Damage Assessment of Multistoried Structures under Seismic Loading using Pushover Analysis

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Abstract- ***The buildings which do not fulfill the requirements of seismic design may be affected by either damage or collapse if shaken by a severe ground motion which results in huge economic and loss of life. A building has the potential to wave back and forth during an earthquake and severe wind storm. This is called fundamental mode, and is the lowest frequency of building response. Most buildings, however, have higher modes of response, which are uniquely activated depending up on the intensity of earthquakes. The purpose of this paper is to assess the damage and to evaluate the performance of the structures which are already designed and analyzed using linear static analysis for seismic loads as per the Indian codes IS-456, IS-1893 and IS-13920. It is proposed to study the performance of the structure before and after the linear state. To make such assessment, simplified linear-elastic methods are not adequate. Thus, the structural engineering community has developed a new generation of design and seismic procedures (ATC-40, FEMA-356 and FEMA-440) that incorporate performance based structures and is moving away from simplified linear elastic methods and towards a more non-linear technique i.e., Pushover analysis which is a series of incremental static analysis. It is carried out on the 12-storied building modal which was designed and analyzed for the earthquake analysis using STAAD for two seismic load cases (Zone-3 and Zone-5) considering both are Special Moment Resisting Frames. Pushover analysis is propounded to perform by SAP to get the extent of damage experienced by the structure at target displacement by the sequence of yielding of components, plastic hinge formation and failure of various structural components. Finally both the frames which were designed to linear static analysis for earthquake loading performed well and the damage is within the limits. Initially, yielding of the beams taken place then yielding of columns. This shows that the analysis theory is based on the strong column and weak beam i.e., both the frames behaving as ductile frames.***

Keywords—fundamental mode; linear static analysis; Non-linear analysis; pushover analysis; Target displacement ; plastic hinge; ductile frame.

1. INTRODUCTION

Indian subcontinent experienced severe earthquakes in the past decades. The major reason for the high frequency and intensity of the earthquakes is that the Indian plate is driving into Asia at a rate of approximately 47 mm per year. Geographical statistics of India shows that almost 54 percent of land is vulnerable to earthquakes. World Bank and United Nations report shows estimates around 200 million city dwellers in India will be exposed to storms and earthquakes by 2050.

The latest version of seismic zoning map of India given in the earthquake resistant design code of India [IS 1893 (part 1) 2002] assigns four levels of seismicity for India in terms of zone factors. In other words, the earthquake zoning map of India divides India into 4 seismic zones (Zone 2, 3, 4, 5). Zone 5 expects the highest level of seismicity whereas Zone 2 is associated with the lowest level of seismicity. The zone factors for the different zones are as follows:

TABLE-I

ZONE FACTORS

S.NO	ZONE	Zone Factor
1.	Zone 2	0.10
2.	Zone3	0.16
3.	Zone 4	0.24
4.	Zone 5	0.36

Seismic analysis is a subset of structural analysis and is the calculation of the response of a building structure to earthquakes. It is part of the process of structural design, earthquake engineering and retrofit in regions where earthquakes are prevalent. Structural analysis methods are classified into following five categories:

a). Linear Static Analysis:

Linear static analysis or Equivalent static analysis can only be used for regular structure with limited height. Elastic analysis gives a good indication of the elasticity capacity of the structures though it cannot predict failure mechanisms but indicates where first yielding occurs. Design forces that are acquired from elastic spectrum are reduced using response modification factor. The larger the value of modification factor, the larger will be the level of energy absorption, resulting in formation of more number of plastic joints.

b). Response Spectrum Method (Dynamic Analysis):

This is an approach to find earthquake response of structures using waves or vibration mode shapes. This method comes under linear dynamic analysis. This method is usually used in conjunction with a response spectrum. The mathematical principles of oscillations in n-degree of freedom systems were adopted from Rayleigh theories. The structures response is determined by mass and stiffness distributions. The stiff building will experience low accelerations relative to the ground. Tall buildings accelerate away from the ground motions.

C.) Time-History Analysis (linear and non-linear):

Time history method of analysis uses appropriate ground motion and shall be performed using accepted principle of dynamics. This is the most rational method available for assessing building performance. There are computer programs available to perform this type of analysis.

d.) Push over Analysis:

The pushover analysis of a structure is a static non-linear analysis under permanent vertical loads and gradually increasing lateral loads. The equivalent static lateral loads approximately represent earthquake induced forces. A plot of the total base shear versus top displacement in a structure is obtained. By this analysis any permanent failure or weakness can be identified. The analysis is carried out up to failure, thus it enables determination of collapse load and ductility capacity. on a building frame, plastic rotation is monitored and lateral inelastic forces versus displacement response for the complete structure is analytically computed. This type of analysis enables us to identify the weakness in the structure. The decision to retrofit can be taken in such studies.

2. SEISMIC EVALUATION BY PUSHOVER ANALYSIS

Pushover analysis is performed by subjecting a structure to a monotonically increasing pattern of lateral loads, that shows the inertial forces which would be experienced by the structure when subjected to ground motion. Under incrementally increasing loads many structural elements may yield sequentially. Therefore, at each event, the structure experiences a decrease in stiffness. Using a non-linear static pushover analysis, a representative non-linear force displacement relationship can be obtained. A two or three dimensional model which includes bi-linear or tri-linear load-deformation diagrams of all lateral force resisting elements is first created and gravity loads are applied initially.

A pre-defined lateral load pattern which is distributed along the building height is then applied. The lateral forces are increased until some members yield. The structural model is modified to account for the reduced stiffness of yielded members and lateral forces are again increased until additional members yield. The process is continued until a control displacement at the top of building reaches a certain level of deformation or structure becomes unstable.

2.1 Types of Pushover Analysis:

Pushover analysis can be performed as force-controlled or displacement-controlled. In force-controlled pushover analysis, full load combination is applied i.e., force-controlled analysis should be used when the load is known (such as gravity loading). Also, in force-controlled pushover analysis some numerical problems that affect the accuracy of results occur, since target displacement may be associated with a very small positive or even a negative lateral stiffness because of the development of mechanisms and P-delta effects.

Generally, pushover analysis is performed as displacement-controlled to overcome these problems. In displacement-controlled procedure, specified drifts are sought (as in seismic loading) where the magnitude of applied load is not known in advance. The magnitude of load combination is increased or decreased as necessary until the control displacement reaches a specified value. Generally, roof displacement at the center of mass of structure is chosen as the control displacement. The internal forces and deformations computed at the target displacement are used as estimates of inelastic strength and deformation demands that have to be compared with available capacities for a performance check.

2.2 Performance Levels of Building:

Pushover analysis gives an insight into the maximum base shear that the structure is capable of resisting. A building performance level is a combination of the performance levels of the structure and the non-structural components. A performance level describes a limiting damage condition for a given building with specific ground motion. The performances levels as per FEMA, ATC 40 are:

Immediate Occupancy (IO):

Damage is relatively less, the structure retains a significant portion of its original stiffness. The risk of life threatening injury as a result of structural damage is very low, and although some minor structural repairs may be appropriate, these would generally not be required prior to re occupancy

Life safety Level (LS):

Substantial damage has occurred to the structure, and it may have lost a significant amount of its original stiffness. However, a substantial margin remains for additional lateral deformation before collapse would occur. It should be possible to repair the structure; however, for economic reasons this may not be practical. While the damaged structure is not an imminent collapse risk, it would be prudent to implement structural repairs or install temporary bracing prior to re-occupancy.

Collapse Prevention (CP):

At this level the building has experienced extreme damage, if laterally deformed beyond this point, the structure can experience instability and collapse. The structure may not be technically practical to repair and is not safe for re-occupancy, as aftershock activity could induce collapse.

2.3 Pushover Curve:

In order to obtain performance points of structure as well as the location of hinges in different stages of analysis, we can use the pushover curve. In this curve, the range AB is the elastic range, B to IO is the range of instant occupancy, IO to LS being the range of life safety and LS to CP being the range of collapse prevention

When a hinge touches point C on its force-displacement curve then that hinge must start to drop load. The load will be released until the pushover force or base shear at point C becomes equal to the force at point D.

As the force is released, all of the elements unload as well as the displacement is decreased .After the yielded hinge touches the point D force level, the magnitude of pushover force is again amplified and the displacement starts to increase again.

If all of the hinges are within the given CP limit then that structure is supposed to be safe. Though, the hinge after IO range may also be required to be retrofitted depending on the significance of structure.

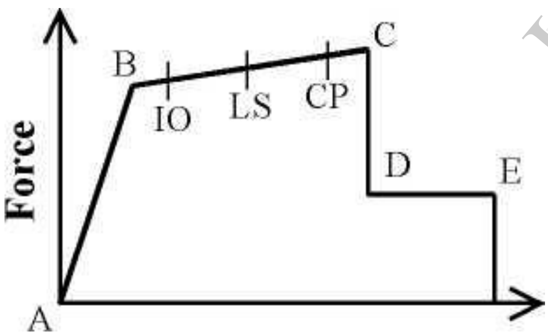


Figure 1 Typical Pushover Curve and Performance Levels

2.4 Key Elements of Pushover Analysis:

Defining Plastic Hinges:

In SAP2000, non-linear behavior is assumed to occur within frame elements at concentrated plastic hinges. The default types include an uncoupled moment hinges, an uncoupled axial hinges, an uncoupled shear hinges and a coupled axial force and biaxial bending moment hinges.

Defining control node:

Control node is the node used to control displacements of the structure. Its displacement versus the base-shear forms the capacity (pushover) curve of the structure. For developing the pushover curve it is important to consider a force displacement that is equal to the expected distribution of the inertial forces. Different forces distributions can be used to represent the earthquake load intensity.

Estimation of Displacement Demand:

This is a crucial step when using pushover analysis. The control node is pushed to reach the demand displacement which represents the maximum expected displacement resulting from the earthquake intensity under consideration.

Evaluation of the Performance Level:

Performance evaluation is the main objective of a performance based design. A component or action is considered satisfactory if it meets a prescribed performance. The main output of a pushover analysis is in terms of response demand versus capacity. If the demand curve intersects the capacity envelope near the elastic range, then the structure has a good resistance. If the demand curve intersects the capacity curve with little reserve of strength and deformation capacity, then it can be concluded that the structure will behave poorly during the imposed seismic excitation and need to be retrofitted to avoid future major damage or collapse.

2.5 Evaluation Procedures:

The procedures for building evaluation are different from one another but their basic principles are all the same. The following are the evaluation procedures according to the respective codes.

ATC 40 – 1996 Capacity Spectrum Method (CSM):

ATC 40 adopts the capacity spectrum method (CSM) which uses the intersection of capacity (pushover) curve and a reduced response spectrum to estimate the maximum displacement. The push over or capacity curve represents the lateral displacement as a function of the force applied to the structure. The important assumption is that inelastic displacement of nonlinear single degree freedom system will be approximately equal to the maximum elastic displacement of linear single degree freedom system, whose damping values are greater than the initial values for those in non- linear system with in natural time period.

FEMA 356 - 2000 Displacement Coefficient Method (DCM):

FEMA 356 adopts the displacement coefficient method that uses pushover analysis and a modified version of the equal displacement approximation to estimate maximum displacement. The DCM is based on the statistical analysis of the results obtained by the time history analysis of SDOF oscillators of various types. The results from various analyses indicate that the capacity spectrum method underestimates the response of the structure in inelastic range while the displacement coefficient method yields reasonable values in most cases.

FEMA 440 - 2005 Equivalent Linearization-Modified CSM:

In equivalent linearization method, the inelastic equivalent single degree of freedom system will be converted to its equivalent elastic single degree of freedom. In Equivalent Linearization method equivalent period and damping is defined in a way that inelastic displacement is nearly close to the elastic displacement of equivalent system. The assumption in capacity spectrum method that the equivalent stiffness of inelastic system will be the same as its secant stiffness is not used here. Instead, the equivalent stiffness is obtained effective from time period and damping properties derived using equations from statistical analyses.

FEMA 440 - 2005 Displacement Modification- Improvement of DCM:

In FEMA 440 Displacement Modification, several improvements to the displacement coefficient procedures in FEMA 356 are made. They relate to the coefficient of target displacement which is used for estimating the maximum inelastic global deformation demands of buildings for earth quake ground motions. The improvement for the angular displacement coefficient method uses advanced equations for different coefficients.

3. MODELING AND ANALYSIS OF STRUCTURE

3.1 Brief overview:

A Twelve storied, 4 x 4 bay regular frame with bay width 5m and floor height 3.2m is to be considered for the analysis. The total height of the building frame is 38.4m. As per IS code 1893 -2002, the natural time period is 1.157 sec. Present project is proposed to study the damage assessment of the multistoried buildings which were already designed for earthquake linear static analysis. Nonlinear static analysis (pushover analysis) is considered for the seismic evaluation of the already designed multistoried buildings using ESA method. Linear Static Analysis is performed using STAAD analysis package, which is a regular practice for most of the professional people and Pushover Analysis is performed using SAP analysis package for the damage assessment.

3.2 Modeling of the structure:

Number of members, nodes and supports of building frames are given in the table 2.

TABLE 2

Building frames	Regularity	Number of members	Number of nodes	Number of supports (fixed)
3D Bare Frame	Regular in plan	780	325	25

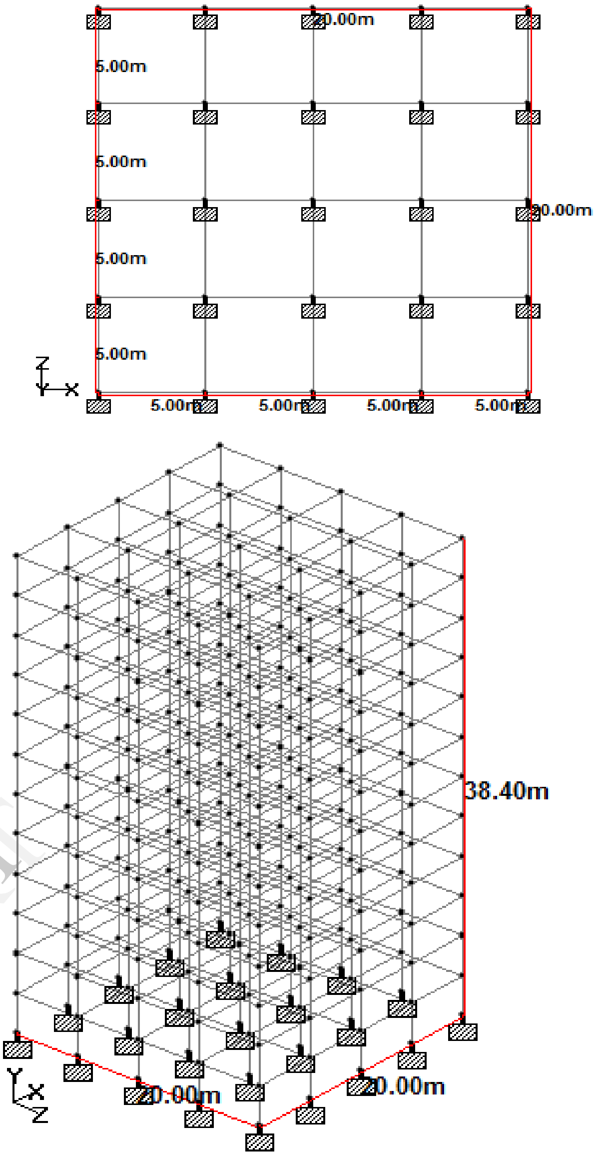


Figure 2 Selected Frame with supports, framing and nodes.

TABLE 3

Material properties considered for analysis

Concrete				
Modulus of elasticity (E) kN/m^2	Poisson ratio	Density kN/m^3	Coefficient of thermal expansion @ / ^{0}K	F_{ck} / f_y kN/m^2
2.73861e+007	200e-003	25	1.17e-005	30
Reinforcing bar (rebar)				
1.999e+008	300e-003	76.97	1.17e-005	415

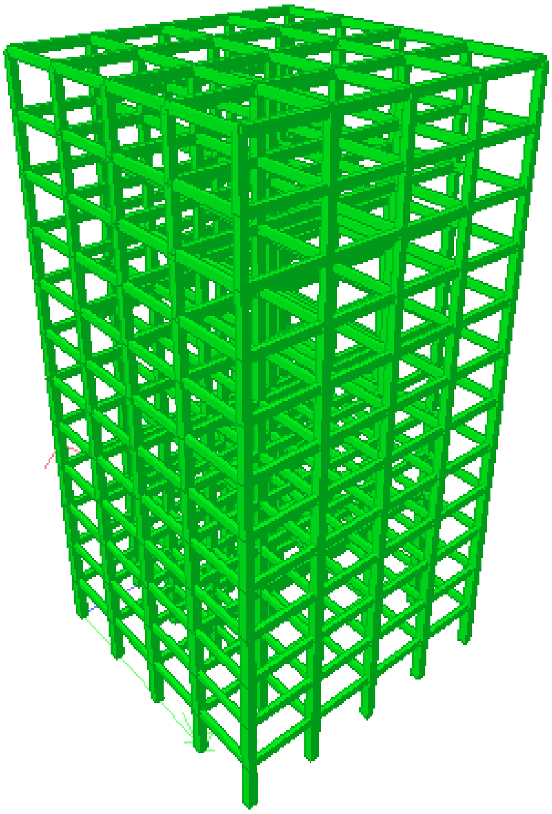


Figure3 3D-Rendered Frame

Table 4

Physical properties of the columns and beams

Member	Size (mm x mm)
Case-1: SMRF and Zone-3	
Beams for all floors	250 x 500
Columns (1,2,3 floors)	470 x 470
Columns (4,5,6 floors)	450 x 450
Columns (7,8,9 floors)	420 x 420
Columns (10,11,12 floors)	410 x 410
Case-2: SMRF and Zone-5	
Beams for all floors	300 x 500
Columns (1,2,3 floors)	600 x 600
Columns (4,5,6 floors)	550 x 550
Columns (7,8,9 floors)	500 x 500
Columns (10,11,12 floors)	450 x 450

3.3 Load Consideration:

Table 5

Dead load and Live loads considered for the analysis

Type of load	Load value
Dead load*	
On floor slabs (member loads)	14.6 kN/m
On roof slabs (member loads)	10.7 kN/m
Live load**	
On floor slabs (member loads)	6.0 kN/m
On roof slabs (member loads)	3.0 kN/m
* which includes self weight, wall load and equivalent slab load ** which is equivalent UDL over the member due to live load on the slab	

Earthquake loads: earthquake loads considered for the calculation of seismic weights are as per the IS 1893(Part 1) : 2002 and are given in the table 6.

Table 6

Loads considered for the calculation of seismic weights

Loads on the floors
Full dead load acting on the floor plus 25 percent of live load(since, as per clause 7.3.1 Table 8 of IS 1893(Part 1):2002, for imposed uniformly distributed floor loads of 3 kN/m^2 or below, the percentage of imposed load is 25 percent).
Loads on the roof slab
Full dead load acting on the roof (since, as per clause 7.3.2, for calculating the design seismic forces of the structure, the imposed load on roof need not be considered).

Seismic Load Case1:

For the analysis purpose, structure is assumed to be located in zone-III (zone factor-0.16) on site with medium soil and S_a/g value taken from the figure 2 of IS-1893: 2002 i.e., Response spectra for rock and soil sites for 5% damping. Structure is taken as a general building and hence Importance factor is taken as 1 and the frame is proposed to design as Special moment resisting frame (SMRF) and hence the Reduction factor is taken as 5. Ductile detailing is adopted as per the IS Code 13920-1993.

Seismic Load Case2:

For the analysis purpose, structure is assumed to be located in zone-II (zone factor-0.36) on site with medium soil and S_a/g value taken from the figure 2 of IS-1893: 2002 i.e., Response spectra for rock and soil sites for 5% damping. Structure is taken as general building and hence Importance factor is taken as 1 and the frame is proposed to design as Special moment resisting frame (SMRF) and hence the Reduction factor is taken as 5. Ductile detailing is adopted is as per the IS Code 13920 -1993.

3.4 Load Combinations and Envelope:

Earthquake load combination is only considered for the analysis.

TABLE 7

LOAD ENVELOPE

Envelope	
1.0DL+1.0LL	0.9DL+1.5(-ELx)
1.5DL+1.5LL	0.9DL+1.5(ELz)
1.5DL+1.5(ELx)	0.9DL+1.5(-ELz)
1.5DL+1.5(-ELx)	1.2DL+1.2LL+1.2(ELx)
1.5DL+1.5(ELz)	1.2DL+1.2LL+1.2(-ELx)
1.5DL+1.5(-ELz)	1.2DL+1.2LL+1.2(ELz)
0.9DL+1.5(ELx)	1.2DL+1.2LL+1.2(-ELz)

After linear static analysis (as per STAAD) for the above modeling, the design results obtained are given in the following table 8 for the both seismic load cases. The design results obtained are proposed to take as material and sectional properties in the pushover analysis using SAP.

TABLE 8 DESIGN RESULTS

Floor	Section(mm x mm)	Longitudinal Reinforcement	Lateral Reinforcement	Materials
Seismic Load Case 1				
Beams for all floors	250 x 500	3-16mmØ-top of support 2-16mmØ-bottom span	4-legged-8mm Ø @100mm c/c	M30, Fe 415
Columns (1,2,3 floors)	470 x 470	16-20mmØ	4-legged-8mm Ø @100mm c/c	M30, Fe 415
Columns (4,5,6 floors)	450 x 450	16-12mmØ	4-legged-8mm Ø @100mm c/c	M30, Fe 415
Columns (7,8,9 floors)	420 x 420	16-12mmØ	4-legged-8mm Ø@100mm c/c	M30, Fe 415
Columns (10,11,12 floors)	410 x 410	16-12mmØ	4-legged-8mm Ø @100mm c/c	M30, Fe 415
SEISMIC LOAD CASE 2				
Beams for all floors	300 x 500	6-16mmØ-top of support 3-16mmØ-bottom span	4-legged-8mm Ø @100mm c/c	M30, Fe 415
Columns (1,2,3 floors)	600 x 600	16-16mmØ	4-legged-8mm Ø @100mm c/c	M30, Fe 415
Columns (4,5,6 floors)	550 x 550	16-16mmØ	4-legged-12mm Ø @100mm c/c	M30, Fe 415
Columns (7,8,9 floors)	500 x 500	12-16mmØ	4-legged-8mm Ø @100mm c/c	M30, Fe 415
Columns (10,11,12 floors)	450 x 450	12-16mmØ	4-legged-10mm Ø @100mm c/c	M30, Fe 415

SAP 2000 which is a finite element analysis package has been used for the analyses. SAP 2000 provides default-hinge properties and recommends PMM hinges for columns and M3 hinges for beams as described in FEMA-356.After designing and detailing the reinforced concrete frame structures as given in table 8, a nonlinear pushover analysis is carried out for evaluating the structural seismic response. Pushover analysis consists of the application of gravity loads, dead and live loads and a representative lateral load pattern. In the non-linear analysis, lateral loads were applied monotonically in a step-by-step procedure. The lateral loads were taken as accelerations in the respective direction in lieu of the forces that would be experienced by the structures when subjected to ground motion. Under monotonic loading, elements may yield one after the other. As a result, at each stage, the structure experiences a stiffness change because of damage. The analysis results are shown in the following tables and graphs. Sequence of damages and their intensity of damage are shown in 3.7 and 3.10 for Zone-3 and Zone-5.

3.5 Analysis Results of Seismic Load Case-1 (SMRF-Z3):

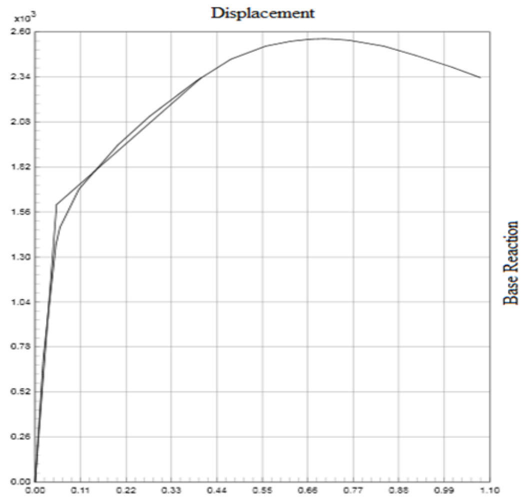
TABLE 9 Base Shear Vs Displacement

Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	m	KN									
0	1.244E-16	0	1560	0	0	0	0	0	0	0	1560
1	0.019756	715.872	1557	3	0	0	0	0	0	0	1560
2	0.050159	1370.683	1409	151	0	0	0	0	0	0	1560
3	0.059979	1470.679	1348	212	0	0	0	0	0	0	1560
4	0.106322	1692.751	1285	275	0	0	0	0	0	0	1560
5	0.199621	1945.678	1245	270	45	0	0	0	0	0	1560
6	0.277679	2113.73	1220	175	165	0	0	0	0	0	1560
7	0.389795	2316.517	1210	115	215	20	0	0	0	0	1560
8	0.473219	2441.913	1160	120	157	123	0	0	0	0	1560
9	0.558203	2517.876	1155	120	85	100	0	100	0	0	1560
10	0.618948	2546.316	1147	113	100	47	0	153	0	0	1560
11	0.666508	2558.006	1139	101	115	28	0	177	0	0	1560
12	0.672213	2558.75	1139	101	115	13	0	192	0	0	1560
13	0.683658	2559.619	1137	103	105	15	0	200	0	0	1560
14	0.695088	2559.792	1130	110	102	18	0	200	0	0	1560
15	0.703374	2559.79	1120	119	101	20	0	200	0	0	1560
16	0.719715	2558.709	1109	128	103	20	0	200	0	0	1560
17	0.758937	2551.973	1093	125	102	40	0	200	0	0	1560
18	0.843341	2517.476	1087	111	102	30	0	230	0	0	1560
19	0.941016	2449.331	1070	120	91	39	0	240	0	0	1560
20	1.021375	2385.052	1062	128	68	62	0	240	0	0	1560
21	1.080204	2332.447	1062	128	65	54	1	250	0	0	1560

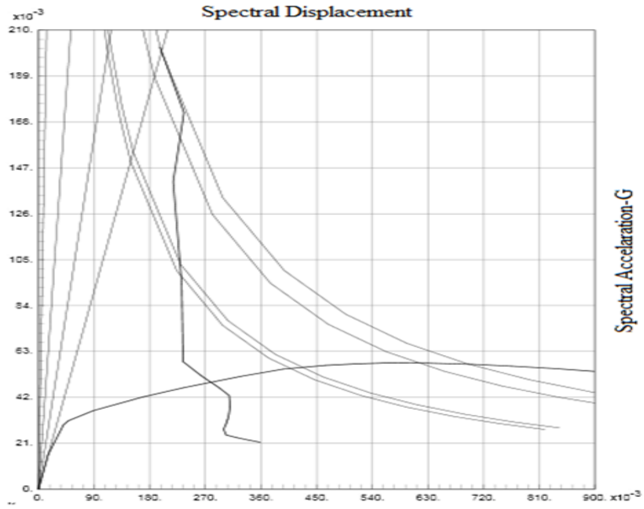
TABLE10 S_d/ S_a (ATC 40) Capacity and Demand Spectrum

Step	Teff	Beff	SdCapacity	SaCapacity	SdDemand	SaDemand	Alpha	PFPhi
			m		m			
0	1.979791	0.05	0	0	0.196716	0.202042	1	1
1	1.979791	0.05	0.015202	0.015614	0.196716	0.202042	0.811176	1.299524
2	2.345436	0.115397	0.040031	0.029294	0.184634	0.135115	0.827823	1.253007
3	2.4958	0.147264	0.048433	0.031301	0.181444	0.117263	0.831262	1.238378
4	3.156608	0.230237	0.088966	0.035944	0.194656	0.078644	0.833209	1.195077
5	4.017324	0.253235	0.169141	0.04219	0.23829	0.059439	0.815911	1.18021
6	4.51382	0.255334	0.235001	0.046432	0.26682	0.052719	0.805407	1.181611
7	5.055533	0.25391	0.327899	0.051647	0.299539	0.04718	0.793551	1.188765
8	5.38511	0.253079	0.395437	0.054894	0.319503	0.044353	0.787025	1.196699
9	5.745822	0.259398	0.464519	0.056642	0.337406	0.041142	0.786469	1.20168
10	6.009117	0.265054	0.514332	0.05734	0.349666	0.038983	0.785663	1.203402
11	6.216357	0.2696	0.553366	0.057647	0.359115	0.037411	0.785068	1.204462
12	6.241257	0.270146	0.558041	0.057671	0.360242	0.03723	0.784968	1.204595
13	6.29176	0.271283	0.567429	0.057704	0.362505	0.036864	0.784791	1.204834
14	6.343028	0.272478	0.576824	0.057715	0.36477	0.036498	0.784695	1.205027
15	6.379773	0.273276	0.583674	0.05773	0.366423	0.036242	0.784496	1.20508
16	6.453228	0.274915	0.597191	0.05773	0.369689	0.035737	0.784166	1.205168
17	6.633155	0.278893	0.629895	0.057632	0.377644	0.034553	0.783419	1.204862
18	7.032138	0.286978	0.701213	0.057084	0.395397	0.032188	0.780254	1.202688
19	7.504566	0.306447	0.783163	0.055981	0.417575	0.029848	0.774092	1.201558
20	7.905409	0.325126	0.85094	0.054814	0.439879	0.028335	0.769828	1.20029
21	8.211612	0.34013	0.900726	0.053774	0.456917	0.027278	0.767401	1.199259

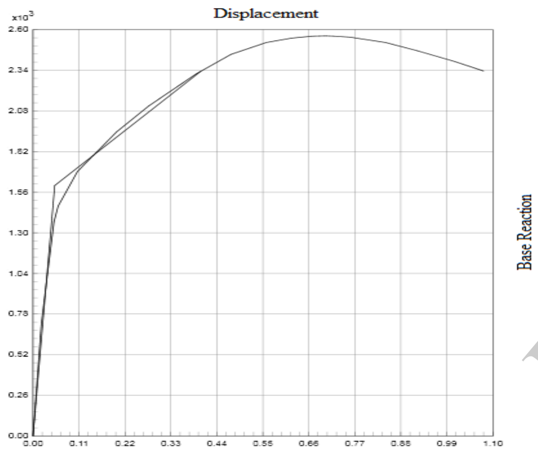
3.6 Capacity and Demand Curves (Frame designed for Zone 3 and SMRF):



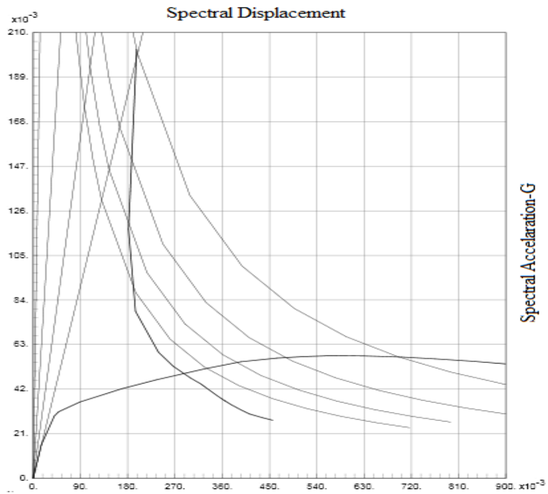
Capacity Curve (FEMA – 356)



Capacity and Demand Curves FEMA-440

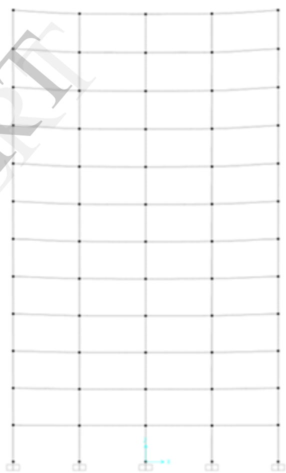


Capacity Curve (FEMA440)

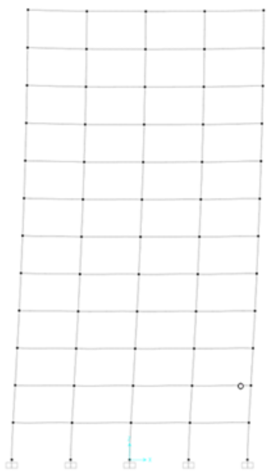


Capacity and Demand Curve ATC-40

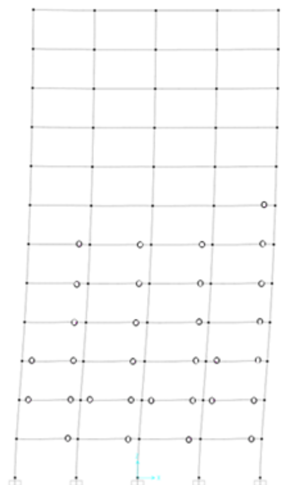
3.7 Damage at different stages in Zone-3:



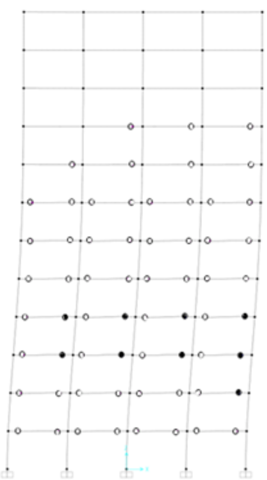
Stage-0



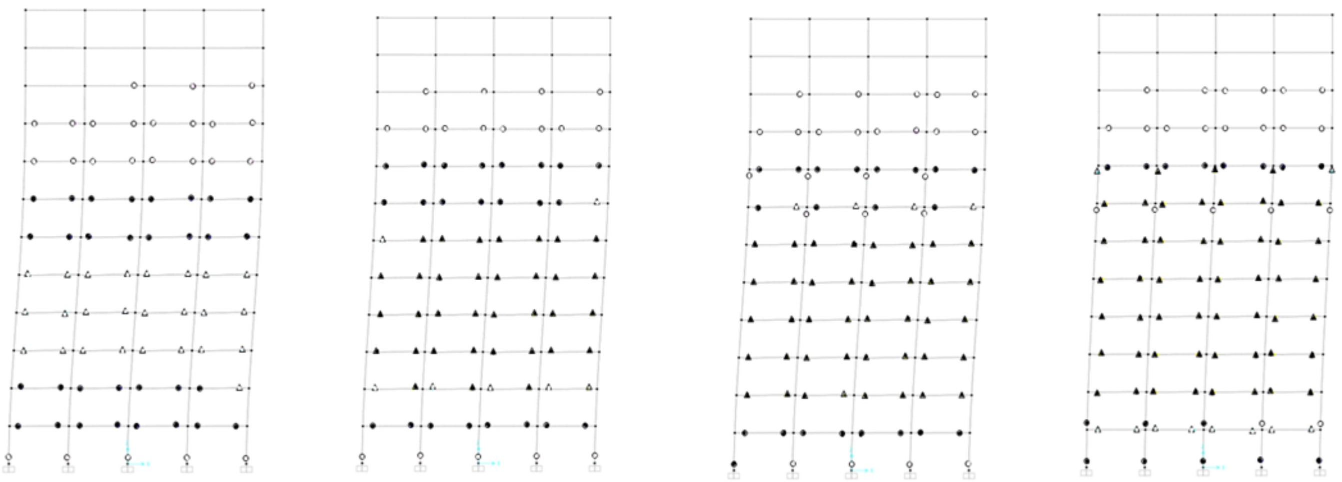
Stage-1



Stage-2



Stage-5



Stage-8 Stage-11 Stage-16 Stage-21

B IO-Immediate occupancy LS-Life Safety C-Collapse

3.8 Analysis Results of Seismic Load case-1 (SMRF-Z5)

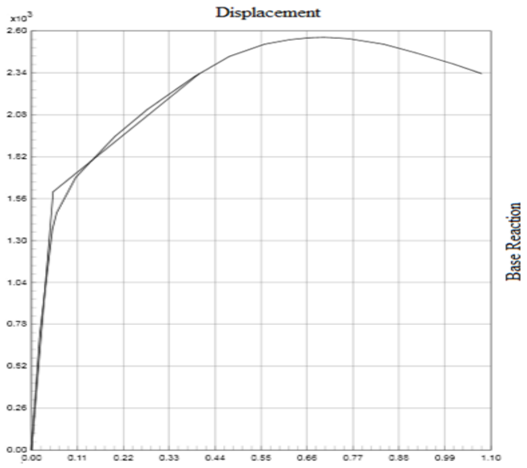
TABLE 11 Base Shear Vs Displacement

Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	m	KN									
0	-3.039E-17	0	1560	0	0	0	0	0	0	0	1560
1	0.019026	1021.213	1554	6	0	0	0	0	0	0	1560
2	0.044606	1817.152	1393	167	0	0	0	0	0	0	1560
3	0.073534	2146.977	1294	266	0	0	0	0	0	0	1560
4	0.160343	2619.492	1240	320	0	0	0	0	0	0	1560
5	0.239706	2923.628	1205	275	80	0	0	0	0	0	1560
6	0.268226	3002.263	1161	241	158	0	0	0	0	0	1560
7	0.270064	3004.35	1160	240	160	0	0	0	0	0	1560
8	0.423409	3108.298	1155	100	305	0	0	0	0	0	1560
9	0.500209	3159.858	1155	100	191	114	0	0	0	0	1560
10	0.57701	3206.975	1135	100	145	142	0	38	0	0	1560
11	0.613654	3220.69	1130	85	100	151	0	94	0	0	1560
12	0.620523	3221.472	1130	85	100	127	0	118	0	0	1560
13	0.627067	3221.421	1130	85	85	140	0	120	0	0	1560
14	0.652347	3220.913	1120	95	80	136	0	129	0	0	1560
15	0.659596	3220.364	1120	95	80	112	0	153	0	0	1560
16	0.702468	3210.582	1120	95	80	75	0	190	0	0	1560
17	0.816618	3133.194	1092	108	55	40	0	265	0	0	1560
18	0.896815	3056.198	1065	132	58	35	0	270	0	0	1560
19	0.938299	3006.887	1062	135	58	22	0	283	0	0	1560

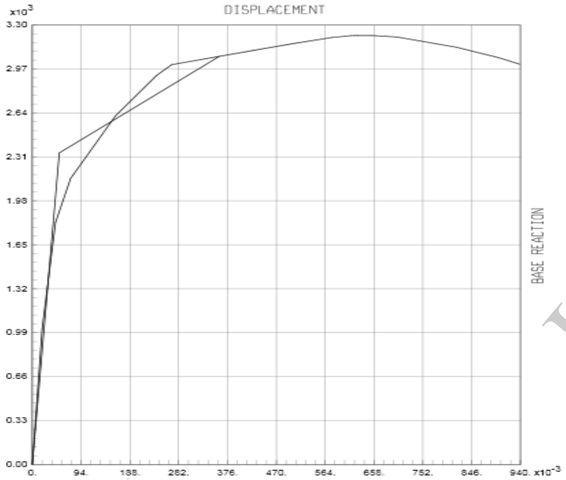
TABLE 12 S_d/S_a (ATC40) Capacity and Demand Spectrum

Step	Teff	Beff	SdCapacity	SaCapacity	SdDemand	SaDemand	Alpha	PFPhi
			m		m			
0	1.708664	0.05	0	0	0.169776	0.234101	1	1
1	1.708664	0.05	0.014516	0.020016	0.169776	0.234101	0.790011	1.310731
2	2.012268	0.117982	0.035262	0.035057	0.157306	0.156391	0.8026	1.264983
3	2.415031	0.194554	0.059907	0.04135	0.158967	0.109724	0.803974	1.227474
4	3.216989	0.24051	0.132933	0.05171	0.194913	0.075819	0.784385	1.206192
5	3.680369	0.244618	0.19768	0.058751	0.221449	0.065816	0.770528	1.212595
6	3.830114	0.247593	0.220479	0.060504	0.229316	0.062929	0.768335	1.216559
7	3.842313	0.248329	0.221951	0.060522	0.229765	0.062652	0.768644	1.216772
8	4.78471	0.283754	0.34555	0.060763	0.270365	0.047542	0.79208	1.225318
9	5.172842	0.289601	0.407517	0.061309	0.289692	0.043583	0.798042	1.227454
10	5.521286	0.296947	0.468994	0.061934	0.30722	0.04057	0.801779	1.230313
11	5.682493	0.30068	0.498156	0.062105	0.31619	0.039419	0.802985	1.231851
12	5.713772	0.301645	0.50364	0.062103	0.31793	0.039204	0.803205	1.232076
13	5.744212	0.302694	0.508878	0.062086	0.319624	0.038996	0.803417	1.232255
14	5.860583	0.3066	0.529104	0.062015	0.326099	0.038221	0.804202	1.232927
15	5.893871	0.30774	0.534894	0.061988	0.327951	0.038006	0.804424	1.233134
16	6.093456	0.315142	0.569191	0.061712	0.339057	0.036761	0.805562	1.234152
17	6.666266	0.342025	0.661634	0.059937	0.37093	0.033602	0.809433	1.234244
18	7.091628	0.363167	0.728264	0.058296	0.394598	0.031587	0.811766	1.231443
19	7.32287	0.375303	0.763323	0.057304	0.407465	0.030589	0.812489	1.22923

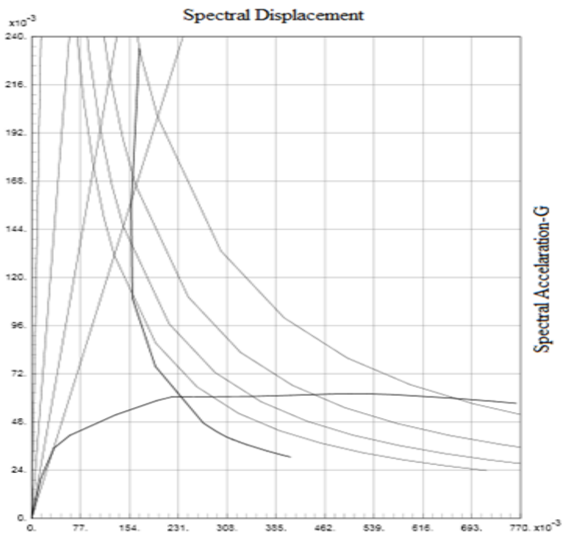
3.9 Capacity and Demand Curves (Frame designed for Zone 5 and SMRF):



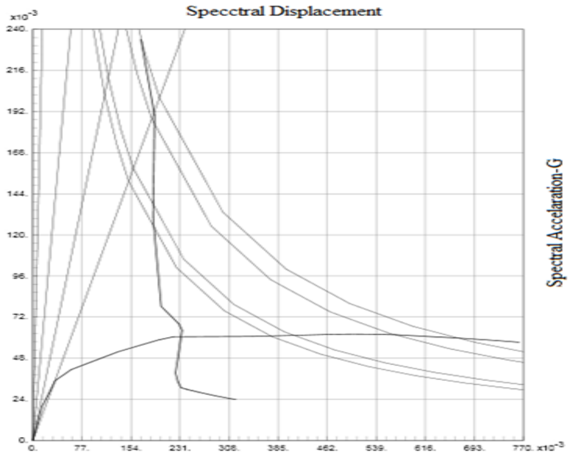
Capacity Curve (FEMA-356)



Capacity Curve (FEMA - 440)

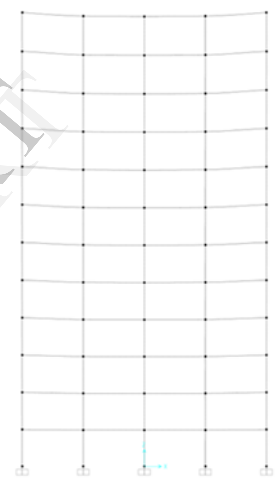


Capacity and Demand Curve (ATC-40)

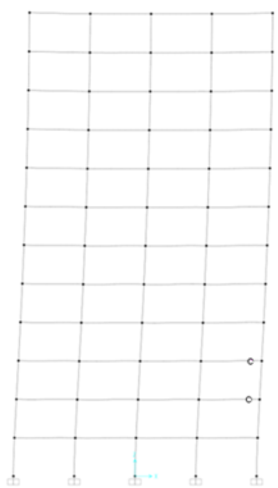


Capacity and Demand Curve (FEMA-440)

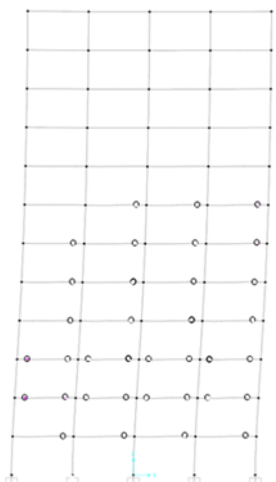
3.10 Damage at different stages in Zone-5:



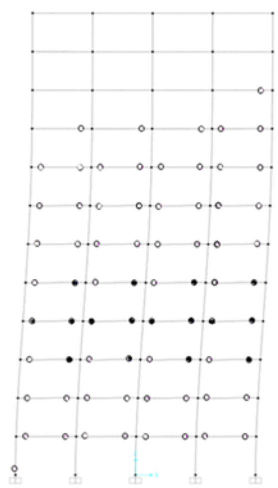
Stage-0



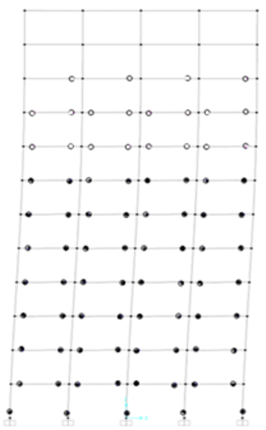
Stage-1



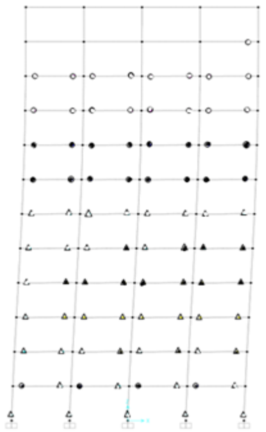
Stage-2



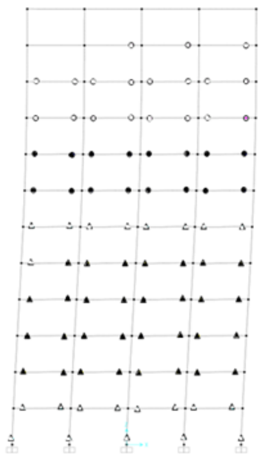
Stage-5



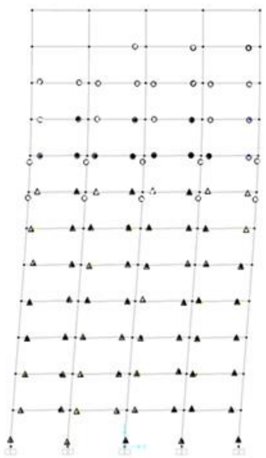
Stage-8



Stage-11



Stage-15



Stage-19

○ B ● IO-Immediate occupancy △ LS- Life Safety ▲ C-Collapse

4. OBSERVATIONS AND CONCLUSIONS

In the present study it is proposed to assess the damage and to evaluate the performance of designed structure for earth quake loads. The frames are designed for the two zones i.e., zone-3 and zone-5 considering both are Special Moment Resisting Frames, whose response reduction factor is 5. The zone factors for the zone-III is 0.16 and zone-V is 0.36 as per IS code 1893-2002. Physical properties of the model will change in the analysis and design because of zone. Hence, two building frame models are available for the non-linear static analysis i.e., pushover analysis. We performed the push over analysis for the displacement control using analysis package SAP. The target displacement values are obtained from four evaluation procedures:

1. ATC-40 Capacity Spectrum Method.
2. FEMA 356 Coefficient Method.
3. FEMA 440 Equivalent Linearization.
4. FEMA 440 Displacement Modification.

The base shear and target displacement values are obtained as shown in table below.

Table 13 Target Displacement and Base Shear

ZONE-3

Evaluation Procedure	ATC-40	FEMA-356	FEMA-440 EL	FEMA-440DM
Target Displacement (m)	0.337	0.402	0.327	0.402
Base Shear (kN)	2220.95	2334.25	2203.82	2334.45

TABLE 14 ZONE-4

Evaluation Procedure	ATC-40	FEMA-356	FEMA-440 EL	FEMA-440DM
Target Displacement (m)	0.285	0.360	0.284	0.360
Base Shear (kN)	3014.13	3065.27	30.13.67	3065.27

From the Tables 9 to12, Graphs 3.6 & 3.9 and Deformed shapes with hinge locations 3.7 & 3.10 shows that damage of the structure in stage wise. This damage assessment shows that performance of the structure under seismic loading. Firstly it is observed the damage of the building frame for the non-linear static analysis for dead and live loads i.e., the initial stage of the push over analysis for the both frames there is no hinge formation or there is no damage after the completion of non-linear static analysis for the dead and live loads. This is shown as stage 0 in the Figure 3.7 & 3.10. The target displacement may vary according to the evaluation procedures i.e., ATC-40(CSM), FEMA-356(CM), FEMA-440 (EL), FEMA-440(DM). The Target displacement considered is the maximum of four evaluation procedures. Now in the case of ZONE-3 the maximum value of target displacement for the damage assessment considered is 0.402 seconds where the base shear is 2334.25 kN. In case of ZONE-5 the maximum value of target displacement is 0.360 seconds and the corresponding base shear is 3065.27 kN.

The Graphs 3.6 & 3.9 shows that the capacity and demand curves for zone-3 and zone-5. Figures 3.7 & 3.10 shows stage wise hinge formation and damage sequence for zone-3 and zone-5. Tables 9 & 11 shows the number of hinge formations at every stage i.e., damage level at every step. In case of ZONE-3 design, the stiffness of the frame is less, hence the damage appeared up to CP level with in the target displacement i.e., 0.402 seconds. There is a formation of hinges up to CP (Figure 3.7). In case of ZONE-5 design, the stiffness of the frame is higher than in ZONE 3 frame, hence the damage appeared up to LS level with in the target displacement i.e., 0.360 seconds (Figure 3.10). Finally both the frames which were designed to linear static analysis for earth quake loading performed well. The damage is within the limits and it is observed by conducting the push over analysis. Initially, the yielding of the beams takes place and then yielding of columns. This shows that the analysis theory is based on the strong column and weak beam i.e., both the frames behaving as ductile frames

5. FUTURE SCOPE OF WORK

Pushover analysis is an efficient method to understand the performance of the structure during earthquakes; however, it is not a dynamic phenomenon and lacks accuracy. This may not consider all the deformation within the structure. To know the complete behavior of the structure from initial stage to collapse stage, knowledge of non-linear analysis for the numerical modals using Finite Element Method (FEM) and Applied Element Method (AEM) is most essential.

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