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Non-Linear Seismic Analysis of RC Setback Frame Structure

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Summary- Dynamic characteristics of setback frames building are different from regular frame building structures in this regards various experimental studies have been conducted and ascertain method of improving their performance to understand the behaviour of setback building. In this work various types of setback frame structures are tested. Seismic response of these selected setback frame structures have been compared with regular frame structure using finite element method based software SAP2000 v20.0.0 in this work one regular and six setback models of G+9 storey RC frames namely Model R, S1-type and S2-type are considered for nonlinear static and dynamic analysis by considering El Centro earthquake. The pushover curve i.e base shear v/s roof displacement is plotted for S1-type, S2-type and regular frame. Target displacements of each model are obtained and pattern of hinges formation at performance point are also determined by pushover analysis and base shear, base moment, storey displacement and storey drift are compared by Nonlinear dynamic analysis. as per all these observations we found that the Setback frame structures are not seen to be high susceptible to damage or high susceptible to higher mode effect as compared to regular frame structure.

Keyword- setback, El Centro earthquake, NDP, NSP

Introduction

Nonlinear static analysis referred as pushover analysis, is the method of determining the maximum load deformation and deflection capability of a structure. Non-linear static analysis procedure was developed by Kilar v and Fajfar P. [1996]. Nonlinear analysis is not essential for capacity design. SAP2000 have various nonlinearities such as material nonlinearity, geometry nonlinearity and staged construction. Mathematically nonlinear static analysis does not always guarantee a unique solution [12]. as per IS 1893: 2016 the structure is to be a irregular if the horizontal dimension of the building frame in any storey are greater than 150 % of the adjacent storey [8]. Various experimental studies have been done for setback structure under various earthquake and researcher found various results, some studies are presented. Sharon L.Wood (1992) Symmetrical and unsymmetrical configuration of setback structures are tested. The result of setback frame structure such as displacement, acceleration, and shear responses during earthquake simulations are compared with those seven previously tested frames with uniform profiles. They conclude that the Setback frames are not observed to be more susceptible to damage or more susceptible to higher mode effects than the frames with uniform profiles. He also conclude that regular frame structure and setback frame structure does not require to different design procedure [1]. Peter Fajfar and M.Eeri (2000) they studied and describe the N2 method and pushover (FEMA 273 and ATC-40) method and they discussed similarities and difference between N2 method and FEMA 27 and ATC-40. They conclude that the N2 method and FEMA-273 are very similar to each other and can yield exactly the same results if the same displacement shape and lateral load distribution are assumed. The major difference lies in visualization provided by the N2 method [15]. Chatpan Chintanapakdee and Anil K. Chopra (2004) studied seismic demands for vertically irregular and regular frames determined by rigorous nonlinear response history analysis NRHA due to an ensemble of 20 ground motions. Forty-eight irregular frames all 12 Storey high with strong columns and weak beams, were designed with three types of irregularities such as- stiffness, strength, and combined stiffness and strength introduced in eight different locations along the height using two modification factors. The effects of vertical irregularity on the median values of Storey drifts and floor displacements are documented and values determined by MPA and NRHA and compared and found there results as, MPA procedure is less accurate relative to the regular frame in estimating the seismic demands of frames with strong or stiff-and-strong first Storey, soft, weak, or soft-and-weak lower half; stiff, strong, or stiff-and-strong lower half [7] . Rui Carneiro Barros and Ricardo Almeida (2005) studied that the effect of higher modes of vibration at nonlinear dynamic response of the structure is very important and have unsolved problems. To simplify this, the nonlinear static analysis was proposed associated with the capacity spectrum method utilizing the load pattern proportional to the shape of fundamental mode of vibration of structure the result of pushover with this load pattern are correct for structures that responses primarily in the fundamental mode but when higher mode of vibration becomes for the total response of the structure this load pattern loss its accuracy. To reduce this problem a new multimode load pattern suggested based on the relative each mode of vibration in the elastic response of the structure subjected to earthquake ground motion. This load pattern applies to the analysis of symmetric frame as well as asymmetric and irregular building frame [14]. The setback response as compared to regular frame structure will identify and best observation regarding this work using SAP2000 have present.

Geometrical Configuration

In this work all model have same material properties and rest on the plain ground and G+9 storey frame structure is considered which have the ground height and typical floor height is 3.5m so the total height is 35.0m is taken in this analysis. The size of the beam, column and thickness of the slab are 350mmx500mm, 350mmx450mm and 150mm is considered in all setbacks as well as regular frame structure. The number of bays along X and Y direction are 6 and 4. The bay width along X and Y direction are 3.5m and 5.0m geometry are considered. In this configuration, M25 grade concrete in beam and slab are used but in column M30 grade concrete are taken. Longitudinal bar Fe-500D and confinement bar Fe-250D is used in all beam and column configuration. The thickness of external wall is 200mm and internal wall is 100mm have taken and the floor finish load 1 kN/m^2 and roof treatment load 1.5 kN/m^2. As per IS 875 part-1 & 2 the Live load 3.0 kN/m^2 and roof live load is 1.5 kN/m^2 is considered for residential building. So the super dead and live load distribution are as follows in Table -1

Table-1 Load distribution

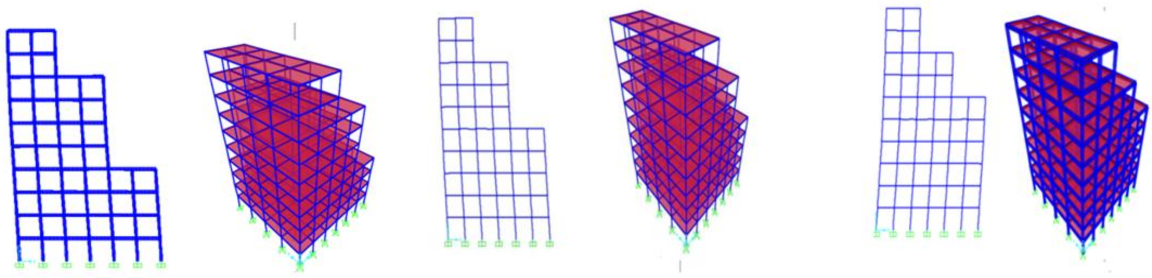
Beams	Type	Super dead Load		Live load	
		Floor (kN/m)	Roof (kN/m)	Floor (kN/m)	Roof (kN/m)
Long Beam	External	13.009	6.669	3.675	1.837
	Internal	8.446	3.675	7.35	3.675
Short Beam	External	12.659	6.144	2.625	1.312
	Internal	7.746	2.624	5.25	2.624

Seismic Load Parameters

Analysis have carried out by considering El Centro Earthquake 19.5.1940 at 4:36:41 UTC earthquake having PGA is 3.304 m/s^2 using SAP2000 v 20.0.0. Frame building structures are to be assumed in Zone IV and importance factor is 1.0 for residential building and response reduction factor is taken as 5 for Simple moment resistance frame system of frame structure. And as per the Indian code IS 1893-2016 the medium soil (Type 2) and structural damping 5% is considered in this analysis. The response of setback building frame structure subjected to seismic loads (El Centro) are compared with regular model by pushover and Nonlinear dynamic analysis using finite element method, software SAP2000 v 20.0.0.

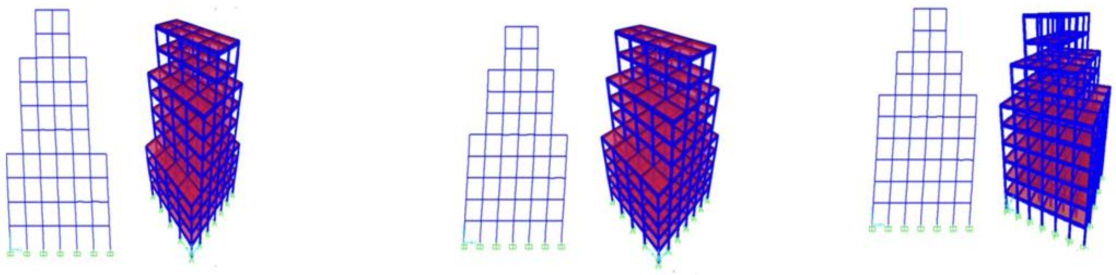
Description of test structures

In this analysis following model and their modal deformed shape are presented. S1-4-8, S1-5-8 and S1-6-8 means the frame structure are setback at fourth level, fifth level and sixth level in one side. Other second model shapes have S2-4-8, S2-5-8 and S2-6-8 means frame structure are setback at fourth level, fifth level and sixth level. These mode shape are obtained by modal analysis with the help of SAP 2000v 20. All S1, S2-type structures are setback at eighth level. Property for all model are same. Models are present here with their first three fundamental time period and deformed shape. Test structure is shown in Fig 1(a, b, c)



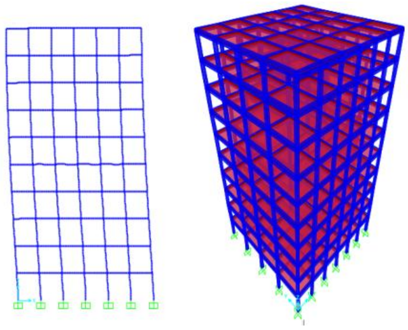
Model S1-4-8 (T$_1$ = 1.854s, T$_2$ = 1.50s, T$_3$ = 1.3s) Model S1-5-8 (T$_1$ =1.855s, T$_2$ = 1.502s, T$_3$ = 1.31s) Model S1-6-8 (T$_1$ =1.856s, T$_2$= 1.508s, T$_3$ = 1.384s)

Fig 1(a) 3D view of frame structure of S1-type models with deformed shape



Model S2-4-8 (T$_1$ =1.742s, T$_2$ =1.48s, T$_3$ = 1.335s) Model S2-5-8 (T$_1$ =1.742s, T$_2$ =1.491s, T$_3$ =1.347s) Model S2-6-8 (T$_1$ =1.766s, T$_2$ =1.501s, T$_3$ = 1.395s)

Fig 1(b) 3D view of frame structure of S2-type models with deformed shape



Model R - (T_1 = 2.091s, T_2 = 1.792s, T_3 = 1.791s)
Fig 1(c) 3D view of regular frame structure with deformed shape

Non-Linear static Analysis (Pushover Analysis)

Non-linear static analysis procedure was developed by Kilar V and Fajfar P [1996]. Nonlinear analysis is not essential for capacity design. SAP2000 have various nonlinearities such as material nonlinearity, geometry nonlinearity and staged construction. Mathematically nonlinear static analysis does not always guarantee a unique solution [20]. ASCE 41 (FEMA 356) applies to performance based design for seismic rehabilitation of existing building and it provides deformation capacities for wide range of structural components for IO, LS and CP levels as shown in Fig 2 the five points A, B, C, D, E used to define the force deflection behaviour of hinges and IO, LS and CP used to define the acceptance criteria for the hinges.

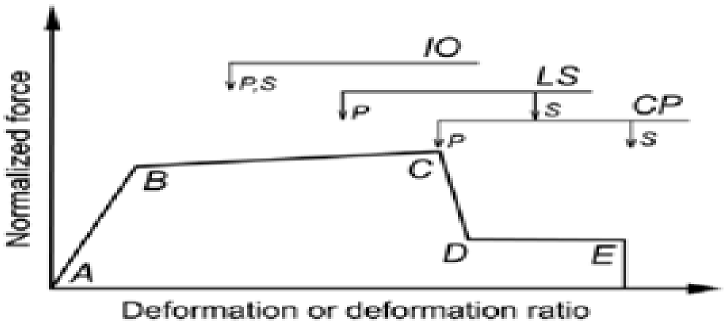


Fig 2 acceptance criteria on a force versus deformation diagram

IO (Immediate Occupancy) little or no damage, allow little inelastic behavior, LS (Life safety) some damage, few or no injuries, allow substantial inelastic behavior and CP (collapse prevention) more damage, no collapse, pushes the limits of the ductile capacities of the components. And other performance level may be used. The above are the most common. In this analysis a plasticity approach is selected for modelling nonlinearity wherein the plastic hinge is assumed to be concentrated at a specific point in the frame member under consideration. Beam and column elements in this study were modeled with flexure (M3 for beams and P-M2-M3 for columns) hinges at possible plastic regions under lateral load *i.e* both ends of the beams and columns. The hinges selected for the analysis proposed in ATC-40 and FEMA-356 and this type of hinges accounts for the interaction between the axial force and bending moment on behaviour of the structure.

Capacity Spectrum curve for S1, S2 and Regular frame

The performance point of all models is determined using ATC-40 capacity spectrum curve.ATC-40 curve are shown in Fig 3 (a to g).

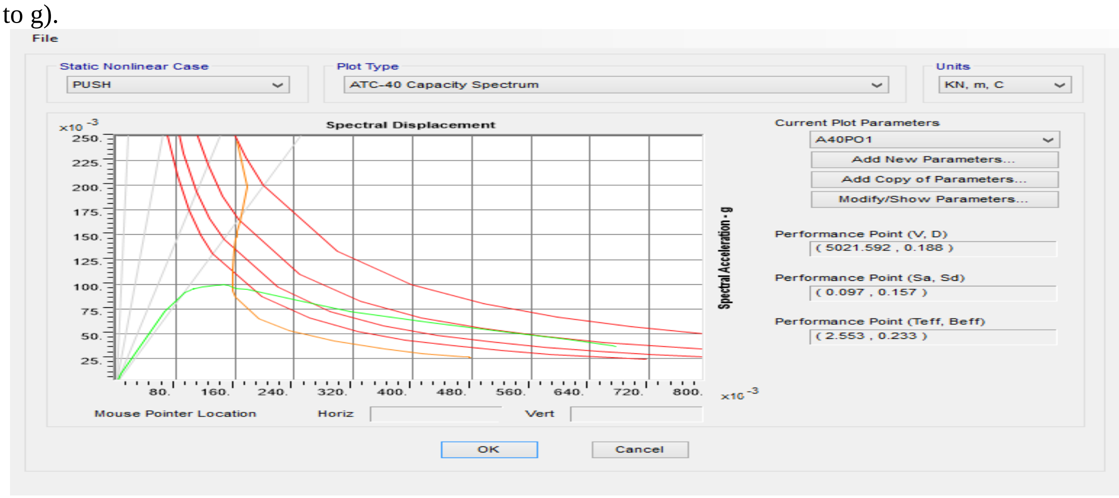


Fig 3 (a) Capacity spectrum curve for regular model

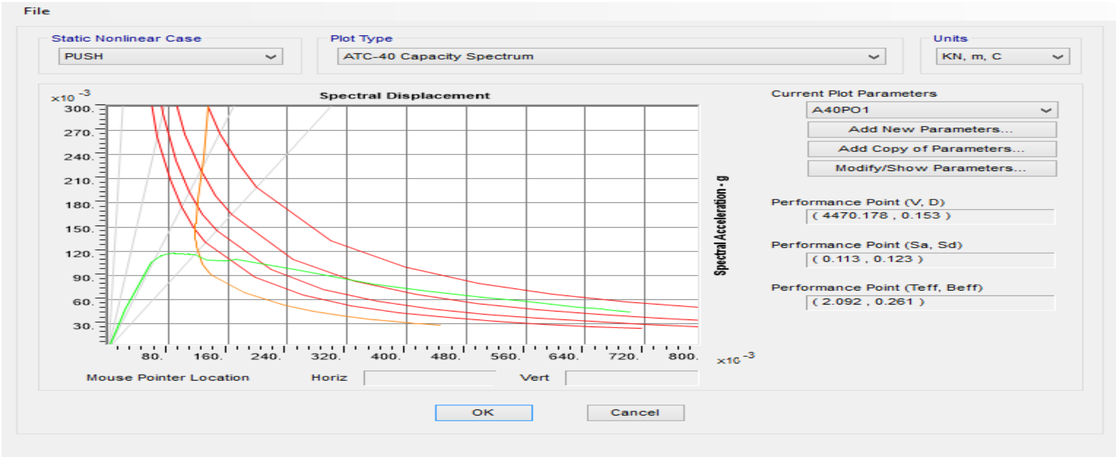


Fig 3 (b) Capacity spectrum curve for S1-4-8 model

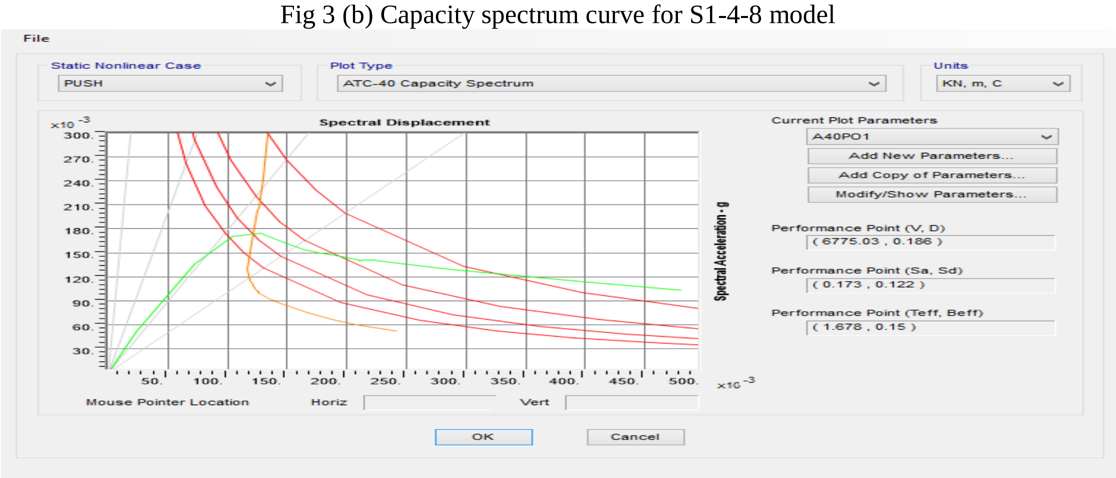


Fig 3 (c) Capacity spectrum curve for S1-5-8 model

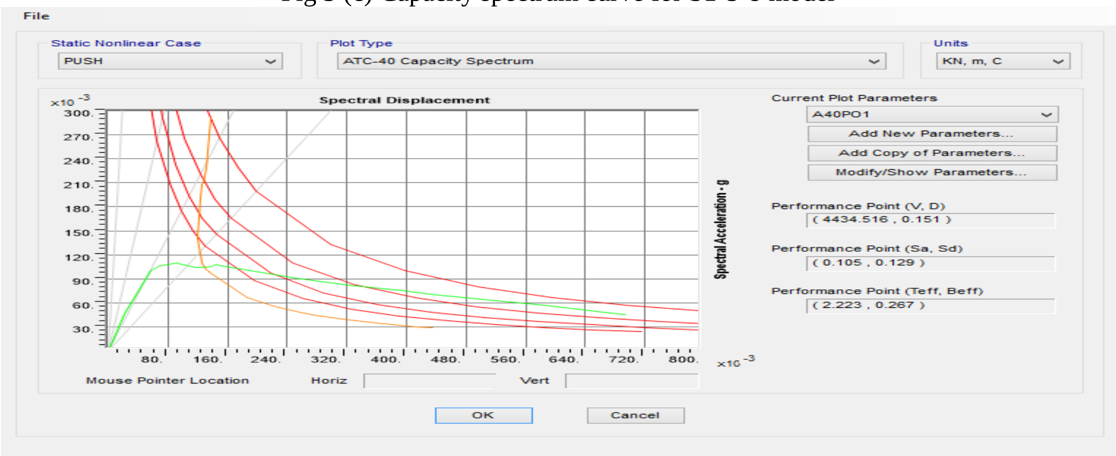


Fig 3 (d) Capacity spectrum curve for S1-6-8 model

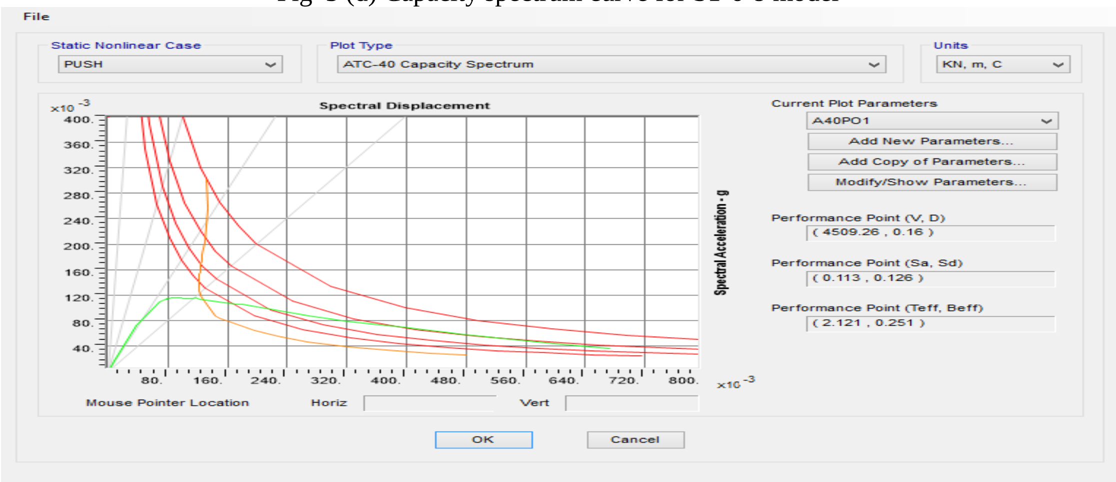


Fig 3 (e) Capacity spectrum curve for S2-4-8 model

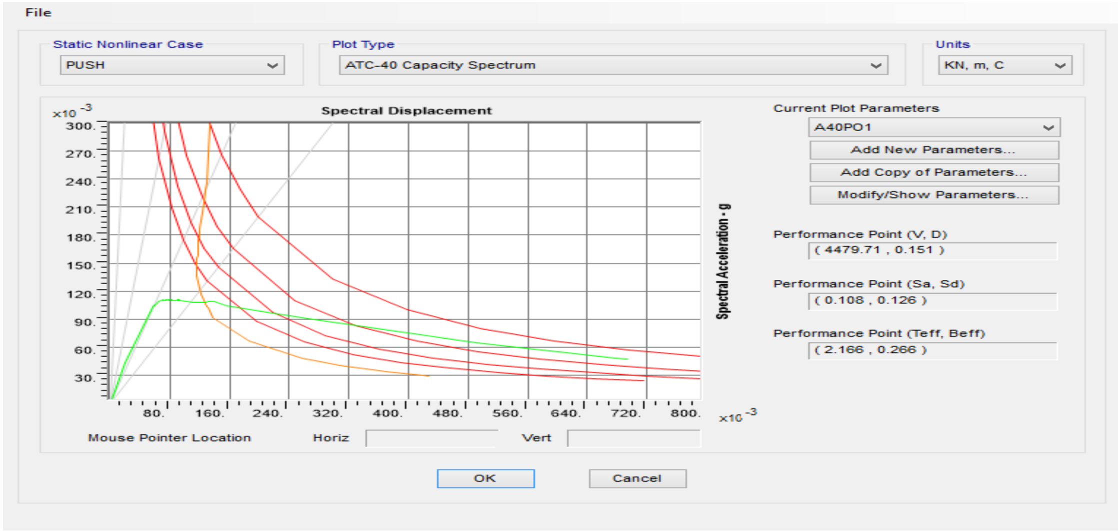


Fig 3 (f) Capacity spectrum curve for S2-5-8 model

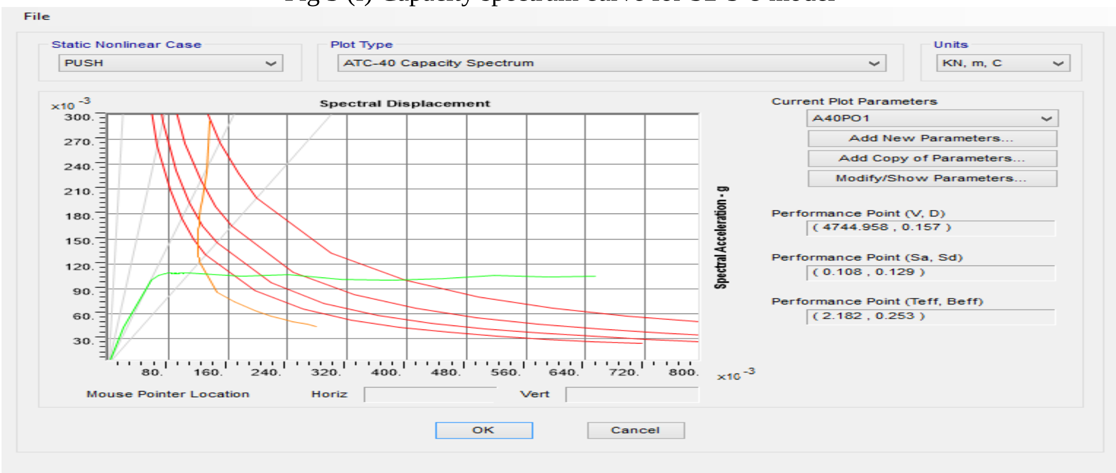


Fig 3 (g) Capacity spectrum curve for S2-6-8 model

Target displacement of S1, S2 and regular frame

Target displacement of frame is shown in Table 2. The target displacement is an estimate of the global displacement of the structure is expected to experience in a design earthquake and it is the roof displacement of the structures. The target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake.

Table 2 Target displacement of frame structures

Models	Performance point Base shear (kN)	Target Displacement (mm)
Model R	5021.592	188
Model S1-4-8	4470.178	153
Model S1-5-8	6775.03	186
Model S1-6-8	4434.518	151
Model S2-4-8	4509.28	160
Model S2-5-8	4479.71	151
Model S2-6-8	4744.958	157

Hinge pattern for S1-type, S2-type and regular frame

The distribution of hinges in the frame structures along the height at performance point are shown in Fig 4 (a to g). Hinge formation shows the performance level for regular as well as setback frame structures. Hinge formations along X-direction are taken at performance point for all models and maximum performance level attained by hinge is immediate occupancy (IO). The performance level is defined as that the damage which may be considered satisfactory for a given building and a given ground motion intensity is known as a performance level. Performance point is obtaining from intersection of demand and capacity curves. Capacity is obtained from pushover analysis, where it is tried to capture real response by assigning nonlinear behaviour to plastic hinge (like FEMA hinge). And demand is gained by using spectrum.

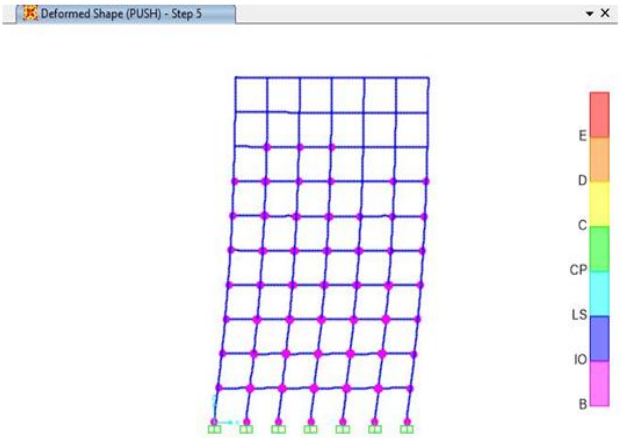


Fig 4 (a) Formation of plastic hinge

at performance point for regular frame

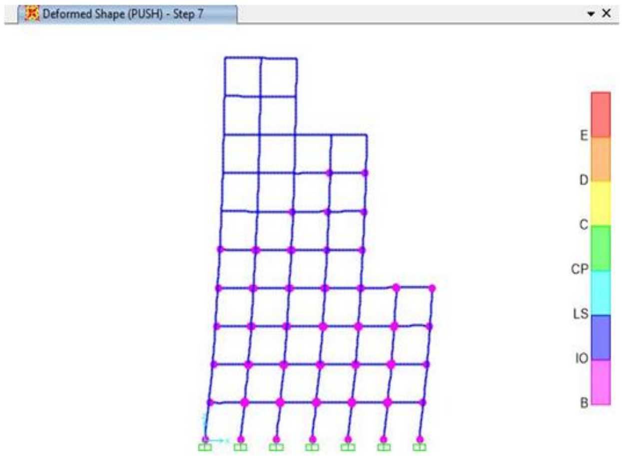


Fig 4 (b) Formation of plastic hinge

at performance point for S1-4-8

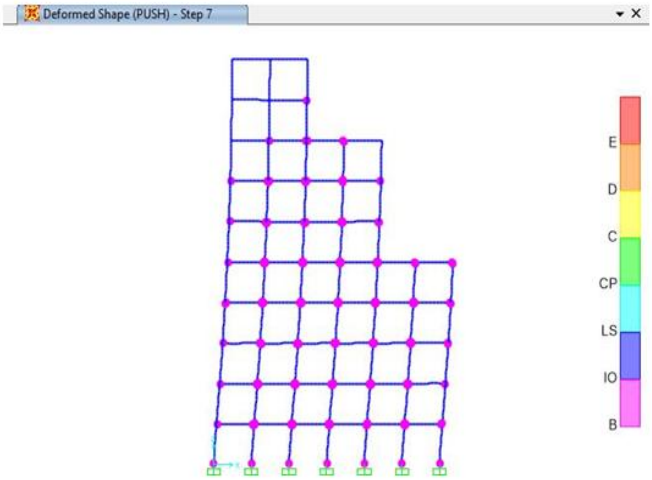


Fig 4 (c) Formation of plastic hinge

performance point for S1-5-8

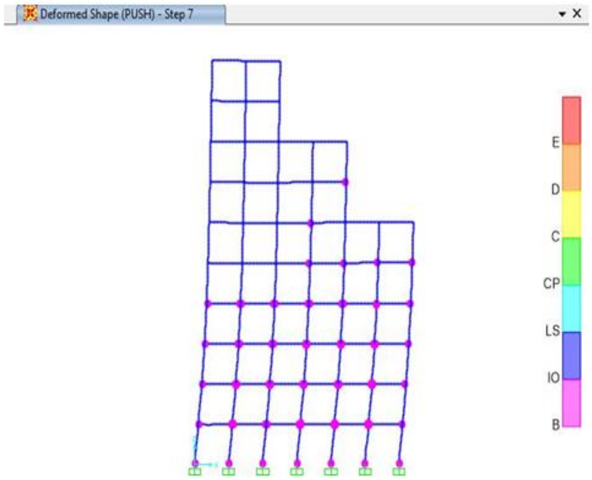


Fig 4 (d) Formation of plastic hinge at at

at performance point for S1-6-8

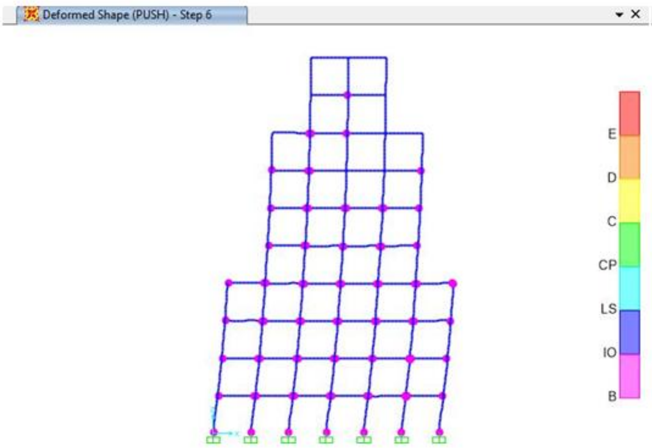


Fig 4 (e) Formation of plastic hinge at

performance point for S2-4-8

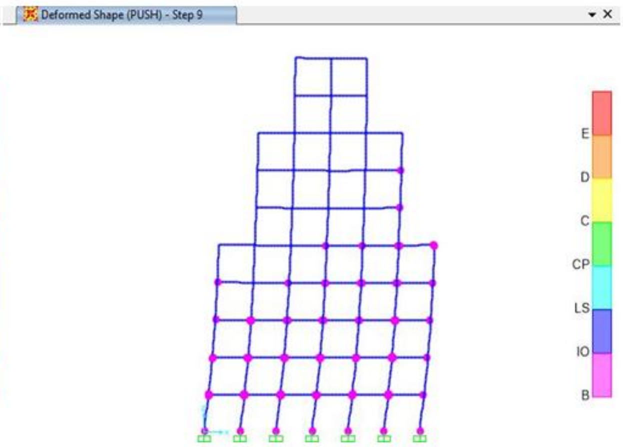


Fig 4 (f) Formation of plastic hinge at

performance point for S2-5-8

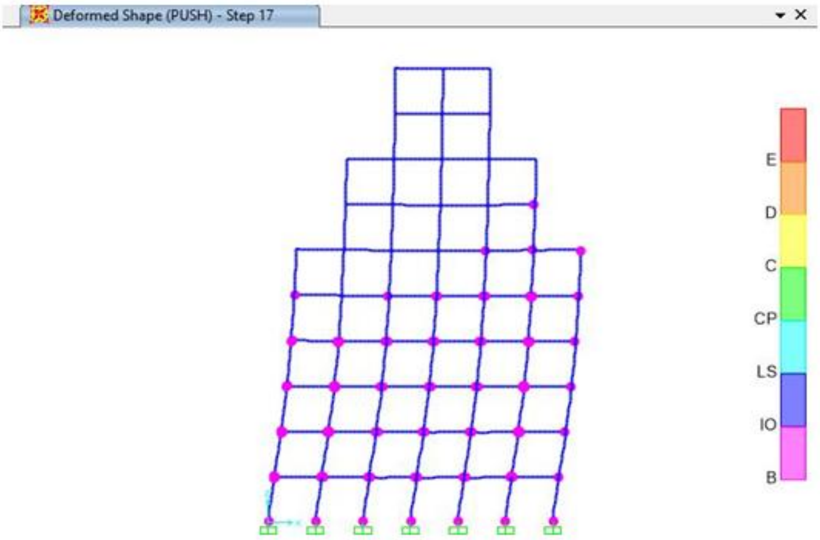


Fig 4 (g) Formation of plastic hinge at performance point for S2-6-8

Nonlinear Dynamic Analysis

The Linear Dynamic Procedure (LDP) ignores nonlinearity and the Nonlinear Static Procedure (pushover) ignores dynamic effects and the Linear Static Procedure (LSP) ignores both [9, 10]. So, Non-linear Dynamic Procedure (NDP) or Non linear time history analysis attempts to fully represent the seismic response of buildings without any of these major simplifying assumptions. This method is used to determine the dynamic response of a structure to arbitrary loading. In nonlinear analysis the stiffness, damping and load depends upon the velocities, displacement and time [20]. In which material nonlinearity and geometric nonlinearity are present in software for performed the nonlinear direct-integration time history. Direct integration does gives the following advantages such as, full damping that couples the modes can be considered and all type of nonlinearity are present in SAP2000 v20.0.0 Direct integration results are extremely sensitive to time step size in a way that is not true for modal superposition.

Base Shear Effect for S1, S2 and regular type structure.

When El Centro dynamic load is applied in X direction then the response were found only in force direction (x-component) but the response in y-component was approximate equal to zero. Similarly when dynamic load is applied in Y-direction then the response was found only in y-component but response in x-component was approximate equal to zero. The obtained response due to dynamic loading in X and Y direction of the frame structures can be seen in Table 3. From these values i found that there is no eccentricity present in the structure means center of mass coincides with center of stiffness in S1, S2 and regular frame structures.

Table 3 Base Shear (kN) by Non-linear Dynamic

Models	**EQ-X**		**EQ-Y**	
	EQ-x	**EQ-y**	**EQ-x**	**EQ-y**
Model R	5064	3.5	1.795	4055
Model S1-4-8	4500.6	9.064	32.88	3867.011
Model S1-5-8	5170.802	877	641.02	5601.44
Model S1-6-8	4585.718	6.58	22.066	3849.61
Model S2-4-8	3950.8	13.98	5.03	3647.182
Model S2-5-8	4566	7.5	2.6	3896.5
Model S2-6-8	4936.45	4.729	1.64	4278.3

Effect of base moment for S1, S2 and regular frame structure

The base moment of the frame structures due to El Centro Earthquake loading in X direction have find for all models is shown in the Table 4. From these obtained values of base moment we can seen that the response in x-component and y-component are nearly approx equal due to the earthquake loading in X-direction. The base moment value of S1-type frame structure is low as compared to regular frame structure similarly base moment of S2-type frame structure is low as compared to regular frame structure.

Table 4 Base Moment by Non-linear Dynamic

Models	ELCENTRO EQ-X	
	EQ-x	EQ-y
Model R	563980	703644
Model S1-4-8	431010	448540
Model S1-5-8	444480	494404
Model S1-6-8	466400	520100
Model S2-4-8	432773	535527
Model S2-5-8	449242	557172
Model S2-6-8	467034	586889

Effect of Storey displacement for S1, S2 and Regular frame

The Storey displacement of the frame structures have find by the earthquake loading in X- direction and found that the storey displacement for S1-type and S2-type frame structures are low as compared to Model R (regular profile). Storey displacement are presented in Table 5

Table 5 Story Displacement (mm) in X- Direction by Non-linear Dynamic

Height, m	Model R	Model S1-4-8	Model S1-5-8	Model S1-6-8	Model S2-4-8	Model S2-5-8	Model S2-6-8
0	0	0	0	0	0	0	0
3.5	23.317	21.644	16.803	20.302	24.187	22.085	17.491
7	48.529	41.207	37.304	41.354	45.92	41.787	38.087
10.5	72.138	55.105	54.557	57.347	61.949	56.327	55.57
14	97.057	65.761	68.413	70.347	74.804	68.23	70.46
17.5	121.6	82.472	80.326	82.897	93.261	79.051	84.46
21	143.903	103.606	97.882	94.804	115.531	101.107	97.462
24.5	161.397	121.239	118.653	112.065	133.431	120.905	116.64
28	173.277	132.198	133.534	124.969	144.233	131.239	128.03
31.5	180.436	142.383	146.958	136.671	151.553	138.925	136.842
35	184.147	148.543	153.877	143.393	155.294	143.288	141.917

Storey Drift of S1, S2 and regular frame

The Storey displacement is the actual displacement however Storey drift is the relative displacement. The Storey drift is decreases due to reduction in stiffness of setback structure having aspect ratio 0.4, 0.5 and 0.6 storey drift value for regular frame are higher as compared to the setback frame structures. The Storey drift values are presented in Table- 6

Table 6 Story Drift (mm) in X- Direction by Non-linear Dynamic

Height, m	Model R	Model S1-4-8	Model S1-5-8	Model S1-6-8	Model S2-4-8	Model S2-5-8	Model S2-6-8
0	0	0	0	0	0	0	0
3.5	23.317	21.644	16.803	20.302	24.187	22.085	17.491
7	23.609	19.563	20.501	21.052	21.733	19.702	20.596
10.5	72.138	13.898	17.253	15.993	16.029	14.54	17.483
14	24.919	10.656	13.856	12.55	12.855	11.903	14.89
17.5	24.543	16.711	11.913	82.897	18.457	10.821	14
21	22.303	21.134	17.556	11.907	22.27	22.056	13.002
24.5	17.494	17.633	20.771	17.261	17.9	19.798	19.178
28	11.88	10.959	14.881	12.904	10.802	10.334	11.39
31.5	7.159	10.185	13.424	11.702	7.32	7.686	8.812
35	3.711	6.16	6.919	6.722	3.741	4.363	5.075

Conclusion

The study is done for one regular and six setback models of G+9 storey RC frames namely Model R, S1-type and S2-type by Nonlinear static and Nonlinear dynamic analysis for El Centro Earthquake 19.5.1940 at 4:36:41 UTC earthquake having PGA is 3.304 m/s^2 using SAP2000 v 20.0.0 and the results are as follows

1. For Non-Linear static analysis, The performance point is obtained from intersection of demand and capacity curves as shown in Fig-3 (a to g) and the performance point are shown in Table-2 for regular and setback frame structures. This is real captured response of regular frame as well as setback frames by assigning the nonlinear behaviour to plastic hinges.
2. For Non-Linear static analysis, The target displacement captured from the pushover curve and it is the global displacement of the regular as well as setback frame structures as shown in table-2 and Fig3 (a to g). The displacements of the setback frame structures are low as compare to the regular frame and this displacement is likely to be knowledgeable for the design.
3. For Non-Linear static analysis, Hinges for frame structures at performance point are presented and shown in Fig 4 (a to g). Hinges shows the performance level for regular frame as well as setback frame structures. Hinge generations at performance point in X-direction are taken for regular frame as well as setback frames and maximum performance level gain by hinge is immediate occupancy (IO) and at IO the damage which may be considered all right.
4. For Non-Linear dynamic analysis, the base shear responses of the frames are obtained by considering El Centro earthquake and the obtained responses due to dynamic loading in X and Y direction of the frame structures can be seen in Table 3 and we found that the frame structures has no eccentricity means center of mass coincides with center of stiffness in S1-type and S2-type frames and regular frame structures.
5. For Non-Linear dynamic analysis, the base moment response in x-component and y- component is nearly approximately equal when El Centro earthquake is applied in X-direction. The base moment value of S1-type and S2-type frame structures is low as compared to regular frame structure and base moment values are shown in Table 4.
6. For Non-Linear dynamic analysis, Storey displacement of regular frame is high as compared to S1-type and S2-type frame structures and the storey displacement is given in Table 5 and the storey drift value for regular frame is higher as compared to the setback S1-type and S2-type frame structures. The Storey drift values are shown in Table 6 as per all these observations we found that the Setback frame structures are not seen to be high susceptible to damage or high susceptible to higher mode effect as compared to regular frame structure.

NOTATIONS

NDP- Nonlinear Dynamic provedure
NRHA- Nonlinear response history analysis
MPA- Modal pushover analysis
NSP- Nonlinear static procedure
All units are taken in MKS system

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