Performance of RC Frames with Different Moment Resisting Type

by

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in

STRUCTURAL ENGINEERING DIGEST

Report No: IIIT/TR/2020/-1

Centre for Earthquake Engineering
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Hyderabad - 500 032, INDIA
June 2020

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Abstract

Indian seismic code IS 1893 divides the estimation of seismic forces based on; Ordinary moment resisting frame and Special moment resisting frame, the classification differs based on reinforcement detailing and response reduction factor (R). The performance of ductile detailed building is expected to be better than non ductile detailed and gravity load designed building. It is expected that the capacity of ductile detailed structure is more and damage will be less compared to non ductile detailed and gravity load designed building. In this view, the present study aims at comparing performance of structure designed considering gravity load, non ductile detailing as per IS 456:2002 and ductile detailing as per IS 13920 :2016, in terms of capacity, damage, ductility and drift. A study has been carried out by considering a 5 storey building designed for Gravity loads as well as lateral load as per IS 1893:2016 for seismic zone III. Static Non Linear (Pushover) analysis and fragility analysis are performed for estimation of post damage yielding behavior of structure. The change in non linear behaviour of structure based on assumed load patterns in pushover analysis is done. This paper also provides other significant conclusions on seismic design provisions and displacement amplification factor.

Introduction

India in past two decades faced 9 major earthquakes, caused huge amount of loss in terms of fatalities and economy (Table 1). From the earthquake reconnaissance reports of past earthquakes, it is clear that the provisions of design and detailing in Indian codes are in line with international practices (EERI Special Earthquake Report, 2001). In spite of this fact, the casualties in India are very large when compared to that of other countries for similar level of ground shaking. This is mainly due to not following code provisions in design and improper execution. One of the major reasons for not following code provisions is lack of awareness about earthquake resistant design and myth that earthquake resistant design is costlier. This results in most common type of problems i.e., 1) slender column to make them flush with infill walls, 2) buildings with open first storey, 3) torsion induced due to more number of infill panel on one side, 4) strong beam weak column, 5) lapping of column reinforcement above beam-column joint, 6) inadequate

Table 1 : Fatalities and damage in India

S.No.	Earthquake	Year	Magnitude(Mw)	Fatalities	Houses damaged
1	Uttarkashi	1991	6.6	1,000	42,400
2	Killari	1993	6.3	12,000	NA
3	Jabalpur	1997	6.0	38	8,546
4	Chamoli	1999	6.8	103	50,000
5	Bhuj	2001	7.7	19,200	3,48,000
6	Sumatra-Andaman	2004	9.3	10,136	NA
7	Kashmir	2005	7.8	1350	32,335
8	Andaman Islands	2009	7.5	NA	NA
9	Nepal Sikkim	2011	6.8	94	4,300

lapping of column reinforcement, 7) abrupt reduction in column dimensions and 8) Improper detailing (Hafeez & Ramancharla, 2009).

Seismic design philosophy states that there will be some amount of damage in the structure when subjected to the design intensity of earthquake. Design of structures is based on Elastic force, the nonlinear response of structure in incorporated in design by using appropriate response reduction factor (R). The concept of response reduction factor is to de-amplify the seismic force and incorporate nonlinearity with the help of over strength, redundancy and ductility.

In seismic design code, two types of reinforcement detailing are specified i.e., 1) non-ductile and 2) ductile detailing, based on type of detailing value of R changes. This leads to change in design base shear, which ultimately leads to change in member cross section. Ductile detailing is done in structure to increase the ductility and to reduce the amount of damage, compared to Non-Ductile detailed structure. The non linear response of the building can be determined by Non-Linear Static Pushover analysis (POA) using displacement control approach. Pushover curve has three major components i.e., initial stiffness, strength and ductility. If ductile detailing is required to be done for a building than only ductility should be increased and other two parameters to be kept same for comparatively less damage, above mentioned behaviour cannot be achieved using provisions given in current seismic code.

In the present study, a non-linear static pushover analysis and damage estimation of a five storey reinforced concrete building designed with different values of R is done. The work presented in this paper focuses on the design provisions of ductile detailing using POA and fragility analysis, effect of load patterns considered in POA and determination of damage based on storey drift.

Building Details

For the current study a 5 storey building is considered. Fig. 1 shows center line diagram, beam location, column orientation. Building consists of four 2BHK flats on each floor. Building does not have any horizontal or vertical irregularities, cantilever projections or heavy overhangs. It is also symmetric about X and Y axes to avoid torsion. All the walls are supported on beams and every beam

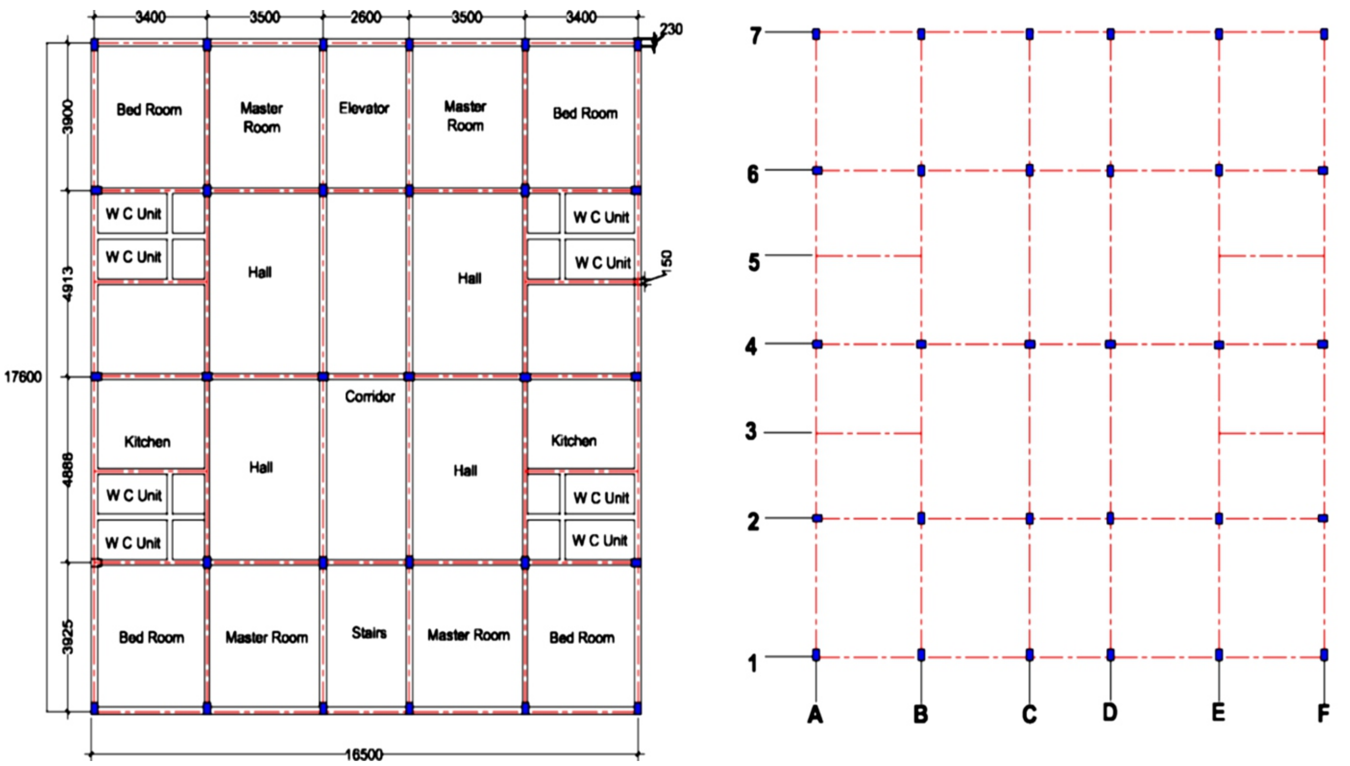

Fig. 1 : Building plan with column orientation and grid line

intersection is supported by a column. Dog legged type staircase is considered with flight and landing width is 1.25 m, riser and trade are 150 and 250 mm, respectively. Mid Landing of staircase is resting on beam connected to the column. Elevator is also provided as per NBC norms.

The building is located in the seismic zone III. External, internal wall thickness and slab thickness are considered as 230 mm, 100 mm and 120 mm, respectively. Floor finish of 1 kN/m^2 is considered. Design live loads are assumed as 2.5 kN/m^2, 1.25 kN/m^2 and 5 kN/m^2 on floors, roof and staircase, respectively. M20 and Fe415 grade of concrete and steel (HYSD) are considered, respectively. Depth of foundation is considered as 2 m from ground level.

Following 4 cases have been considered in the study:

Model I : Building designed for Gravity Loads only.

Model II : Building designed for Gravity and Seismic Loads of Zone III (Ordinary Moment Resisting Frame)

Model III : Building designed for Gravity and Seismic Loads of Zone III (Special Moment Resisting Frame)

Model IV : Building designed for Gravity and Seismic Loads of Zone III (Special Moment Resisting Frame) with same member sizes as model II.

For analysis, dead load, imposed load and seismic loads were considered as per IS 875 (1987) and IS 1893 (2016), respectively. Design of structure was done as per IS 456 (2000). For Lateral load analysis seismic forces applied along +X, -X, +Y, -Y directions and load combinations were considered as per IS 1893. Fig. 2 shows the structural model.

Fundamental natural time period of the building was found to be 0.339 sec and 0.319 sec along X and Y directions, respectively as per IS 1893 (2016). Base

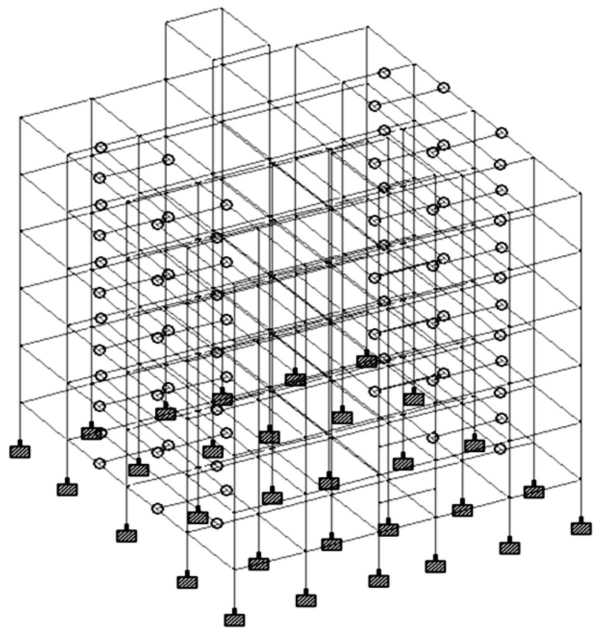

Fig. 1 : Building Model

shear values for structure and Frame 4 are given in Table 2.

Table 2 : Base Shear values on building and frame 4

Model	II	III	IV
R Factor	3	5	5
Building Base Shear (kN)	1196	717	717
Frame Base Shear (kN)	396	238	238

Design

All considered models are designed as per IS design codes. Model I and II were designed as per IS 456:2000 (Normal Detailing), and model III and IV are designed as per IS 456:2000 and IS 13920:2016.

Increase in R factor lead to significant decrease in base shear which ultimately lead to significant amount of decrease in member dimensions and reinforcement. Beam column dimensions are given in Table 3. Reinforcement detailing of column C-4 (First floor) and beam connecting columns of C-4 and B-4 (First floor) are given in Fig. 3.

Pushover Analysis

Non-linear seismic response of the building can be estimated by Nonlinear time historey analysis (NTHA) and Nonlinear static analysis (NSA). Literature shows that for actual response NTHA is required to be done but it requires high computation time and lots of parameters involved in it makes it difficult for analysis. Pushover analysis is not as complicated as nonlinear time historey analysis and can use response spectrum as demand diagram to estimate the seismic response of structures (Chopra & Goel, 1999). Pushover analysis is Non Linear Static Analysis done to

Table 3 : Dimensions of Beams and Columns for different models

Model	Column Dim. (mm)		Beam Dimension (mm)					
	Exterior	Interior	Plinth	I Floor	II Floor	III Floor	IV Floor	Terrace
I	350 x 230	400 x 230	230 x 300	230 x 300	230 x 300	230 x 300	230 x 300	230 x 300
II, IV	450 x 300	450 x 300	250 x 300	250 x 450	250 x 350	250 x 350	250 x 300	250 x 300
III	350 x 300	350 x 300	230 x 300	250 x 350	230 x 325	230 x 325	230 x 300	230 x 300

Model	Beam Dimension (mm)					
	Plinth	I Floor	II Floor	III Floor	IV Floor	Terrace
I	230 x 300	230 x 300	230 x 300	230 x 300	230 x 300	230 x 300
II, IV	250 x 300	250 x 450	250 x 350	250 x 350	250 x 300	250 x 300
III	230 x 300	250 x 350	230 x 325	230 x 325	230 x 300	230 x 300

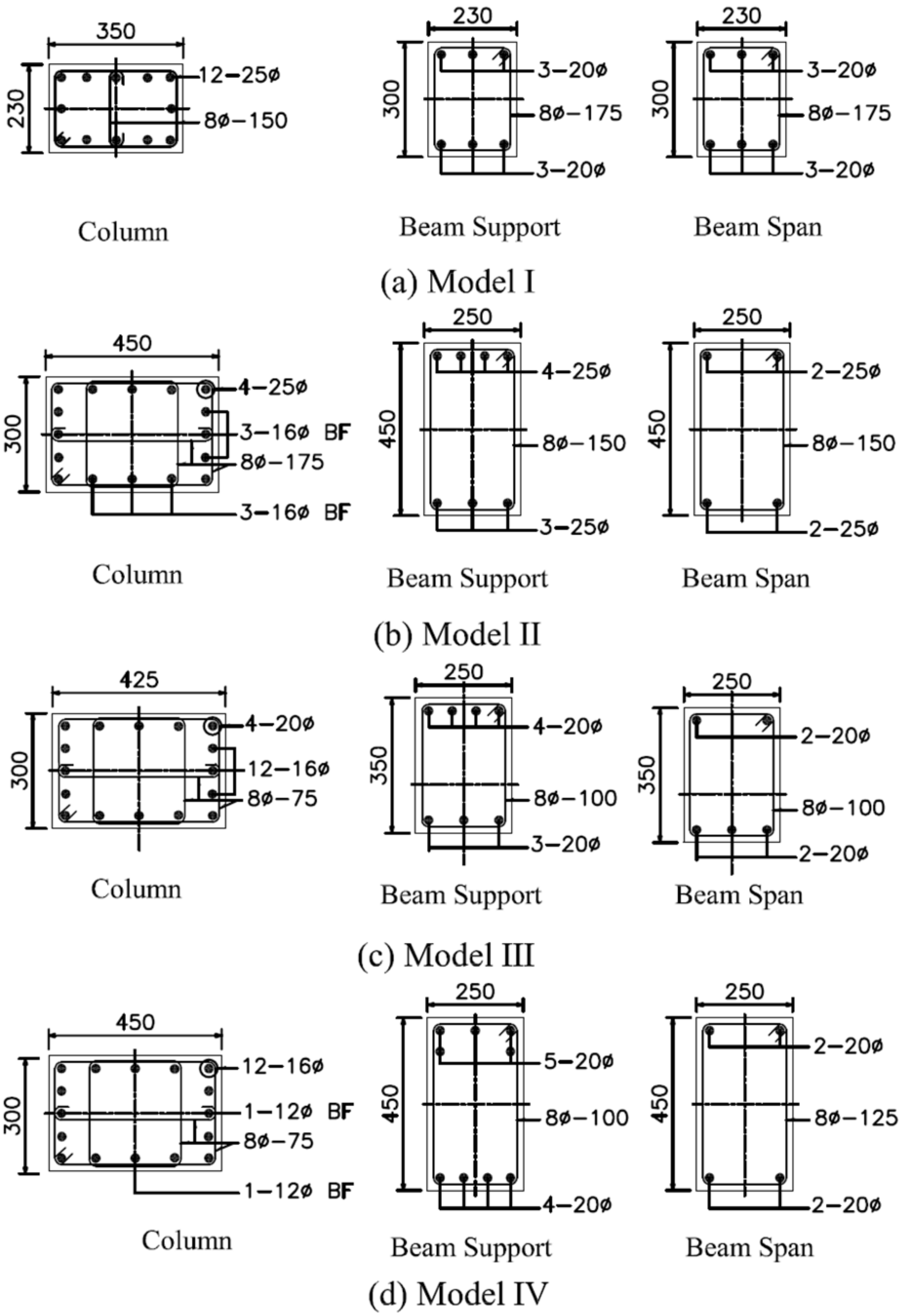

Fig. 3 : Reinforcement Detailing of Column and Beam

determine the capacity of structure. With the help of pushover curve non linear behavior of structure can be observed. Literature shows that assumed lateral load pattern applied for the structure to perform pushover analysis affects the capacity of the structure (Khoshnoudian, S. Mestri & Abedinik, 2011). If the curve over or underestimates the seismic capacity of the building, then results would not be realistic. Therefore, the selection of a reasonable lateral load pattern is particularly important in pushover analysis (Jingjiang, Ono & Yangang, 2003). Several load patterns suggested in the literature i.e., (1) Uniform distribution (FEMA356, 2000), (2) Inverted triangular distribution, (3) Distribution based on response spectrum analysis (Tso & Maghadam, 1997), 4) Load Distribution based on product of mass and fundamental mode shape (Fajfar & Fischinger, 1988), (5) Load pattern based on codal lateral load distribution. The first two patterns result in the upper and lower bound of pushover curves, respectively (Tsopelas, Constantinou, Kircher & Whittaker, 1997). The objective of different load patterns is to obtain results closer to NTHA. In the present study four lateral load patterns were considered based on Mode I, IS-1893, ASCE07 and Uniform along with Triangular.

Non-linear Static Analysis was performed using SAP2000 Version 16. Non-linear static analysis requires the knowledge of material property, stress-strain model, plastic hinge property, types of hinges, hinge location, hinge length and moment-curvature relationship.

SAP defines plastic hinge properties as per FEMA-356. Hinge property defined in the form of force-deformation curve with five points labeled A, B, C, D, and E (Fig. 4). The value of these points obtained from moment curvature relationship of element depends on the type of geometry, material property, longitudinal reinforcement, shear reinforcement and loads subjected to particular member.

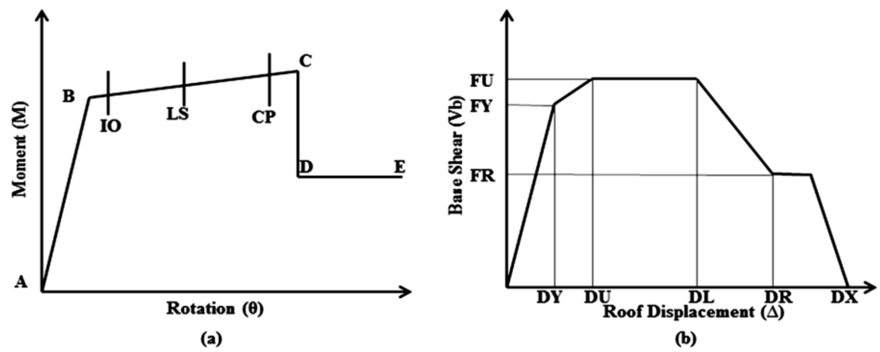

Fig. 4 : (a) A-B-C-D-E Curve for Moment vs. Rotation, (b) Idealized Monotonic Backbone Curve

Table 4 : Normalized distribution of lateral load for different load patterns

Floor\Load Pattern	Modal	IS	ASCE	Tri. + Uni.
Top	18	43	32	25
IV	29	27	24	22
III	24	17	19	18
II	17	9	14	15
I	10	3	9	12
Plinth	2	0	2	8

For the present study a two dimensional model of each structure along Grid C (Fig. 1) was modeled in SAP to perform Non-Linear Static analysis. Equivalent Loads from third dimension were applied on considered frame. For pushover analysis 100 % Dead load and 25 % of Live loads were considered as initial load. Reinforcement in the members was defined using Section Designer based on provided reinforcement. Auto hinges with hinge type P-M2-M3 and M3 hinges were assigned to columns and beams, respectively. Several formulas for plastic hinge length were proposed, in current study hinge length given by Park and Paulay Eqs. (1) was used. Locations of hinges (Fig. 5) were calculated using Eqs. 2-4 (Inel & Ozmen, 2006).

$$L_p = 0.5 \times H \tag{1}$$

$$l_1 = \frac{L_p}{2} \tag{2}$$

$$l_2 = H_{Beam} - \frac{L_p}{2} \tag{3}$$

$$l_3 = \frac{H_{Column}}{2} - \frac{L_p}{2} \tag{4}$$

L_p = Length of Plastic Hinge

H = Depth of Section

H_{Beam} = Depth of Beam

H_{Column} = Depth of Column

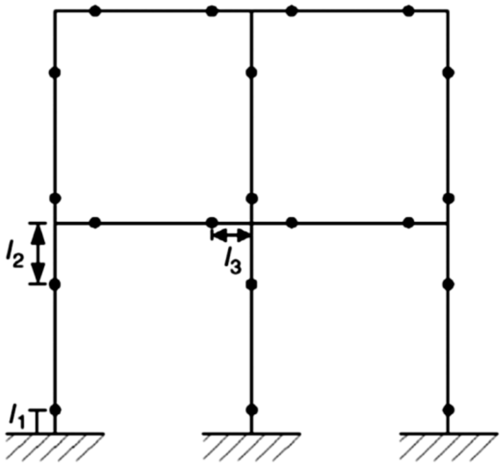

Fig. 5 : Hinge location at column and beam

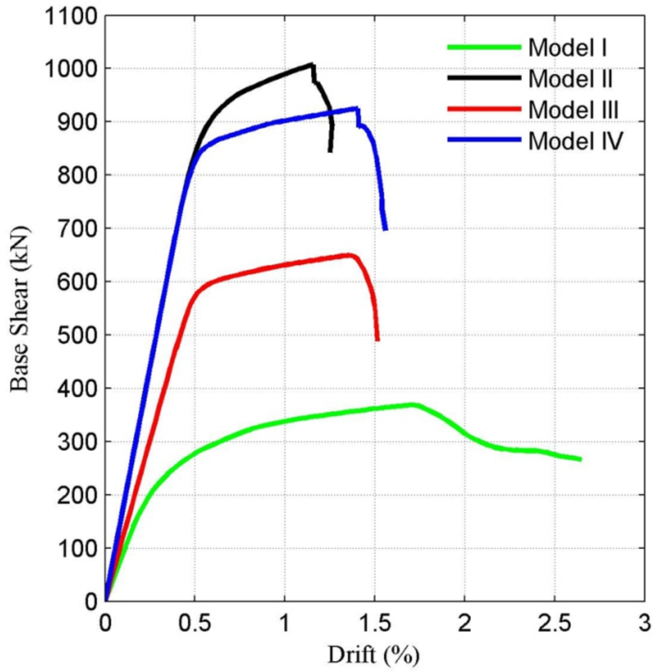

Fig. 6 : Pushover curves of building considering modal load pattern

Mander model for confined concrete and Park model for steel stress-strain were considered. The points B and C on Fig. 4 are related to yield and ultimate curvatures values.

Fig. 6 shows that model II and model IV has same stiffness and are higher than model I and model III. Strength of Model III is significantly less that of model II and IV, where as the max displacement of model III and IV are almost same. Decrease in strength and increase in ductility of model II with respect to model I is because of decrease in longitudinal reinforcement and increase in shear reinforcement, respectively. Model III and model IV are designed for same seismic force, but because of reduction in member sizes of model III leads to significant decrease in stiffness and strength, thus capacity of model III is very less than model IV.

Table 5 shows the pushover parameters in terms of elastic stiffness ($K_{Elastic}$), Yield base shear (V_y), maximum base shear (Vmax) and Ductility, values in bracket of column 3 and 4 shows the corresponding displacement.

Table 5 : Comparison of pushover parameters

Parameter	Type/ Model	I	II	III	IV
Initial Stiffness (kN/m)	Mode I	5319	10364	7113	10367
	IS Load	4203	8170	5691	8171
	ASCE Load	4858	9317	6409	9318
	Tri. + Uni.	5483.02	10554.7	7232.18	10591.5
First Yield (kN)	Mode I	100 (0.02)	684 (0.07)	445 (0.06)	687 (0.07)
	IS Load	120 (0.03)	515 (0.07)	410 (0.07)	687 (0.07)
	ASCE Load	101 (0.02)	643 (0.07)	443 (0.07)	665 (0.07)
	Tri. + Uni.	136 (0.03)	633 (0.06)	461 (0.06)	753 (0.07)
Maximum Capacity (kN)	Mode I	369 (0.29)	1008 (0.20)	649 (0.23)	926 (0.24)
	IS Load	314 (0.31)	835 (0.29)	601 (0.36)	804 (0.35)
	ASCE Load	350 (0.30)	956 (0.23)	642 (0.27)	904 (0.33)
	Tri. + Uni.	389 (0.30)	875 (0.23)	682 (0.22)	981 (0.23)
Ultimate Displacement (m)	Mode I	0.45	0.21	0.25	0.26
	IS Load	0.35	0.33	0.4	0.37
	ASCE Load	0.35	0.25	0.3	0.34
	Tri. + Uni.	0.33	0.26	0.24	0.26

Fig. 7 shows the pushover curves obtained for all models considering different load patterns.

The distribution of lateral load considering different load patterns (Fig. 7) clearly shows the change in pushover curve. The change in pushover parameters corresponding to load patterns is shown in Table 5. In Triangular along with Uniform load pattern, strength and maximum displacement had higher and greater value, respectively compared to other load patterns. Fig. 8 shows the formation of hinges at ultimate displacement. More number of hinges formed in model IV shows that the strength of each member was utilized properly compared to other models. Fig. 7 (d) shows with change in load pattern the initial stiffness of the structure also changes.

Ductility

In the last decade extensive work was done to determine the ductility factor by Newmark and Hall, Nassar and Newmark, Vidic et al. and Krawinkler and Nassr. In the present study relationships developed by Pristley is used. Ductility is defined as the ratio of maximum

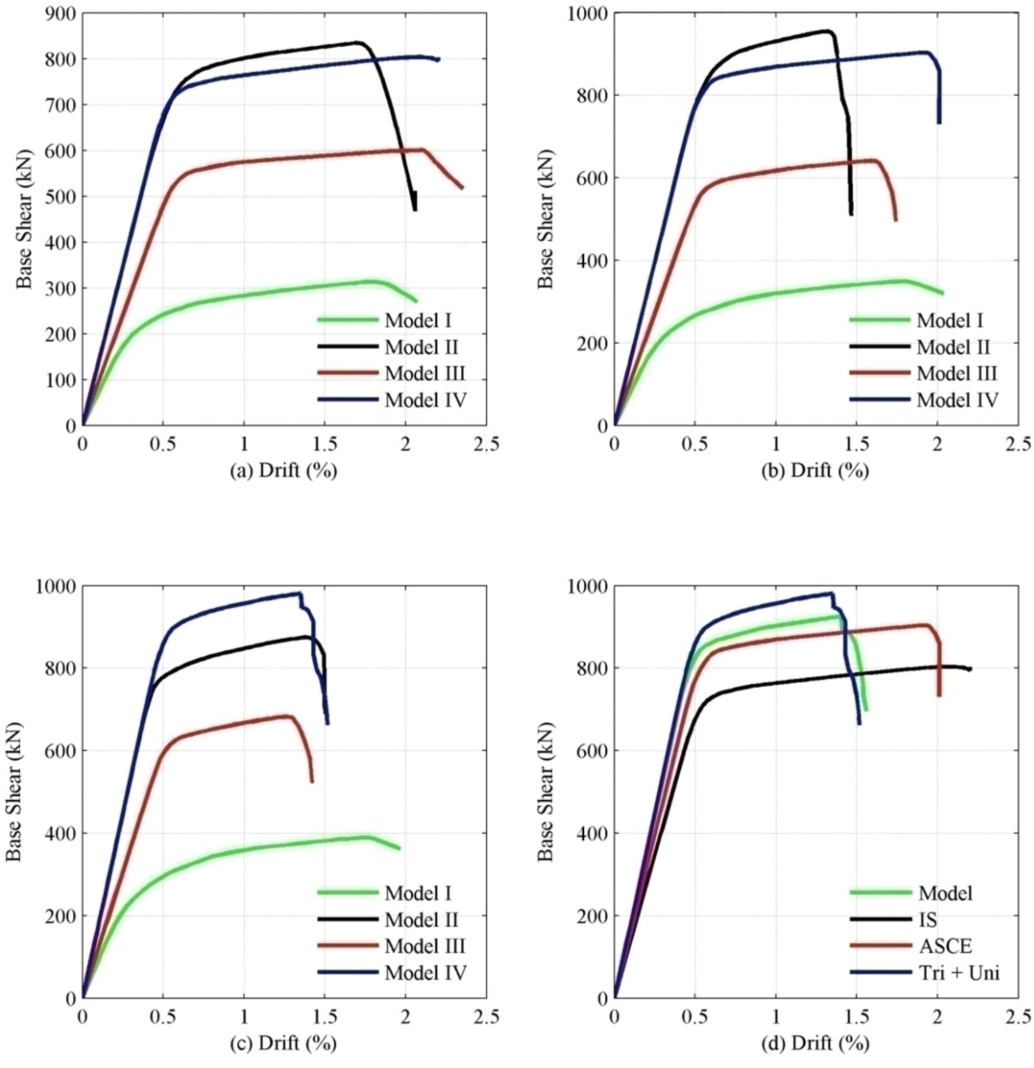

Fig. 7 : Pushover curves (a) IS load pattern, (b) ASCE load pattern, (c) Triangular along with Uniform load pattern and (d) Comparison for different load patterns for Model IV.

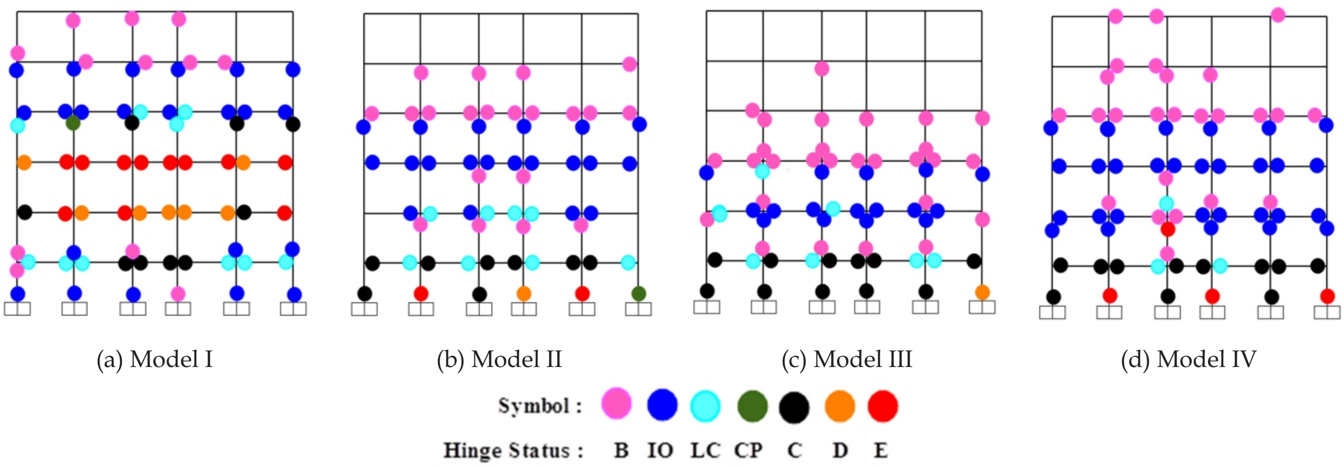

Fig. 8 : Hinge status at ultimate displacement

Table 6 : Ductility factor for calculation for all models

Load Pattern	Mode I				IS 1893				ASCE 07				Tri. + Uni.			
Model	I	II	III	IV	I	II	III	IV	I	II	III	IV	I	II	III	IV
Δ_y (m)	0.02	0.07	0.06	0.07	0.03	0.06	0.07	0.07	0.02	0.07	0.07	0.07	0.03	0.06	0.06	0.07
Δ_m (m)	0.45	0.21	0.25	0.26	0.35	0.33	0.4	0.37	0.35	0.25	0.3	0.34	0.33	0.26	0.24	0.26
$\mu= \Delta m/\Delta y$	22.50	3.00	4.14	4.08	11.67	5.52	5.76	5.39	17.50	3.71	4.43	4.92	4.14	4.28	3.78	3.61
R_μ= sqrt(2μ-1)	6.63	2.24	2.70	2.68	4.73	3.17	3.24	3.13	5.83	2.53	2.80	2.97	2.70	2.75	2.56	2.49

displacement to yield displacement. Ductility factor in the present paper is calculated as per the relation given by Pristley. Table 6 shows the values of ductility and ductility factor for different models with respect to different load patterns.

Drift

IS code specifies a max drift of 0.04 % of storey height for lateral load. ASCE specifies max drift as the product of elastic drift and C_d, if obtained drift

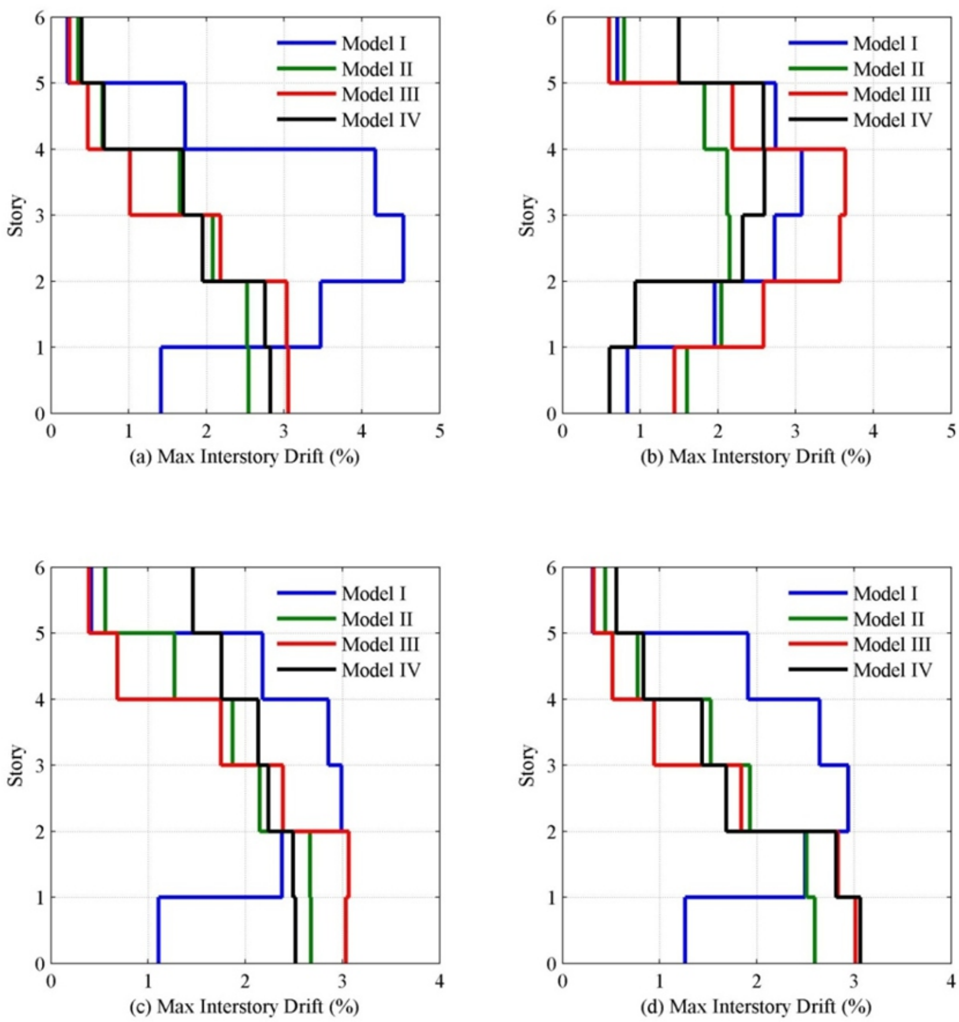

Fig. 9 : Interstorey Drift profile for (a) Modal Load Pattern, (b) IS load pattern, (c) ASCE load pattern and (d) Triangular along with Uniform load pattern

is more than above specified drift the member dimensions are required to be increased to achieve drift in permissible limit. As per the C_d factor given in ASCE for max drift a constant if multiplied, means that the relation between linear and non linear drift is same but the nonlinear drift may change depending upon the damage in storey. In this section the differences in drift at elastic force, yield force and at ultimate displacement and the change in damage in building with respect to different load patterns was studied.

Fig. 9 shows the interstorey drift profile of all models. Interstorey drift is direct estimation of damage occurred in the structure. Fig. 9 shows that drift is more in model I compared to other models. Model III has higher maximum interstorey drift compared to model II and IV, which clearly shows that non ductile design with reduced member dimension, will have more damage. The interstorey drift in floors will not have the same proportion, as it depends on the relative strength of storey which ultimately depends on the design. The values of interstorey drift also depend on load pattern, the change in load pattern leads to significant change in the damage pattern of the building. Thus assumption of lateral load pattern plays a significant role in estimating the response of structure. Fig. 10 shows the displacement profile of

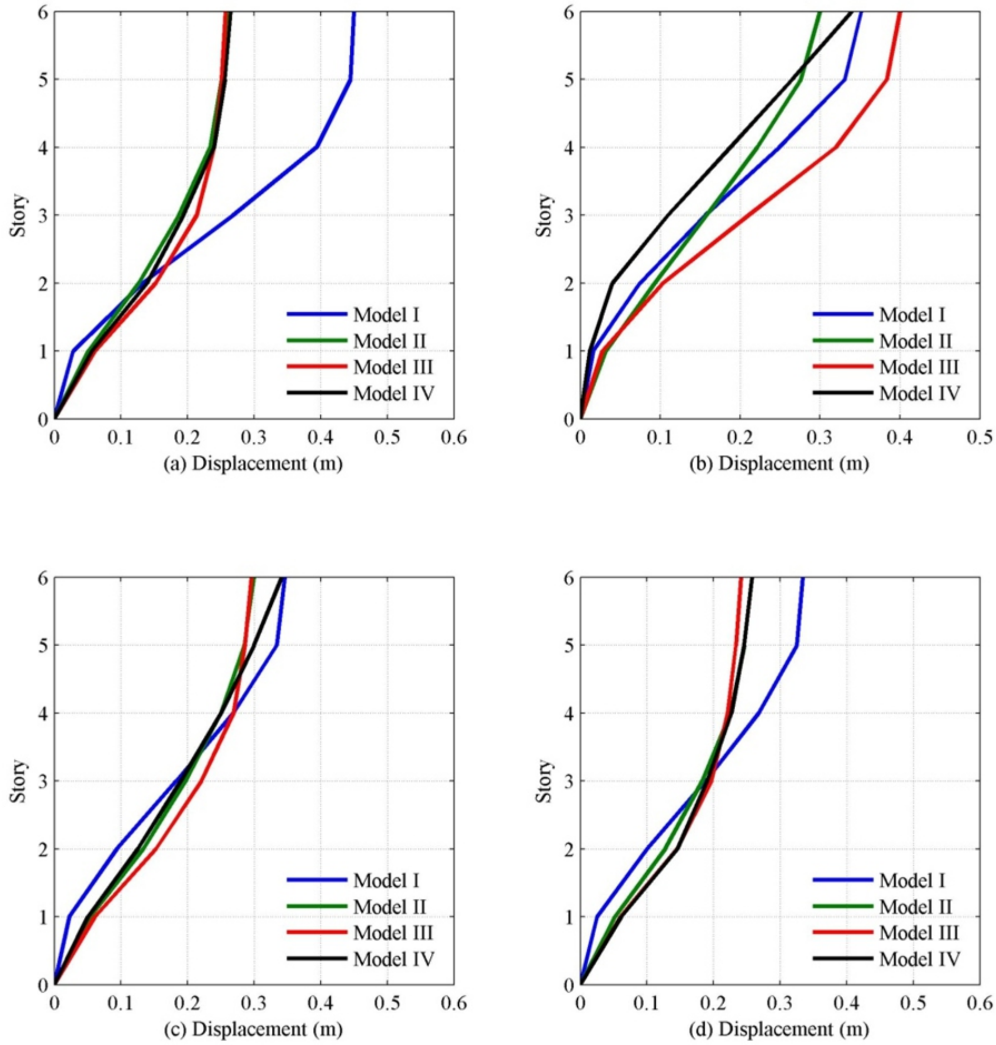

Fig. 10 : Displacement profile for (a) Modal Load Pattern, (b) IS load pattern, (c) ASCE load pattern and (d) Triangular along with Uniform load pattern

buildings. The maximum storey drift (Fig. 7) ranges from 1.5 % to 2.5 % for all buildings considering different load patterns, indicates that structure is safe, whereas the interstorey drift (Fig. 9) ranges from 2.5% to 4%. Thus it is important to determine the interstorey drift also, as the structure may be safe in storey drift but may fail in interstorey drift criteria specified in ATC-40 and FEMA-356.

Displacement amplification factor (Cd) as specified in ASCE07, shows a constant relation between the elastic drift and maximum allowable drift. Fig. 10 shows interstorey drift and displacement profile of all buildings at different stages. ND, RS and SS correspond to model II, III and IV, respectively. Y, UL and MD correspond to three stages in pushover curve; first yield, maximum base shear and maximum displacement, respectively. It shows that the ratio between elastic and maximum drift is not constant and alter significantly from floor to floor.

Fragility Analysis

Fragility curves are used for representing extent of damage structure can have when subjected to seismic forces. Vertical and Horizontal axis of fragility curve consists of Damage and spectral acceleration, respectively. If maximum PGA value of ground motion is known than expected level of damage from that earthquake can be approximately estimated. In the current study fragility curve were developed from Pushover curve. Conversion factors [ATC 40, 1996].

Conversion form Base Shear to Damage

$$\text{Damage} = \frac{E_i}{E_{max}} \tag{5}$$

E_{max} = Area under the pushover curve with line dropped parallel to initial stiffness at the end point.

E_i = Energy dissipated at every displacement (Area under the curve at every displacement with line dropped parallel to initial stiffness.

Conversion from Roof Displacement to Spectral Acceleration:

$$S_{di} = \frac{\Delta_{roof}}{PF_1 \times \phi_{1,\,roof}} \tag{6}$$

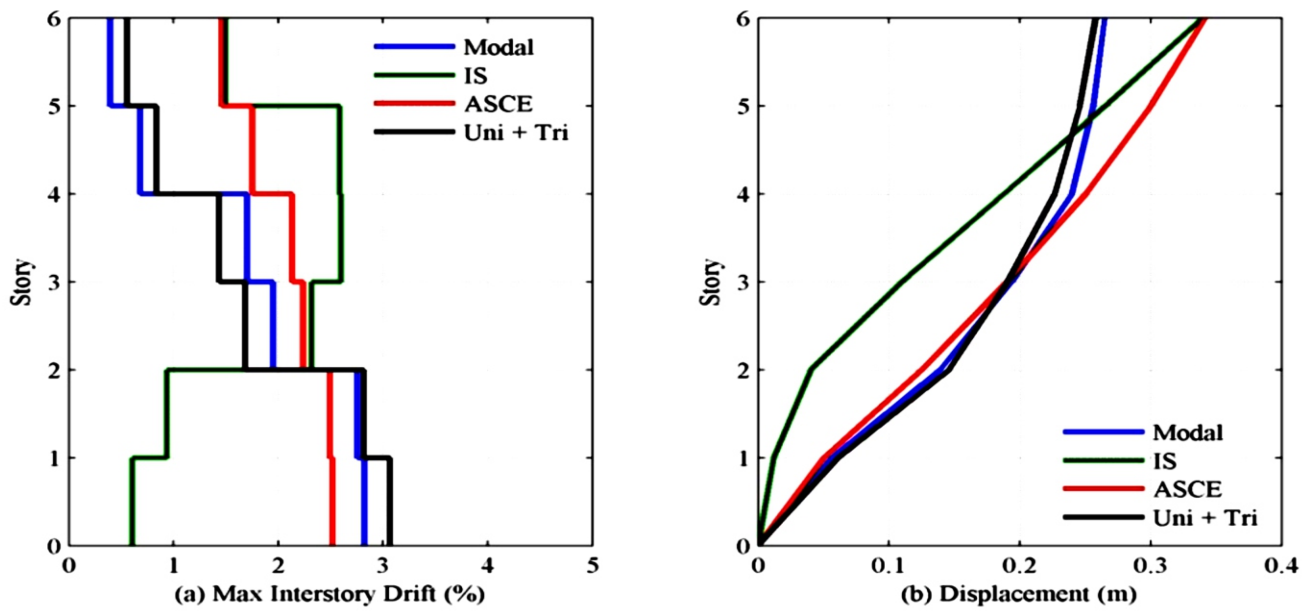

Fig. 11 : Drift and Displacement profile comparison for load patterns of Model IV

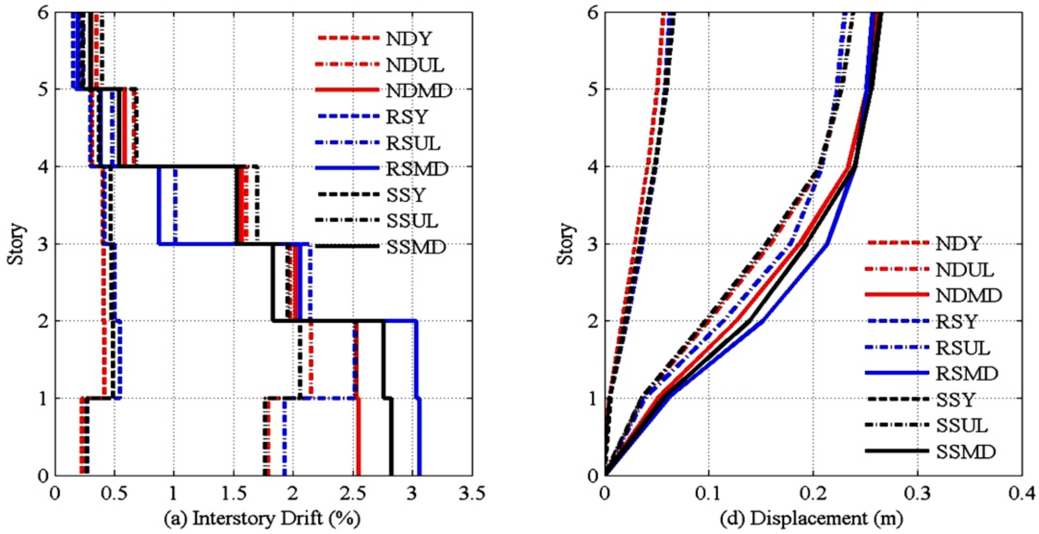

Fig. 12 : Comparison of drift and displacement at first yield, ultimate load and maximum displacement for modal load pattern

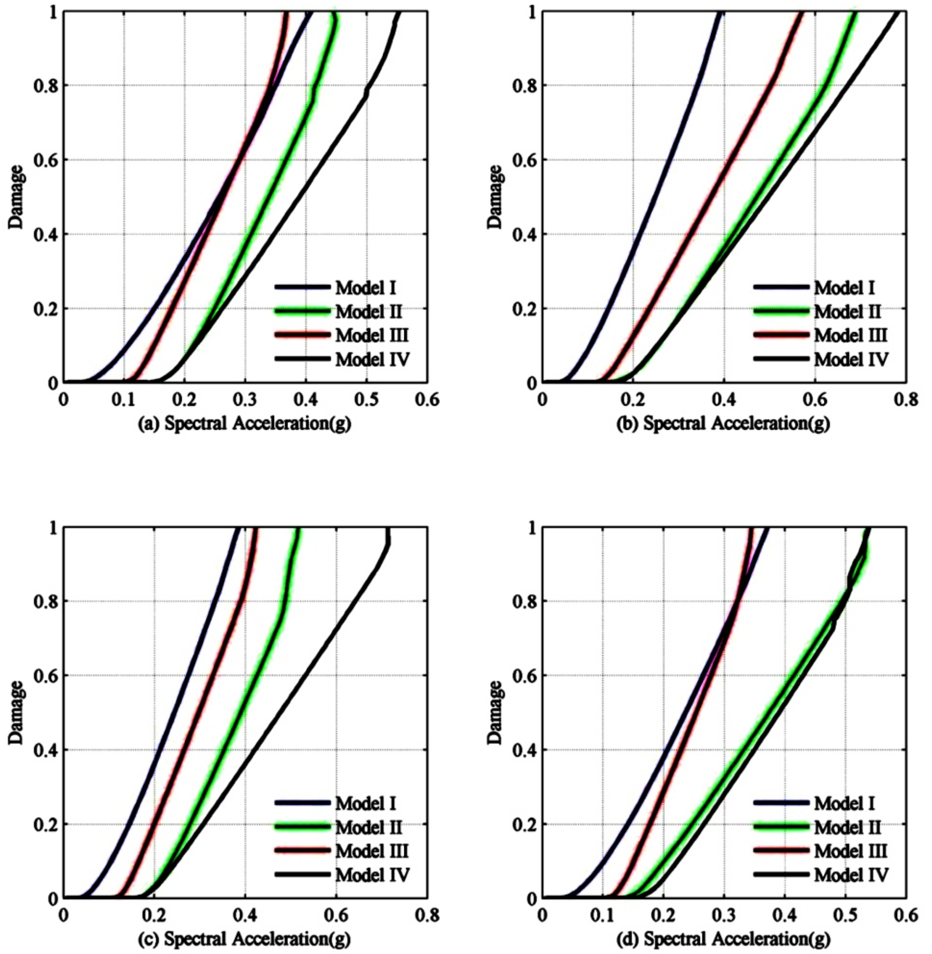

Fig. 13 : Fragility curve for (a) Modal Load Pattern, (b) IS load pattern, (c) ASCE load pattern and (d) Triangular along with Uniform load pattern

S_{di} : Spectral Displacement

ϕ_{roof} : Roof displacement obtained from pushover curve

PF_1 : Participation factors for the first natural mode of the structure

$\varnothing_{1,roof}$: Roof level amplitude of the first mode

$$S_{ai} = \frac{4\pi^2 S_{di}}{T^2 g} \quad (7)$$

T : Time Period of the structure

g : Acceleration due to gravity

Values of Δ_{roof}, PF_1, $\varnothing_{1,roof}$ and T are given in Table 9.

Fig. 13 shows the obtained fragility curves for all models. Extent of damage in buildings was computed from fragility curves with respect to Design Basis Earthquake (DBE) Table 7. It was clearly observed from Table 7 that extent of damage was more for model I, difference in damage up to DBE was not significant but after DBE the damage was very high model I. Model III is having more damage compared to model II, thus ductile detailed structure will give less damage only if member dimensions are kept more than or equal to normal detailed structures. Model IV has the least damage compared other models, thus by providing ductile detailing in the building the damage can be reduced to greater extent.

Table 7 : Amount of damage at different seismic hazard levels

Load Pattern	Model	DBE	2 DBE	3 DBE	4 DBE
Modal	I	5.0	22.4	69.2	>100
	II	0.0	0.6	43.5	>100
	III	0.0	13.5	71.2	>100
	IV	0.0	0.6	33.4	72.2
IS	I	4.7	23.9	72.9	>100
	II	0.0	0.3	21.4	51.6
	III	0.0	4.4	38.6	74.9
	IV	0.0	0.1	20.7	47.2
ASCE	I	5.0	24.2	75.3	>100
	II	0.0	0.2	30.9	76.0
	III	0.0	8.3	57.2	>100
	IV	0.0	0.1	22.2	50.6
Tri.+ Uni.	I	5.4	25.4	79.3	>100
	II	0.0	2.1	37.0	76.4
	III	0.0	13.4	78.8	>100
	IV	0.0	0.4	32.7	73.0

Results

Ductile detailed building (Mode III), considering higher values of R factor leads to decrease in member size had shown higher amount of damage compared to non ductile detailed structure (Model II).

Ductile detailed building (Mode IV), considering higher values of R factor and keeping member size same as in non ductile detailed structure (Model II) had shown less damage compared to ductile detailed reduced member size (Model III) and non ductile detailed structure (Model II).

Response of the structure changes completely based on the load pattern assumed in pushover analysis, thus it becomes important to assume load pattern, which gives results closer to actual capacity of structure.

Increase in drift in ductile detailed structure ranges from 0.5% to 0.75%, but max interstorey drift was reduced by 2% to 3% compared to non ductile detailed structure.

Interstorey drift value changes based on the assumed lateral load pattern in pushover analysis, Damage in top floors was more for IS and ASCE load patterns and less for modal and triangular along with uniform load pattern. Damage in ground floors was more for modal, ASCE and triangular along with uniform load pattern, and less for IS load pattern.

Relation between elastic interstorey drift and maximum allowable drift is constant as per ASCE code, results obtained shows the ratio of interstorey drift and maximum drift is not constant and varies depending upon the relative damage between the storeys.

Damage in the structure depends on combination initial stiffness, strength and ductility. Model I had maximum displacement but less stiffness and strength shown higher damage. Model III had equal stiffness and higher strength but less maximum displacement shown higher damage compared to model IV. Model IV had equal stiffness and higher maximum displacement but slightly less strength shown lesser damage compared to all other models.

Conclusions

Static non-linear analysis and fragility analysis is carried out on a 2D Frame of a 5 storey structure considering four different cases based on seismic force and type of detailing. Results show that earthquake resistant design is to be followed to decrease loss in terms of fatalities and economy. The damage in the structures can further be reduced by ductile detailing provisions. Assumed load pattern in pushover analysis plays an important role in the non linear response of structure. Design provisions for ductile detailing need to be modified as it has been observed that with increased R values, the member size decreases and lead to structures having more damage compared to normal detailed structures. Whereas damage is less when member sizes of ductile detailed structure is kept same or more compared to normal detailed structures.

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