Published by :
http://www.ijert.org

International Journal of Engineering Research & Technology (IJERT)
ISSN: 2278-0181
Vol. 6 Issue 06, June - 2017

Comparative Study of Pushover Analysis on RCC Structures

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***Abstract*: Nowadays non-linear static analysis is gaining its importance for structural design and seismic assessment of reinforced concrete members. Overall shape, size and geometry of the building determine the behaviour of a building during earthquakes. Progressive collapse refers to a phenomenon in which local damage in a primary structural element leads to total or partial structural system failure. The method can be used to study the behaviour of reinforced concrete structures including force redistribution. In this study, four, eight and twelve storied buildings are analysed and compared in seismic zone-V using Response Spectrum Method and Non linear static method (Pushover method). The base shear, roof displacements and various structural forces are tabulated and the performance point is determined using SAP2000 which gives information about the global behaviour of the structure.**

Key words*: Response *Spectrum method, Pushover Analysis, ATC-40, Performance Point.

1. INTRODUCTION

Non linear static procedure (pushover analysis) has been widely used for evaluating the performance of existing buildings and verifying the design of seismic retrofits. Various methods, both elastic (linear) and inelastic (non-linear) are available for the analysis of existing concrete buildings. Elastic analysis methods available include code static lateral force procedure, code dynamic lateral force procedure and elastic procedure using demand capacity ratios. The most basic inelastic analysis method is the complete non-linear time history analysis which is at this time is considered overly complex as it requires accurate acceleration data of previous earthquake data. Available simplified non-linear static analysis procedures include the capacity spectrum (CSM) that uses the intersection of the capacity (pushover) curve and a reduced response spectrum to estimate maximum displacement of the structure as per ATC-40 guidelines. The objective of this study is to emphasize the use of non-linear static procedure in general and focus on the capacity spectrum method as per ATC-40.

2. DESCRIPTION OF THE WORK UNDER STUDY

The RCC structures chosen for the study are 4, 8, 12 storeys of each storey height 3m subjected to earthquake forces in the form of site specific spectra. Different types of earthquake analysis carried out are equivalent static method, response spectra method and non-linear static pushover method

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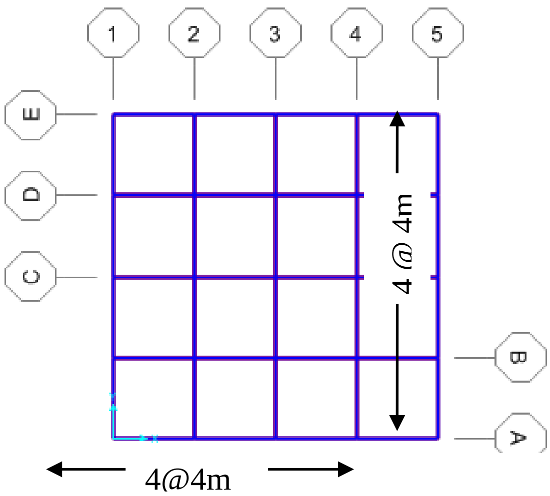

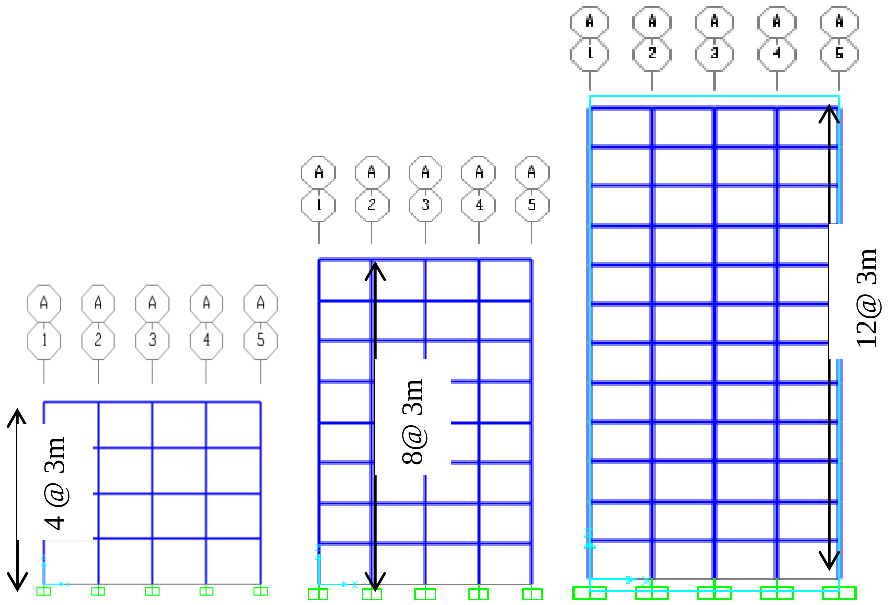

Fig1. Plan of 4, 8 12 storey Elevation

3. METHODOLOGY

A. Modelling of the structure

The RCC structures are modelled as three dimensional finite element using analysis software SAP2000. The structures considered are 4, 8 and 12 storeys of 4 bay symmetric in both directions. The structures are analysed for equivalent static method, response spectrum method, pushover analysis.

Properties of the structures:

Grade of Concerte	M30
Grade of Steel	Fe 500
column size	350X350 mm
beam size	230X400 mm
Slab thickness	175 mm
Live Load	$2kN/m^2$
Super dead load	$1.5kN/m^2$
Project Site	Bongaigoan, Assam
Zone factor	0.36(very severe)
Importance factor (I)	1.5
Response reduction factor (R)	5

B. Method of Analysis

Modal Analysis

Modal analysis is used to determine the dynamic properties of the structure such as amplitudes, frequency and mode shapes which depends on the overall mass and stiffness of a structure.

Equivalent Static Method

This method of analysis is based on the assumption that the fundamental mode of the building makes the most significant contribution to the base shear and the total building mass is considered as against the dynamic procedure**.** For this to be true, the building must be low-rise and must not twist significantly when the ground moves. 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.

International Journal of Engineering Research & Technology (IJERT)
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RESPONSE SPECTRUM METHOD

This is the most common linear dynamic method of analysis suitable for problems involving the structural design of new structures. Response spectrum analysis uses the vibration properties such as natural frequencies, natural modes, and modal damping ratios of the structure and the dynamic characteristics of the ground motion through its response spectrum. This is required in many building codes for all except for very simple or very complex structures. The response of a structure can be defined as a combination of many mode shapes. For each mode, a response is read from the design spectrum, based on the modal frequency and the modal mass, and they are then combined to provide an estimate of the total response of the structure. In this we have to calculate the magnitude of forces in all directions i.e. X, Y & Z and then see the effects on the building. Combination methods include the following:

- Absolute –peak values are added together
- Square root of the sum of the squares (SRSS)
- Complete quadratic combination (CQC) – a method that is an improvement on SRSS for closely spaced modes

This method doesn't hold good for too tall and irregular structure. In the present study, site specific spectrum is used instead of the design spectra specified in IS: 1893-2002 and the type used is SRSS method.

NON-LINEAR STATIC ANALYSIS

Available simplified nonlinear analysis methods referred to as non-linear static analysis procedures such as Capacity Spectrum method (ATC-40), displacement co-efficient method (FEMA -273) and secant method.

EVALUATION PROCEDURES

The basic principles of all non linear procedures are same i.e they all use bilinear approximation of the pushover curve. In this static procedure, the properties of every multi degree of freedom (MDOF) structures is equated to corresponding single degree of freedom (SDOF) equivalents, and the expected maximum displacement is approximated using the response spectrum of relevant earthquake intensity .

ATC 40[1] - 1996 - Capacity Spectrum Method (CSM)

This method is based on the equivalent linearization of a nonlinear system. The important assumption here is that inelastic displacement of a nonlinear SDF system will be approximately equal to the maximum elastic displacement of linear SDF system with natural time period and damping values greater than the initial values for those in nonlinear system. ATC 40 describes three procedures (A, B and C) for the CSM and the second one is used in this study

LOADS CONSIDERED:

Dead load: Self weight of the structure

Live load: $2kN/m^2$ Superimposed dead load: $1.5kN/m^2$

Seismic loads: The structure shall be analysed for site specific design acceleration spectra instead that given in figure-2 of IS: 1893 (Part1). The site specific acceleration spectra along with multiplying factors include the effect of the seismic environment of the site, the importance factor related to the structures and the response reduction factor. Hence, the design spectra do not require any further consideration of the zone factor (Z), the importance factor (I) and response reduction factor (R) as used in the IS: 1893(Part 1 and Part 4). Horizontal seismic acceleration spectral coefficients (in units of 'g')

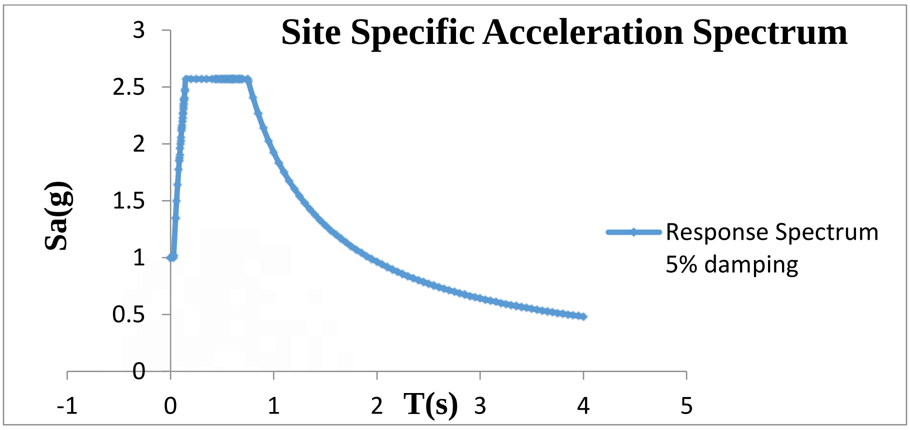

Fig 2. Site Specific Acceleration Spectrum

To convert acceleration spectra to ADRS format, following relation is used as per ATC-40 guidelines. Hence spectral displacement is given by

$$S_d = \frac{Sa * T^2}{4\pi^2}$$

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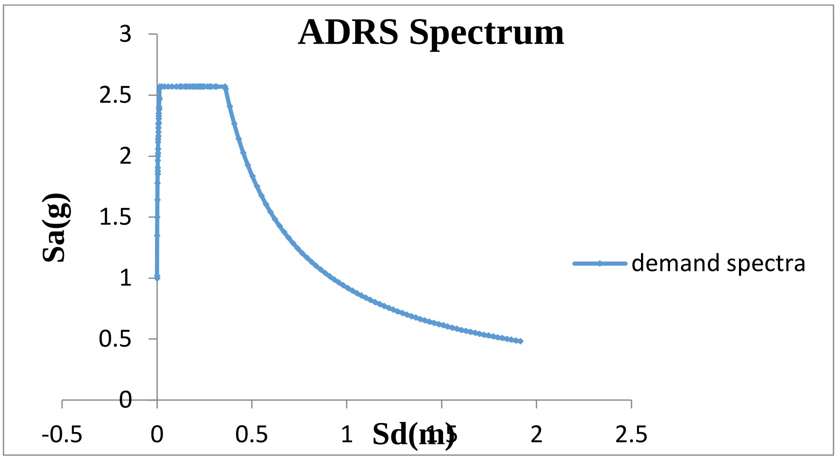


Fig 3. Acceleration Displacement Response Spectrum

4. RESULTS AND DISCUSSIONS

The RCC structures are analysed using SAP2000. The base shear, roof displacements of 4, 8 and 12 stories are obtained for response spectrum method, pushover analysis are plotted for purpose of comparison.

Analysis results

		4-storey		8-storey		12-storey	
StepType	StepNum	Period, sec	UX	Period, sec	UX	Period, sec	UX
Mode	1	0.6203	0.84247	0.987095	0.66	1.495811	0.39754
Mode	2	0.6203	0.00576	0.987095	0.17	1.495811	0.41849
Mode	3	0.57	2.08E-16	0.904633	1.79E-20	1.353848	0
Mode	4	0.4227	1.04E-16	0.508243	4.08E-16	0.555972	1.3E-16
Mode	5	0.3257	0.00334	0.362795	0.000284	0.497833	0.03005
Mode	6	0.3257	2.43E-05	0.362795	0.000174	0.497833	0.07231
Mode	7	0.2572	1.36E-16	0.331378	0.09111	0.451185	2.3E-15
Mode	8	0.2294	4.77E-15	0.331378	0.004808	0.379983	8.3E-05
Mode	9	0.2112	0.00367	0.302836	1.38E-16	0.379983	1.5E-05
Mode	10	0.2112	0.08203	0.282369	6.31E-14	0.377693	1.6E-17
Mode	11	0.203	6.56E-05	0.272649	3.09E-16	0.301559	6.5E-06
Mode	12	0.203	4.02E-05	0.242599	0.002913	0.301559	0.00053

Table 1.Modal properties of the structures

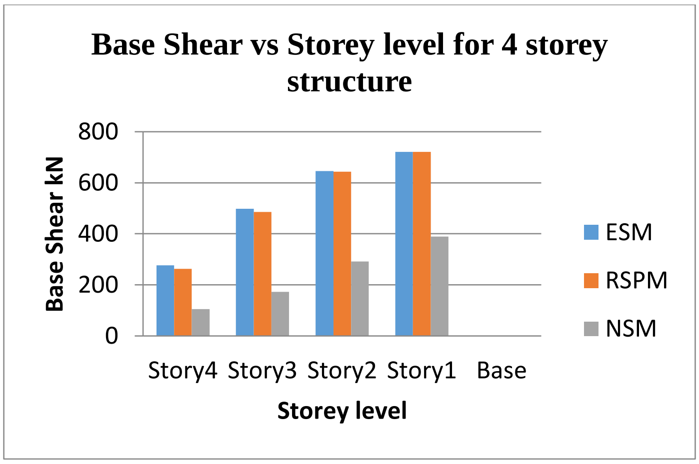


Fig4. Base shear variation vs storey

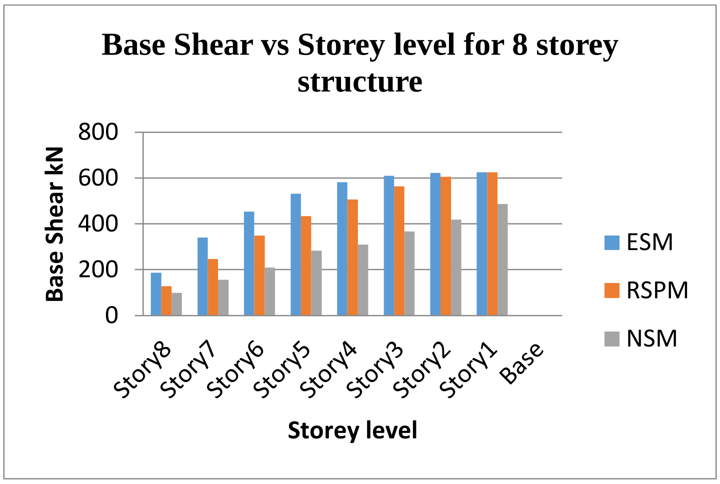


Fig5. Base shear variation vs storey

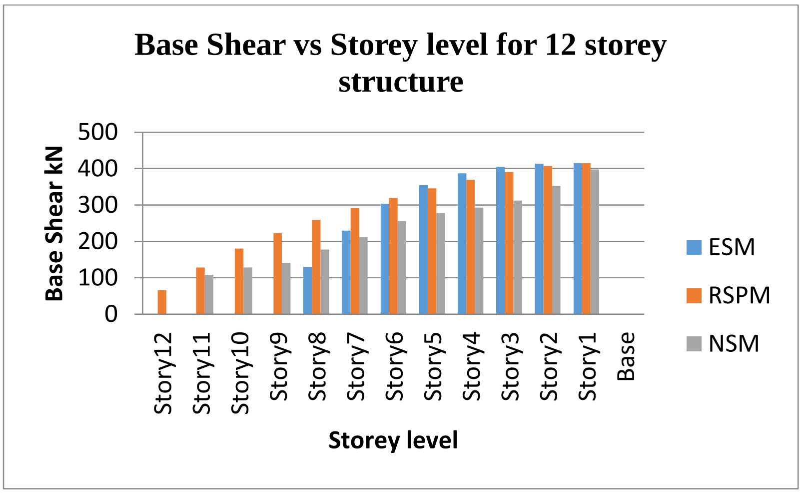


Fig6. Base shear variation vs storey

Displacement	BaseForce
m	KN
0.031273	3113.707
0.042256	3925.926
0.044486	4029.447
0.045922	4077.245
0.045931	4042.097
0.04618	4050.352
0.04619	3929.134
0.047264	4006.019
0.047278	4006.645

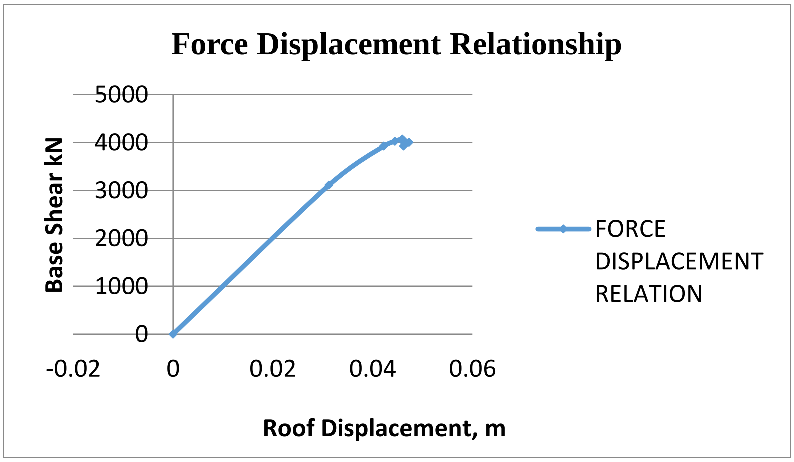


Table 2. Base Shear Vs Roof displacement for 4-storey

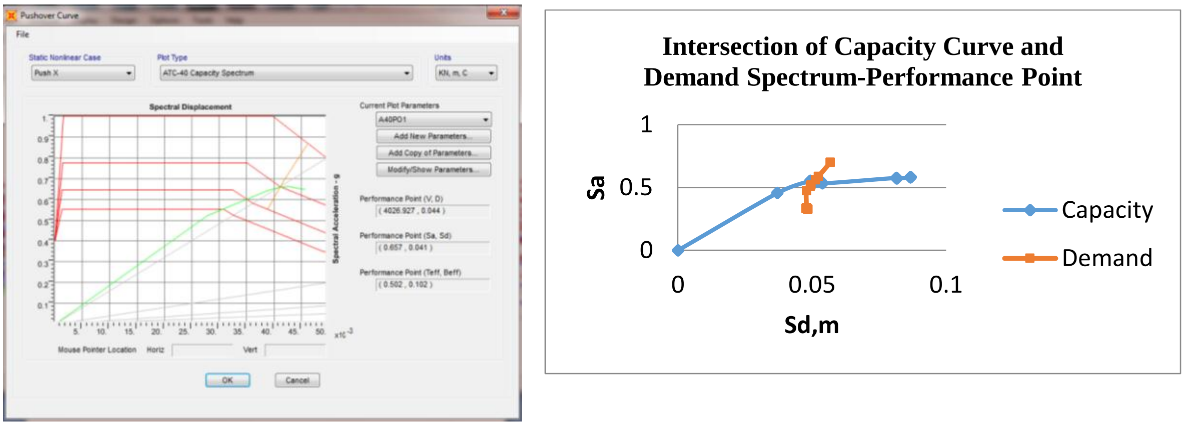


Fig7. Capacity Demand Curve of 4 Storey

Step	Teff	Beff	SdCapacity	SaCapacity	SdDemand	SaDemand
Unitless	Sec	Unitless	m	Unitless	m	Unitless
0	0.463385	0.05	0	0	0.046043	0.863214
1	0.463385	0.05	0.027814	0.521452	0.046043	0.863214
2	0.492804	0.090066	0.038815	0.643407	0.041809	0.693048
3	0.501841	0.101939	0.041116	0.657235	0.041042	0.65604
4	0.50906	0.111674	0.042619	0.662064	0.040486	0.628929
5	0.531073	0.144463	0.045425	0.64838	0.038861	0.554678
6	0.532508	0.146056	0.045718	0.64905	0.038822	0.551136

Table 3. Summary of Capacity and Demand curve as per ATC-40 procedure-4 storey

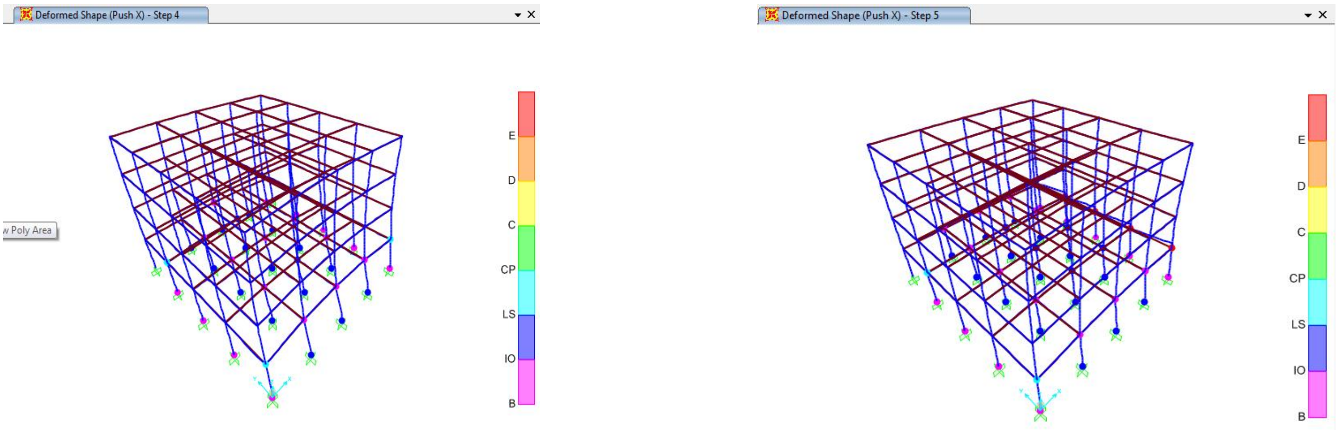


Fig8. Plastic hinge formation corresponding to performance point

- The performance point is 0.66m/s^2 acceleration at 0.0404m displacement with 0.502sec effective time period which lies between 3rd and 4th step.
- The highest plastic hinge in 4th and 5th step obtained is at life safety level.
- Hence the structure is safe for the above specified base shear of 4027KN at roof displacement of 0.0404m
- Therefore minor retrofitting may be required for few beams at lower storey level.

Displacement (m)	Base Force (KN)
2.70E-05	0
0.085896	3838.31
0.101658	4434.98

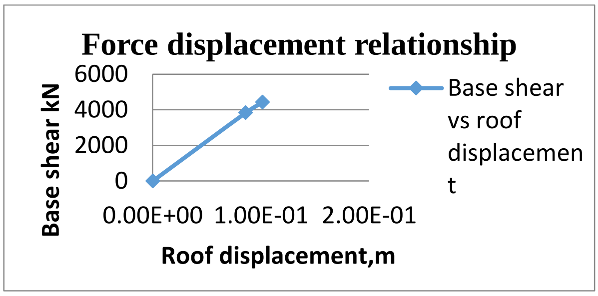
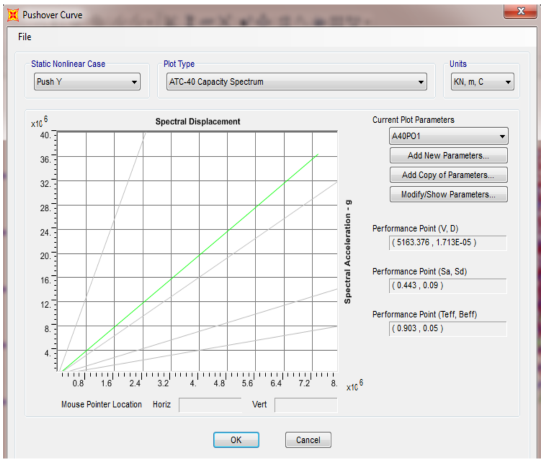


Table 4. Base Shear Vs Roof displacement for 8-storey



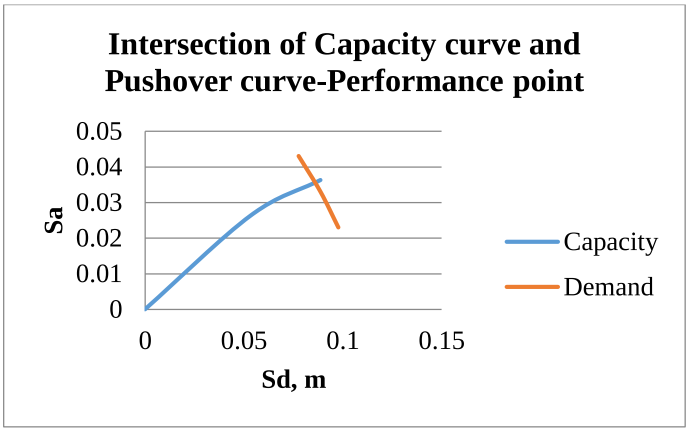


Fig9. Capacity Demand Curve of 8 Storey

Step	Teff	Beff	SdCapacity	SaCapacity	SdDemand	SaDemand
Unitless	Sec	Unitless	m	Unitless	m	Unitless
0	0.902823	0.05	0	0	0.077706	0.043055
1	0.902823	0.05	0.05347166	0.026409349	0.088706	0.033055
2	0.902823	0.05	0.08862825	0.036364573	0.097706	0.023055

Table5. Summary of Capacity and Demand curve of 8 storey as per ATC-40 procedure

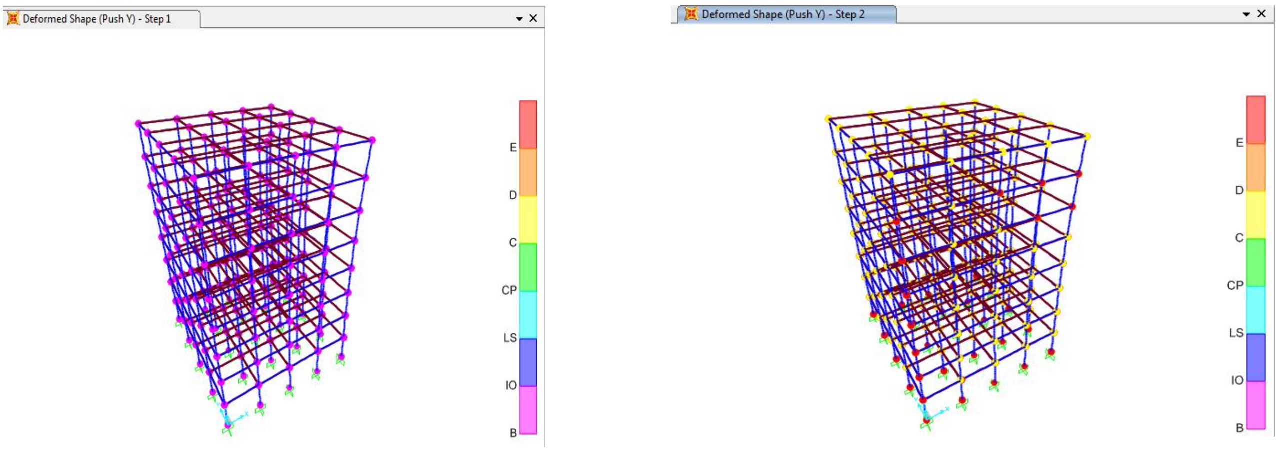


Fig10. Plastic hinge formation corresponding to performance point

- The performance point is $0.033m/s^2$ acceleration at 0.088m displacement with 0.903sec effective time period which lies between 1st and 2nd step.
- The highest plastic hinge in 1st and 2nd step obtained has crossed little bit beyond collapse prevention level.
- Hence the structure is subjected to failure of few peripheral beams and columns for the above specified base shear of 4434KN at roof displacement of 0.101m.
- Therefore most the peripheral beams and columns need to be retrofitted for the revised forces.

International Journal of Engineering Research & Technology (IJERT)
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Displacement	BaseForce
m	KN
0.000041	0
0.231246	6799.436
0.628941	17079.51
0.628948	17030.41
0.634539	17168.95
0.629408	16963.64

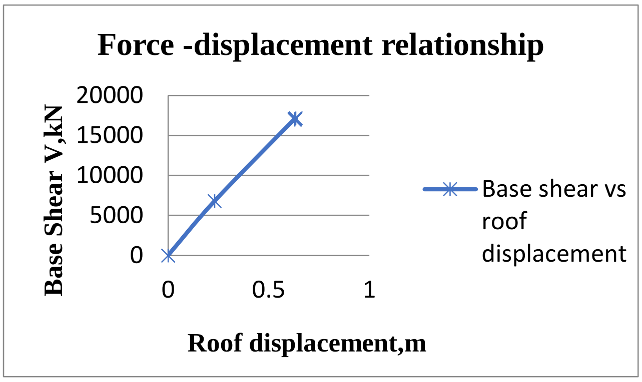
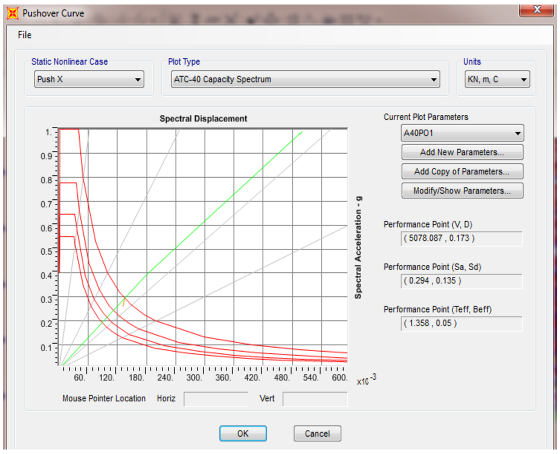


Table 6. Base Shear Vs Roof displacement for 12-storey

Step	Teff	Beff	SdCapacity	SaCapacity	SdDemand	SaDemand
Unitless	Sec	Unitless	M	Unitless	M	Unitless
0	1.358341	0.05	0	0	0.134968	0.294477
1	1.358341	0.05	0.046145	0.100681	0.134968	0.294477
2	1.422072	0.065048	0.125691	0.250208	0.132074	0.262913
3	1.465308	0.071499	0.209953	0.393645	0.132668	0.248741
4	1.487736	0.070395	0.296875	0.53996	0.135271	0.246031
5	1.499725	0.068548	0.373917	0.669254	0.137345	0.245827
6	1.501421	0.069551	0.37396	0.66782	0.136962	0.244587
7	1.502028	0.069168	0.381217	0.680229	0.137222	0.244854

Table7. Summary of Capacity and Demand curve of 8 storey as per ATC-40 procedure



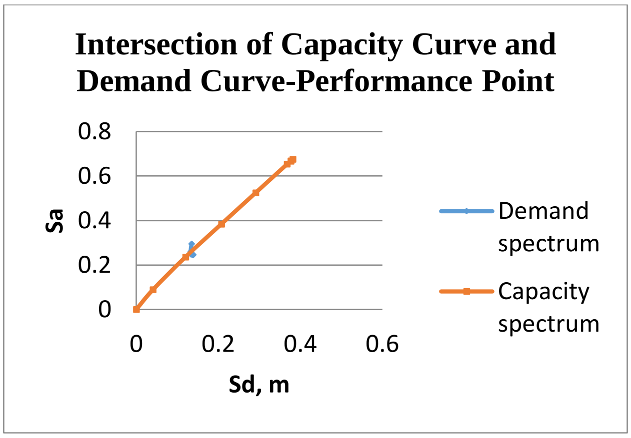


Fig11. Capacity Demand Curve of 12 storey

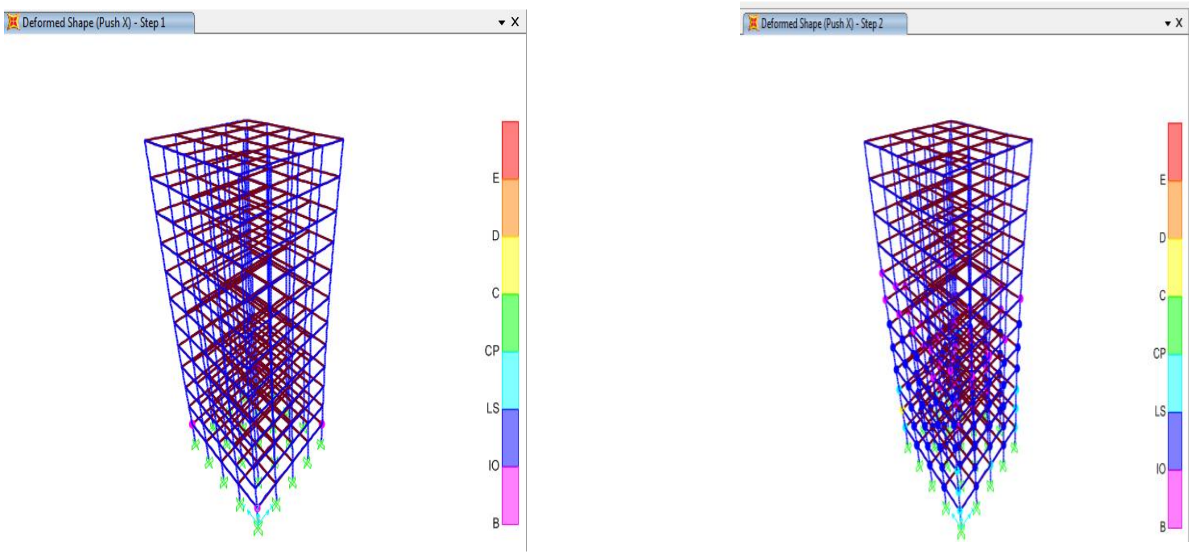


Fig12. Plastic hinge formation corresponding to performance point

- The performance point is 0.294m/s^2 acceleration at 0.134m roof displacement with 1.358sec effective time period which lies between 1st and 2nd step.
- The highest plastic hinge in 1st and 2nd step obtained is between Operational and Immediate occupancy zone. Hence there won't be localized collapse for this level of earthquake.
- Hence the structure is safe for the above specified base shear of 16963KN at roof displacement of 0.629m.

5. CONCLUSIONS

The results have shown clear information about the evaluation methods which can be concluded as follows:

1. The base shear obtained from equivalent static and response spectrum is more than that of the pushover method of analysis.
2. In both 4 & 8 storey structures, performance point is figured in the non linear region. Therefore elastic method of assessment doesn't hold good for seismic evaluation of structures in severe ground motions.
3. The 4 storey structure is in "life safety level" after locating the performance point. Therefore damages may occur in the non-structural members but serviceable.
4. In 8 storey structure, few hinges have crossed "collapse prevention level" i.e large damage to structural members, therefore its not serviceable and requires major retrofitting to structural elements.
5. The performance point in 12 storey structure is figured in the elastic region which shows more strength and stiffness towards lateral loading.
6. In 12 storeyed structure most of the plastic hinges generated are in "Immediate occupancy level i.e less damage but serviceable". Hence no retrofitting is required.
7. From the results of pushover analysis, 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. The above results have showed that intersection of demand curve with capacity curve near the elastic range, the structure has a good resistance and high safety against collapse.
9. Intersection of demand and capacity curve indicates that the properly detailed reinforced concrete frame building is adequate

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