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Pushover and Time-History Analyses for Seismic Performance Based Analysis of Reinforced Concrete Frames

Dr.P.Asha
Professor and Head, Department of Civil Engineering, Jerusalem College of Engineering, Chennai - 600100

Due to the inelastic behavior intended in most structures subjected to infrequent earthquake loading, the use of nonlinear analyses is essential to capture behavior of structures under seismic effects. This paper presents nonlinear pushover and time-history analysis techniques for performance evaluation of 2D reinforced concrete frames subjected to earthquake loading. The performance of the reinforced concrete frame is evaluated in terms of maximum base shear, maximum displacement, ductility, performance point and sequence of plastic hinge formation. The results from pushover analysis are compared with that obtained from nonlinear time-history analysis.

Keywords: *Pushover analysis, nonlinear time-history analysis, performance point, plastic hinge*

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I. Introduction

While conventional limit-states design is typically a two-level design approach having concern for the service-operational and ultimate-strength limit states for a building, performance-based design can be viewed as a multi-level design approach that additionally has explicit concern for the performance of a building at intermediate limit states related to such issues as occupancy and life-safety standards. With the emergence of the performance-based approach to design, there is a need to develop corresponding analysis tools.

There is a hierarchy of structural analysis appropriate for performance based analysis of structures. Each higher level procedure provides a more accurate method of the actual performance of a building subjected to earthquake loads, but requires greater effort in terms of data preparation, time and computational efforts.

This paper explains the performance based evaluation of reinforced concrete frames by two advanced analysis techniques, Non-linear Static Procedure and Non-linear Dynamic Procedure. Pushover analysis is a simplified, static, nonlinear procedure in which a predefined pattern of earthquake loads is applied incrementally to frameworks until a collapse mechanism is reached. Nonlinear dynamic procedure is the time-history method of analysis through which both inelastic behaviour and earthquake induced actions changing with time can be accounted. It is a step by step analysis of the dynamical response of a structure to a specified loading that may vary with time.

The performance based analysis is based on quantifying the deformation of the members and the building as a whole, under the lateral forces of an earthquake of a certain level of seismic hazard. Existing codes are based on elastic analysis which has no measure of the deformation capability of members or of building. The performance based analysis gives the analyst more choice of 'performance' of the building as compared to the limit states of collapse and serviceability in a design based on limit state method. Performance-based methods require reasonable estimates of inelastic deformation or damage in structures which are better quantities to assess damage than stress or forces.

The objectives of the study were (i) To carry out performance based analysis of 2D RC frame by pushover analysis. (ii) To make a comparison of the parameters obtained from pushover analysis with that of time-history analysis. (ii) To study the effect of infill on seismic performance of frame.

II. Literature Review

Zameeruddin and Sangle (2021) attempts to address these primary concerns by evaluating the performance of reinforced concrete frame using nonlinear static procedures. For this, fifteen-moment resisting frames designed following the guidelines of Indian seismic codes were subjected to different lateral load patterns. The seismic performance is investigated in terms of fundamental periods, roof displacements, interstory drift ratio, base shear, and modification factor and was compared with various performance limits. The obtained results showed disagreement with Indian seismic code provisions, especially, towards the fundamental time period, upper and lower bound values of base shear drift ratio and modification factor.

Easa and Yan (2019) paper presents a critical review of PBA applications in three civil engineering fields: transportation, environmental, and structural engineering. The applications are grouped into a wide array of civil engineering areas, including highway transportation, pavement design and management, air transportation, water-structures design and operation, landfill design, building architectural design for evacuation, urban energy design, building earthquake-based design, building wind-based design, and bridge design and management. A total of 187 publications on PBA were reviewed and details on 122 application papers (from 23 countries/regions) are presented. The review consists of vertical and horizontal scans of PBA applications. In the vertical scan, the applications in each civil engineering area are summarized in tabular format that shows the system element modeled, analysis objective, performance criteria, analytical tool, and specifications/codes. The horizontal scan (discussion and lessons learned) addresses the following aspects of PBA: (1) the wide array of analytical tools used, (2) the broad functional and process-related areas, (3) the advantages, challenges, and opportunities, and (4) potential future applications. It is hoped that the state-of-the-art review presented in this paper will help researchers/practitioners quickly find useful information about PBA and promote its development in their respective fields.

Bari and Nirkhe (2019) presents study of two different configuration of G+5 story building with step back and step back set back configuration modeled in which slope angle is varied. Models are then analyzed for preliminary design by linear static analysis and Response spectrum analysis according to Indian seismic code and evaluated using Performance based Design approach by Nonlinear Static Pushover analysis. This study aims to create awareness about Performance based design approach a method other than conventional prescriptive codes.

Alashker et al (2015) used nonlinear pushover analysis to evaluate the seismic performance of three buildings with three different plans having same area and height. This method determines the base shear capacity of the building and performance level of each part of building under varying intensity of seismic force. The results of effects of different plan on seismic response of buildings have been presented in terms of displacement, base shear and plastic hinge pattern.

Wu and Wu (2013) designed a reinforced concrete frame model was by PKPM software and then performed a push-over analysis. Values of plastic hinges were calculated by Section Builder software, which based on constitutive relations of material and the section forms, then the data was written into corresponding components in the model to carry out push-over analysis. The plastic hinges first appeared at the ends of beams in the first story ,spread to the second story and the ends of columns in the first story. At last plastic hinges spread to the top story. The story drift and interstory displacement rotation of the model in different cases tended to decrease as structural height increase. It can be demonstrated that failure mechanism satisfies the design requirements of strong column weak beam.

Sadjadi et al (2007) et al discussed about Moment resisting frames (MRF) which are typically classified as “ductile”, “nominally ductile”, and “GLD” (Gravity Load Designed). The seismic performance of these structures can be evaluated in terms of its lateral load resistance, distribution of interstory drift, and the sequence of yielding of the members. In this study a typical 5-story frame is designed as (a) ductile, (b) nominally ductile, (c) GLD, and (d) retrofitted GLD. This study presents an analytical approach for seismic assessment of RC frames using nonlinear time history analysis and push-over analysis. The analytical models are validated against available experimental results and used in a study to evaluate the seismic behavior of these 5-story frames. It is concluded that both the ductile and the nominally ductile frames behaved very well under the considered earthquake, while the seismic performance of the GLD structure was not satisfactory. After the damaged GLD frame was retrofitted the seismic performance was improved.

Zou and Chan (2005) presented an effective computer-based technique that incorporates pushover analysis together with numerical optimization procedures to automate the pushover drift performance design of reinforced concrete (RC) buildings. Steel reinforcement, as compared with concrete materials, appears to be the more cost-effective material, that can be effectively used to control drift beyond the occurrence of first yielding and to provide the required ductility of RC building frameworks. In this study, steel reinforcement ratios are taken as design variables during the design optimization process. Using the principle of virtual work, the nonlinear inelastic seismic drift responses generated by the pushover analysis can be explicitly expressed in terms of element design variables. An optimality criteria technique is presented in this paper for solving the explicit performance-based seismic design optimization problem for RC buildings. Two building frame examples are presented to illustrate the effectiveness and practicality of the proposed optimal design method.

Kappos and Panagopoulos (2004) suggested the use of two alternative tools i.e. either time-history analysis for appropriately scaled input motions, or inelastic static (pushover) analysis, both for two different levels of earthquake loading depending on the building configuration,. The critical issues of defining appropriate input for inelastic dynamic analysis, setting up the analytical model that should account for post-yield behaviour of the plastic hinge zones, defining loading in two directions and target displacement for the pushover analysis, and detailing in a way consistent with the deformations derived from the advanced analysis, are discussed. The proposed method is then applied to a regular multistorey reinforced concrete 3D frame building and is found to lead to better seismic performance than the standard code (Eurocode 8) procedure, and in addition leads to a more economic design of transverse reinforcement in the members that develop very little inelastic behaviour even for very strong earthquakes.

Kunnath and Kalkan (2004) investigate the correlation between demand estimates for various lateral load patterns used in non-linear static analysis. It also examines the rationale for using component demands over story and system demands. Results reported in the paper are based on a comprehensive set of pushover and non-linear time-history analyses carried out on eight- and twelve-story steel and concrete moment frames. Findings from this study point to inconsistencies in the demands predicted by different lateral load patterns when using pushover analysis and also highlight some issues in the current understanding of local demand estimates using FEMA-based procedures.

Hasan et al (2002) presents a simple computer-based push-over analysis technique for performance-based design of building frameworks subject to earthquake loading. The technique is based on the conventional displacement method of elastic analysis. Through the use of a plasticity-factor that measures the degree of plastification, the standard elastic and geometric stiffness matrices for frame elements (beams, columns, etc.) are progressively modified to account for nonlinear elastic–plastic behavior under constant gravity loads and incrementally increasing lateral loads. The behavior model accounts for material inelasticity due to both single and combined stress states, and provides the ability to monitor the progressive plastification of frame elements and structural systems under increasing intensity of earthquake ground motion. The proposed analysis technique is illustrated for two building framework examples. Whittaker et al stated blast, earthquakes, fire and hurricanes are extreme events for buildings and infrastructure and warrant innovative structural engineering solutions. The state-of-the-practice and new developments in performance-based earthquake engineering (PBEE) are discussed, with emphasis on hazard intensity measures, engineering demand parameters, and performance levels. The new performance-based earthquake engineering methodology is extended to performance-based blast engineering. Sample intensity measures, engineering demand parameters, and performance levels are proposed for blast engineering. Some similarities and differences between performance approaches for blast and earthquake engineering are identified.

Memari et al (2001) presented the results of seismic damage evaluation of a tall reinforced concrete building. Plastic hinge formation patterns obtained by using DRAIN-2D and IDARC computer programs for dynamic analysis are compared. Damage indices given by IDARC are interpreted and their implications compared with those of drift ratios. Results of static push-over analysis are compared with those of inelastic dynamic time history analysis. Moreover, the result of collapse mechanism approach is compared with that of static push-over analysis. It is shown that simple collapse mechanism approach can predict the failure mode given by static push-over analysis for this building. It is concluded that drift limits in codes do not necessarily predict the degree of damage that this type of construction can sustain in severe earthquakes.

Kappos and Manafpour (2001) presented a seismic design procedure considering performance criteria for two distinct limit states, involving analysis of a feasible partial inelastic model of the structure using currently available powerful tools. The procedure is developed in a format appropriate for incorporation into modern design codes, such as the Eurocode 8, and two alternatives are explored, one involving time–history analysis for appropriately scaled input motions, and a simpler one involving inelastic static (pushover) analysis. The proposed method is found to lead to better seismic performance than the standard code procedure, at least in the case of regular multistorey reinforced concrete frame structures studied herein, and in addition leads to a more economic design of transverse reinforcement in the members that develop very little inelastic behaviour even for very strong earthquakes.

III. Methodology

3.1 Tasks in Performance Based Analysis

Performance-based seismic analysis requires that the engineer should complete the tasks indicated in the flowchart shown in Figure 1.

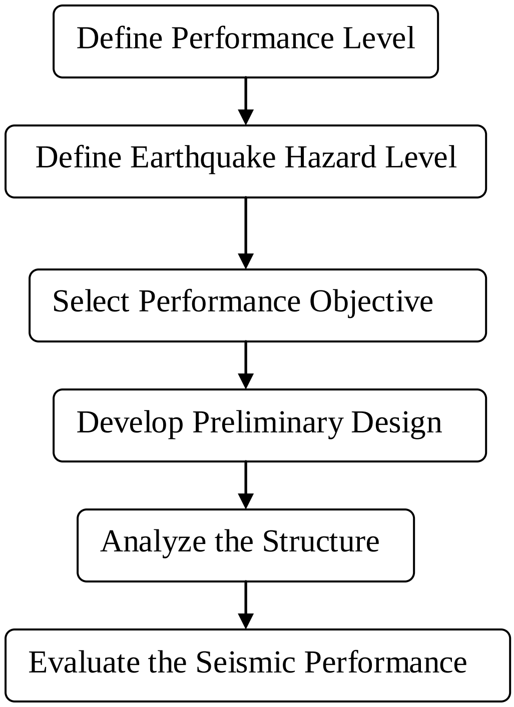


Figure 1. Performance-based Analysis Procedure

The following sections summarize recommendations for performance level, earthquake hazard level and performance objective within the context of performance-based analysis. Since two types of nonlinear analyses methods i.e. nonlinear pushover and nonlinear time-history analyses.

The desired condition of the structure after a range of ground shakings, or building performance level, is decided by structural engineer. The building performance level is a function of the post event conditions of the structural and non-structural components of the structure. The performance levels as per FEMA 356 are as follows: (i) Immediate Occupancy (ii) Life Safety (iii) Collapse Prevention

3.2 Seismic Hazard Levels

In the performance based analysis, seismic hazard level (or earthquake hazard level or simply, earthquake level) refers to the level of ground motion. The earthquake level can be described by two types of methods, deterministic method and describing the earthquake level is the probabilistic method.

3.3. Performance Objectives

A performance objective is the pairing of a building performance level and a seismic hazard level. If the objective includes two building performance levels under two earthquake levels, then it is a dual level performance objective. Similarly there can be multiple level performance objectives. A basic safety objective (BSO) satisfies the dual requirement of Life Safety under DBE and Collapse Prevention under MCE (combinations k+p in below table.2.). The aim of BSO is to have a low risk of life threatening injury during a moderate earthquake (as defined by DBE) and to check the collapse of the vertical load resisting system during a severe earthquake (as defined by MCE)

3.4 Calculation of Target Displacement

The target displacement i.e. the maximum displacement the structure is expected to undergo during a design event is now calculated. The target displacement is calculated as per the following equation of FEMA 356.

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^{2}}{4\pi^{2}} g \qquad \text{----- (1)}$$

C_0 is Modification factor to relate spectral displacement of an equivalent SDOF system to the roof displacement of the building MDOF system. C_1 is a modification factor to relate expected maximum inelastic displacement to displacement calculated for linear elastic response. C_2 is modification factor to represent the effect of pinched hysteretic shape, stiffness degradation and strength deterioration on maximum displacement response . C_3 is modification factor to represent increased displacement due to dynamic P-Δ effect. For buildings with positive post-yield stiffness, C_3 shall be set equal to 1.0. For buildings with negative post-yield stiffness, values of C_3 shall be calculated using equation. α = Ratio of post-yield stiffness to effective elastic stiffness, where the nonlinear force-displacement relation shall be characterized by a bilinear relation.

3.5 Pushover Analysis

Pushover analysis is a technique by which a computer model of the building is subjected to a lateral load of a certain shape (i.e., parabolic, inverted triangular or uniform). In such analysis, a monotonic steadily increasing lateral load is applied to the structure, in the presence of the full gravity dead load, until a predetermined level of roof displacement is approached. The magnitude of lateral loads at floor levels do not affect the response of the structure in displacement-controlled pushover analysis, but the ratio in which they are applied at each floor level alters the response of the structures.

Pushover analysis is an efficient way to analyze the behavior of the structure, highlighting the sequence of member cracking and yielding as the base shear value increases. This information then can be used for the evaluation of the performance of the structure and the locations with inelastic deformation. The primary benefit of pushover analysis is to obtain a measure of over strength and to obtain a sense of the general capacity of the structure to sustain inelastic deformation.

The loads acting on the structure are contributed from slabs, beams, columns, walls, ceilings and finishes. They are calculated by conventional methods according to IS 456 – 2000 and are applied as gravity loads along with live loads as per IS 875 (Part II) in the structural model. The lateral loads and their vertical distribution on each floor level are determined as per IS 1893 – 2002 and then they are applied in “PUSH - Analysis case” during the analysis.

3.6 Time History Analysis

Time-history analysis is a step by step analysis of the dynamical response of a structure to a specified loading that may vary with time. A time history function may be a list of time and function values or just a list of function values that are assumed to occur at equally spaced intervals. The validity of pushover procedure is examined using the results of non-linear time-history analyses as a benchmark. The peak displacements obtained in time-history analysis do not correspond to the ultimate displacement capacity on the push over curve. To facilitate the comparison with pushover analyses, the ground motions are scaled in such a manner so that the resulting peak roof displacement is equal to the target displacement computed for each building.

In this paper, Northridge earthquake motion record of 0.344g Peak Ground Acceleration was selected from ATC40 for analysis. Here loading is dynamic and the frame moves laterally in both directions. Hence struts are required in both directions

IV. Modelling of Structure

4.1 Modeling of Slabs

Conventionally slabs are not modeled. Instead its load contribution is transferred to the adjacent beams as equivalent trapezoidal and triangular loads. But its in plane stiffness contribution is very large. So its effect should be modeled, especially when lateral analysis is carried out. It is assumed that slabs are rigid in its plane. This in plane rigidity of slab is modelled by assigning rigid diaphragm behaviour in that plane by connecting all the column beam joints in that floor. In SAP, this option is available as diaphragm constraint.

4.2. Modeling of Beams and Columns

The building considered for analysis is a typical 6- storey RC frame designed only for gravity loads as per IS 456 – 2000. The seismic performance of the frame is evaluated in terms of interstorey drift ratio, ductility, maximum base shear, roof displacement and plastic hinge formation. Material properties are assumed to be 25MPa for the concrete compressive strength and 415MPa for the yield strength of longitudinal and shear reinforcement. The labels of beam and column along with the frame dimensions are shown in Figure 2. The beam in all storey levels is of size 300mm x 600mm with tension and compression reinforcements of $3885mm^2$ and $2412mm^2$ respectively. The column dimensions and area of longitudinal reinforcement (A_{col}) details are presented in Table 1.

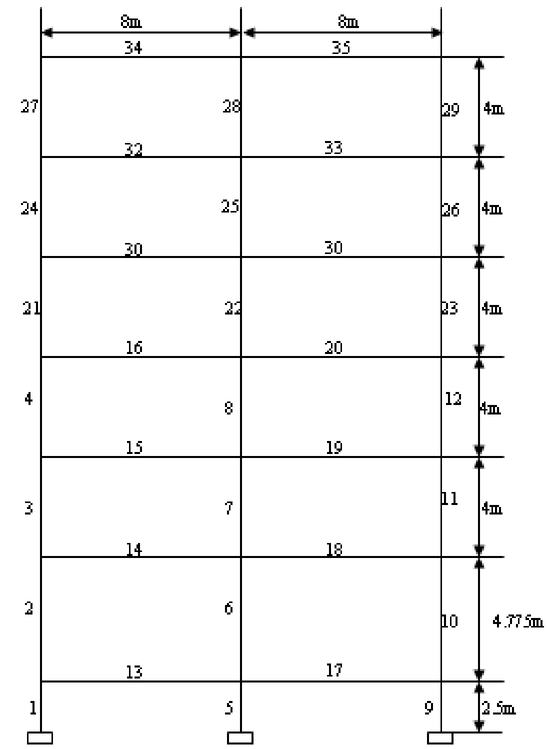


Figure 2 6-storey Frame with Dimensions

Table 1 Column Dimensions and Area of Longitudinal Reinforcement

Column Label	Cross Section mm x mm	A_{col} (mm^2)
1 & 9	300 x 500	5892
2 & 10	300 x 500	4020
3 & 11	300 x 400	3216
4 & 12	300 x 300	3080
21& 23	300 x 300	1232
24& 26	300 x 300	905
27& 29	300 x 300	905
5	650 x 650	14784
6	600 x 600	12744
7	550 x 550	10620
8	500 x 500	7856
22	450 x 450	6372
25	300 x 300	4928
28	300 x 300	804

A_{col} = Area of longitudinal reinforcement in column

The validity of pushover procedures based on the load distributions is examined using the results of non-linear time-history analyses as a benchmark. To facilitate the comparison with pushover analyses, the ground motions are scaled in such a manner so that the resulting peak roof displacement is equal to the target displacement computed for each building. A conventional technique is to scale ground motions such that the spectral acceleration at the fundamental period matches a given design spectrum.

4.3. Modeling of Infill Walls

Infill walls are non-structural elements in a building. The walls are built after the construction of frames. No gap is expected between a wall and the bounding columns. Figure 3 shows a typical panel of an infilled frame. Any gap between the top of the wall and the soffit of the beam above is expected to be packed with mortar. The dead loads of the slabs are carried by the beams to the supporting columns and hence, it is not transferred to the wall. The live load also will not cause substantial deflection of the beam for the load to be transferred to the wall. Hence the infill wall is not considered to be gravity load bearing and is not designed But infill walls are modeled to incorporate its stiffness contribution in the lateral direction. Finite element modeling of infill although rigorous is time consuming to develop. Hence an approximate method based on equivalent strut is adopted.

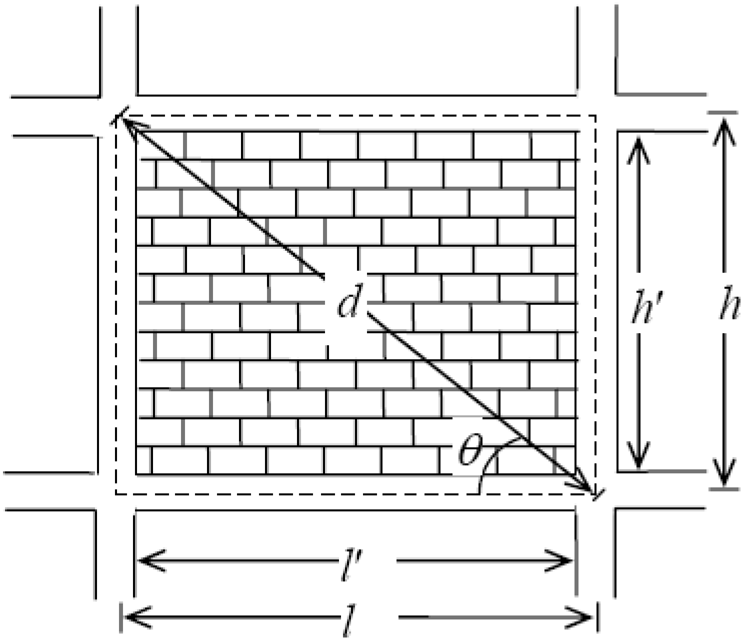


Figure 3 A typical panel of an infilled frame

Strut is a compression member similar to frame element. It will carry only axial compressive forces. Hence both the ends of the strut are assigned pin connection or it can be modeled as truss member.

4.4. Properties of Strut

The properties of infill that should be assigned to equivalent strut while modeling it for linear analysis are modulus of elasticity (equation 2), cross-sectional dimension of the equivalent strut (equation 3) and the diagonal length of infill panel.

$$E_m = k f'_m \qquad (2)$$

Here

E_m = modulus of elasticity of infill material

f'_m = compressive strength of infill

k = 550 (IS:1905)

Width of strut is calculated based on equation proposed by Holmes.

$$w = d/3 \qquad (3)$$

Here

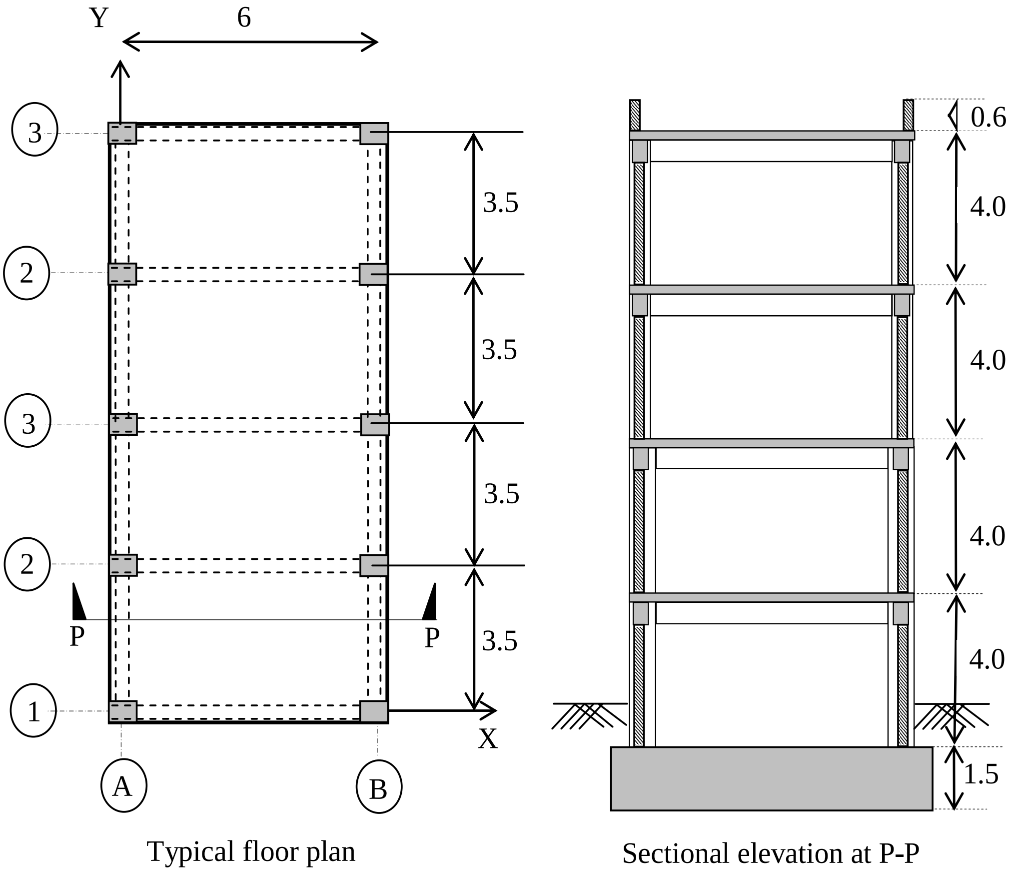
d = diagonal length of infill

Thickness of strut is equal to the thickness of infill. In the case of pushover analysis, the structure is pushed laterally in one direction only. Hence compressive force will be developed in infill between one set of opposite corners only. So strut is modeled along that direction with above calculated properties.

In the conventional seismic analysis of framed structures, stiffness contribution due to infill walls is not considered. The presence of infill increases the demand and capacity of the structure. Even though we are considering the increase in demand due to infill, we are neglecting the increase in capacity due to infill. Thus we are under estimating the actual lateral strength of the structural system. Hence modeling of infill wall is necessary. This paper studies the behaviour of 2D frames with and without infill action under lateral loads using pushover and time history analysis.

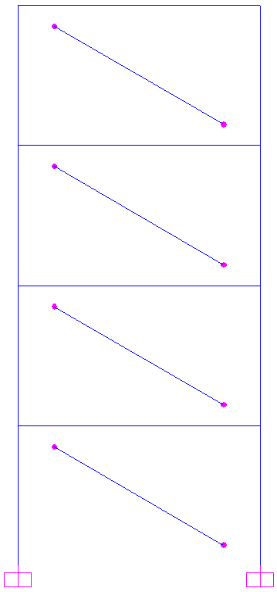
To study the difference in behaviour of structures with and without infill action, a single bay 2D frame (fame 2-2) is selected from all the building described above. The plan view and sectional elevation of a G+3 building is shown in Figure 3. The X and Y direction were selected along the width and length of the building respectively.

Figure 4(a) shows a G+3 storey 2D frame with infill modelled as strut for pushover analysis. In the case of non-linear time-history analysis, the structure will be laterally pushed in both directions alternatively. Hence in infill compressive forces will be developed along both diagonals alternatively. So strut has to be modeled in both directions. The depth of strut can be reduced to half in both directions. Figure 4(b) shows a G+3 2D frame with infill modelled as strut for non-linear time history analysis.

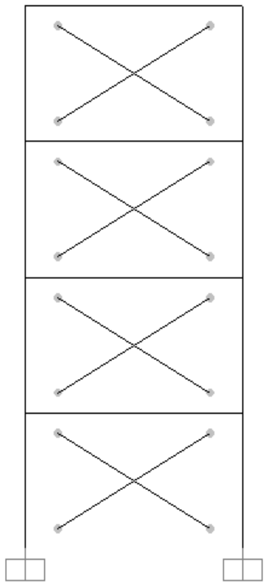


(All dimensions in meters)

Figure 3 Typical Floor Plan and Sectional Elevation of The Building



(a) Infill Model for Pushover Analysis



(b) Infill Model for Timehistory Analysis

Figure 4 Infill Models for Analyses

4.5. Modeling of Hinges

Beam and column elements are modeled as nonlinear frame elements with lumped plasticity by defining plastic hinges at both ends of the beams and columns. The location of hinges in beams (L_{beam}) and columns (L_{col}) are obtained from following equations.

$$L_{beam} = \frac{D_{col}}{2} - \frac{L_{pb}}{2} \quad \text{------ (4)}$$

$$L_{col} = D_{beam} - \frac{L_{pc}}{2} \quad \text{----- (5)}$$

$$L_{pb} = 0.5\, D_{Beam} \quad \text{----- (6)}$$

$$L_{pc} = 0.5\, D_{Col} \quad \text{----- (7)}$$

Where D_{col} = Column depth
D_{beam} = Beam Depth
L_{pb} = Length of plastic hinge in beam
L_{pc} = Length of plastic hinge in column

SAP2000 implements the plastic hinge properties described in FEMA-356 (or ATC-40). The values assigned to each of these points vary depending on the type of element, material properties, longitudinal and transverse steel content, and the axial load level on the element. SAP2000 provides default-hinge properties and recommends PMM hinges for columns and M3 hinges for beams. Once the structure is modeled with section properties, steel content and the loads on it, default hinges are assigned to the elements (PMM for columns and M3 for beams).

V. Results and Discussion

The pushover analysis consists of the application of gravity loads and a representative lateral load pattern. The frame was subjected to gravity loads and simultaneous lateral loading. Gravity loads were in place during lateral loading. Lateral forces calculated according to IS 1893 – 2002 were applied monotonically in a step-by-step nonlinear static analysis. P-Delta effect was not taken into consideration. In pushover analysis, the behavior of the structure is characterized by a capacity curve that represents the relationship between the base shear force and the displacement of the roof. This is a very convenient representation in practice, and can be visualized easily by the engineer.

5.1 Base Shear

The pushover curve is shown in Figure 5. It is observed that maximum base shear was 571kN which is about 10% of seismic weight of frame and the maximum displacement corresponding to this base shear is 1.02m. The displacement ductility of frame is 2.3. The frame is pushed to a maximum displacement of 4% of its height. The base shear obtained at DBE and MCE levels from push over analysis were 116kN and 171kN respectively. The corresponding values obtained from nonlinear time-history analysis were 151kN and 251kN respectively. The results from time-history analysis were 23% and 32% higher than that of the pushover analysis results.

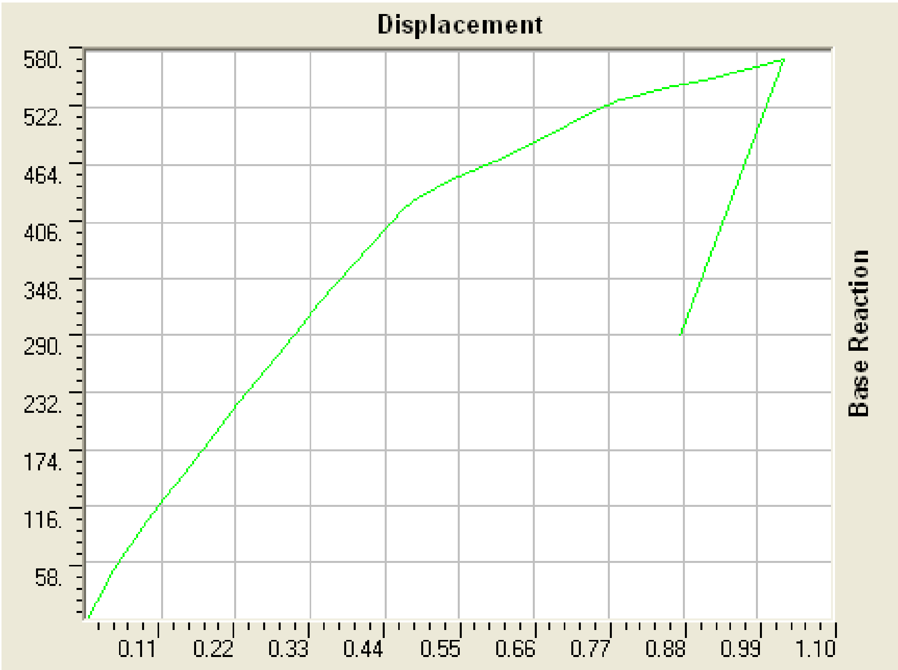


Figure 5 Pushover Curve of 6 Storey Frame

5.2 Performance Point

The performance point of frame is obtained from the intersection of capacity and demand spectra from SAP analysis. The performance is assessed for two levels of performance objectives, Life Safety (LS) under design basis earthquake (DBE) and Collapse Prevention (CP) under maximum considered earthquake (MCE). The capacity vs. demand spectrum for the frame under DBE and MCE is shown in Figures 6 and 7 respectively. The base shear, roof displacement, spectral acceleration, spectral displacement, effective time period and effective damping corresponding to the performance point is shown in same figures. The displacement at performance point at DBE level is 0.123m (Figure 6) and it is greater than target displacement given by FEMA 356 for life safety which is 0.119m. The displacement at performance point at MCE level is 0.171m (Figure 7) and is lesser than corresponding target displacement as per FEMA 356 which is 0.177m.

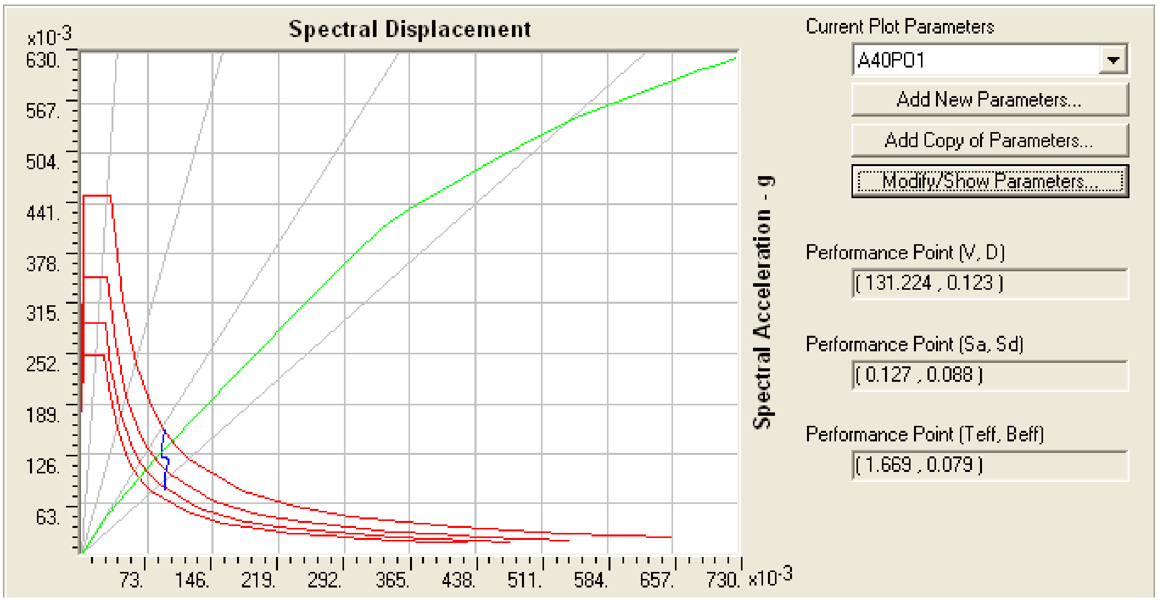


Figure 6 Demand Vs Capacity Spectrum for Design Basis Earthquake

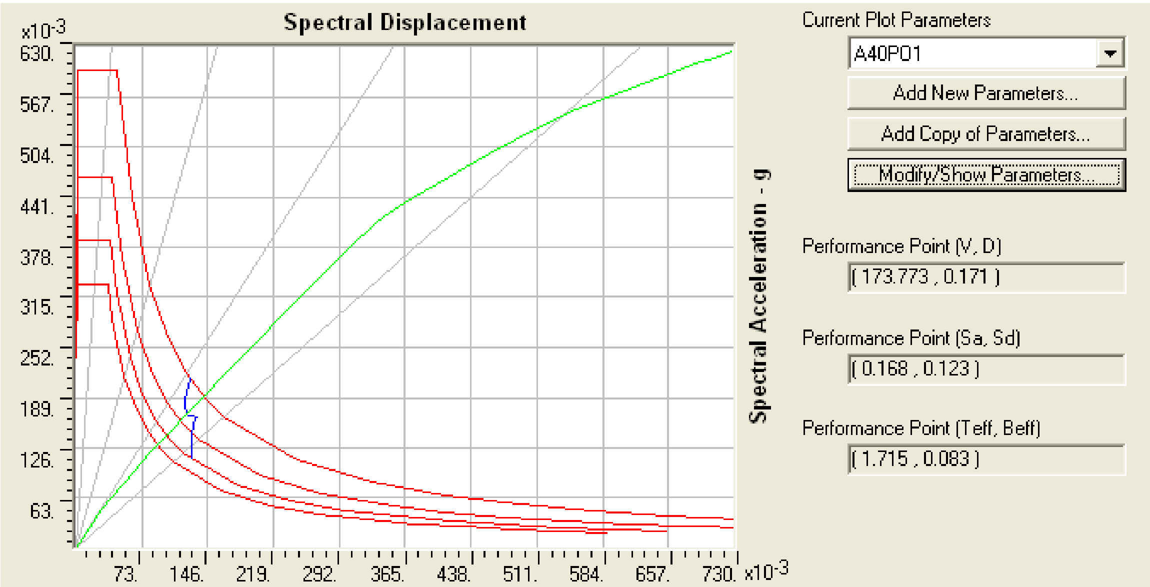


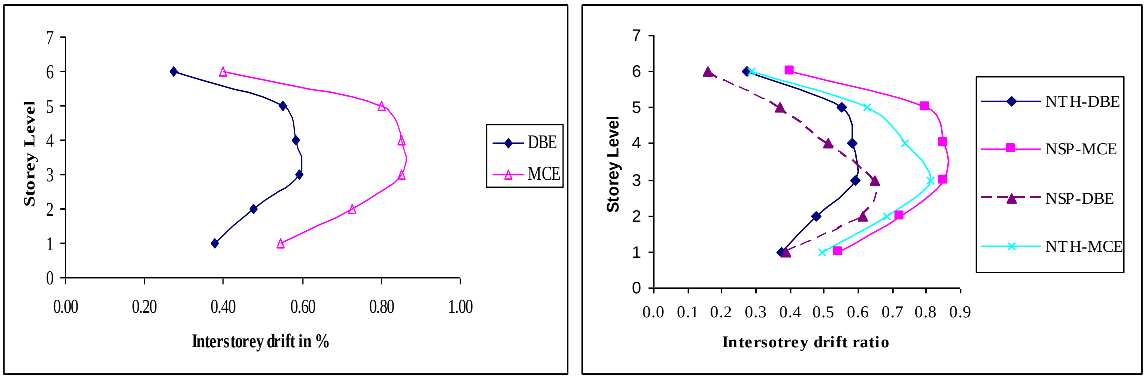
Figure 10 Demand Vs Capacity Spectrum for Maximum Considered Earthquake

7.3 Interstorey Drift

The interstorey drift has long been recognized as an important indicator of building performance. Interstorey drift is defined as the ratio of relative horizontal displacement of two adjacent floors and corresponding storey height. Interstorey drift ratio from pushover analysis at DBE and MCE levels is presented in Figure 8(a). It is observed that 3rd storey level experienced the largest interstorey drift values of 0.58% and 0.85% at both DBE and MCE levels. It is seen that the interstorey drift ratio increased with increase in storey level up to first 4 stories and thereafter showed a reverse trend at both levels of earthquake.

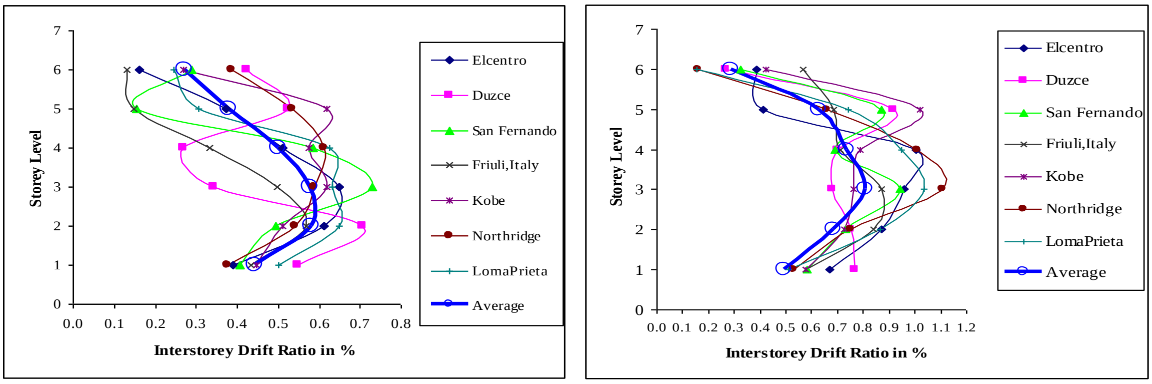
The interstorey drift ratio from pushover analysis is compared with that of time-history analysis as shown in Figure 8(b). At DBE level, pushover analysis over-estimated the interstorey drift ratio at lower storey levels and underestimates the same at upper storey levels. At MCE level, pushover analysis over-estimated the interstorey drift ratio at almost all storey levels.

The interstorey drift ratios from time-history analyses for the seven earthquake ground motions at DBE and MCE levels are shown in Figures 9(a) and 9(b) respectively.The average interstorey drift ratio is also shown in same figures which were compared with the interstorey drift ratio from pushover analysis.



(a) Results from Pushover Analysis at DBE & MCE Levels **(b) Comparison between Pushover & Time-history Results at DBE & MCE Levels**

Figure 8 Interstorey Drift Ratios



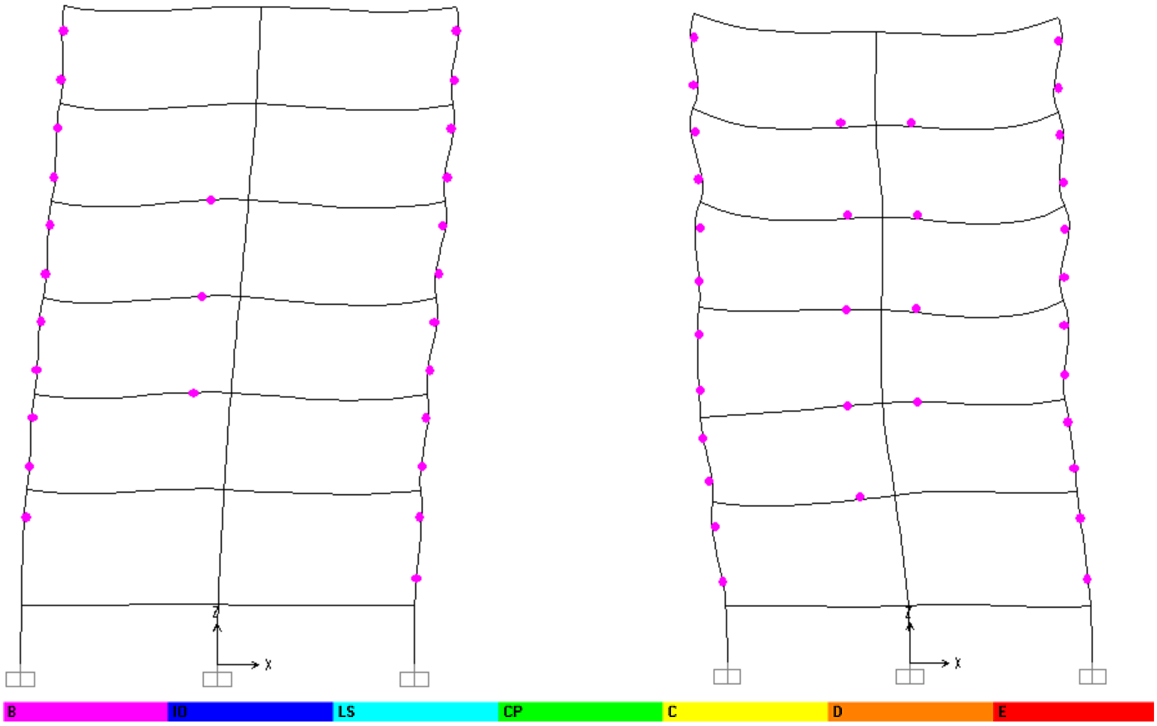
(a) Results from Time-history Analysis at DBE Level **(b) Results from Time-history Analysis at MCE Level**

Figure 9 Interstorey Drift Ratios from Time – history Analysis

7.4 Plastic Hinge Patterns

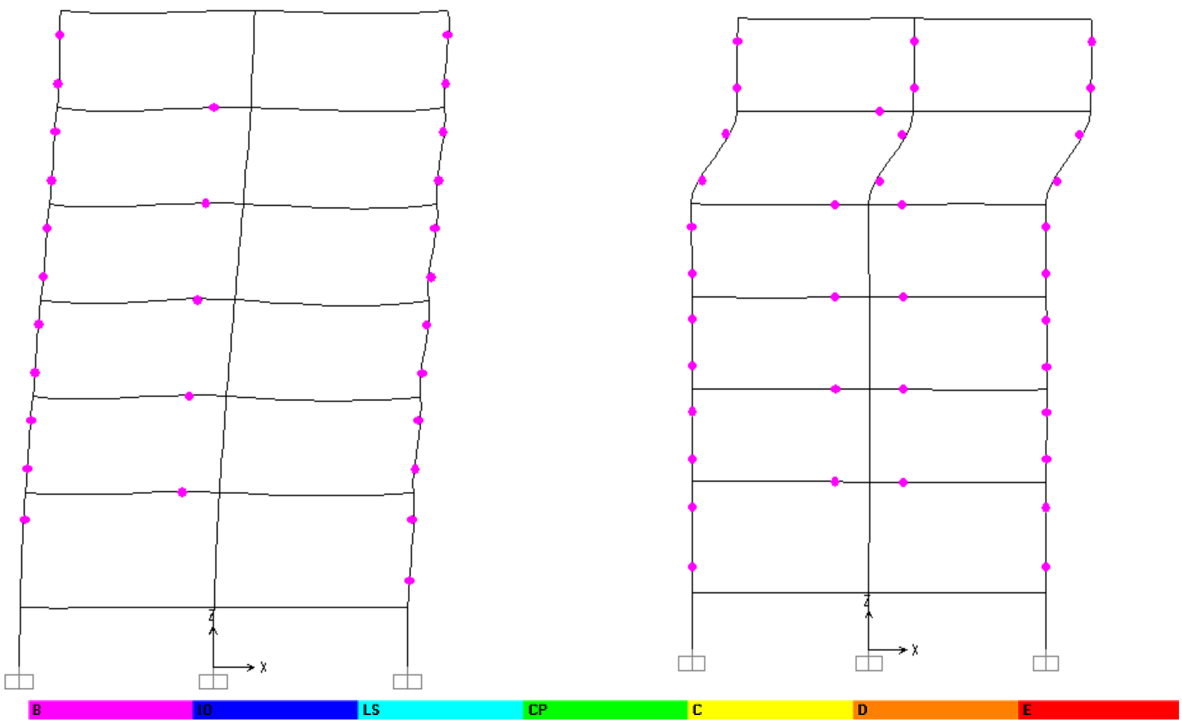
The plastic hinge patterns of frame at DBE and MCE levels from pushover analysis are shown in Figures 10 and 11. In both the analyses, it is observed that more number of columns underwent yielding than beams at the displacement levels corresponding to DBE and MCE levels. It is also seen that more number of beam ends showed hinges at yielding level in model of time-history analysis than the model from pushover analysis at both DBE and MCE levels. Comparison of plastic hinging pattern at MCE level indicates that middle columns in 5^{th} and 6^{th} stories yielded in the model from time-history analysis whereas there was no hinge formation in the middle columns in the model from pushover analysis.

The plastic hinge pattern from pushover analysis at last step i.e. when roof of frame is pushed to 4% of total height is shown in Figure 12. Plastic hinge formation started with yielding of outer columns at all stories with yielding of few beam ends in upper stories. Then middle columns at upper stories start to yield with simultaneous yielding of base columns. Although the beams experienced less number of hinges than columns, they were all at significant damage or failure stage. All the hinges in columns were only at the yielding stage. Thus the model with default hinge properties shows significant damage in beams, though such mechanism is not guaranteed for structures designed only for gravity loads as per IS 456-2000.



(a) Pushover Analysis **(a) Time – history Analysis**

Figure 10 Plastic Hinge Pattern at DBE Level



(a) Pushover Analysis **(b) Time - history Analysis**

Figure 11 Plastic Hinge Pattern at MCE Level

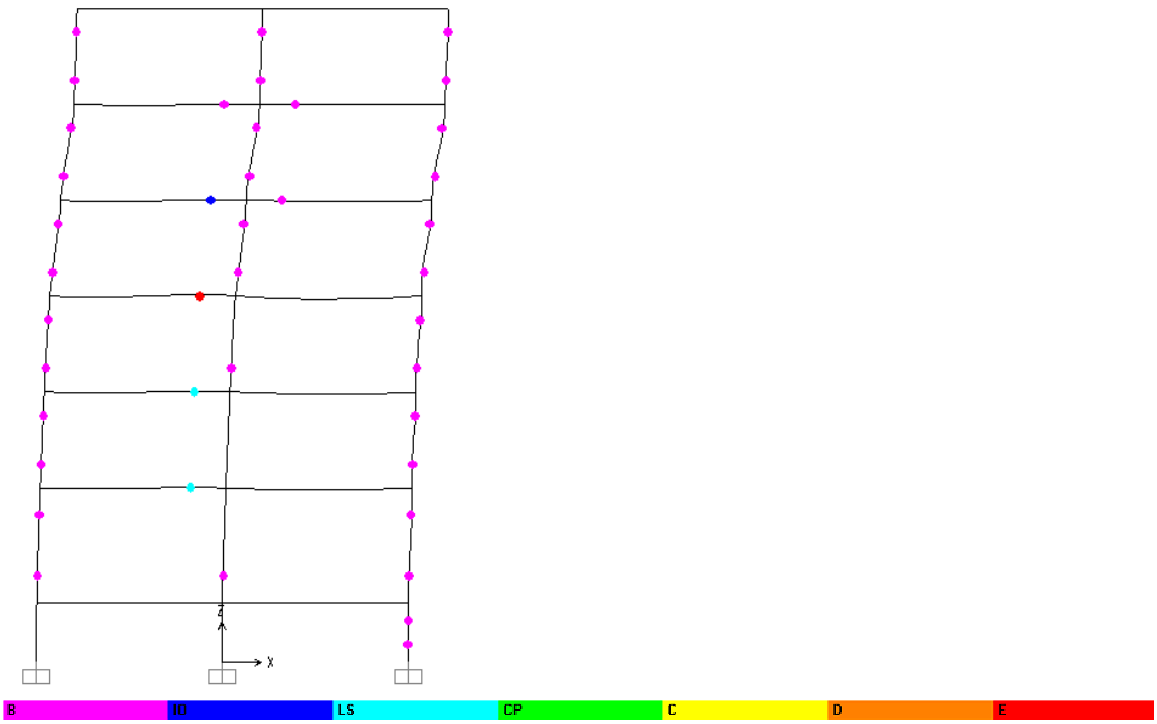


Figure 12 Plastic Hinge Pattern at Last Step from Pushover Analysis

7.5 Effect of Infill

Analysis results shows that, hinges will be formed earlier in frames of structures without strut action than frames of structures with strut action. This is due to the additional stiffness offered by the strut in the lateral direction. From the pushover analysis, it is found that performance point parameters such as roof displacement and base shear get reduced due to strut action. It is observed that roof displacement get considerably (50%) reduced with strut action (50mm for framwe without strut and 101.2mm for frame with strut).

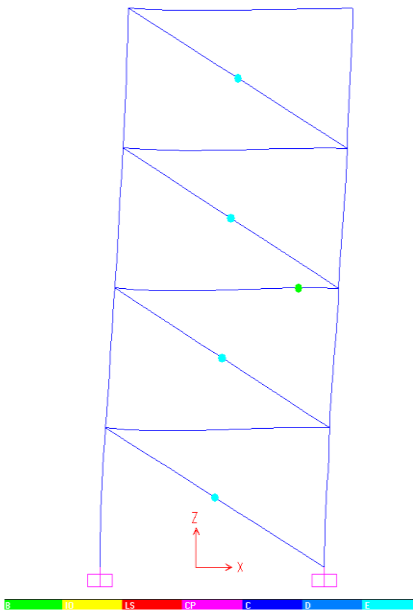


Figure 13 2D model showing typical hinge formation in infill of a G+3 storey frame, using pushover analysis.

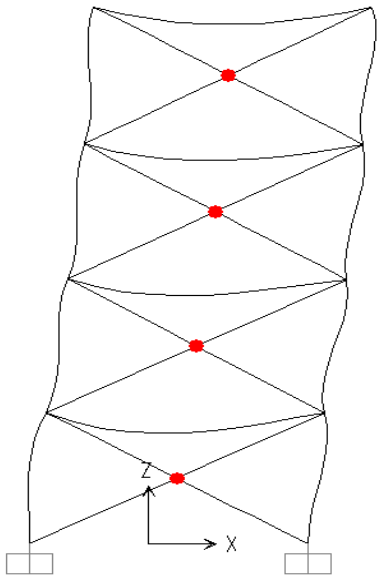


Figure 14 2D model showing typical hinge formation in infill of a G+3 storey frame using time history analysis.

VI. Conclusion

A plane RC frame with 6 stories designed only for gravity loads as per IS 456-2000 was considered and nonlinear static pushover and time-history analyses were carried out to evaluate seismic performance of the frame. Beam and column elements were modeled as nonlinear frame elements with lumped plasticity by defining plastic hinges at both ends of the beams and columns. The frame was modeled with default hinge properties. The following findings were observed:

1. The time-history analysis gave 23% and 32% higher values of base shear at DBE and MCE levels than pushover analysis.
2. The roof displacement of frame at DBE and MCE levels indicates that the frame satisfies the requirement for Life Safety performance at DBE level whereas it does not satisfy the requirement for Collapse Prevention performance at MCE level. The satisfactory performance at DBE level may be attributed to the default hinge properties assigned to structural members; an observation consistent with that noticed by others.
3. From pushover and time-history analyses, it is seen that 3rd storey experienced the maximum interstorey drift ratio at both DBE and MCE levels. At MCE level, pushover analysis over-estimated the interstorey drift when compared to time-history analysis.
4. There is no significant difference in the plastic hinge pattern for the frame at DBE and MCE levels from both the analyses; but time-history analysis gave more number of beam hinges than pushover analysis at both levels. At MCE level, the middle columns in the upper stories of time-history model showed yielding which was not observed in the model of pushover analysis.
5. The behaviour of frame was as expected for one designed only for gravity loads showing column side sway mechanism.

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