soil Spring

Shear modulus (G) used in stiffness formula of FEMA356 is computed in accordance with N-value of Standard Penetration Test (S.P.T). Now, N-value is computed in accordance with soil bearing capacity (S.B.C) by settlement criteria and shear criteria. The procedure of computing N-values by settlement criteria and shear criteria are discussed below in FEMA1 and FEMA2 respectively.

By using this N-value further calculation is made for compute soil property in following way,

– The relation between modulus of elasticity (E) of sandy soil and N-value.

$$E = 500\,(N + 15)\ \text{kN/m}^2$$

– The relation between modulus of elasticity (E) and Shear modulus (G),

$$G = \frac{E}{2(1+\mu)}$$

Where, μ = Poisson's ratio (For sand μ = 0.3 is considered)

This soil property are used in formula given by fema356,

$$K_i = K_{isur} * \beta_i$$

Where, K_i = Equivalent soil spring stiffness
K_{isur} = Soil spring stiffness at surface
β_i = Correction factor for embedment
i = x, y, z, xx, yy and zz;

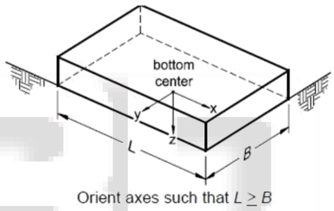
Degree of freedom	Stiffness of foundation at surface
Translation along x-axis	$K_{x,sur} = \frac{GB}{2-\mu}\left[3.4\left(\frac{L}{B}\right)^{0.65} + 1.2\right]$
Translation along y-axis	$K_{y,sur} = \frac{GB}{2-\mu}\left[3.4\left(\frac{L}{B}\right)^{0.65} + 0.4\frac{L}{B} + 0.8\right]$
Translation along z-axis	$K_{z,sur} = \frac{GB}{1-\mu}\left[1.55\left(\frac{L}{B}\right)^{0.75} + 0.8\right]$
Rocking about x-axis	$K_{xx,sur} = \frac{GB^3}{1-\mu}\left[0.4\left(\frac{L}{B}\right) + 0.1\right]$
Rocking about y-axis	$K_{yy,sur} = \frac{GB^3}{1-\mu}\left[0.47\left(\frac{L}{B}\right)^{2.4} + 0.034\right]$
Torsion about z-axis	$K_{zz,sur} = GB^3\left[0.53\left(\frac{L}{B}\right)^{2.45} + 0.51\right]$
Note	 L= Length of footing B= Width of footing G= Shear modulus of soil μ = Poisson's ratio of soil

Table 4: Stiffness of foundation at surface

Degree of freedom	Correction factor for embedment
Translation along x-axis	$\beta_x = \left(1 + 0.21\sqrt{\frac{D}{B}}\right)\left[1 + 1.6\left(\frac{hd(B+L)}{BL^2}\right)^{0.4}\right]$
Translation along y-axis	$\beta_y = \beta_x$
Translation along z-axis	$\beta_z = \left[1 + \frac{1}{21}\frac{D}{B}\left(2 + 2.6\frac{B}{L}\right)\right]\left[1 + 0.32\left(\frac{d(B+L)}{BL}\right)^{\frac{2}{3}}\right]$
Rocking about x-axis	$\beta_{xx} = 1 + 2.5\frac{d}{B}\left[1 + \frac{2d}{B}\left(\frac{d}{D}\right)^{-0.2}\sqrt{\frac{B}{L}}\right]$
Rocking about y-axis	$\beta_{yy} = 1 + 1.4\left(\frac{d}{L}\right)^{0.6}\left[1.5 + 3.7\left(\frac{d}{L}\right)^{1.9}\left(\frac{d}{D}\right)^{-0.6}\right]$

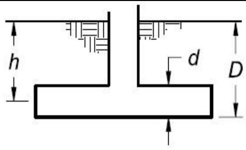
Torsion about z-axis	$\beta_{zz} = 1 + 2.6\left(1 + \frac{B}{L}\right)\left(\frac{d}{B}\right)^{0.9}$
Note	 d = height of effective sidewall contact h = depth of centroid of effective sidewall contact

Table 5: Correction factor for embedment

D. FEMA1 (N-value by settlement criteria) :

In FEMA1 N-values are computing according to settlement criteria as per IS 8009 (Part-1):1976 code for calculation of settlements of shallow foundations.

IS 8009 (Part-1):1976 gives relation among width of footing, N-values from standard penetration test and settlement per unit pressure in graphical form. This graph represents settlement in meter per unit pressure of 100kN/m^2. Here, max allowable settlement is 40mm (for isolated footing) for our assumed soil bearing capacity. From this data, settlement for unit pressure of 100kN/m^2 is calculated. Then, N-value is computed in accordance with that settlement and width of footing from graph.

E. FEMA2 (N-value by shear criteria) :

In FEMA2 N-values are computing according to shear criteria as per IS 6403:1981.

IS 6403:1981 gives relationship between angle of internal friction (Ø) and N-values of standard penetration test. This code also gives equation to obtain ultimate bearing capacity. To obtain safe bearing capacity, divide ultimate bearing capacity by factor of safety (In this paper f.o.s. is 3).

The bearing capacity factors depend upon angle of internal friction (Ø). So, Ø is calculated such that the value of safe bearing capacity becomes equal to our assumed soil bearing capacity. For that particular value of Ø, N-value is computed from graph.

III. ANALYSIS RESULTS

A. Time Period of Structure:

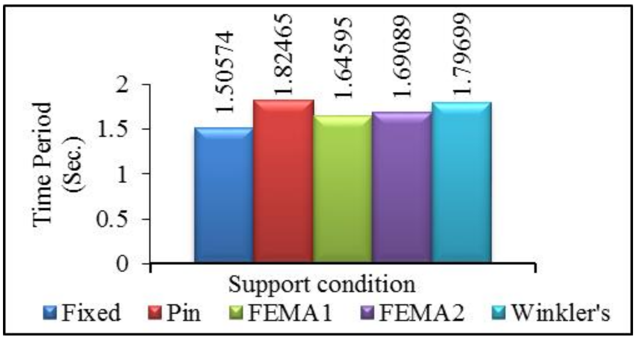


Fig. 4: Time period of structure

B. Storey Displacement:

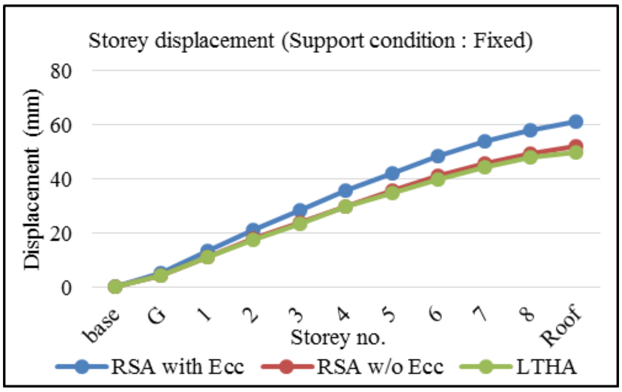


Fig. 5: Story displacement with fix support

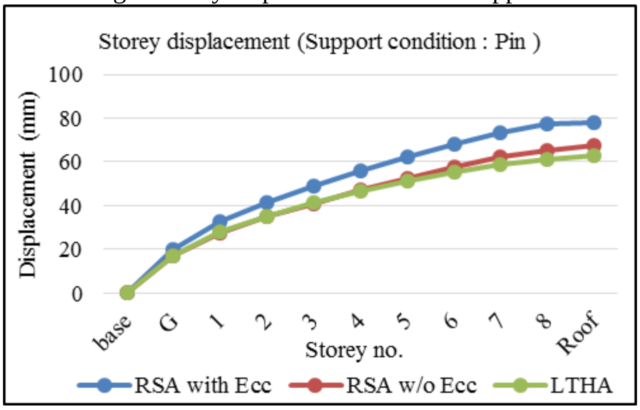


Fig. 6: Story displacement with pin support

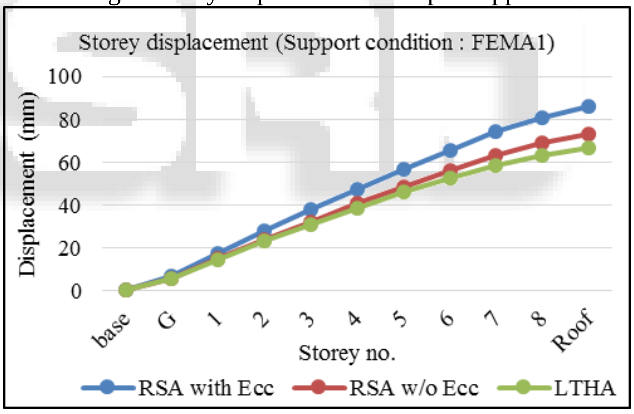


Fig. 7: Story displacement with FEMA1 support

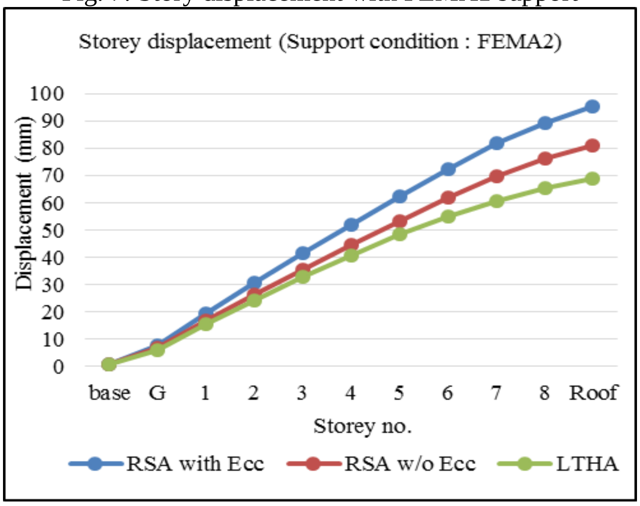


Fig. 8: Story displacement with FEMA2 support

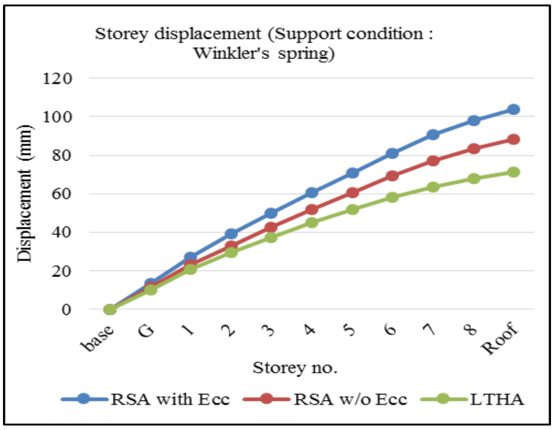


Fig. 9: Story displacement with Winkler's spring

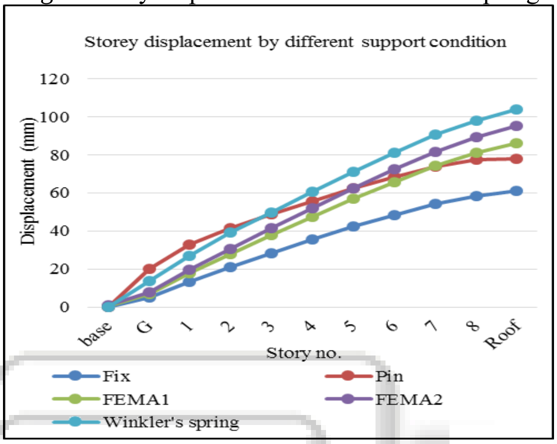


Fig. 10: Story displacement by RSA with design ecc.

C. *Bending Moment:*

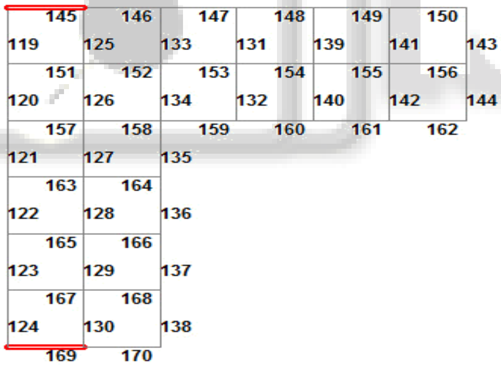


Fig. 11: Selected beam

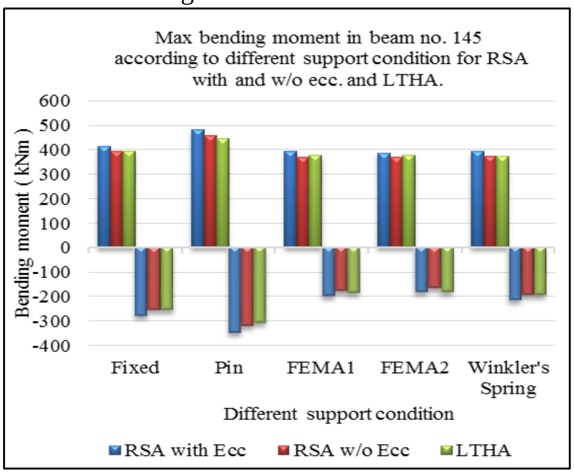


Fig. 12: Max bending moment in beam no. 145

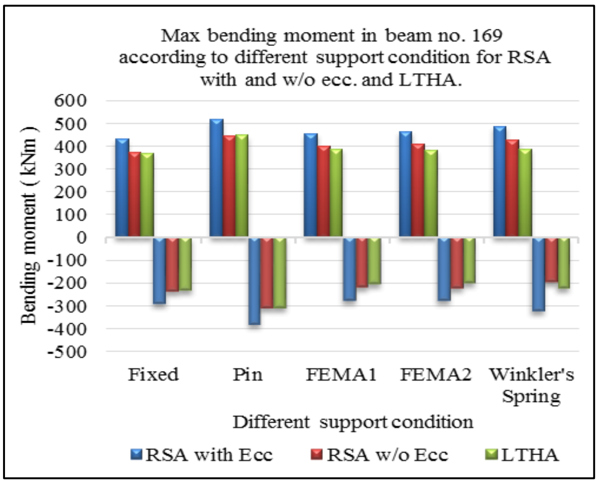


Fig. 13: Max bending moment in beam no. 169

D. *Column End Forces*

The comparison of column end forces is done on the base of unity check for column capacity. The iteration ratio is derived for required percentage of steel (Pt) by RSA with design eccentricity and shall be compared with iteration ratio derived by RSA without design eccentricity and linear time history analysis (LTHA) for the same required percentage of steel (Pt).

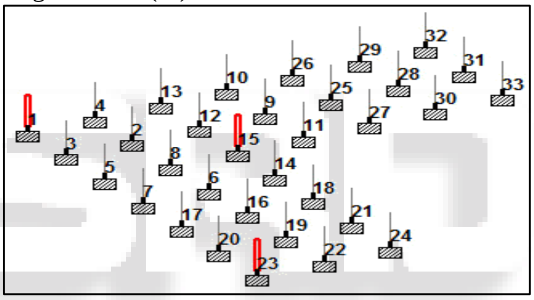


Fig. 14: Selected column

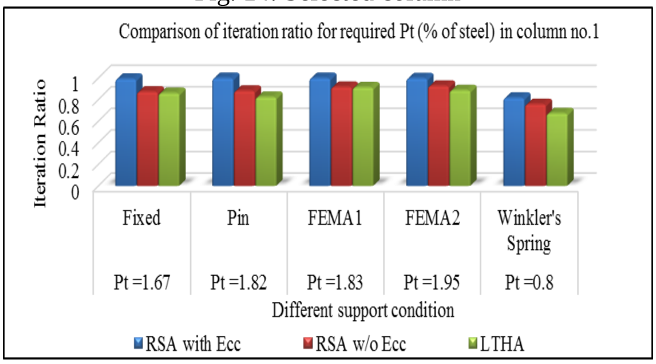


Fig. 15: Comparison of iteration ratio for required Pt (% of steel) in column no.1

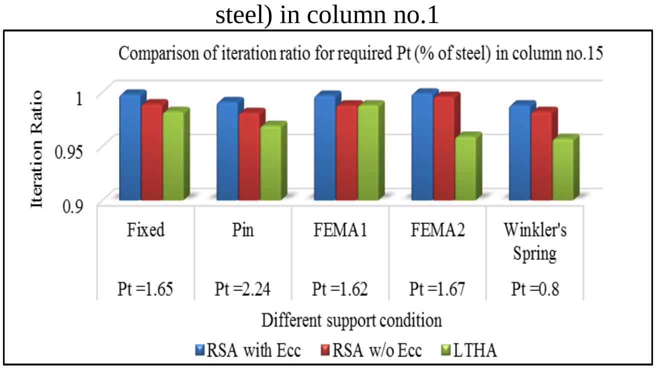


Fig. 16: Comparison of iteration ratio for required Pt (% of steel) in column no.15

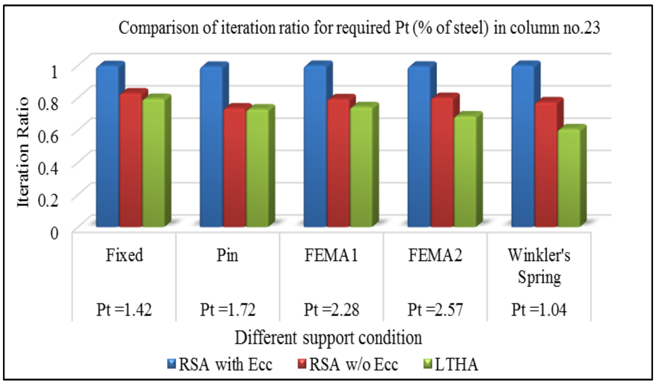


Fig. 17: Comparison of iteration ratio for required Pt (% of steel) in column no.23

IV. CONCLUSION

This study reveals that, consideration of design eccentricity and soil structure interaction in seismic analysis has great impact on response of structure. From observation and results of this study it can be conclude that,

1) Time period of structure is least in fixed support condition and for pin support it is maximum. For both approach of the soil structure interaction it lies between these two extremes. In case of soil structure interaction Winkler's approach give higher time period than approach of equivalent spring stiffness by FEMA356.
2) Storey displacement is least for fixed support and for Winkler's approach it is maximum. In case of equivalent spring stiffness by FEMA356 it is lies between these two extremes. In case of pin support, lower stories (stories up to 3) displacement is higher than all other support condition but for higher stories it gives lower value than SSI approaches.
3) RSA (with design eccentricity) gives maximum storey displacement & LTHA gives minimum storey displacement in all support condition.
4) RSA (with design eccentricity) give max value of required percentage of steel (Pt) in column for all support condition.
5) RSA (with design ecc.) gives slightly higher values of bending moment than LTHA and RSA (w/o design ecc.).
   So, Consideration of design eccentricity in seismic analysis gives higher value displacement and forces of structure.
6) Equivalent spring stiffness approach by FEMA356 gives higher value of required percentage of steel (Pt) in column as compare to Winkler's approach.
7) Time history analysis is an elegant tool to visualize the performance level of a building under the earthquake, and from comparison of LTHA and RSA it is concluded that design forces by RSA are very acceptable.

Note: The natural / inherent torsion, generated due to the difference between CM and CR when dynamic loading is acting along Z direction, was not added to the original load vector. This problem was found in STAAD.Pro V8i SS5 during this research work. So, the results of only X direction are discussed here.

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