PERFORMANCE BASED ANALYSIS OF 10 STOREY, 5x5 BAY MODEL WITH fixed base subjected to different monitored displacement

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ABSTRACT

Pushover analysis is usually performed to find the week points and capacity of the building. The hinges shows various failure state of the building whereas, the capacity curve shows the capacity of the building. In the present study a 5x5 bay, 10 storey building with fixed base is analyzed for different monitored displacement. Indian standard code IS 1893-(Part-1):2002 and American standard ASCE 41-13 are referred for nonlinear analysis. Totally 6 models are analyzed using SAP2000 V19.2.1 software. The model is pushed for a monitored displacement of 0.1m, 0.3m,0.5m 1.0m and 4.0m. The result shows displacement, base force and hinge formation at different locations for different monitored displacement. The study gives clear idea about the safety state and failure state of the model. It also indicates the variations occurring in the behavior of the model when it is pushed for different monitored displacement.

Keywords: *Nonlinear analysis , Fixed base, Monitored displacement.*

1. INTRODUCTION

One of the promising field in seismic design of structures is the Performance Based Design. Seismic design is transforming from a phase of linear elastic analysis design, to a phase of non-linear analysis, which influences the seismic design as a whole.

"The basis for the linear approach lies in the perception of the Response Reduction factor R. When a building is designed for a Response Reduction factor of, say, R = 5, it means that only 1/5th of the seismic force is taken by the limit state capacity of the structure. Further deflection is in its ductile behavior and is taken by the ductile capacity of the structure. In reinforced concrete (RC) structures, the members (ie., beams and columns) are detailed to make sure that the structure can take the full impact without collapse beyond its limit state capacity up to its ductile capacity.

The drawback is that the response beyond the limit state is neither a simple, nor a perfectly ductile behavior with predeterminable deformation capacity. This is due to various reasons such as change in stiffness of member due to cracking and yielding, P-delta effects, change in the final seismic force estimated, etc. Although elastic analysis gives a good indication of elastic capacity of structures and shows where yielding might first occur, it cannot account for redistribution of forces during the progressive yielding that follows and predict its failure mechanism, or detect possibility and location of any premature failure. A non-linear static analysis can predict these more accurately since it considers the inelastic behavior of the structure. It can help identify critical members likely to reach critical states during an earthquake for which attention should be given during design and detailing.

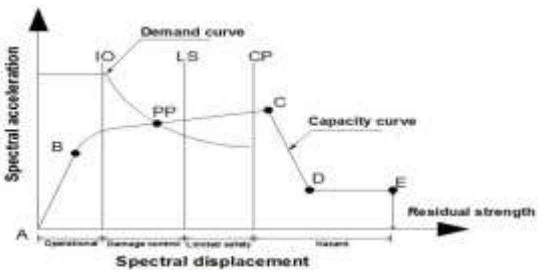

Fig.1.Pushover curve

In Fig.1. AB represents the linear elastic range from unloaded state A to its effective yield B, followed by an inelastic but linear response of reduced (ductile) stiffness from B to C. CD shows a sudden reduction in load resistance, followed by a reduced resistance from D to E, and finally a total loss of resistance from E to F. Hinges are inserted in the structural members of a framed structure. These hinges have non-linear states defined as 'Immediate Occupancy' (IO), 'Life Safety' (LS) and 'Collapse Prevention' (CP) within its ductile range. This is usually done by dividing B-C into four parts and denoting IO, LS and CP, which are states of each individual hinges (in spite of the fact that the structure as a whole too have these states defined by drift limits). There are different criteria for dividing the segment BC. For instance, one such specification is at 10%, 60%, and 90% of the segment BC for IO, LS and CP respectively (courtesy Inel & Ozmen, 2006).
One of the method adopted is
Capacity Spectrum Method (CSM) of ATC-40, here the load is incremented and checked at each stage, until what is called the 'Performance Point' is reached.. Equations developed (ATC-40, FEMA 440) to arrive at this 'equivalent' damping ratio β (see Appendix), and also time period T (both continuously changing due to the weakening of hinges in course of the analysis) at any particular point in the course of the progress of the analysis, having known only the instantaneous $\Delta_{\text{roof top}}$ and V_b of the structure.
The Acceleration Displacement Response spectra(ADRS): Once the Performance Point is found, the overall performance of the structure can be checked to see whether it matches the required performance level of IO, LS or CP, based on drift limits specified in ATC-40 which are 0.01h, 0.02h and 0.33(Vb/W)·h respectively (h being the height of the building). The performance level is based on the importance and function of the building. For example, hospitals and emergency service buildings are expected to meet a performance level of IO. In fact these limits are more stringent than those specified in IS:1893-2002. The 'Limit State' drifts of 0.004 specified in the latter, when accounted for R (= 5 for ductile design) and I (taken as 1.5 for important structures which demand an IO performance level) gives 0.004·R/I = 0.0133, which is more relaxed than the 0.01 allowed in ATC-40. This 0.004·R/I may be taken as the IS:1893-2002 limits for pushover drift, where I takes the value corresponding to Important and Ordinary structures for limits of IO and LS respectively.

Pushover analysis, is for checking how much load (when using force controlled push over) or displacement (when using displacement controlled pushover) in either X or Y direction can a building take under monotonically increasing load or displacement. In push over analysis, the structure should be loaded to full collapse, if you use displacement controlled push over, you should be sure that target displacement is large enough to cause full collapse.
Spectral acceleration (SA) is a unit measured in g (the acceleration due to Earth's gravity, equivalent to g-force) that describes the maximum acceleration in an earthquake on an object specifically a damped, harmonic oscillator moving in one physical dimension. This can be measured at (or specified for) different oscillation frequencies and with different degrees of damping, although 5% damping is commonly applied. The SA at different frequencies may be plotted to form a response spectrum.
Spectral acceleration, with a value related to the natural frequency of vibration of the building, is used in earthquake engineering and gives a closer approximation to the motion of a building or other structure in an earthquake than the peak ground acceleration value, although there is normally a correlation between [short period] SA and PGA. Some seismic hazard maps are also produced using spectral acceleration. Spectral acceleration (SA) PGA (peak acceleration) is what is experienced by a particle on the ground.
SA(spectral acceleration) is approximately what is experienced by a building, as modeled by a particle on a mass vertical rod having the same natural period of vibration as the building.
Peak Ground Acceleration (PGA) is equal to the maximum ground acceleration that occurred during earthquake shaking at a location. PGA is equal to the amplitude of the largest absolute acceleration recorded on an accelerogram at a site during a particular earthquake. Earthquake shaking generally occurs in all three directions. Therefore, PGA is often split into the horizontal and vertical components. Horizontal PGAs are generally larger than those in the vertical direction but this is not always true, especially close to large earthquakes. PGA is an important parameter (also known as an intensity measure) for earthquake engineering, The design basis earthquake ground motion (DBEGM) is often defined in terms of PGA.
Displacement-based seismic design and assessment of structures require the reliable definition of displacement spectra for a wide range of periods and damping levels. The displacement spectra derived from acceleration spectra in existing seismic codes do not provide a suitable answer and there are no existing frequency-dependent attenuation relationships derived specifically for this purpose."

2.METHODOLOGY

In the present study, Nonlinear analysis is performed on Bare frame with fixed base for different monitored displacement of 0.1m, 0.3m, 0.5m, 1m and 4.0m in order to observe the changes in the behavior of the building. (The maximum allowable displacement is 0.12m (as per IS code IS 1893:2002, 0.004H given in clause no. 7.11.1 Page no.27), but for Pushover analysis the maximum monitored displacement is obtained by trail analysis until beyond E point is reached in pushover curve.)

3. RESULT AND DISCUSSION

The results of Pushover analysis is performed for various monitored displacement.(0.1m to 4.0m)
Below given is the discussion on monitored displacement of 0.1m, the output is given by the capacity of the building (in terms of base force and displacement) .

3.1. Pushover Capacity curve

The Table 1.1 and Fig.1.2 shows Pushover capacity results for a monitored displacement of 0.10 m. The curve shows the plot of Base shear in 'kN' along Y axis and Displacement in 'm' along X-axis.

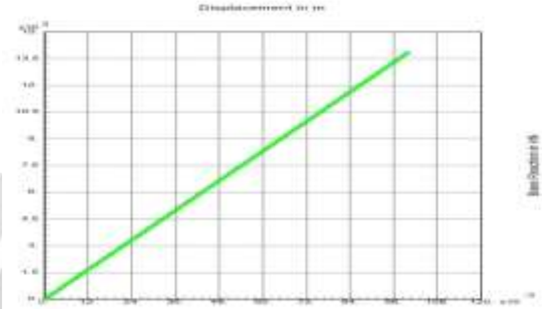

Fig.1.2 Pushover capacity curve-0.1m - Monitored Displacement

The shape of the curve obtained is linear (where stress is proportional to strain). The results of the curve are tabulated in Table 1.1 with total number 1920 hinges lying in A-B state. The curve is Linear Elastic from unloaded state A to its effective yield state B. The maximum displacement is 0.1m with a Base Force of 13869.51kN when the building is pushed for a monitored displacement of 0.1m. It is observed that all the hinges lie in A-B state and zero hinges in other states of failure. This indicates that the building is within elastic limit. This shows the building doesn't need retrofitting.(note: displacement is within limit 0.12m specified by IS code 1893:2002)

Table: 1.1 Pushover Capacity Curve - 0.1m											
Step	Displace 'm'	Base .Force 'kN'	AB	B IO	IO LS	LS CP	CP C	C D	D E	Bey E	Total
0	2.80E-05	0	1920	0	0	0	0	0	0	0	1920
1	0.00015	17.337	1920	0	0	0	0	0	0	0	1920
800	0.10003	13869.514	1920	0	0	0	0	0	0	0	1920

3.2. ATC40 - Demand Capacity spectrum

The Fig.1.3 shows Demand Capacity curve with Performance point. The Table 1.2 shows Demand Capacity values for a monitored displacement of 0.1m.

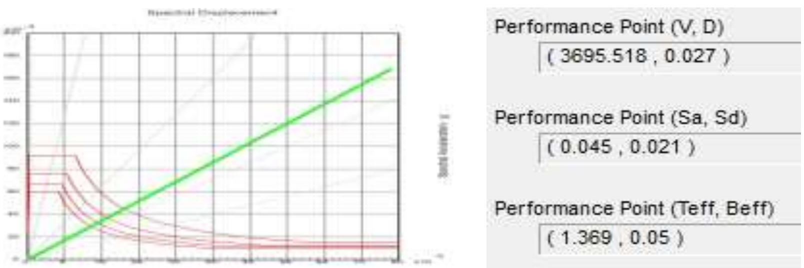

Fig. 1.3 Demand - Capacity curve with Performance point

The graph shows performance point magnitude obtained for the values of Spectral Acceleration demand (Sa-

0.045), Spectral Displacement demand (Sd-0.021), Effective Time Period (Teff -1.369sec) and Effective Damping (Beff-0.05). The Base shear divided by displacement at performance point gives the stiffness of the building.

Table :1.2 Pushover Curve Demand Capacity - ATC40

Step	Teff ‘Sec’	β	Sd Capacity ‘m’	Sa Capacity	Sd Demand ‘m’	Sa Demand
0	1.3688	0.0500	0.0000	0.0000	0.0209	0.0449
1			0.0001	0.0002	0.0209	0.0449
213			0.0209	0.0449	0.0209	0.0449
214	1.3688	0.0500	0.0210	0.0451	0.0209	0.0449
799	1.3688	0.0500	0.0784	0.1684	0.0209	0.0449
800			0.0785	0.1686	0.0209	0.0449

It is observed that the Effective Time period (Teff), Spectral Displacement Demand (Sd) and Spectral Acceleration Demand (Sa) is same up to step 800. The Spectral Displacement Capacity (Sd) and Spectral Acceleration Capacity (Sa) increases up to step 213. At step 213 Capacity is almost equal to Demand. This point where the Demand and Capacity are same is Performance point. The results show that as the capacity curve is linear and there is no hinge formation in the structure. Hence the structure does not require any retrofitting.

Deformed shape of building: As observed in Fig.1.3 the curve is straight and linear. The Fig.1.4 shows deformed shape of the building at step 800 for monitored displacement of 0.1m and there is no hinges observed in the building. The deformed shape for monitored displacement of 0.1m shows no hinge formation

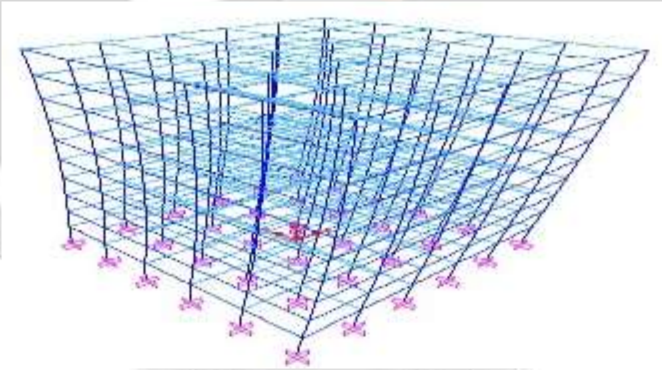

Fig.1.4 step 800(A-B)State -0 Hinges

3.3. COMPARISON

Comparison of different monitored displacement

In previous discussion was given individually about the monitored displacement of 0.1m was discussed.. If the monitored displacement is increased by 0.1m, 0.3m, 0.5m, 1.0m and 4.0m what is the changes in behavior of the building. What is the difference in displacement the building undergoes with maximum load carrying capacity is discussed. The building is pushed up to a monitored displacement where the hinges are formed in all failure state. (Beyond E state). The Pushover capacity curve of various monitored displacement is given as shown in Fig.1.5 to Fig.1.9.

It can be observed that there is a change in base force when there is a change in monitored displacement, however in some case there is sudden decrease in base force value, which indicates that the structure no longer has the ability to take load and ultimately collapse state is reached if the application of displacement is continued.

As seen in Fig.1.5 the curve is linear for a monitored displacement of 0.1m and the program me terminates reaching the allocated monitored displacement of 0.1m and assigned steps of 800. The linear curve shows that no hinge formation has taken place and the building has more capacity.

As the building is pushed to a monitored displacement of 0.3m as shown in Fig.1.6, The curve is initially linear which matches with the curve initially obtained for MD of 0.1m, till the displacement is proportional to the base force. However the curve drops down after reaching the max. load carrying capacity. The programme terminates after reaching allocated monitored displacement of 0.3m with assigned total steps of 800. At this stage the hinges are formed only till C-D state. However the building has not reached total collapse state (Beyond E). Further the building is analyzed for a monitored displacement of 0.5m, and curve is shown in Fig.1.7. The curve shape is similar to that obtained earlier however it drops further till it reaches displacement of 0.5m. The max. displacement of 0.5m is achieved at 795 steps itself where the program terminates without reaching maximum of 800 steps. It is also worth to note that more number of hinge is formed in C-D state only and still the building has not reached total collapse state (Beyond E).

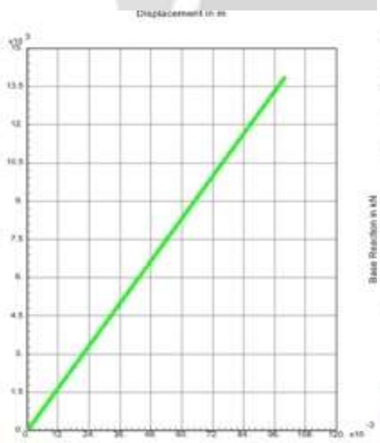

Fig. 1.5 - 0.1m

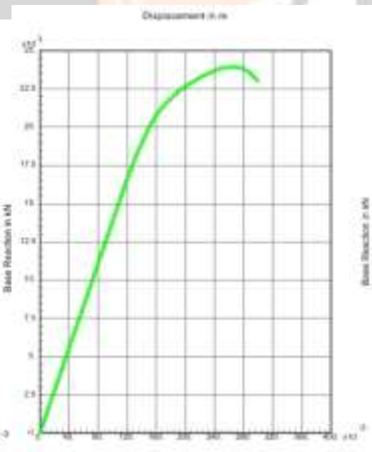

Fig. 1.6 - 0.3m

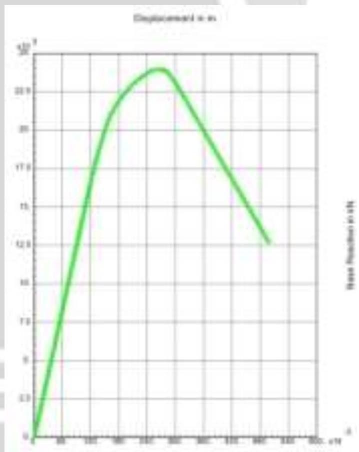

Fig. 1.7 - 0.5m

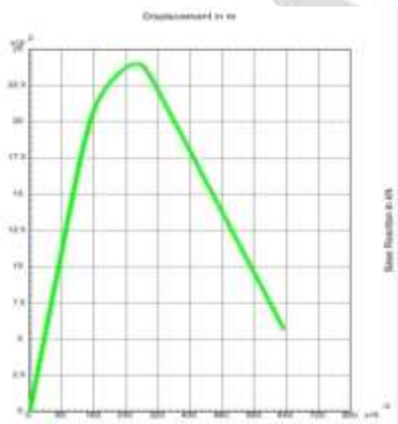

Fig. 1.8 - 1.0m

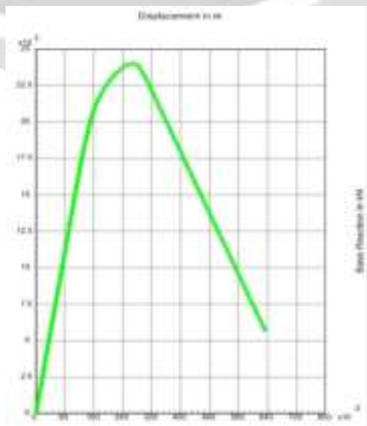

Fig. 1.9 - 4.0m

The Fig.1.8 shows the pushover capacity curve for a monitored displacement of 1.0m where the program terminates for a maximum displacement of 0.634m at 499 steps without reaching monitored displacement of 1.0m and without completing the total number of 800 steps assigned. This is because the hinge formation occurs beyond E state at 499 steps.

The Fig.1.9 shows the pushover capacity curve for a monitored displacement of 4.0m where the curve bends fully and program me terminates for a same displacement of 0.634m at step124 itself without reaching the allocated monitored displacement of 4.0m and without completing the total number of 800 steps assigned. This is again because the hinges have occurred Beyond E state at 124 steps.

Performance point for different monitored displacement :The performance point for different monitored displacement is given the Table 1.3

It is observed that except in case of monitored displacement of 0.10 m almost same results of displacement and base force is noticed in all the monitored displacement at failure states up to LS-CP. The result shows that the building reaches a state of collapse for a maximum displacement of 0.194m and base force of 22534kN for monitored displacement of 0.3m,0.5m ,1m to 4.0m. It is also noticed that for a monitored displacement of 0.30m the analysis terminates at C-D failure state reaching maximum displacement of 0.3m with max. base force of 23020kN. For a monitored displacement of 0.5m the analysis terminates at C-D state reaching a maximum displacement of 0.5m with decrease in base force of 12629kN but however, the max. base force is 23940.31kN for monitored displacement of 0.5m. it is observed that the analysis beyond E point is obtained only when the monitored displacement is 1.0m and 4.0m for a displacement of 0.634m with reduction in base force. Therefore further analysis of the building is carried out keeping the monitored displacement as 4.0m so that the complete displacement and base force curve is obtained beyond E failure state.

Table 1.3 Linear and Nonlinear analysis results for different monitored displacement.

Base Condition	Linear Analysis(ESLM)		Nonlinear analysis													
			Monitored Displacement 'm'	At P. point		B-IO		IO-LS		LS-CP		C-D		Beyond E	
	Max. Displacement 'm'	Max. Base Force ‘kN’		Max. Displacement 'm'	Max. Base Force 'kN'	Max. Displacement 'm'	Max. Base Force 'kN’	Max. Displacement 'm'	Max. Base Force 'kN’	Max. Displacement 'm'	Max. Base Force 'kN'	Max. Displacement 'm'	Max. Base Force 'kN'	Max. Displacement 'm'	Max. Base Force 'kN'
BF Regular building Fixed base	0.148	9183	0.10	0.027	3695	0.100	13869	-	-	-	-	-	-	-	-
			0.30	0.027	3695	0.165	21114	0.188	22203	0.197	22549	0.300	23020	-	-
			0.50	0.027	3695	0.166	21164	0.188	22218	0.196	22556	0.500	12629	-	-
			1.00	0.027	3695	0.165	21156	0.188	22247	0.196	22585	0.615	6646	0.634	5658
			4.00	0.027	3695	0.166	21206	0.187	22250	0.194	22534	0.615	6671	0.635	5667

4. ATC 40 Demand Capacity curve

Different monitored displacement in case of Bare frame with Fixed base for soft soil. The Fig.1.9 to Fig. 1.13 shows performance point obtained for different monitored displacement of 0.1m,0.3m,0.5m, 1.0m and 4.0m. The Table 1.4 shows demand capacity values for different monitored displacement after performance point.

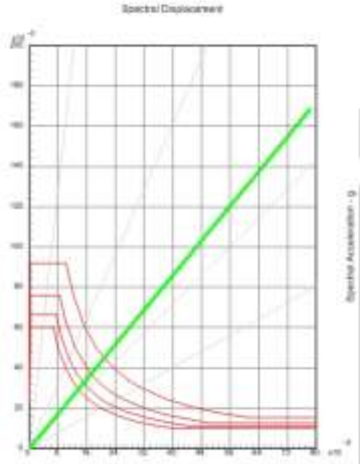

Fig.1.90 - 0.1m

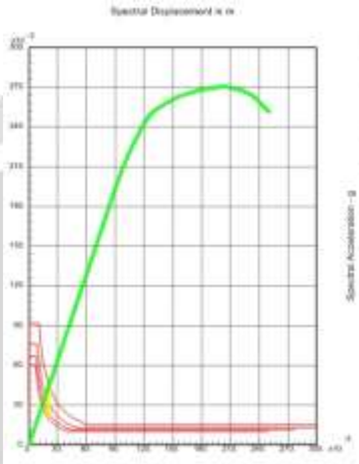

Fig. 1.10 - 0.3m

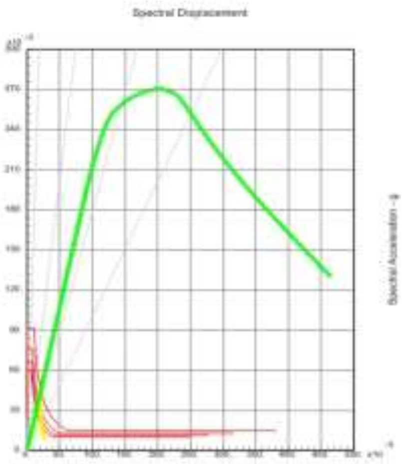

Fig. 1.11 - 0.5m

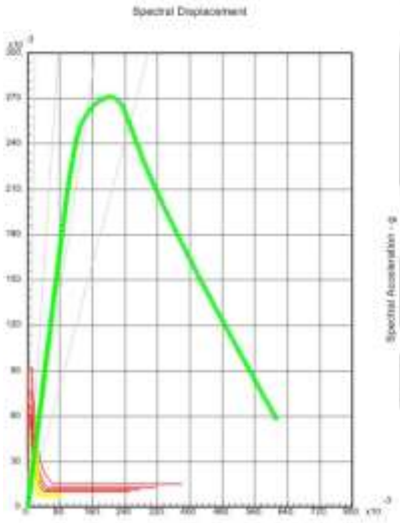

Fig. 1.12 - 1.0m

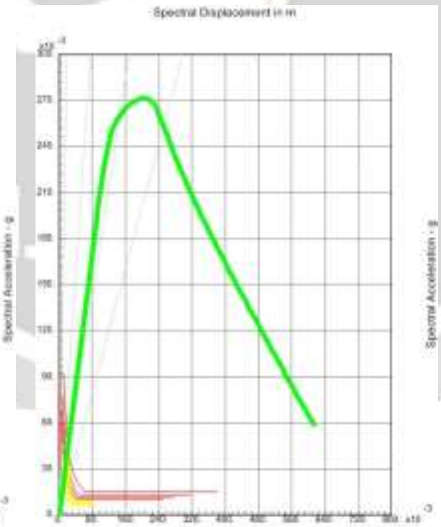

Fig. 1.13 - 4.0m

The Fig.1.9 to Fig.1.13 shows the performance point is reached in between effective time period line between 1.0sec and 1.5sec. The yellow line indicates the performance point where both capacity curve and demand curve meets.

Values for different Monitored Displacement corresponding to Max. Base force

Max. Base force obtained for different monitored displacement considered is given in Table 1.5 and the other data indicated are corresponding values of max. base force.

Table: 1.5 Pushover Capacity Magnitude for different Monitored Displacement (MD) corresponding to Max. Base force

MD	Step	Displacement 'm'	Base Force 'kN'	AB	B IO	IOLS	LSCP	CPC	CD	DE	BeyE	Total
0.1	800	0.100	13869.51	1920	0	0	0	0	0	0	0	1920
0.3	716	0.269	23917.37	1554	220	68	4	0	74	0	0	1920
0.5	425	0.269	23940.31	1554	220	68	6	0	72	0	0	1920
1.0	209	0.270	23994.77	1554	220	68	4	0	74	0	0	1920
4.0	48	0.271	24042.36	1554	222	66	4	0	74	0	0	1920

It is observed that even though the monitored displacement increases the max. base force and the max. displacement is almost same in all case except for 0.10m this is because, the monitored displacement value is very much less than max. displacement value. It is observed that the building can undergo a max.displacement of 0.271m no matter what the increase in max. base force is. The hinge reaches failure state of C-D with number of hinges same for different monitored displacement.

Values for different Monitored Displacement corresponding to Max. Displacement

Table:1.6 Pushover Capacity Magnitude for different Monitored Displacement corresponding to Max. Displacement

MD	Step	Displace 'm'	B.Force 'kN	AB	B IO	IO LS	LS CP	CPC	CD	DE	Bey E	Total
0.1	800	0.100	13869.51	1920	0	0	0	0	0	0	0	1920
0.3	800	0.300	23019.52	1554	212	56	4	0	94	0	0	1920
0.5	795	0.500	12629.31	1554	212	50	4	0	100	0	0	1920
1.0	499	0.634	5657.670	1554	210	52	2	0	94	2	6	1920
4.0	124	0.635	5666.870	1554	212	50	4	0	92	2	6	1920

It is observed that for a monitored displacement of 0.1m, 0.3m and 0.5m the max.displacement reaches assigned monitored displacement value when the last step is reached. Further for monitored displacement of 1.0m and 4.0m the displacement does not reach assigned monitored displacement values because the model reaches the state Beyond E (Total collapse) for a displacement of 0.634m. Table 1.6, it is observed that as max.displacement increases the base force also increases up to monitored displacement of 0.3m later, for the monitored displacement of 0.5m, 1.0m and 4.0m the Base force decreases after reaching maximum value indicating that the building can no longer take up load. Also as max.displacement increases the hinge formation reaches failure state beyond E. The maximum value of displacement reached is noticed at last step of analysis which is very much less than maximum steps assigned.

For monitored displacement of 0.1m the structure remains in elastic state as no hinge formation is observed. For monitored displacement of 0.3m and 0.5m the base force decreases and building shifts to inelastic state with hinge formation in C-D failure state (elasto plastic state). However, in case of monitored displacement of 1.0m and 4.0m the required monitored displacement is not reached as it has reaches failure state E for less value of displacement.

For monitored displacement of 4.0m the programme terminates at step 124 itself and does not reach assigned 800 steps to perform the analysis. It Is also observed that the analysis terminated for a displacement of 0.634 m with a Base force of 5666.87 kN. This is because of the fact that the hinges provided allows the load to drop hence the analysis shows reduction in base force with hinges formed in C-D, D-E and Beyond E state.

Deformed shape of the building for monitored displacement of 0.3m

Fig.1.14 shows the monitored displacement of 0.3m from step 309 upto 800 step. The hinges reaching various failure stages are represented by various colours. It can be seen that the hinge formation is found more in number at bottom half of the building. The hinges formed in each state are discussed. Fig.1.14 shows the building lies in (B-IO) state at Step 309 with 4 hinges formed in first floor last bay beams indicated by green colour. Fig.1.15 at step 441 the hinges lie in (IO-LS) state with 2 hinges found in the middle core denoted by yellow colour. Fig.1.16 at step 501 with 4 hinges in (LS-CP) state in middle core denoted by red colour. Fig. 1.17 at step 525 lies in (C-D) state with 6 hinge formed in middle core denoted by blue colour. Fig.1.18 shows deformed shape at step 800 C-D state with 94 hinges formed denoted by dark blue colour at all the columns supports.

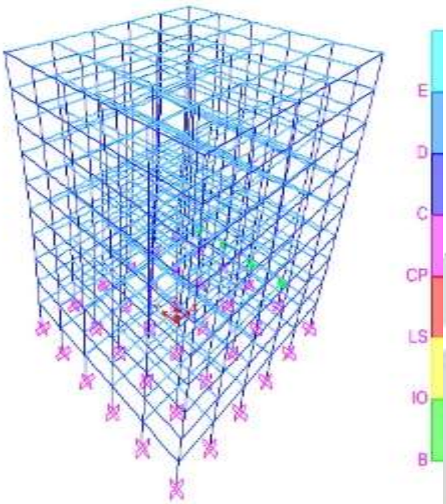

Fig. 1.14 Step 309 (B-IO) state
- 4 hinge

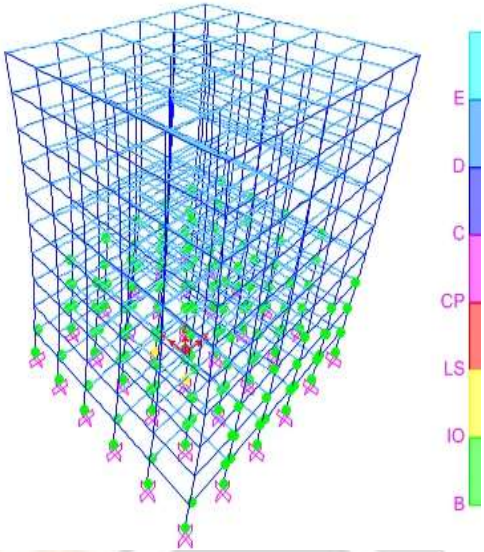

Fig. 1.15 Step 441 (IO-LS) state
-2 hinge

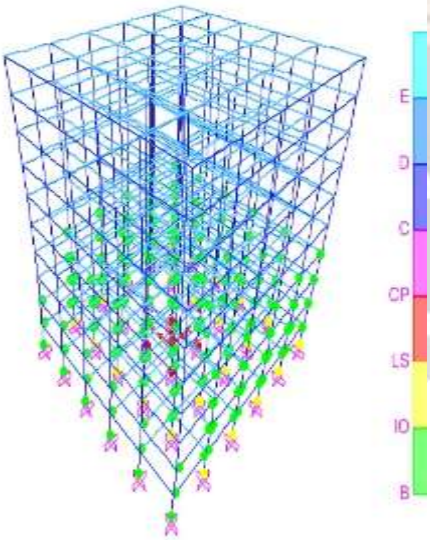

Fig. 1.16 Step 501 (LS-CP) state
-4 hinge

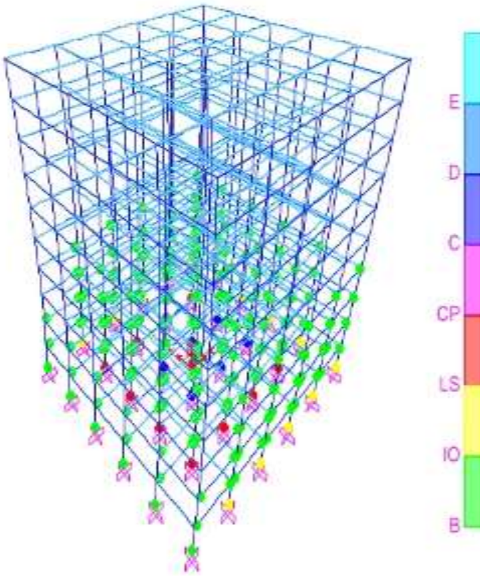

Fig. 1.17 Step 525 (C-D) state
- 6 hinge formation

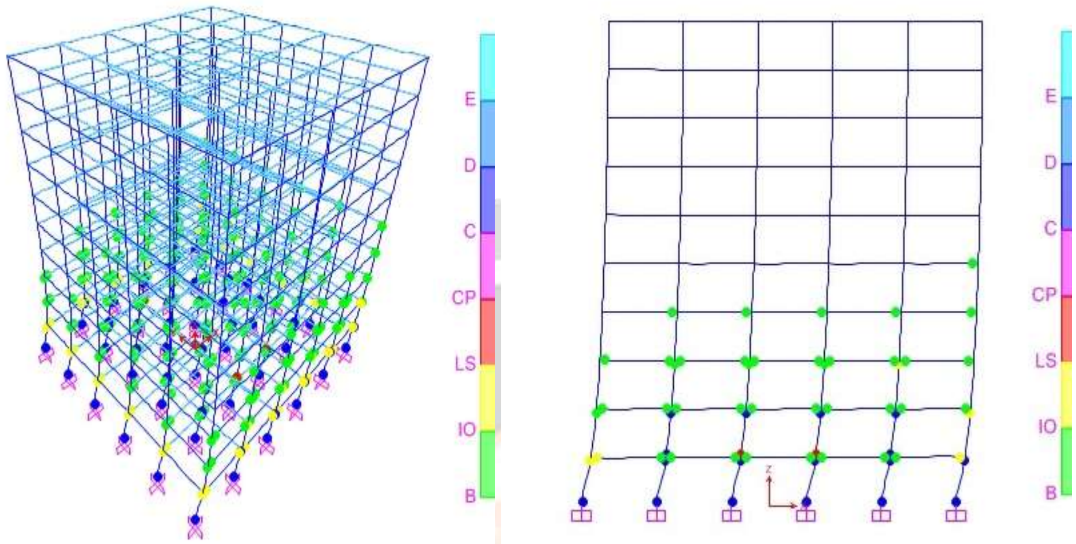

Fig. 1.18 Deformed shape at step 800 (C-D) state -94 hinges

5. CONCLUSION

1. The above discussion indicates that it is important to assign monitored displacement while performing nonlinear analysis.
2. The result shows that for monitored displacement of 0.1m the building is safe and is within elastic limit.
3. Results also indicate that as the building is pushed further up to beyond E state, the formation of hinges starts.
4. The hinge locations give a idea of weaker sections which requires retrofitting as observed in the results. It shows that as the push of building is greater than 0.1m it weakens sections of the building.

6. REFERENCES

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