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Pushover Analysis of Post-tensioned Flat Slab Building Using Equivalent Frame Method

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Abstract: This study presents the seismic capacity of a G+7 office building having post-tensioned flat slab using ETABS. It is possible to estimate the potential of future earthquake losses by predicting the performance of a building. The equivalent frame method for post-tensioned flat slab is used in this parametric study considering IS1893(Part-I):2016, SP 24 and ACI318-17. Linear Static analysis and Response Spectrum analysis are carried out as per Indian code followed by the Pushover analysis. ETABS measures the performance of non-linear hinges. As per ASCE41-13 NSP, it is determined that the base shear values are nearly double and target displacements are half the values of models with URM infill walls compared to without URM infill walls. IO level of M3 hinges is found in the slab-beam. IO, LS and CP levels of P-M2-M3 hinges are found in the columns.

Keywords: Pushover Analysis, Non-linear Static Procedure, Equivalent Frame Method, Equivalent Diagonal Strut Method, Post-tensioned Flat Slab, Office Building, Non-linear hinge, ETABS

I. INTRODUCTION

The Non-linear analysis is used to determine the seismic capacity evaluation of existing and new buildings. A nonlinear analysis of a building should be carried out as Static or Dynamic. Pushover analysis can be defined as a Non-linear Static analysis by which a computer model of the building is subjected to a gravity loading and a monotonic displacement-controlled lateral loading in a specific shape of load pattern. The nonlinear analysis under static condition consumes less time compared to dynamic condition. Non-linear Static Procedure converts a multi degree of freedom (MDOF) system to a single degree of freedom (SDOF) system to determine capacity curves typically in the form of Base Shear versus the Displacement at the roof. Building is pushed in the same direction of lateral load applied, as shown in Fig. 1. The intensity of the lateral load is continuously increased step by step through the elastic and inelastic behaviour and damage state of each member is recorded until the member is collapsed or failed. Material nonlinearity is modelled by assigning the non-linear hinge at potential location. 'Performance Level' of the member, with the help of hinges, predicts the building's service condition after the seismic hazard in terms of damage state that will be expected. ASCE41 provides the modelling parameters and acceptance criteria for estimation of hinge rotation values in steel and reinforced concrete members. It is presumed that the target displacement could be the actual displacement obtained if the MDOF system was subjected to non-linear analysis.

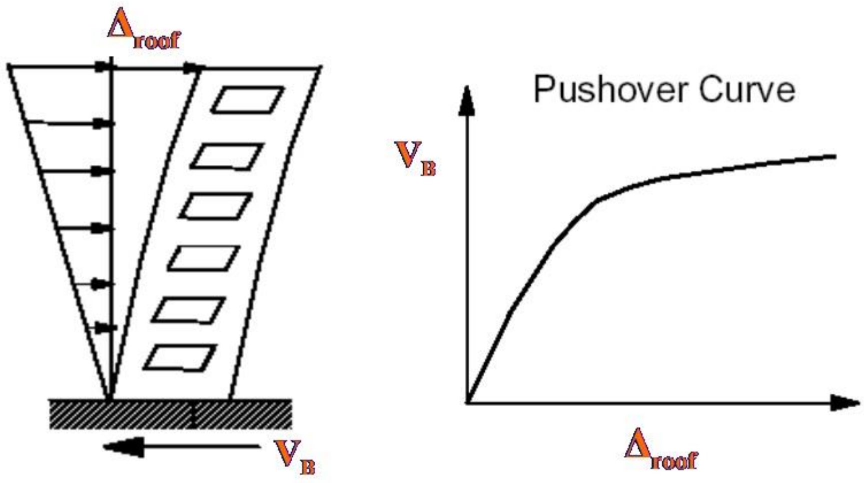

Fig. 1 Schematic Representation of Pushover Analysis

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There are structural and non-structural performance levels of a whole building. The followings are the types of building performance levels:

- Immediate Occupancy (IO): Limited damage with the basic resisting capacities but may not be functional, as shown in Fig. 2.

Fig. 2 IO level in an actual building

- Life Safety (LS): Considerable damage with the partial or no collapse, as shown in Fig. 3. Injuries may occur with the risk of life-threatening injury being low. Repair or retrofit cost may not be economically feasible.

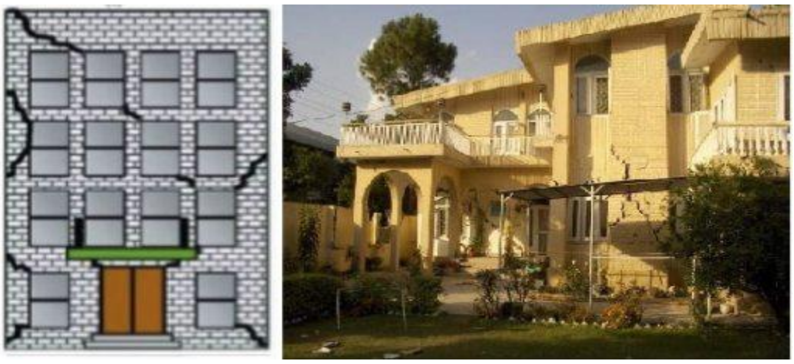

Fig. 3 LS level in an actual building

- Collapse Prevention (CP): The building damage is severe and about to fully collapse. The building suffers Large permanent drifts. Non-structural components damage extensively. The structural components retain little residual stiffness and strength.

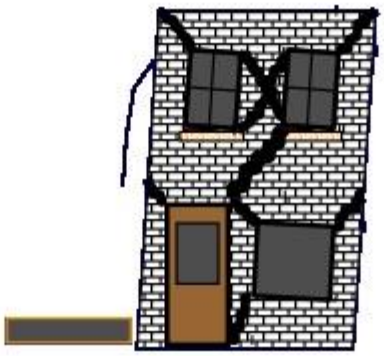

Fig. 4 Representation of a CP level

All the structural and non-structural components in a building must have expected deformation capacities not less than the maximum deformation demands obtained at the target displacement for the selected performance levels. 'Point of Inelastic' location in the member is called as 'Non-linear Plastic Hinge'. In this state structural member starts losing strength to come back in original state. The overall building performance levels are, as shown in Fig. 5.

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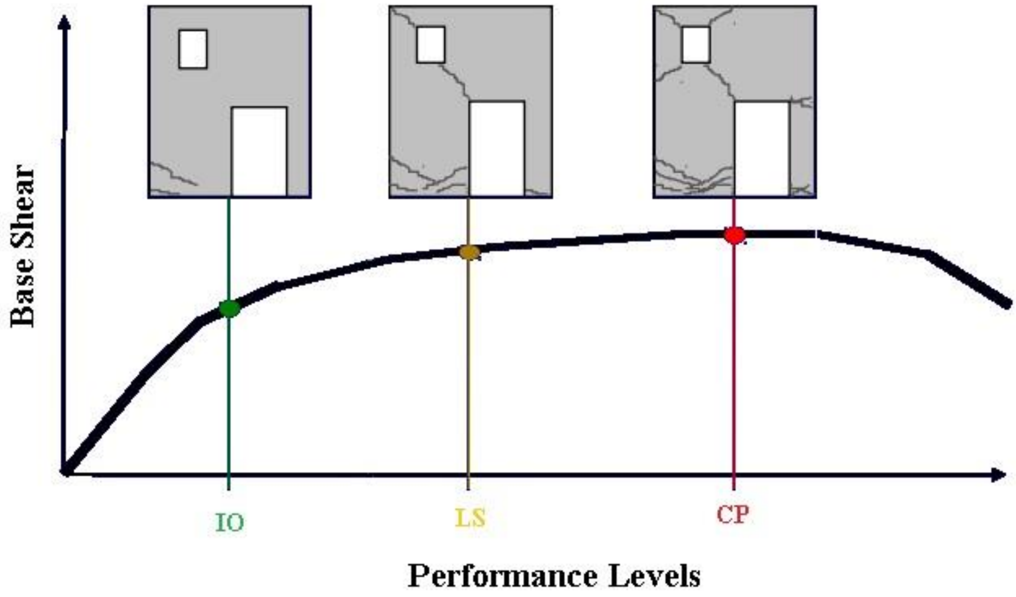

Fig. 5 Overall Building Performance Level

The nonlinear hinge property can be represented as the plot of moment vs rotation capacity, as shown in Fig. 6. Where, Point A is the original state of the member. Point B represents the yield point. There is no significant deformation occurs in the hinge up to point B. Point C represents the ultimate state of the lateral resistant capacity for Pushover analysis. Point D represents the residual strength limit measured in the vertical axis direction. After this limit, member initialize collapsing. Point E represents total failure of the member. After this point hinges break down in the mechanism form.

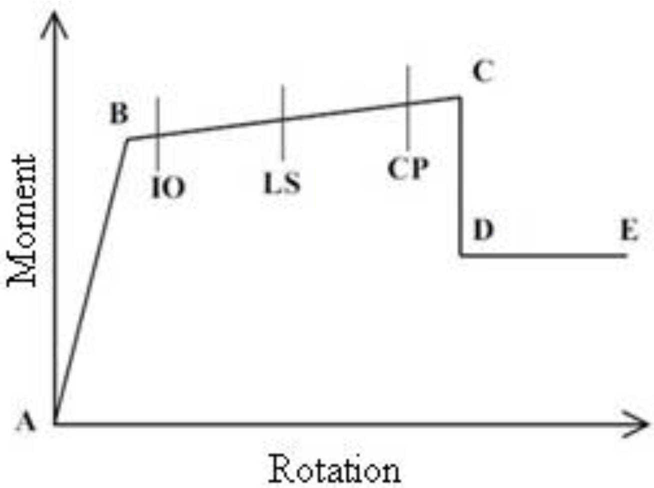

Fig. 6 Moment vs Rotation diagram of hinge

Nowadays, the Prestressed concrete is used more in office building and other critical civil structures. Typical applications of prestressed concrete are: high-rise buildings, residential and parking slabs, bridge deck, silos and nuclear containment structures. The compression in prestressed concrete is produced by the tensioning of high-strength "tendons" through dedicated hollow ducts after hardening of concrete. The post-tensioned flat slabs are as shown in Fig. 7. The advantages of PT flat slab are as follows:

- Flexible in building plan layout
- Savings in the building floor height as well as savings in the construction materials
- Shorter construction time and simple formwork
- Ease of installation of M&E services
- Possible of long span having small deflection
- High shear strength capacity against gravity and lateral loads

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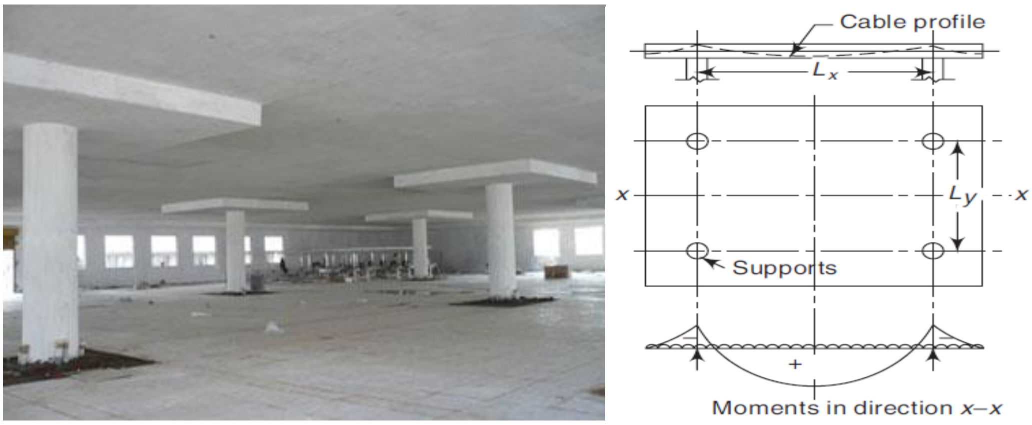

Fig. 7 PT flat slab

II. LITERATURE REVIEW

i. Boonyapinyo et. al. (2004) presented the non-linear pushover analysis using SAP2000, as in [4]. The modelling of the post-tensioned flat slab is carried out using Equivalent Frame Method and the masonry infill walls as Equivalent Strut considering ATC-40 and FEMA 273/274. A case study of Post-Tensioned flat slab building is taken. In this paper, it is found that the lateral capacity of building is increased to 40% with shear wall compared to without shear wall. From the results of the capacity spectrum, it is observed that lateral capacity is increased by 18%. The conclusion of the study reveals that the building will not be collapsed when it is subjected to highest intensity of earthquake in Bangkok.

ii. Hueste et. al. (2007) presented an equation of Gravity Shear ratio to drift ratio, as in [5]. The effort to compare the experiments data for slab-column connections with recommended limits as per ACI 318-05 is made in this paper. The plot of Drift Ratio to the Gravity Shear Ratio is derived from the experiments data for interior slab-column connection specimens with and without shear reinforcement. It has been shown that the punching shear failure occurred in the range of 0.1 to 0.9 for the specimen with no shear reinforcement; whereas the specimen with shear reinforcement reached the large drift ratio.

iii. Zameeruddini and Sangle (2016) have carried out pros and cons and provided an updated review of developments in PBSD, as in [6]. The overall effort of this study was to give the concept of Damage Indices (DI) for reinforced concrete structures. An example of M.R.F. of reinforced concrete model was taken. The models are subjected to pushover analysis using SAP2000. Model was designed as per Indian codes. The plots obtained after the pushover analysis for FEMA 440, give a variation of 3.30% in displacement with respect to ATC 40. The plots obtained as per ASCE 41, give a variation of 13.60% in displacement with respect to the FEMA 356. The authors have introduced an equation of DI which represents an extension of the DI given by Ghorbah et al. (1999).

iv. Satish Kumar and Srinivasan (2012) presented the performance of Open Ground Storey (OGS) buildings, as in [7]. The OGS frames are modelled as per Indian Codes and the pushover analysis is carried out using SAP2000. In this paper, the Inter-storey Drift Index (IDI) is given. Design parameters like column stiffness ratios, % of longitudinal reinforcement, seismic zones and response reduction factor were considered for parametric study. Response parameters considered as IDI and ground floor column's ductility demand. The results are presented as values of IDI plotted against % of reinforcement. From the plots given in results, it is observed that the IDI increases with the increments in % of reinforcement; IDI decreases with Z/R; storey stiffness ratio has no significant effect on the IDI.

v. Choi et. al. (2017) presented the concept of the 'Unified Equivalent Frame Method' for the post-tensioned flat slab considering the shear ratios and different values of prestressed, as in [8]. In this paper, the stiffness reduction factors for PT flat slab were introduced to consider the effects of post-tensioning. From the graphs of the experiments result, it is determined that the stiffness reduction factors of the transverse torsional members in the PT slab are more than the RC slab. From the research of Kwon et al. (2007), Kee et al. (2006), Gayed and Ghali (2006), Han et al. (2006), Ritchie and Ghali (2005), it is concluded that the proposed concept gave accurate results of PT slab for the analysis under lateral loading.

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III. METHODOLOGY

The Equivalent Frame Method (EFM) represents the 3D flat slab building model by a series of 2D frames up to the full height of the building. The flat slab system should be considered as equivalent frame on the centre line of columns in longitudinal direction having the transverse width of span. ACI 318-11 code strongly suggests to use EFM for the flat slab buildings under lateral loading like wind load and earthquakes, as shown in Fig. 8. Basically, the flat slab is very weak in the lateral load resistance capacity - that is the main disadvantage. The primary assumption of EFM is that the value of the ratio of longer to shorter span length should not to exceed 2. The followings are the procedure steps for EFM as per ACI 318-11:

(i) Divide the structure into design strips in longitudinal direction.
(ii) Each equivalent 2D frame shall consists of a row of columns and slab-beam strips bounded by the panels.
(iii) Columns shall be assumed to be attached to slab-beam strips by transverse torsional members.
(iv) The slab-beam is assumed to be continuous at all intermediate supports.
(v) Determine the new stiffness for the columns and slab-beams considering the equations given in ACI 318.

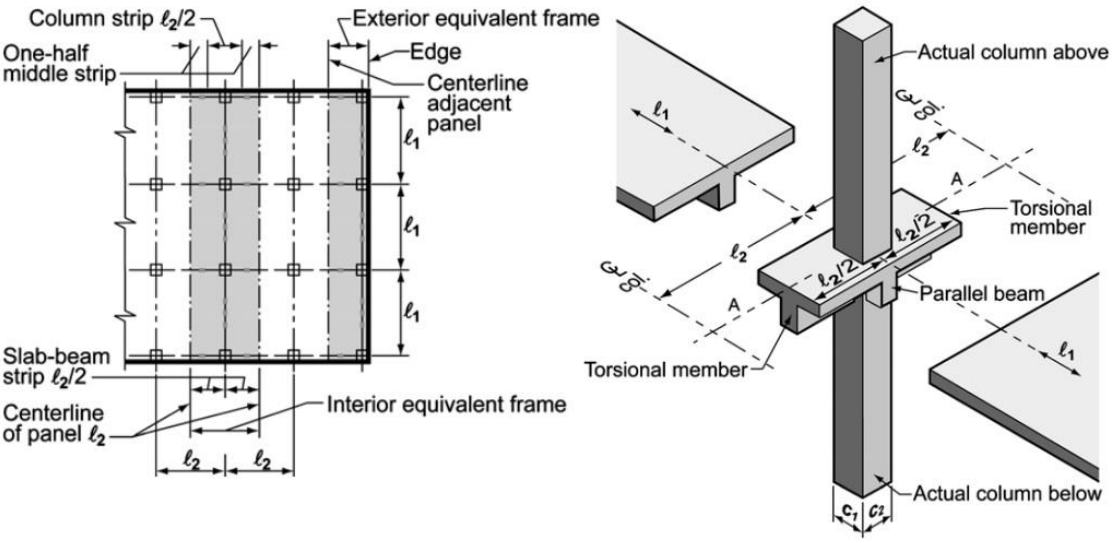

Fig. 8 Representation of EFM as per ACI 318-11

All RC buildings with Unreinforced Masonry (URM) infill walls should be examined for the in-plane storey stiffness and strength as per IS1893(part 1):2016. URM infill walls should model as the 'Equivalent Diagonal Strut' having compression limit in ETABS, as shown in Fig. 9.

Value of Modulus of Elasticity of Masonry shall be taken as: $E_m = 550 f_m$

Width of the Strut: $\omega_{ds} = \frac{0.175 L_{ds}}{\alpha_h^{0.4}}$

where; $\alpha_h = h\left(\frac{E_m t \sin 2\theta}{4EIh}\right)^{\frac{1}{4}}$

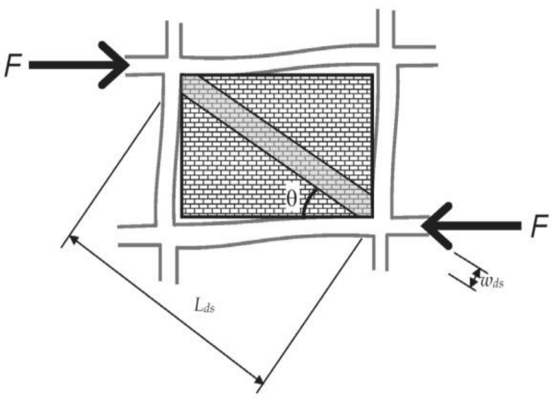

Fig. 9 Representation of strut as per IS1893(Part I):2016

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There are two useful load applications provided in ETABS for Pushover analysis i.e., Force-controlled and Displacement-controlled. In Force-controlled, the total loads are applied to the whole structure continuously increasing. Whereas in Displacement-controlled, the displacement of 4% of total height of building is applied to the roof level at centre of diaphragm step by step in positive X-direction. The flowchart of the pushover analysis is, as shown in Fig. 10.

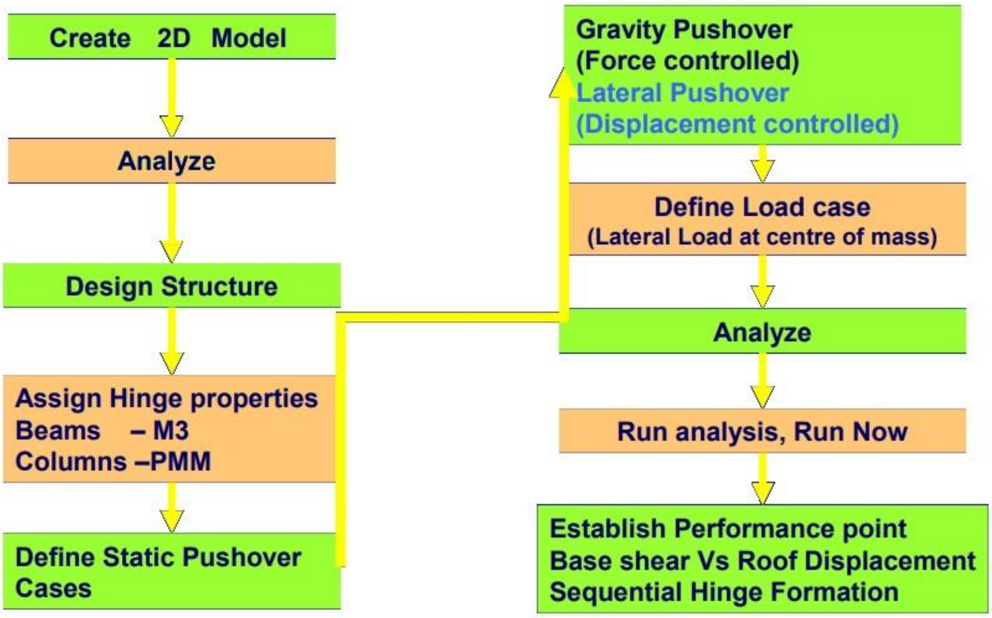

Fig. 10 Flowchart of pushover analysis in ETABS

IV. MODEL DESCRIPTION

ETABS v17 software is used for the preparation of G+7 storey office building models having post-tensioned flat slab to determine the Performance Point using Pushover analysis. The floor height is taken as 3.3 m and 4 bays in both directions. All the models have Slab Drop and Column Capitals. The geometric specifications of the models prepared are, as shown in Table I.

TABLE I GEOMETRIC SPECIFICATIONS

Panel Size (in m)	Optimum Slab Thickness (in mm)	Drop Size (in m)	Drop Thickness (in mm)	Least Possible Column Size (in mm)	Column Capital Size (in m)
7.5 X 6	200	2.5 X 2	300	450 X 450	1.5
				600 X 600	1.5
6 X 6	150	2 X 2	225	375 X 375	1.2
				450 X 450	1.5

In the present study, the design strip is taken in the longitudinal direction only. The calculations to find new Moment of Inertia (M.O.I) of various Equivalent Frame members are done using MS Excel. Stiffness constant 'k' is taken from the tables of SP24 for the ease of calculations. The constant 'k' depends on the ratios c_1/L_1 and c_2/L_2; where c = Size of column or column capital and L = Panel span length. As per ACI 318, the ratio of longer span length to the overall depth of slab should be less than 42 considering deflection criteria. The slab-beam is defined as the beam section in ETABS having the flat slab properties. The URM infill wall is modelled as the strut having the calculated width using the feature of 'Section Designer' in ETABS. The various materials used in the models along with their grades are, as shown in Table II.

TABLE II MATERIAL PROPERTY

Characteristic Strength of Concrete f_{ck}	30 MPa
Yield Strength of Rebar f_y	415 MPa
Compressive Strength of Second-Class Brick f_b	6.9 MPa
Compressive Strength of Mortar MM3 (1:6) f_m	5 MPa

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The followings are the general conditions of construction site considered as per IS1893(part I):2016:

- Zone Factor: 0.36 (Zone 5)
- Soil type: II (Medium or stiff soils)
- Importance factor: 1.2 (Office building)
- Response Reduction factor: 3 (Flat Slab)
- Damping Ratio: 0.05

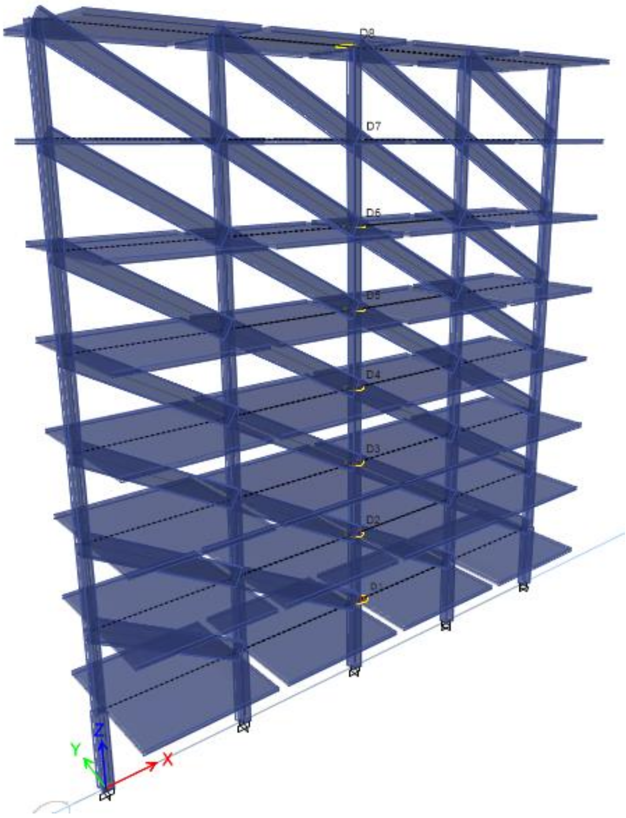

Fig. 11 G+7 equivalent frame model in ETABS

The design strip of G+7 equivalent frame model in ETABS is, as shown in Fig. 11. The total numbers of tendon required for post-tensioning is determined considering all the losses as per IS1343. The upward balanced UDL (w_{bal}) is assigned to each slab-beam along with the compressive member joint force at the both ends of the equivalent frame.

Here, the equation of upward balanced UDL for prestressed force is: $w_{bal} = 8Pe/L^2$
where; P = Effective Force in each tendon = 110 kN
e = eccentricity between top kern and bottom effective cover (take 40 mm of effective cover for tendons)
L = Longitudinal Span Length

The first stage of analysis is carried for Static lateral load case and Response Spectrum load case considering all the possible ultimate load combinations as per Indian code to check the stability of the model. Then, the Deformation-controlled M3 hinges are assigned from the values given in Table 15 of ASCE 41-17 for Slab-beam member and P-M2-M3 hinges are assigned as per Table 10-8 and 10-9 of ASCE 41-17 for columns. The ratio of Gravity Shear (V_g) to the Punching Shear (V_o) is calculated nearly equal to zero for PT flat slab panel. The Force-controlled Axial P hinge is assigned to equivalent diagonal strut at the middle length.

Now, the Gravity Pushover case is defined as Force-controlled having the load pattern of Dead Load, 25% of Live Load, Balanced Upward UDL and Compressive Frame Point Load. Then, Lateral Pushover case is defined as Displacement-controlled having the load pattern of acceleration UX with a scale factor of -1 and continued from Gravity Pushover case. The second stage of analysis is carried out only for the Pushover cases to determine the non-linear behaviour of the model with the help of hinges assigned.

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V. RESULTS

The states of hinge in terms of performance levels of each member are mentioned, as shown in Table III.

TABLE III STATES OF HINGE

Panel Size:	6mX6m		6mX6m	6mX6m		7.5mX6m	7.5mX6m	
URM wall:	No		Yes	No		Yes	No	
Column Size:	Same		Different	Different		Same	Same	
Hinge State:	M3	Axial P	Axial P	M3	PM2M3	Axial P	M3	PM2M3
Floor No.								
7	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-
5	-	IO=1	IO=1	-	-	IO=1	-	-
4	-	LS=1	LS=1	-	-	IO=1	-	-
3	-	LS=1	LS=1	-	IO=5	LS=1, IO=1	-	LS=5
2	IO=8	IO=3	IO=3	IO=8	-	IO=3	IO=8	-
1	IO=8	LS=3	LS=3	IO=8	-	LS=3	IO=8	-
G.F.	IO=8	-	-	-	CP=4, IO=1	-	IO=8	CP=4, IO=1

The deformed structure of model with performance level of hinges is, as shown in Fig. 12.

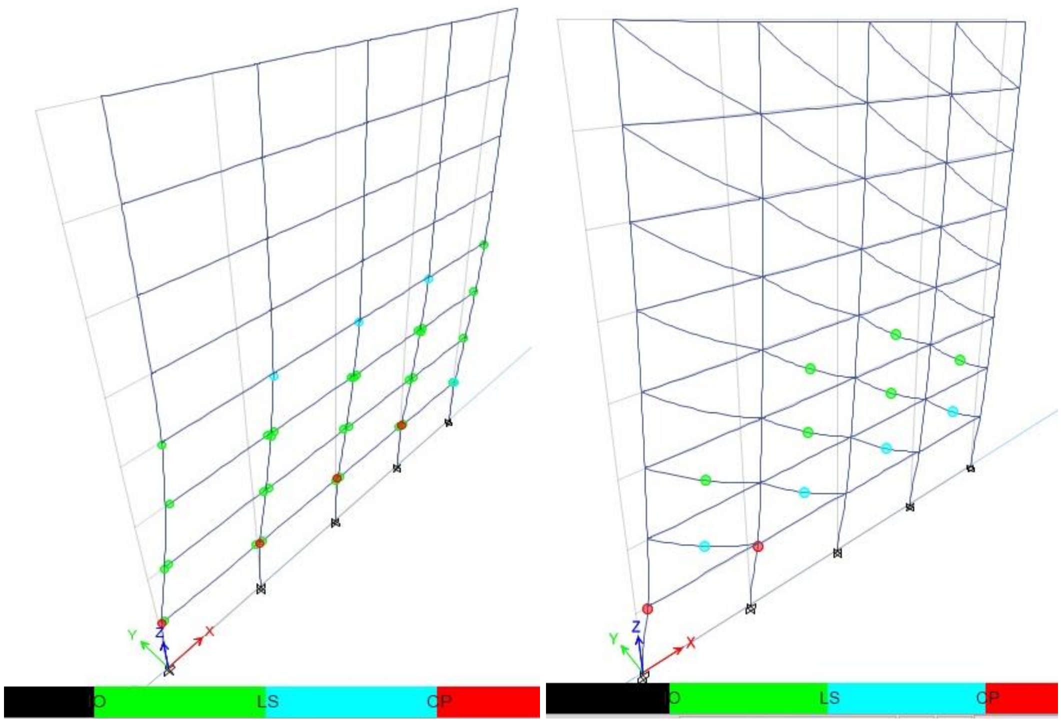

Fig. 12 Performance level of each member in models with and without strut

Now, the values of Base Shear and Target Displacements at the performance point of the models are found as per ASCE41-13 NSP and FEMA440 EL in ETABS, as shown in Table IV. The combined charts from the result of Storey Height vs Drift are determined by using the feature of MS Excel, as shown in Fig. 13. Hence, the first stage of analysis helps to check safety and stability of the design section with detailing. And the second stage of analysis helps in determining the performance points along with the performance levels of each member. The comparison of value of performance points are, as shown in Fig. 14.

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TABLE IV BASE SHEAR AND TARGET DISPLACEMENT AT PERFORMANCE POINT

Panel Size (in meter)	6 X 6	6 X 6	6 X 6	7.5 X 6	7.5 X 6
Masonry wall	No	Yes	No	Yes	No
Condition of Column Size	Same	Different	Different	Same	Same
Base Shear (kN)	706	1389	630	2430	1377
Target Displacement (mm)	225	90	204	229	213

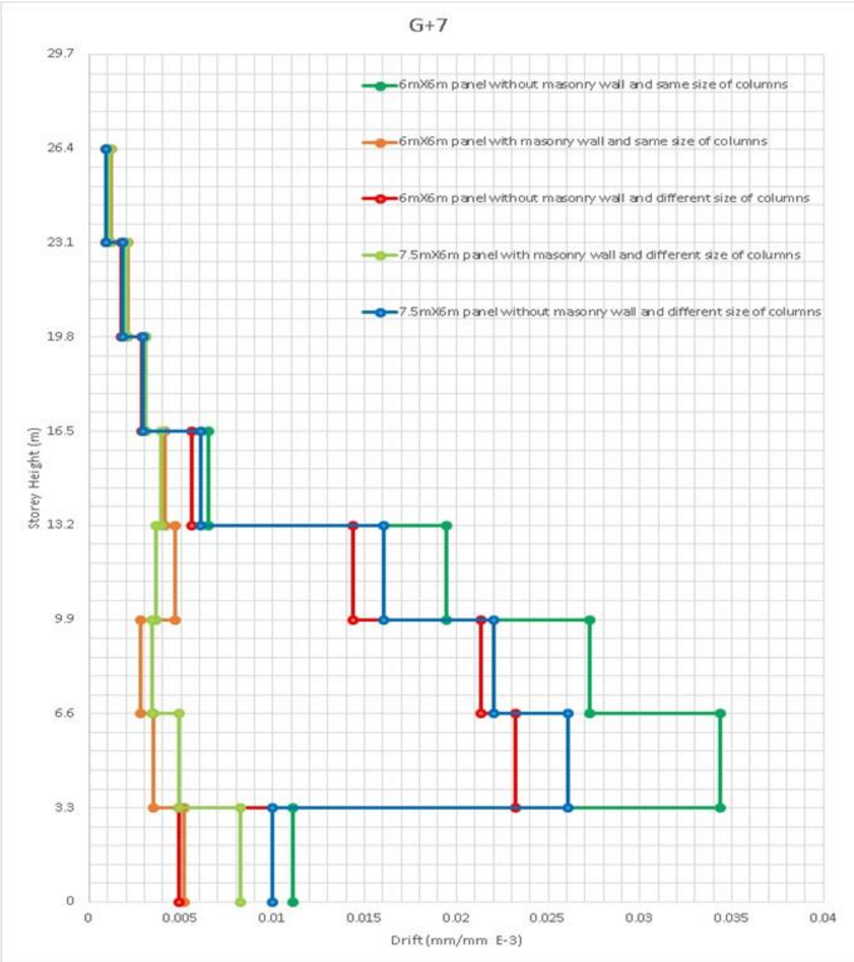

Fig. 13 Storey height vs drift combined chart

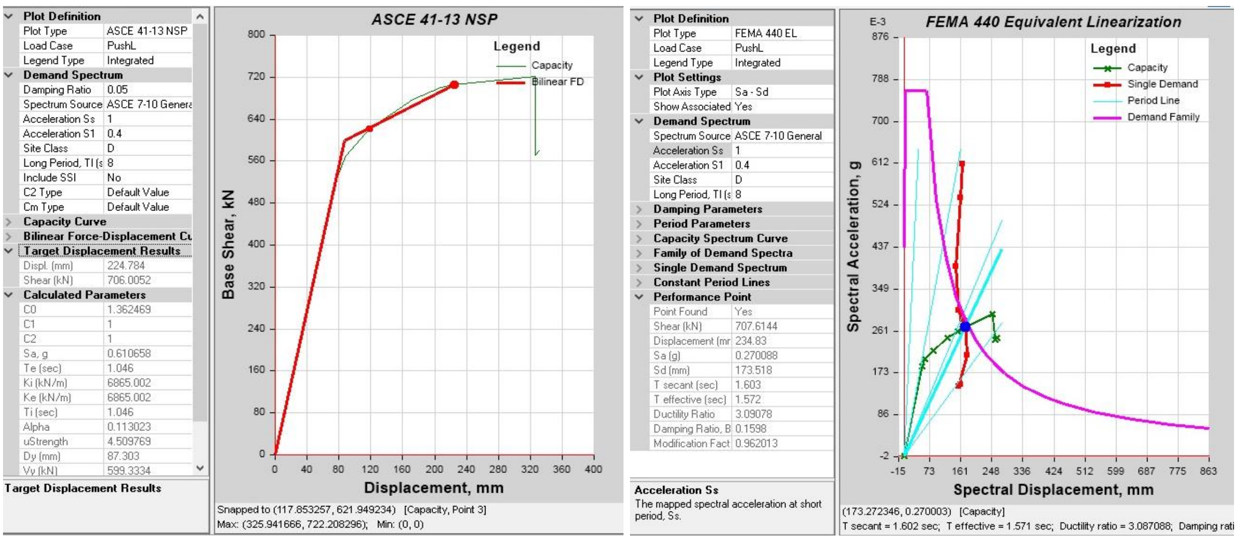

Fig. 14 Comparison of performance point

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VI. CONCLUSIONS

The following conclusions are derived from the results obtained using ETABS:

- Drift values of models without URM walls is about 6 times than the models with URM walls.
- Drift values of models with increase in size of Ground Floor columns reduced by 50% without URM walls while no significant change observed in models having URM walls.
- IO and LS levels of axial P hinges are generated in the strut of models having masonry infill walls up to the fifth floor.
- IO level of M3 hinges is generated in the pot-tensioned slabs up to second floor in G+7.
- CP level of P-M2-M3 hinges is generated in the columns up to first floor and LS, IO levels hinges are generated in columns up to third floor.
- The models having span length of 7.5m have Base Shear values about 50% more than 6m span length whereas no significant changes in the values of Target Displacement.
- The models with URM walls have base shear values nearly double and target displacements half the values of models without URM walls found as per ASCE 41-13 NSP.

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