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Linear and Non-Linear Analysis of Reinforced Concrete Frames with Members of Varying Inertia

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Abstract: *Beams are major media of carrying and transferring loads. A careful approach in its design may lead to good serviceability and optimization of the cost of structure. Prismatic beams are commonly used for medium span and bending moments. As the span increases, bending moments and shear forces increases substantially at the centre of span and over the supports. Hence, prismatic beams may become uneconomical in such cases. Moreover, with the increased depth there is considerable decrease in headroom. Therefore in such cases non-prismatic beams are an appealing solution.In the present study, linear and non-linear analysis of reinforced concrete buildings with members of varying inertia has been carried out for buildings of (G+2), (G+4), (G+6), (G+8) and (G+10) storey. Further, two cases are considered, one is bare frame (without infill walls) and another one is frame with infill (considering infill walls). The buildings are analyzed for severe earthquake load (seismic zone V of India). Linear analysis of frames has been done using two methods Seismic Coefficient Method and Response Spectrum Method. Non-linear analysis of frames has been done using Pushover Analysis as per ATC 40 and FEMA 356 guidelines. Beams in x direction are made non-prismatic, Linear Haunch, Parabolic Haunch and Stepped Haunch are considered. The linear analysis is performed using ETABS 9.7.4 and non-linear analysis is performed using SAP2000.The linear analysis has been performed on the building to identify the effect of varying inertia on various response parameters such as base shear, displacement and member forces. The nonlinear analysis has been performed to determine the capacity spectrum curve, performance levels and hinge formation patterns of the considered buildings. Due to inclusion of non-prismatic members, moments in the members have varied significantly but forces in the members haven't varied much as well as it leads to the formation of strong beam and weak column.*

Keywords: *Non-Prismatic Members, Base Shear, Time Period, Storey Displacement, Seismic Coefficient Method, Response Spectrum Method and Pushover Analysis.*

I. Introduction

In last few years the widespread damage to reinforced concrete building during earthquake generated demand for seismic evaluation and retrofitting of existing buildings in Indian sub-continents. In addition, most of our buildings built in past decades are seismically deficient because of lack of awareness regarding structural behavior during earthquake and reluctance to follow the code guidelines. Due to scarcity of land, there is growing responsiveness of multi-storied reinforced concrete structures to accommodate growing population. In developing countries, multi-storied buildings are generally provided with prismatic sections. Structural engineers should design the structures in such a way that the structural systems perform their functions satisfactorily and at the same time the design should prove to be economical. This helps to choose the right type of sections consistent with economy along with safety of the structure. Beams are major media of carrying and transferring loads. A careful approach in its design may lead to good serviceability and optimization of the cost of structure. Prismatic beams are commonly used for medium span and bending moments. As the span increases, bending moments and shear forces increases substantially at the centre of span and over the supports. Hence, prismatic beams may become uneconomical in such cases. Moreover, with the increased depth there is considerable decrease in headroom. Therefore in such cases non-prismatic beams are an appealing solution.

The non-prismatic members having varying depths are frequently used in the form of haunched beams for bridges, portal frames, cantilever retaining walls etc. The cross-section of the beams can be made non-prismatic by varying width, depth, or by varying both depth and width continuously or discontinuously along their length. Variation in width causes difficulty in construction. Therefore, beams with varying depth are generally provided. Either the soffit or top surface of the beam can be inclined to obtain varying cross-section, but the former practice is more common. The soffit profile may have triangular or parabolic haunches. Effective depth of such beams varies from point to point and the internal compressive and tensile stress resultants are inclined. It makes the analysis of such beams slightly different from prismatic beams. The inclination of internal stress resultant may significantly affect the shear for which the beam should be designed. The aim of the present work is to study the effect of non-prismatic members in multistoried RC frames, with respect to various building performance levels. The work also aims at studying the linear and non-linear behavior of frames with varying inertia.

II. Methodology

2.1 Equivalent Static Method

Seismic analysis of most structures is still carried out on the assumption that the lateral (horizontal) force is equivalent to the actual (dynamic) loading. This method requires less effort because, except for the fundamental period, the periods and shapes of higher natural modes of vibration are not required. The base shear which is the total horizontal force on the structure is calculated on the basis of the structures mass, its fundamental period of vibration, and corresponding shape. The base shear is distributed along the height of the structure in terms of lateral force according to Codal formula. Planar models appropriate for each of the two orthogonal lateral directions are analyzed separately, the results of the two analyses and the various effects, including those due to torsional motions of the structure, are combined. This method is usually conservative for low to medium-height buildings with a regular configuration.

2.2 Response Spectrum Method

This method is also known as Modal Method or Mode Super-Position Method. This method is applicable to those structures where modes other than the fundamental one significantly affect the response of structures. Generally, this method is applicable to analysis of the dynamic response of structures, which are asymmetrical or have geometrical areas of discontinuity or irregularity, in their linear range of behaviour. In particular, it is applicable to analysis of forces and deformation in multi-storey buildings due to intensity of ground shaking, which causes a moderately large but essentially linear response in the structure.

This method is based on the fact that, for certain forms of damping which are reasonable models for many buildings the response in each natural mode of vibration can be computed independently of the others, and the modal responses can be combined to determine the total response. Each mode responds with its own particular pattern of deformation (mode shape), with its own frequency (the modal frequency), and with its own modal damping.

2.3 Non-Liner Static Pushover Analysis

Pushover analysis which is an iterative procedure is looked upon as an alternative for the conventional analysis procedures. Pushover analysis of multi-story RCC framed buildings subjected to increasing lateral forces is carried out until the preset performance level (target displacement) is reached. The promise of performance-based seismic engineering (PBSE) is to produce structures with predictable seismic performance.

The recent advent of performance based design has brought the non-linear static push over analysis procedure to the forefront. Pushover analysis is a static non-linear procedure in which the magnitude of the structural loading along the lateral direction of the structure is incrementally increased in accordance with a certain pre-defined pattern. It is generally assumed that the behaviour of the structure is controlled by its fundamental mode and the predefined pattern is expressed either in terms of story shear or in terms of fundamental mode shape. With the increase in magnitude of lateral loading, the progressive non-linear behaviour of various structural elements is captured, and weak links and failure modes of the structure are identified. In addition, pushover analysis is also used to ascertain the capability of a structure to withstand a certain level of input motion defined in terms of a response spectrum. Pushover analysis is of two types:

(i) Force Controlled

(ii) Displacement Controlled.

In the force control, the total lateral force is applied to the structure in small increments. In the displacement control, the displacement of the top storey of the structure is incremented step by step, such that the required horizontal force pushes the structure laterally. The distance through which the structure is pushed, is proportional to the fundamental horizontal translational mode of the structure. In both types of pushover analysis, for each increment of the load or displacement, the stiffness matrix of the structure may have to be changed, once the structure passes from the elastic state to the inelastic state. The displacement controlled pushover analysis is generally preferred over the force controlled one because the analysis could be carried out up to the desired level of the displacement.

III. Description of Analytical Model

The R.C. moment resisting frame models with prismatic and non-prismatic members are developed. Material properties, geometry and loading conditions of different models are as follows:

3.1 Material Properties

Density of concrete and brick masonry is taken as 25 KN/ m^3 and 20 KN/m^3 respectively. M-25 grade of concrete and Fe 500 grade of reinforcing steel are used for all the frame models considered in this study. The modulus of elasticity for concrete and brick masonry is taken as 25000MPa and 1225MPa respectively.

3.2 Geometry and Loading Conditions

In the present study, Bare frames and Frames with infill situated in seismic zone V are considered with variations of heights, i.e. (G+2), (G+4), (G+6), (G+8) and (G+10). Depending upon different height of building, depth of foundation is taken as 1.5m for (G+2) and (G+4) buildings, 2.0m (G+6), (G+8) and (G+10) buildings. The storey height taken is 4m (for all models). The analytical model consists of single bay of 10m in global X direction and 5 bays of 3m each in Y direction. Beams in X direction are made non-prismatic. Three types of non-prismatic members are developed which includes linear haunch (LH), parabolic haunch (PH) and stepped haunch (SH). In the model, the support condition is assumed to be fixed and soil condition is assumed as medium soil.

The size of beam in X direction is taken as 250mmX710mm (for prismatic member) and 230mmX530mm (medium soil) in Y direction. Length of haunch is taken as 1000mm, depth of haunch at centre as 675mm and depth of haunch at supports as 1000mm, width of haunch is 250mm. Sizes of columns have been varied according to loading conditions. Thickness of slab as well as brick wall is taken as 150 mm; floor finish load is 1 KN/m^2, Live load on floor slabs is 4 KN/m^2. Seismic coefficient method is used for static analysis and Response spectrum method is used for dynamic analysis. And non-linear analysis has been performed by using Static Pushover Analysis.

The plan, elevations in X direction of different frames, elevation in Y direction of frame with prismatic members for G+2 bare frame structure considered in this study are as shown in Figures 4.1 to 4.6. Detailed features of building are shown in Table 4.1.

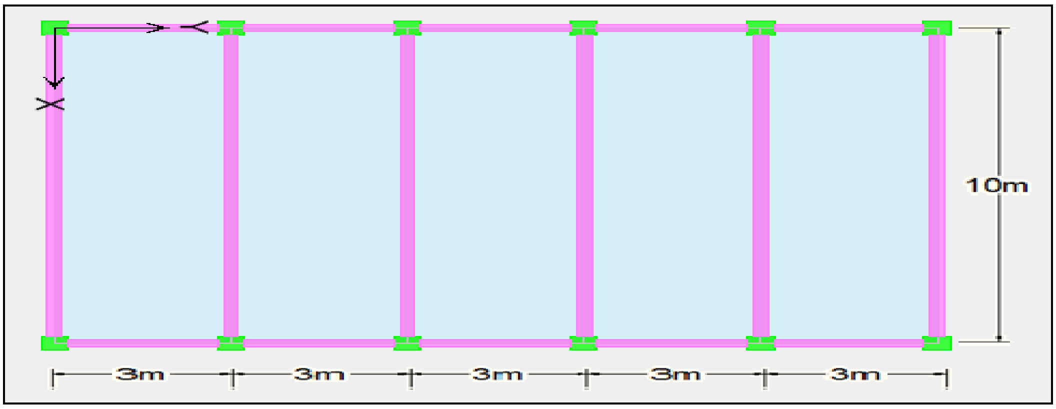

Figure 4.1: Plan of Building

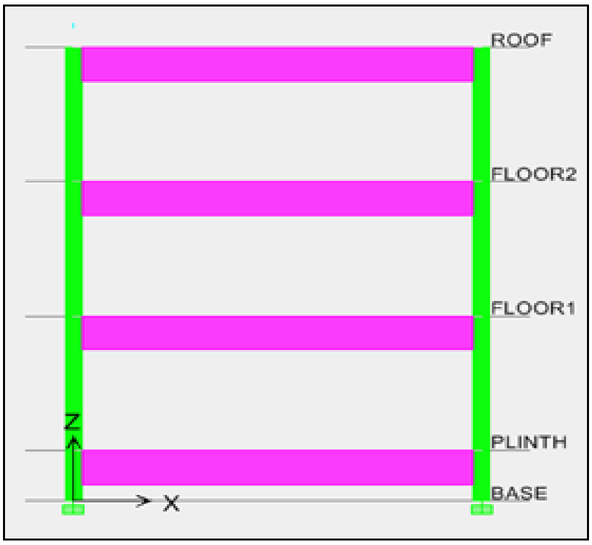

Figure 4.2: Elevation of Frame with Prismatic member in X direction

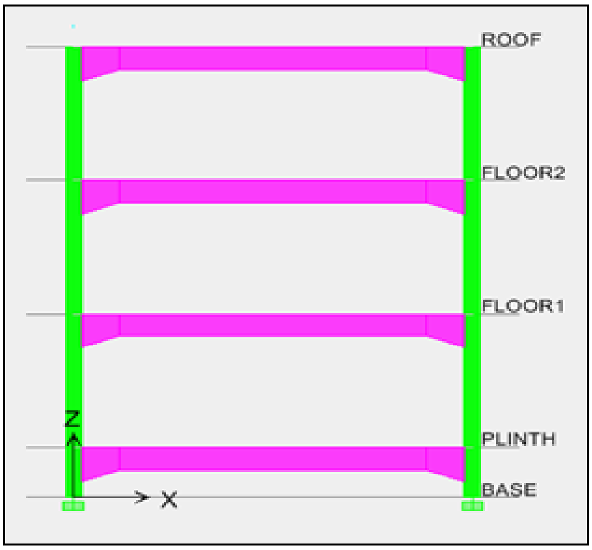

Figure 4.3: Elevation of Frame with Linear haunch in X direction

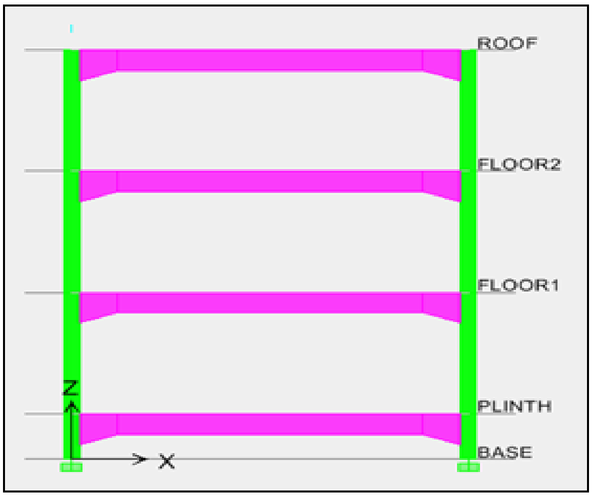

Figure 4.4: Elevation of Frame with Parabolic haunch in X direction

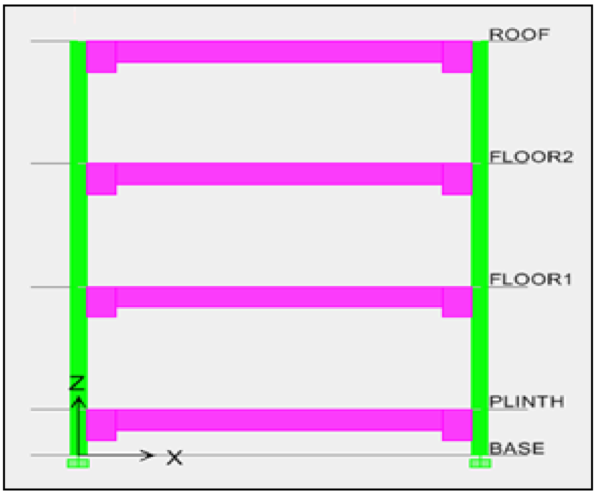

Figure 4.5: Elevation of Frame with Stepped haunch in X direction

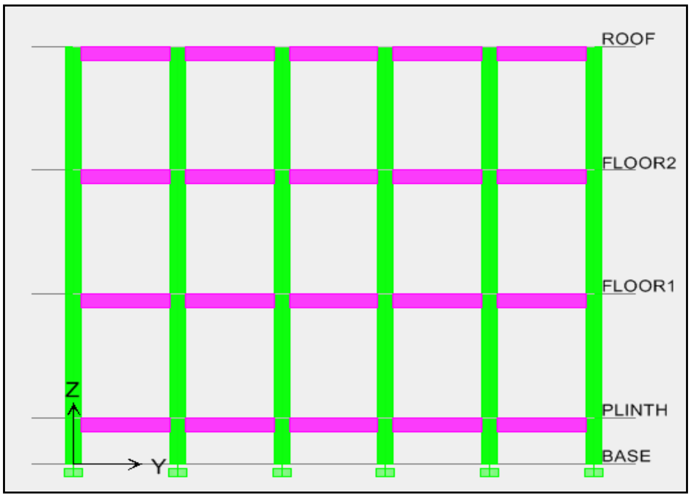

Figure 4.6: Elevation of Frame with Prismatic member in Y direction

IV. Results

The linear analysis is performed using ETABS 9.7.4 and non-linear analysis is performed using SAP2000. The response of structures has been studied in the form of base shear, displacement, effective time period, effective stiffness and pattern of hinge formation. The results of various parameters are presented in the form of Figures from 4.7 to 4.17 and Tables from 4.1 to 4.4 respectively. The observations for each parametric variation are stated as under respective tables and graphs.

4.1 Fundamental Time Period (sec.)

Natural period of a structure is its time period of undamped free vibration. It is the first (longest) modal time period of vibration. Variation of Fundamental Time Period for various height of structure is shown in Figures 4.7 and 4.8.

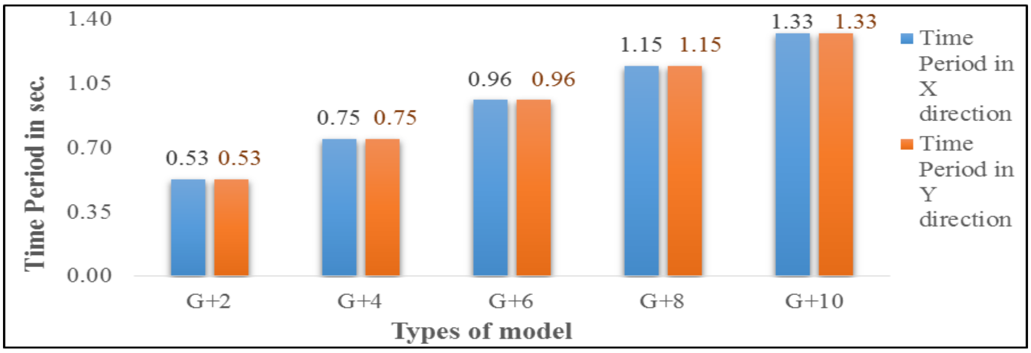

Figure 4.7: Variation of Time Period (sec.) for Bare Frame

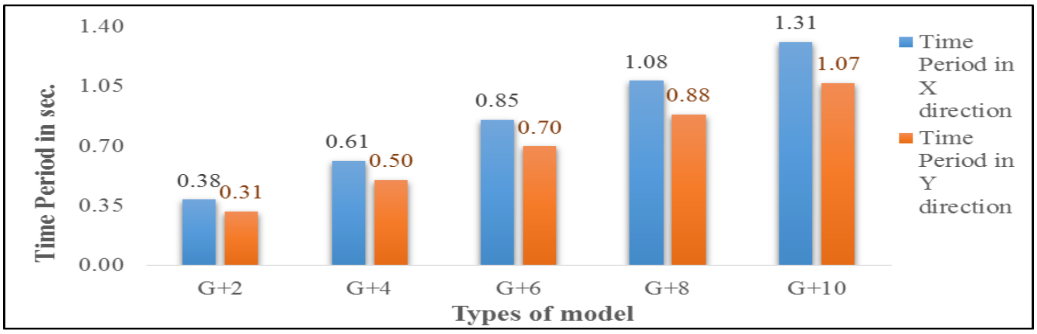

Figure 4.8: Variation of Time Period (sec.) for Frame with Infill

4.2 Base Shear (KN)

It is the total design lateral force at the base of the structure. Variation of Base Shear in X as well as Y direction has been studied.

4.2.1 Variation of Base Shear in X direction for G+8 building is shown in Figures 4.9 and 4.10.

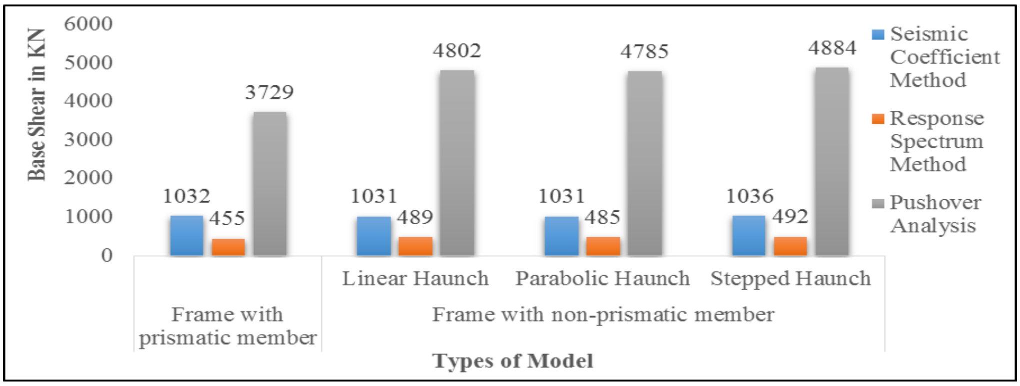

Figure 4.9: Variation of Base Shear in X direction for G+8 Bare Frame in KN

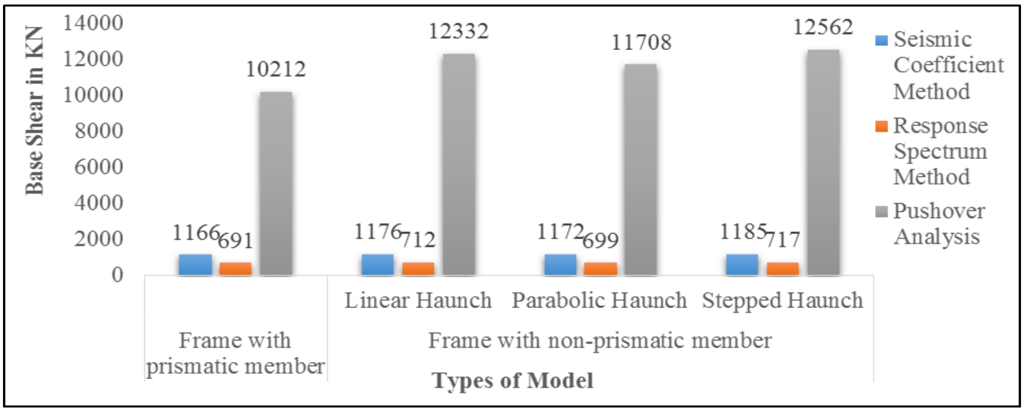

Figure 4.10: Variation of Base Shear in X direction for G+8 Frame with Infill in KN

4.2.2 Variation of Base Shear in Y direction for G+8 building is shown in Figures 4.11 and 4.12.

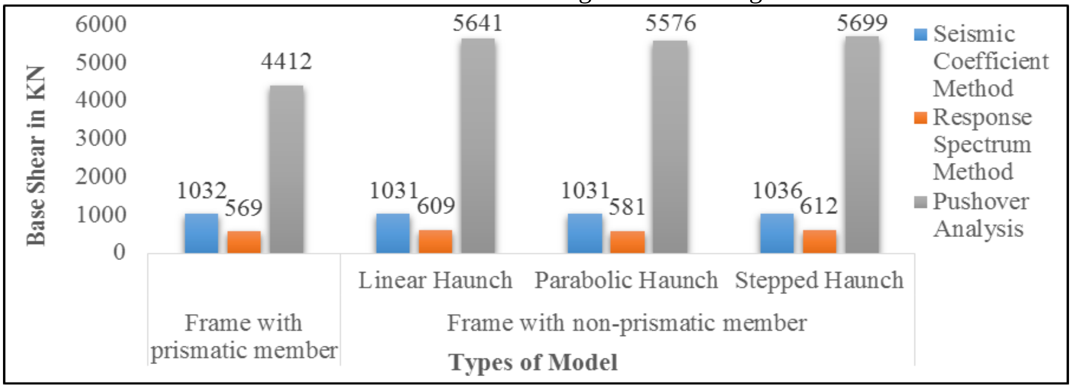

Figure 4.11: Variation of Base Shear in Y direction for G+8 Bare Frame in KN

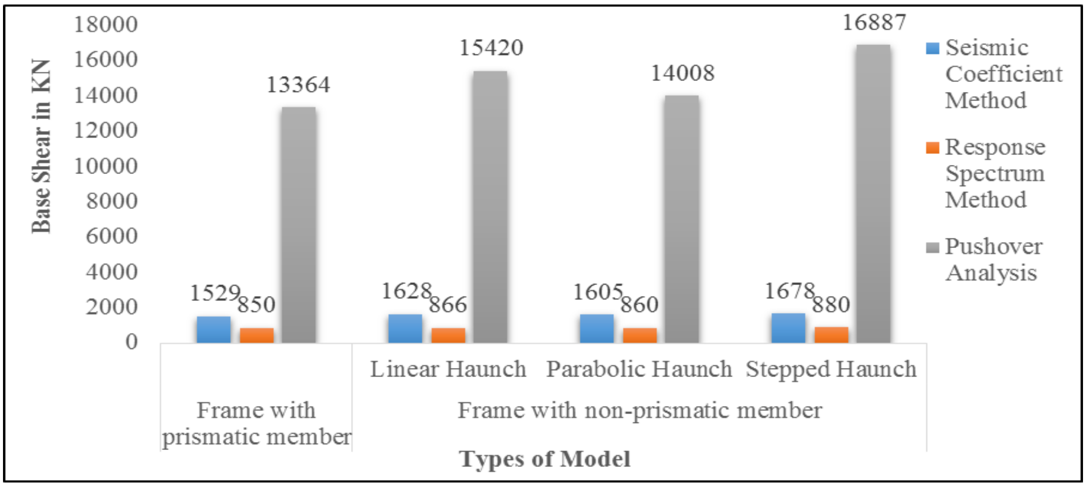

Figure 4.12: Variation of Base Shear in Y direction for G+8 Frame with Infill in KN

4.3 Top Storey Displacements (mm)

It is the lateral displacement at the top floor of frame. The Displacements are observed for EQx case. The variation of displacements for G+8 building is shown in Figures 4.13 and 4.14.

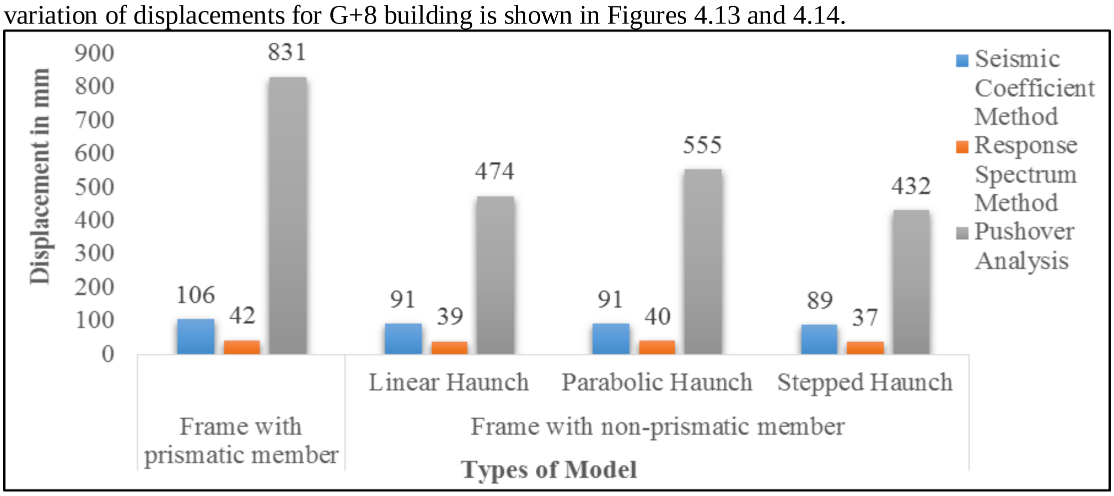

Figure 4.13: Variation of Displacement for G+8 Bare Frame in mm

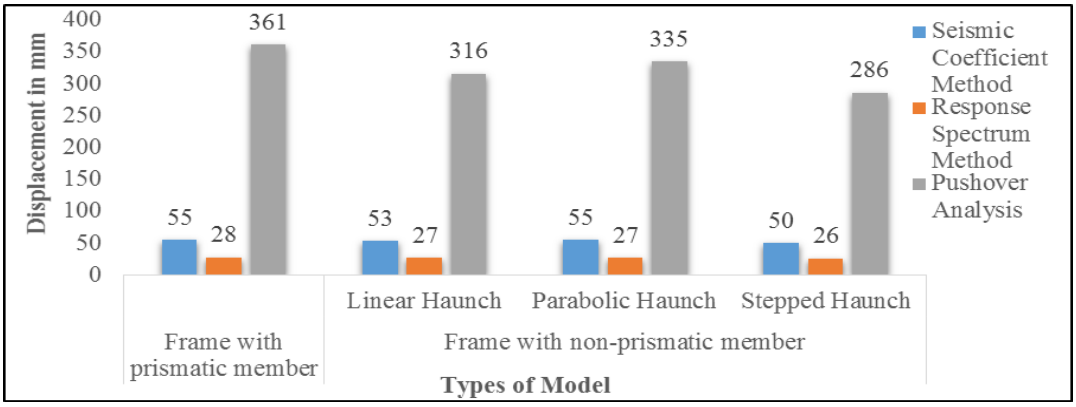

Figure 4.14: Variation of Displacement for G+8 Frame with Infill in mm

4.4 Effective Time Period (sec.)

An effective period, Te, is generated from the initial period, Ti, by a graphical procedure using an idealized force-deformation curve (i.e., pushover curve) relating base shear to roof displacement, which accounts for some stiffness loss as the system begins to behave inelastically. The effective period represents the linear stiffness of the equivalent SDOF system. The effective period is used to determine the equivalent SDOF system's spectral acceleration, Sa, using an elastic response spectrum. The time period is evaluated by coefficient method using FEMA 356. The variation of time period for G+8 building is shown in Figure 4.15.

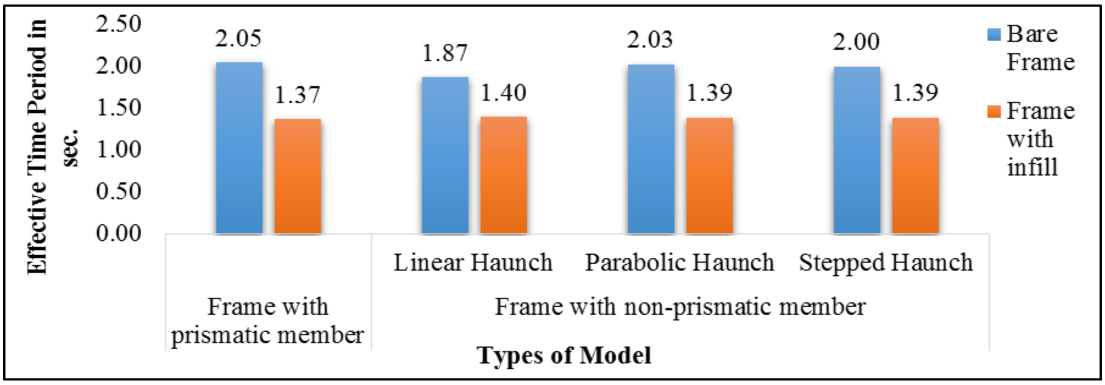

Figure 4.15: Variation of Effective Time Period in Sec. for Both Bare Frame and Frame with Infill for G+8 Building

4.5 Effective Stiffness (KN/m)

An effective stiffness is generated from the effective time period, Te, by a graphical procedure using an idealized force-deformation curve (i.e., pushover curve) relating base shear to roof displacement. The stiffness is evaluated by coefficient method using FEMA 356. The variation of stiffness for G+8 building is shown in Figure 4.16.

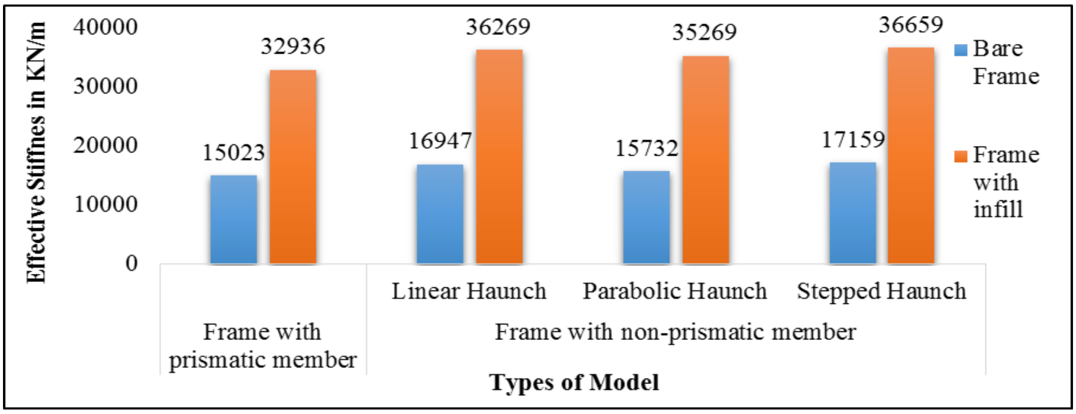

Figure 4.16: Variation of Effective Stiffness in KN/m for Both Bare Frame and Frame with Infill for G+8 Building

4.6 Behavior Factor

Behavior Factor is the ratio of the strength required to maintain the structure elastic to the inelastic design strength of the structure. The Behavior Factor, R, accounts for the inherent ductility and over strength of a structure and the difference in the level of stresses considered in its design. The behavior factor is evaluated by coefficient method using FEMA 356. The variation of behavior factor for G+8 building is shown in Figure 4.17.

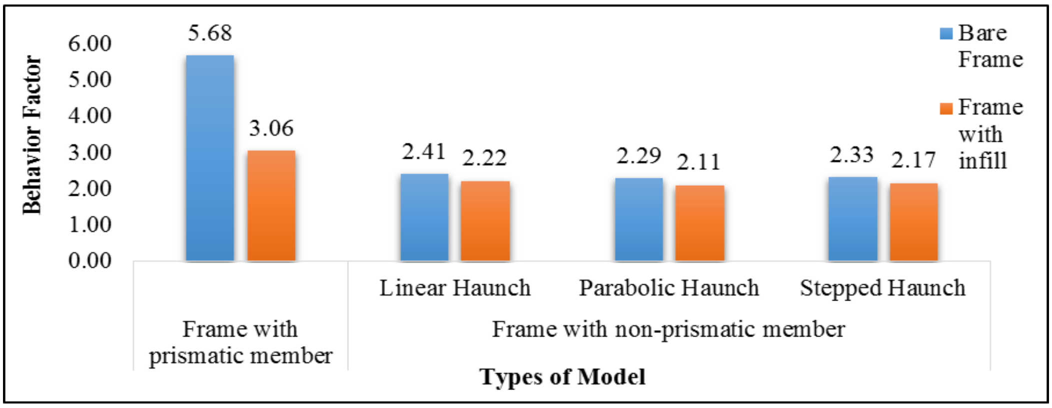

Figure 4.17: Variation of Behavior Factor for Both Bare Frame and Frame with Infill for G+8 Building

4.7 Performance Point

Performance point can be obtained by superimposing capacity spectrum and demand spectrum and the intersection point of these two curves is performance point. The capacity spectrum method by ATC 40 is used for the analysis. The variation of performance point for models with various height is shown in Tables 4.1 to 4.4.

Table 4.1 - Variation of Performance Point (X Direction) for Bare Frame

Height of building	Bare Frame			
	Frame with prismatic member	Frame with non-prismatic member		
		LH	PH	SH
G+2	1784.539, 0.087	3249.912, 0.084	3141.130, 0.084	3268.090, 0.083
G+4	2671.643, 0.141	3379.193, 0.132	3276.421, 0.138	3283.244, 0.137
G+6	2831.661, 0.183	3441.970, 0.179	3436.777, 0.180	3471.096, 0.175
G+8	2857.323, 0.240	3651.904, 0.221	3607.774, 0.244	3619.093, 0.240
G+10	2955.526, 0.284	3729.465, 0.205	3716.732, 0.269	3738.891, 0.264

Table 4.2 - Variation of Performance Point (X Direction) for Frame with Infill

Height of building	Frame with Infill			
	Frame with prismatic member	Frame with non-prismatic member		
		LH	PH	SH
G+2	4623.484, 0.062	5181.029, 0.061	5165.931, 0.061	5204.198, 0.060
G+4	4921.175, 0.101	5208.899, 0.093	5193.205, 0.099	5194.538, 0.098
G+6	4969.660, 0.132	5370.330, 0.129	5323.116, 0.130	5476.917, 0.129
G+8	5418.087, 0.165	5741.666, 0.174	5723.116, 0.173	5753.055, 0.173
G+10	5463.872, 0.206	5944.483, 0.199	5924.067, 0.200	5973.659, 0.198

Table 4.3 - Variation of Performance Point (Y Direction) for Bare Frame

Height of building	Bare Frame			
	Frame with prismatic member	Frame with non-prismatic member		
		LH	PH	SH
G+2	2707.685, 0.0004	3087.336, 0.0005	3081.464, 0.0005	3093.093, 0.0005
G+4	3015.447, 0.0007	3231.153, 0.0004	3226.159, 0.0004	3427.802, 0.0004
G+6	3245.416, 0.0004	3545.293, 0.0004	3425.473, 0.0004	3633.467, 0.0004
G+8	3714.820, 0.0001	3951.829, 0.0004	3902.435, 0.0004	4003.343, 0.0004
G+10	4196.640, 0.0005	4292.717, 0.0004	4277.516, 0.0004	4378.321, 0.0004

Table 4.4 - Variation of Performance Point (Y Direction) for Frame with Infill

Height of building	Frame with Infill			
	Frame with prismatic member	Frame with non-prismatic member		
		LH	PH	SH
G+2	6260.098, 0.00015	6751.468, 0.00013	6561.648, 0.00013	6957.028, 0.00013
G+4	6788.384, 0.00011	7038.806, 0.00009	6932.022, 0.00010	7139.045, 0.00010
G+6	7185.300, 0.00020	7206.429, 0.00010	7177.943, 0.00020	7390.525, 0.00020
G+8	7440.820, 0.00020	7516.904, 0.00030	7423.116, 0.00030	7548.206, 0.00030
G+10	7490.705, 0.00070	7620.195, 0.00060	7524.684, 0.00050	7659.932, 0.00050

V. Conclusions

In the present study, linear and non-linear analysis of reinforced concrete buildings is carried out with varying inertia for different storey height. Further, two cases are considered, one is bare frame analysis (without infill walls) and another one is frame with infill (considering infill walls). The buildings are analyzed for very severe earthquake load (seismic zone V). Comparison is made between various parameters as base shear, storey displacement, member forces, performance levels, patterns of hinge formation.

Based on the analysis results for all cases considered, following conclusions are drawn:

1) Frames with prismatic member have lesser base shear and higher storey displacement as compared to Frames with non-prismatic member as the stiffness of Frames with prismatic member is less than Frames with non-prismatic member.
2) Frames with parabolic haunch have lesser base shear and higher storey displacement as compared to Frames with linear haunch and Frames with stepped haunch as the stiffness of Frames with parabolic haunch is less than Frames with linear haunch.
3) Due to inclusion of non-prismatic members, behaviour and failure modes of buildings change. The results show the importance of considering varying inertia in modeling, to get the real scenario of damage.
4) Response Spectrum Method predicts lesser base shear and lesser storey displacement as compared to Seismic Coefficient Method.
5) Pushover analysis produces higher base shear and higher storey displacement as compared to Seismic Coefficient Method and Response Spectrum Method.
6) Due to absence of strength and stiffness effect of infill in bare frame analysis, it leads to under estimation of base shear as compared to infilled frame.
7) From pushover analysis results the weak links in the structure are identified and the performance level achieved by structure is known. This helps to find the retrofitting location to achieve the performance objective.
8) Frames with prismatic member have higher effective time period, lesser effective stiffness and higher behavior factor as compared to Frames with non-prismatic member.
9) Frames with parabolic haunch have higher effective time period, lesser effective stiffness and lesser behavior factor as compared to Frames with linear haunch and Frames with stepped haunch.
10) The performance point of frames with non-prismatic member is higher than that of frames with prismatic member for bare frames and frames with infill.
11) For bare frames as well as frames with infill analyzed by both PUSH in x direction and PUSH in y direction, the performance point of frames with parabolic haunch are lesser than frames with linear haunch. Whereas the performance point of frames with stepped haunch are higher than frames with linear haunch.

In the present study, variation of haunch dimensions is not considered. Therefore work can be repeated by changing haunch dimensions. Addition of shear wall especially for multistoried building can be done. Variation of storey height is not considered in the present work. Therefore work can be repeated by changing storey height. The study of varying inertia can be done by considering T- beam action. All the analysis can be done for different seismic parameters.

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