Influence of parameters on performance evaluation of designed RC buildings: seismic hazard

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Abstract: Seismic engineering of structures is in discussion since decades while the aspects of risk mitigation and hazard assessment are relatively new in this field concerned with our preparedness for future events. With the advancement in knowledge and advent of performance-based design procedures, it has become possible to safeguard our interests against the fury of nature. In this paper, performance of a seven storey building is evaluated for different hazard levels expected in its lifetime. Four cases are considered for building representing: a) non-seismic code compliant building; b) designed for lateral loading pattern conforming to IS1893:2002; c) IS1893 draft code; d) EC8. The efficient design and detailing of building according to present code of practice is found to satisfy the Life Safety performance objective for the MCE level earthquake of 0.24 g. Non-structural damages is expected for the building under even the 0.1 g MCE level earthquake as the storey-drift exceeds the code limit. The performance of building is evaluated for extreme events at structure level and at non-structural level under each hazard level leading to notification of weaknesses in the structure with respect to the parameters of evaluation.

Keywords: performance-based design; PBD; seismic hazard; detailing; vulnerability; direct displacement-based design; DDBD.

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1 Introduction

Seismic risk in regions prone to earthquakes can be determined by the study of geographic areas and historical earthquakes, based on which seismic hazard maps are generated. The likely PGA values are mentioned with a certain probability of exceedance (MCE-2%, DBE-10%) for that region. Design engineers and state authorities use these values to determine the appropriate earthquake loading for buildings in particular zone to survive MCE level earthquake. The damage and devastation produced by an earthquake depend on its location, depth, proximity to populated regions and its true size.

India is a region experiencing seismic activities since ancient times. According to BIS seismic zoning map, over 65% of the country is prone to earthquakes of intensity MSK VII or more, putting 38 cities in high risk zones. In 2011, there were 80 earthquakes in India, with magnitude ranges between 3.5 M to 6.5 M on Richter scale. The year 2012 was an equally eventful year with 19 earthquakes by 5th March. Majority of these quakes occurred in the northern regions of Jammu and Kashmir, Haryana, Punjab and Gujarat. Earthquakes with magnitude of about 2.0 M or less are called micro-earthquakes and are not commonly felt. Events with magnitudes of 4.5 M or greater, occurring in large numbers per year, are strong enough to be recorded by seismographs all over the world. While, great earthquake having magnitude of 8.0 M or higher occurs somewhere in the world every year (http://earthquake.usgs.gov).

Many stable continent regions of the country have suffered the jolt of nature and were caught off-guard. The 1993 Killari earthquake, 1997 Jabalpur and the 2001 Bhuj earthquakes are the Stable Continental Region (SCR) earthquakes in peninsular India. The 1991 Uttarkashi, the 1999 Chamoli and the 2005 Kashmir earthquakes are the Himalayan collision zone earthquakes. Various studies have been done in past to assess the vulnerability of different regions of the country like Mumbai (Raghukanth and Iyengar, 2006), Tamil Nadu (Menon et al., 2010), Kalpakam (Kanagarathinam et al., 2008), Karnataka (Sitharaman et al., 2012), Delhi (Iyengar and Ghosh, 2004) and Gujarat (Chopra et al., 2012). These studies press to the fact that majority of damage is due to low-moderate level earthquakes in addition of vast devastation caused by high return period major earthquakes (MCE) for which the performance of building needs to be evaluated.

Estimation of seismic demand for a structure is the most important aspect of performance evaluation and subsequent mitigation actions. A constant check of the seismicity of the regions of the country would provide right input to the engineers and lead to safer establishments. The engineering parameter, EPA, is defined as the peak value of the truncated ground acceleration record for which the spectrum intensity is 90% of that computed for the original time history (Watabe and Tohdo, 1979). Based on EPA, the earthquake destructiveness potential factor P_D is commonly used for comparing the severity of ground shaking (Araya and Saragoni, 1984). IS1893:2002(P1) specifies EPGA values for various regions of country {0.1 g, 0.16 g, 0.24 g, 0.36 g} in terms of zone factor for calculation of seismic demand on structure based on its seismic weight and acceleration coefficient. List of severe earthquakes in India are mentioned in Table 1 along with the hazard in terms of magnitude, intensity and PGA values for each event.

Table 1 List of severe earthquakes in India: magnitude-PGA-intensity

Event	*Year*	*Magnitude*	*PGA*	*Intensity*	*Casualty*
Bihar-Nepal	1934	8.2	0.3 g	IX	> 10,000
Assam	1950	8.7	-	X	1,500
India-Burma	1988	7.2	0.34 g	VIII	709
India-Bangladesh	1988	5.8	0.1 g	VI–VII	-
Garhwal	1991	7.1	0.3 g	VIII	768
Uttarkashi	1991	7	0.29 g	IX	> 2,000
Koyna	1967	6.5	0.4 g	VIII	1,500
Chamoli	1999	6.6	0.34 g	VIII	103
Bhuj	2001	7.7	0.38 g	VIII	20,000
Kashmir	2005	7.6	0.23 g	VIII	> 80,000
Sikkim	2011	6.9	0.35 g	VI	111
Nepal	2015	7.9	-	IX	> 8,000
Chamoli Aftershock	1999	5.4	0.06 g	VI	-

An attempt has been made in this paper to check the performance of a seven storey building designed as per code of practice for the next seismic event in the region. The hazard considered for the building range from frequent minor, moderate earthquakes, design basis earthquake (DBE) to MCE expected in its lifetime. Seismic hazard is represented by a measure of ground shaking and its effect on the structure is evaluated at structural and non-structural levels. The Bhuj earthquake (2001) had caused severe damage to property and more than 20,000 people were killed. The reported PGA of the earthquake was 0.38 g having magnitude 7.7 M. Around 70 multistorey buildings collapsed in Ahmedabad with reported PGA of 0.1 g for which the damage observed was higher than expected. Hence, the influence of present process of design of buildings on the performance of RC structures is checked. The paper covers the aspect of creating resilient structures and measuring our preparedness for a next higher seismic event in Indian subcontinent using performance-based design (PBD) procedures. This study is the extension of earlier study done on influence of parameters on performance evaluation of RC structures (Bhushan, 2014).

2 Problem description

A regular seven storey RC building is considered for performance evaluation. The plan of building is as shown in Figure 1 with 16 m in X direction (4 m c/c) and 12 m in

Y direction (3 m c/c). The typical storey height is 2.8 m and the building overall height is 19.6 m. Building is considered to be located in Zone IV having EPGA of 0.24 g with medium soil conditions. Four cases of buildings are considered for study. First three cases are evaluation of building designed for lateral loading with variation in lateral loading patterns prescribed by

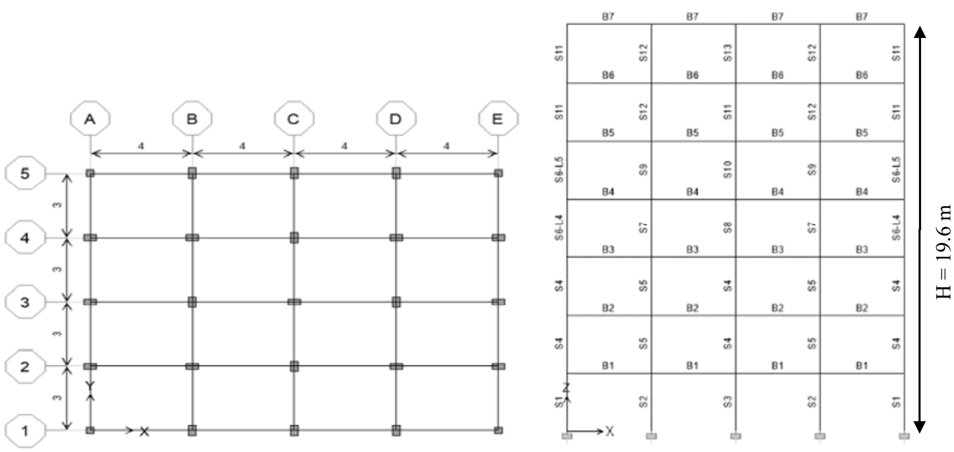
1 IS1893:2002

2 IS1893 draft code

3 EC8.

The fourth case is of performance evaluation of the same building designed only for gravity loads. The design of building is carried out according to IS 456 (2000). The detailing of building is done as per IS456:2000 and IS13920 and the performance are compared. The performance of building is evaluated using nonlinear static analysis as per ATC40 in ETABS software and direct displacement-based design (DDBD) procedure using DBDsoft.

Figure 1 Building details – plan, elevation and member nomenclature of seven storey building



3 Estimation of seismic demand for building

Demand estimation of structures has evolved from the stage of prescribed loading patterns to adaptive loading patterns representing the damage progress in the structure. In this paper, the loading patterns prescribed by the codes are considered as these are used in general practice. Seismic demands for the building are estimated using IS 1893 Part I (2002) and IS1893 latest draft code. The seismic weight of the building is 14,956.2 kN and estimated base shear for zone IV is 1,495.6 kN which is distributed along the height of building as shown in Table 2 as per the equations mentioned below. Time period of building is 0.44s in X direction and 0.51 s in Y direction calculated as per IS1893:2002(P1) for buildings with infill.

IS 1893 2002, Parabolic pattern: $$Q_i = \frac{w_i h_i^2}{\sum_1^n w_i h_i^2} V_b \quad (1)$$

IS 1893 latest draft code: $$Q_i = \frac{w_i h_i^k}{\sum_1^n w_i h_i^k} V_b \quad (2)$$

EC8, 1st mode loading pattern: $$Q_i = \frac{w_i \phi_{1i}}{\sum_1^n w_i \phi_{1i}} V_b \quad (3)$$

where

$k = 1$ $T \le 0.5$ sec (inverted triangular pattern)

$k = 2$ $T \ge 2.5$ sec (parabolic pattern)

ϕ_{1i} Eigen vector in 1st mode for building at level $i = \{1.0, 0.94, 0.83, 0.67, 0.48, 0.29, 0.11\}^T$

Table 2 Distribution of base shear Q_i (kN) in building

Storey	*Distribution pattern*		
	Parabolic	*Triangular*	*1st mode*
7	330.97	350.55	204.54
6	457.59	324.30	362.03
5	320.80	272.83	322.31
4	204.78	217.69	259.38
3	116.28	164.81	189.04
2	51.68	109.88	115.25
1	12.92	54.94	42.46

4 Design and detailing of building

Building is designed for load combination as per IS456:2000 and ductile detailing is done as per IS 13920 (2002). P-Delta effects are considered for design. Capacity ratio between beams and columns are reviewed such that the capacity of columns is higher than that of beams. The result of design and detailing of beams are shown in Table 3 and Table 4 for three lateral loading patterns. The column reinforcement details are mentioned in Table 5. The shear reinforcement as per ductile detailing code requires that the shear resistance in columns at hinge formation shall be 1.4 times the moment capacity at the two ends of the element for sway towards right and sway towards left. Thus the failure mechanism considered for design is flexure mode by providing adequate shear reinforcement at probable plastic hinge locations for design level earthquake demands. Typical reinforcement detailing for column and joint according to IS13920 is shown in Figure 2. For building designed only for gravity loading, design and detailing is as per IS456:2000.

Figure 2 Typical column and joint detailing according to IS13920 (see online version for colours)

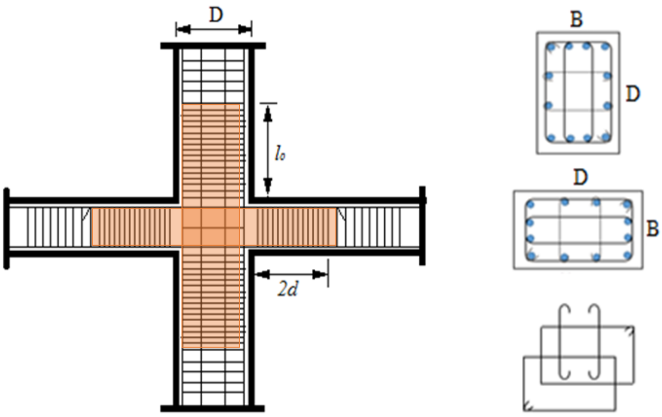


Table 3 Reinforcement details of beams (grid 3) under parabolic and 1st mode loading patterns

Beam	*Size*	*Top R/F*	*Bottom R/F*	*Shear R/F*	
				Ends	*Centre*
B7	0.45 × 0.25	3-12dia HYSD, Ast = 339 mm^2	3-12dia HYSD, Ast = 339 mm^2	8d-2leg-95c/c	8d-2leg-200c/c
B6	0.45 × 0.25	4-16dia HYSD, Ast = 804 mm^2	4-12dia HYSD, Ast = 452 mm^2	8d-2leg-95c/c	8d-2leg-200c/c
B5	0.45 × 0.25	4-20dia HYSD, Ast = 1,257 mm^2	4-16dia HYSD, Ast = 804 mm^2	8d-2leg-100c/c	8d-2leg-180c/c
B4	0.45 × 0.25	4-20dia HYSD, Ast = 1,257 mm^2	3-20dia HYSD, Ast = 942 mm^2	8d-2leg-100c/c	8d-2leg-180c/c
B3	0.5 × 0.25	3-25dia HYSD, Ast = 1,473 mm^2	2-25dia HYSD, Ast = 982 mm^2	8d-2leg-100c/c	8d-2leg-140c/c
B2	0.5 × 0.25	3-25dia HYSD, Ast = 1,473 mm^2	2-25dia HYSD, Ast = 982 mm^2	8d-2leg-100c/c	8d-2leg-140c/c
B1	0.5 × 0.25	3-25dia HYSD, Ast = 1,473 mm^2	2-25dia HYSD, Ast = 982 mm^2	8d-2leg-100c/c	8d-2leg-140c/c

Table 4 Reinforcement details of beams (grid 3) under triangular loading pattern

Beam	*Size*	*Top R/F*	*Bottom R/F*	*Shear R/F*	
				Ends	*Centre*
B7	0.45 × 0.25	3-12dia HYSD, Ast = 339 mm^2	3-12dia HYSD, Ast = 339 mm^2	8d-2leg-95c/c	8d-2leg-200c/c
B6	0.45 × 0.25	3-16dia HYSD, Ast = 603 mm^2	3-12dia HYSD, Ast = 339 mm^2	8d-2leg-95c/c	8d-2leg-200c/c
B5	0.45 × 0.25	3-20dia HYSD, Ast = 942 mm^2	3-16dia HYSD, Ast = 603 mm^2	8d-2leg-100c/c	8d-2leg-180c/c
B4	0.45 × 0.25	4-20dia HYSD, Ast = 1,257 mm^2	3-20dia HYSD, Ast = 942 mm^2	8d-2leg-100c/c	8d-2leg-180c/c

Table 4 Reinforcement details of beams (grid 3) under triangular loading pattern (continued)

Beam	*Size*	*Top R/F*	*Bottom R/F*	*Shear R/F*	
				Ends	*Centre*
B3	0.5 × 0.25	4-20dia HYSD, Ast = 1,257 mm^2	3-20dia HYSD, Ast = 942 mm^2	8d-2leg-100c/c	8d-2leg-140c/c
B2	0.5 × 0.25	4-20dia HYSD, Ast = 1,257 mm^2	3-20dia HYSD, Ast = 942 mm^2	8d-2leg-100c/c	8d-2leg-140c/c
B1	0.5 × 0.25	4-20dia HYSD, Ast = 1,257 mm^2	3-20dia HYSD, Ast = 942 mm^2	8d-2leg-100c/c	8d-2leg-140c/c

Table 5 Reinforcement details of columns under lateral loading patterns

Column	*Size (0°)*	*Size (90°)*	*Longitudinal R/F*	*Transverse R/F*	
				Ends (l0)	*Centre (h-2l0)*
Level 7	0.5 × 0.3	0.3 × 0.50	16-12dia HYSD, Ast = 1,357 mm^2	8d-100c/c	8d-135c/c
Level 6	0.5 × 0.3	0.3 × 0.5	12-16dia HYSD, Ast = 2412.8 mm^2	8d-100c/c	8d-135c/c
Level 5	0.55 × 0.3	0.3 × 0.55	12-20dia HYSD, Ast = 3,770 mm^2	10d-100c/c	10-125c/c
Level 4	0.55 × 0.3	0.3 × 0.55	12-25dia HYSD, Ast = 5,890.5 mm^2	10d-100c/c	10-125c/c
Level 3	0.6 × 0.3	0.3 × 0.6	12-25dia HYSD, Ast = 5,890.5 mm^2	10d-100c/c	10-125c/c
Level 2	0.6 × 0.3	0.3 × 0.6	12-25dia HYSD, Ast = 5,890.5 mm^2	10d-100c/c	10-125c/c
Level 1	0.6 × 0.3	0.3 × 0.6	12-25dia HYSD, Ast = 5,890.5 mm^2	10d-100c/c	10-125c/c

5 Performance evaluation of building

The performance of a building is measured at global and local levels based on the acceptability limits at each performance level prescribed by ATC40. The capacity of building designed for a particular hazard in a region is defined by the capacity curve obtained through pushover analysis procedure (Krawinkler, 1996). The performance is measured on the same platform as the hazard using Capacity Spectrum Method (Freeman et al. 1975; ATC 40). If the capacity curve breaks through a demand envelope, the building survives an earthquake (Freeman, 1998). For the seven storey building, capacity curve is shown in Figure 3. Curve 1 represents capacity of building for 1st mode loading pattern, Curve 2 for triangular loading pattern and Curve 3 for parabolic loading pattern. Curve 4 represents the capacity of existing building designed only for gravity loads. The influence of detailing w.r.t. spacing of ties for column at base of building is shown in Figure 4. The performance point is obtained from ETABS using CSM as shown in Figure 5. Time period of building in 1st mode is 1.22 sec. The performance is evaluated for building under DBE and MCE level earthquakes. For the buildings designed for DBE level earthquake of 0.24 g and the one designed for only gravity loads (T_n = 1.04 s), the shear capacity of the sections are checked for MCE level earthquake using user-defined shear hinges in beams and columns for the detailing done at design stage.

Figure 3 Capacity curve of seven storey building for three loading patterns

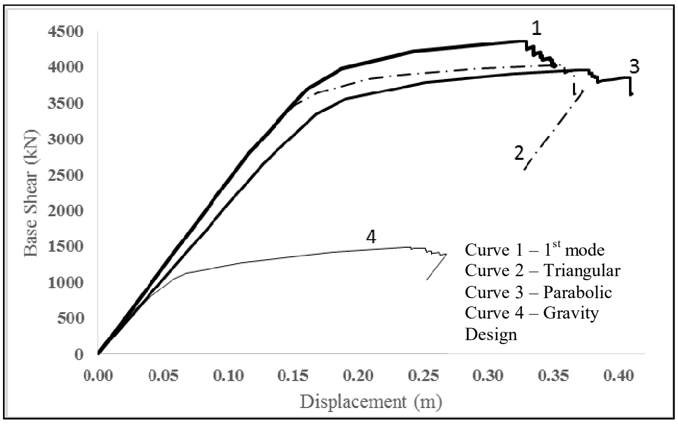
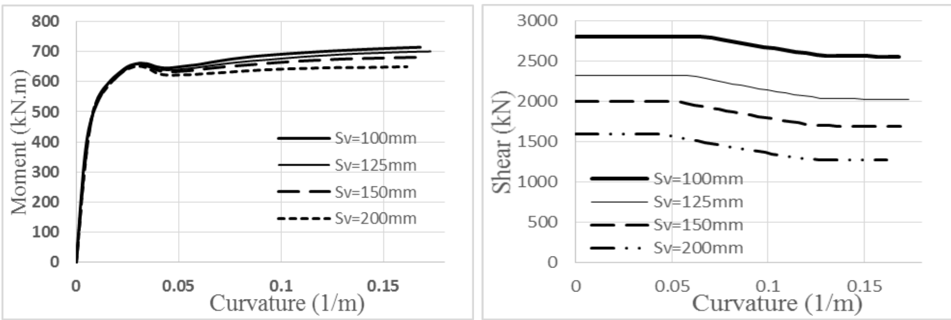
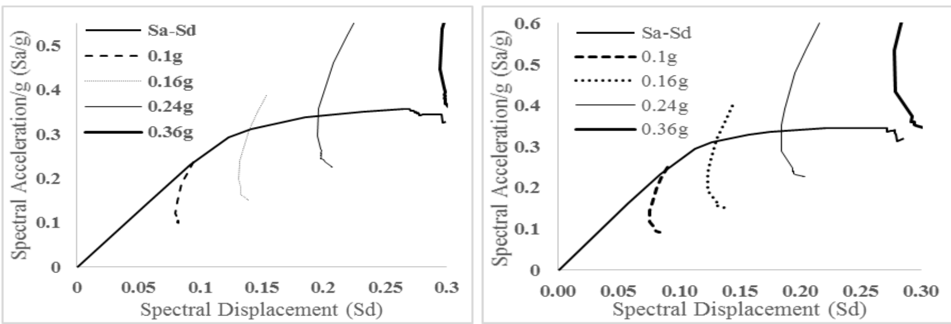


Figure 4 Effect of spacing of ties on performance of column s1 at the base of building



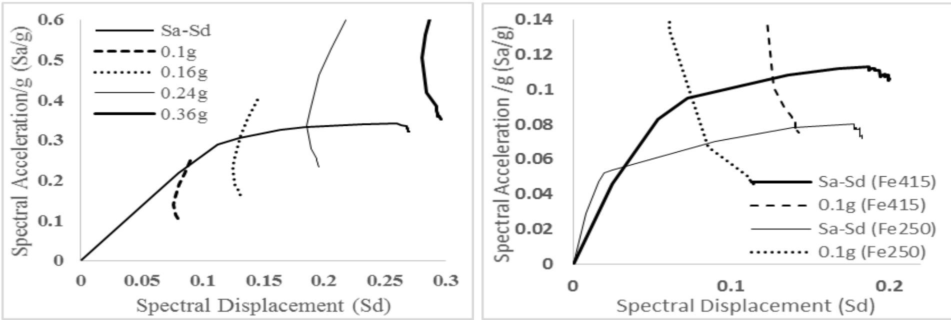
Source: RC Analysis software (2008)

Figure 5 ADRS plots of seven storey building designed for the four cases considered for building, (a) IS 1893 2002 (b) IS 1893 draft code (c) EC8 (d) gravity designed



(a) (b)

Figure 5 ADRS plots of seven storey building designed for the four cases considered for building, (a) IS 1893 2002 (b) IS 1893 draft code (c) EC8 (d) gravity designed (continued)



(c) (d)

Table 6 Performance of seven storey building designed for parabolic load pattern

Hazard	*V (kN)*		*D (m)*		S_a		S_d		β_{eff}	
	DBE	*MCE*	*DBE*	*MCE*	*DBE*	*MCE*	*DBE*	*MCE*	*DBE*	*MCE*
0.1 g	1,354	2,689	0.07	0.13	0.12	0.24	0.047	0.094	5%	5%
0.16 g	2,167	3,544	0.1	0.19	0.2	0.31	0.076	0.14	5%	9%
0.24 g	3,121	3,811	0.15	0.27	0.27	0.34	0.113	0.2	6.1%	16%
0.36 g	3,622	-	0.21	-	0.32	-	0.15	-	11%	-

Table 7 Performance of seven storey building designed for triangular load pattern

Hazard	*V (kN)*		*D (m)*		S_a		S_d		β_{eff}	
	DBE	*MCE*	*DBE*	*MCE*	*DBE*	*MCE*	*DBE*	*MCE*	*DBE*	*MCE*
0.1 g	1,460	2,855	0.06	0.12	0.125	0.24	0.045	0.09	5%	5.3%
0.16 g	2,345	3,657	0.097	0.17	0.2	0.3	0.072	0.13	5%	9.8%
0.24 g	3,233	3,908	0.14	0.24	0.28	0.34	0.1	0.18	6.3%	17.7%
0.36 g	3,743	-	0.19	-	0.32	-	0.14	-	12%	-

Table 8 Performance of 7 storey building designed for 1st mode load pattern

Hazard	*V (kN)*		*D (m)*		S_a		S_d		β_{eff}	
	DBE	*MCE*	*DBE*	*MCE*	*DBE*	*MCE*	*DBE*	*MCE*	*DBE*	*MCE*
0.1 g	1,455	2,790	0.06	0.12	0.12	0.24	0.045	0.09	5	5.6
0.16 g	2,347	3,574	0.097	0.17	0.2	0.31	0.07	0.13	5	10.3
0.24 g	3,153	3,831	0.14	0.25	0.27	0.33	0.1	0.19	6.6	17.8
0.36 g	3,654	-	0.2	-	0.3	-	0.145	-	12.3	-

Note: V = base shear at performance point; D = displacement at performance point.

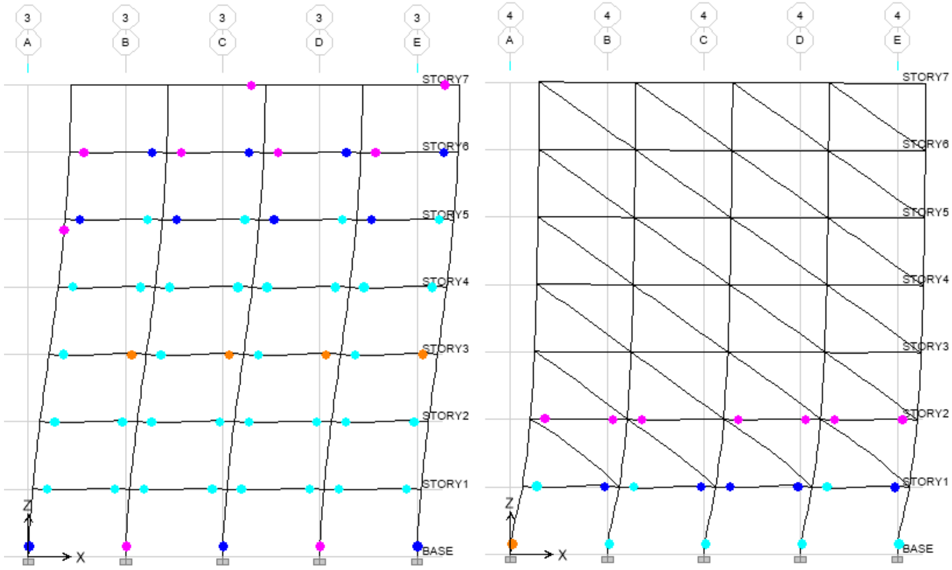
6 Discussion of results

The building is designed and evaluated for three lateral loading patterns to evaluate the difference in performance for variation in seismic demand. The performance of building for each lateral loading pattern is evaluated for DBE and MCE level earthquakes of different intensities defined by IS 1893. Following are the observations on the performance of building:

- The capacity of building designed for parabolic loading pattern is highest considering 1st mode as evaluation loading pattern (Figure 3). However, when the building is designed and evaluated for parabolic loading pattern, the capacity is the least as the intermediate stories experience higher demands. Hence, the parabolic loading patterns impose higher demands on structure.
- Of the ATC40 recommended loading patterns, Triangular loading pattern imposes highest demand for the building and hence the capacity is lower as compared to building designed for 1st mode loading pattern (Figure 7). The storey shear at the lower levels is the highest in this case (Table 2).
- Based on the performance point obtained for different hazard levels for three lateral loading patterns (Table 6, Table 7 and Table 8), the performance of building is less in case of 1st mode loading pattern followed by triangular loading pattern. The performance of building under parabolic loading is the lowest as it results in higher moments in members but is not prescribed by ATC40. Thus, the building if designed as per IS1893:2002 and IS1893 Draft code lateral loading patterns would not suffice the performance requirements when evaluated for 1st mode pattern prescribed by ATC40 as the seismic demand on elements at lower-intermediate storey levels is higher.
- However, the performance of building designed as per IS1893:2002 and detailed as per IS13920 performs better than the building that would be designed for loading pattern prescribed by IS1893 latest draft code for the reason stated above.
- The influence of detailing as per IS13920 is evident from the comparison of shear demand and shear capacity of the section. The shear capacity of the section exceeds the shear demand on the section for all three loading patterns at DBE level earthquake.
- At MCE level earthquake, the seismic demand increases on the structure for which the structure is checked. The structure designed for seismic loads at DBE level is found to cater for the shear demand imposed at MCE level earthquake of 0.24 g and 0.36 g.
- Under DBE level earthquake it is found that no yielding of members occurs at all four hazard levels considered for building.

- Under MCE level earthquake the storey drift limits exceed the maximum permissible by code, i.e., 0.004 h (Figure 11). Displacement at each storey under seismic hazards is shown in Figure 12.
- Under MCE level earthquake the building undergoes damage progressively as the hazard increases. For 0.1 g level earthquake yielding of beams occur at intermediate stories.
- For 0.16 g earthquake beams are at IO performance level at 2nd–3rd stories while the intermediate beams undergo yielding.
- For 0.24 g level earthquake yielding of columns occur at grids A-C-E at base and at 1st storey level. Beams at intermediate levels are under LS performance level and upper storey beams are under IO level performance.
- The building did not survive 0.36 g level earthquake, but shows significant energy dissipation characteristics before final failure. At failure, the hinges in columns at base are under IO performance level and beams of 3rd storey fail. The demand estimated was 4,487 kN while the base shear capacity of building under 1st mode loading pattern was 4,196 kN (6.9% less than demand).
- The building designed as per EC8 loading pattern proportional to 1st mode addresses the issues of damage at upper and lower stories. However, for detailing done for the building according to IS13920 and designed according to IS1893:2002, the chance of building to survive the demand estimated as per 1st mode loading pattern increases. The building designed and evaluated for triangular loading pattern shows lesser performance as the previous case of building designed as per IS1893:2002 at later stages of evaluation as more number of members are damaged as compared to parabolic loading pattern (Figure 3). The severity of loading pattern is shown in Table 9 using ranking system (Bhushan, 2014).
- The building designed for gravity loading can survive an earthquake of 0.1 g only. In case of buildings evaluated for lateral load resistance considering infill walls as single struts excluding ground storey, it is found that the columns of lower stories fail due to insufficient reinforcement provided during design stage due to soft storey effect.
- The influence of seismic hazard on performance of building designed for lateral loads is mentioned in Table 10.
- The effect of spacing of stirrups on the performance of elements of the building is evident from Figure 4. The moment and shear capacity of column S1 reduces with increase in the spacing of stirrups (Murthy et al., 2012).

Figure 6 Member deformation in building designed only for gravity loading, (a) without infill (b) with infill (see online version for colours)



(a) (b)

Figure 7 Member deformation in building under parabolic loading pattern for different seismic hazard (see online version for colours)

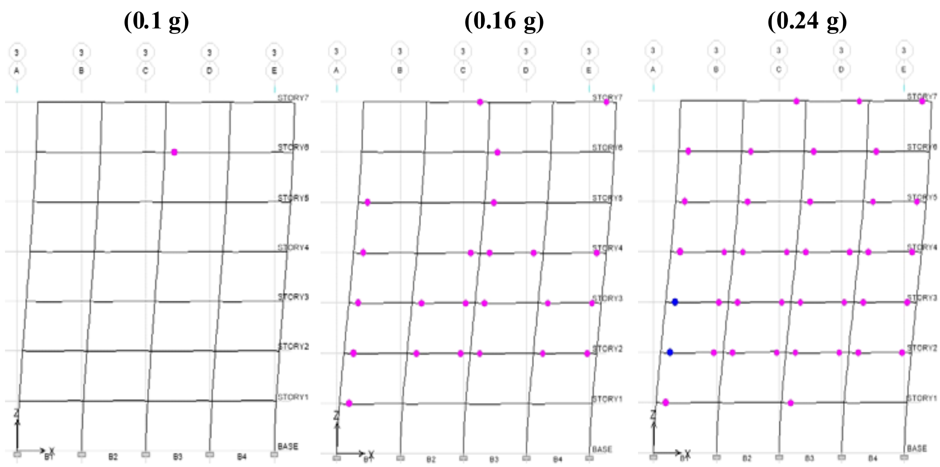


Figure 8 Member deformation in building under triangular loading pattern for different seismic hazard (see online version for colours)

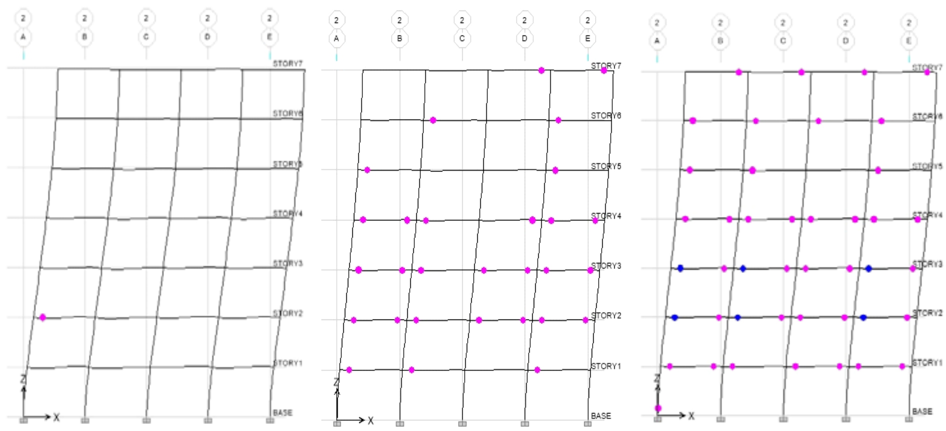


Figure 9 Member deformation in building under 1st mode loading pattern for different seismic hazard (see online version for colours)

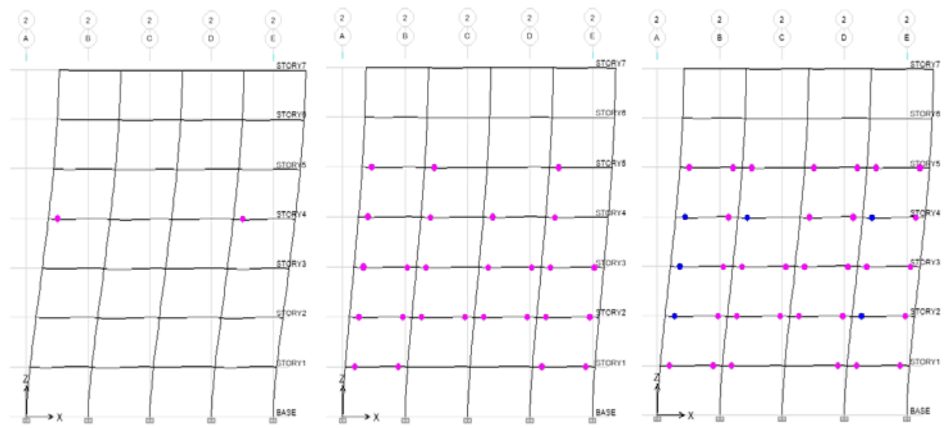


Table 9 Influence of load pattern in estimation of hazard

Parameters	*Seismic hazard (0.24 g)*		
Load pattern	*Parabolic*	*Triangular*	*1st mode*
Demand	1	3	2
Capacity	1	2	3
Storey drift	1	3	2
Performance point	1	3	2
Component damage	3	1	2
Support reactions	1	3	2

Figure 10 Member deformation in building under lateral loading at the end of POA (see online version for colours)

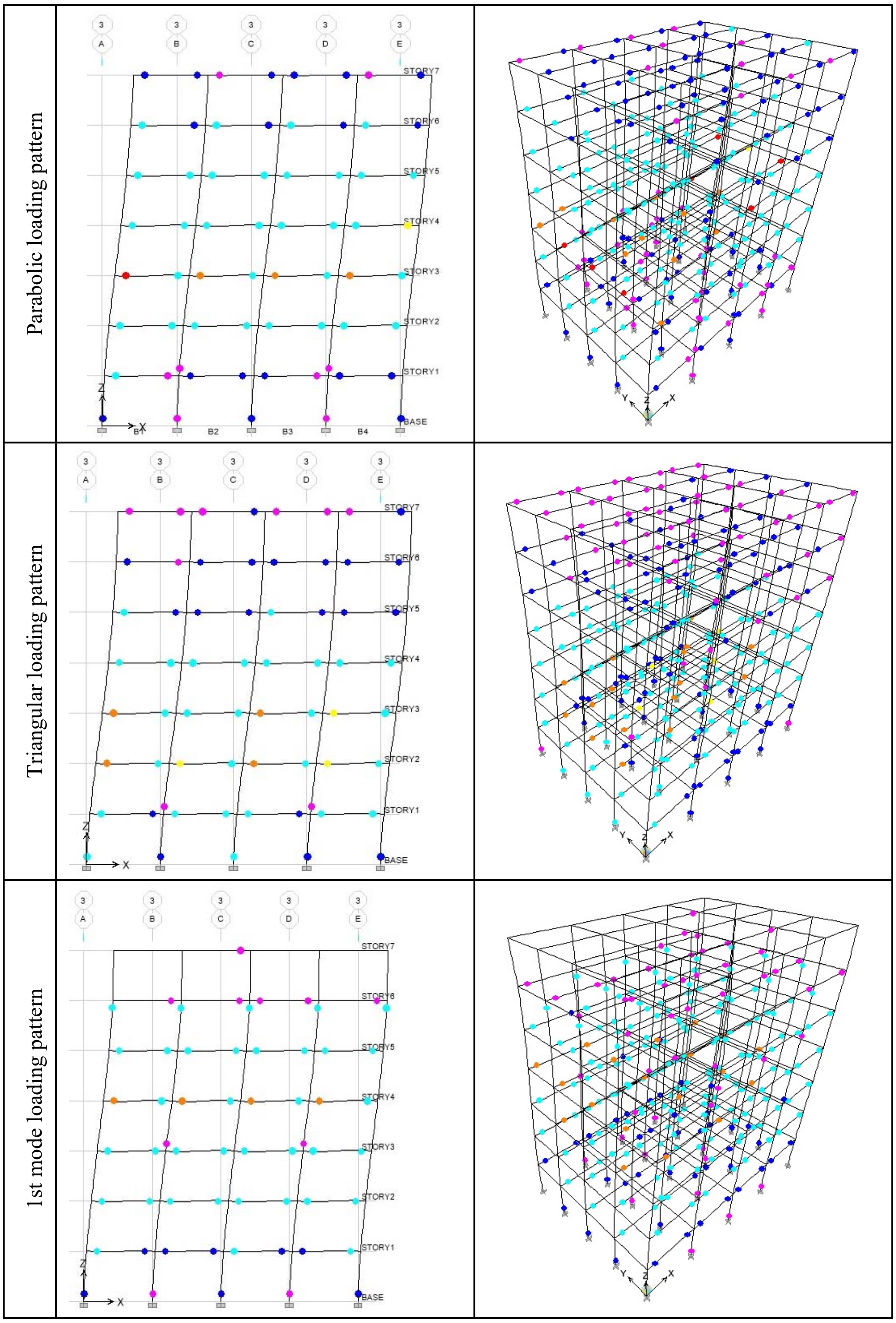


Table 10 Influence of seismic hazard on performance of building

Hazard	*Parabolic – IS 1893*	*Triangular (draft code)*	*1st mode (EC8)*
0.1 g	Non-structural damage expected	Non-structural damage expected	Non-structural damage expected
0.16 g	Yielding of intermediate level beams at all stories	Yielding of intermediate level beams except upper 2 stories	Yielding of intermediate level beams (63 nos.) mostly between levels 2 to 4
0.24 g	12 beams under IO-LS performance level at stories 3 and 2	2 beams reach IO-LS performance level at 2nd storey	14 beams under IO-LS performance level at stories 4 to 2
0.36 g	Collapse	Collapse	Collapse

Figure 11 Storey drift (1st mode pattern)

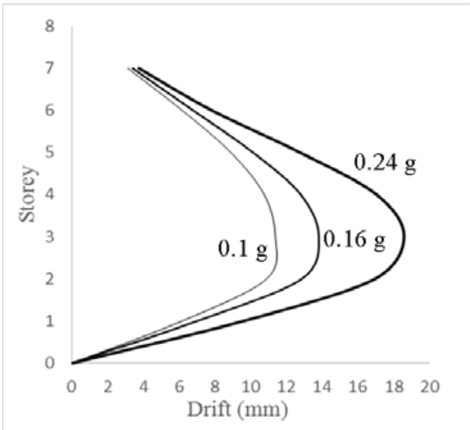
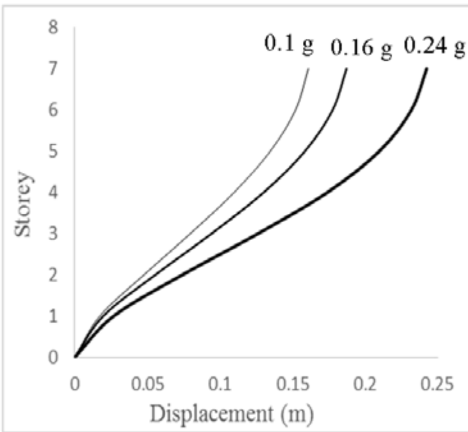


Figure 12 Storey displacement (1st mode pattern)



7 Performance evaluation of building using DDBD procedure

Displacement-based design and evaluation procedures are also considered as PBD procedure by which performance can be regulated based on the expected displacement of building structure. Direct displacement-based design (DDBD) of building structures was developed by Priestley et al. (2007) owing to the deficiencies of force-based design (FBD) procedures to provide realistic estimate of the demands on structure. During earthquakes structure basically undergoes displacement which can be represented by forces that cause them which forms the very basis of this design procedure. It utilises the substitute structure principle developed by Shibata and Sozen (1976) and secant stiffness for calculation of base shear. This procedure is fully developed for regular reinforced concrete building structures and hence can be used for PBD and evaluation of present building considered for the study.

Maximum storey drift considered for the building due to severe earthquake of 0.24g is 2.4%. The complete procedure for DDBD is referred for the estimation of base shear on the building and comparison with the code procedure is carried out. *DBDsoft* (v9.0.3) is used for displacement-based design of building structure for the said drift limit. *DBDsoft* is a program developed by EUCENTRE for the application of DDBD procedure developed by Priestley et al. (2007). Hence, the influence of seismic hazard on building structure designed as per DDBD procedure is also evaluated and compared with the performance of building designed as per code of practice.

The effective mass of the system is 1,267.24 ton (12,672.4 kN), effective height of system is 13.06 m, design displacement is 0.264 m, effective time period is 1.24 sec (nearby: T_n = 1.22 s) and effective stiffness is 32,812.72 kN.m. *DBDsoft* (Sullivan et al., 2012) calculates base shear (V_b) as well as maximum base shear ($V_{b\max}$) of building which are 8,653.6 kN and 4,720.33 kN respectively. Thus, the design base shear comes out to be 4,720.33 kN in X-direction. This is the base shear on the capacity curve where the demand and capacity coincide. Hence, this can be considered as the capacity of designed building as well as the performance point (4,720.33kN, 0.264 m) for 0.24 g seismic hazard. For Takeda hysteresis response, a positive stabilised second slope stiffness (K_s) of at least 5% of the initial elastic stiffness is assured provided the stability index is less or equal to 0.3 (Priestley et al., 2007). In Y-direction, the software shows P-Δ error as the stability index is greater than 0.3 which was not recommended by ETABS. Model of building generated in DBDsoft is shown in Figure 13. The capacity curve of the building obtained by DDBD procedure, a bilinear curve, is shown in Figure 14.

The capacity of building required for 0.24 g level of seismic hazard is highest in case of DDBD procedure with 2.4% storey drift as compared to the code-based design of building (Figure 14). The time period of building in both cases are almost equal, i.e., 1.22 sec as 1st mode time period and 1.23sec as effective time period of system in DDBD. The comparison of maximum base shear that the building can resist before collapse is shown in Table 11. Comparison of performance point of building for various hazard levels obtained using DDBD procedure with that of building evaluated for 1st mode loading pattern in ETABS is shown in Table 12.

Figure 13 3D model of seven storey building generated in DBDsoft (see online version for colours)

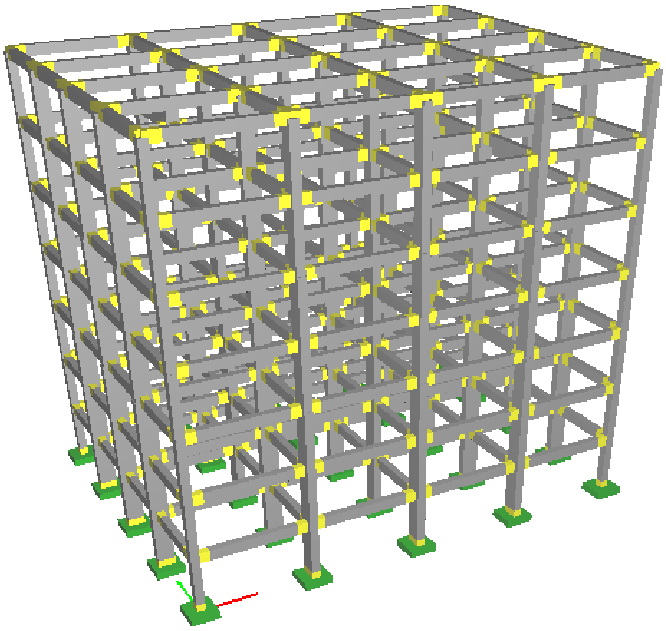


Figure 14 Capacity curve of building obtained by ETABS and DDBD procedure

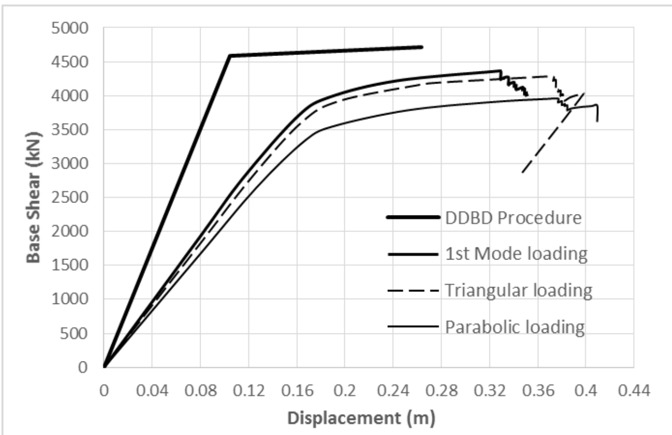


Table 11 Comparison of maximum values obtained from capacity curves

Procedure	*Parabolic loading*	*Triangular loading*	*1st mode loading*	*DDBD*
Max. base shear	3,959.5 kN	4,290.3 kN	4,272.8 kN	4,720.3 kN
Ultimate displacement	0.41 m	0.397 m	0.35 m	0.264 m

Table 12 Comparison of performance point of building

Hazard	*Vb (kN)*	*Performance point*	
		ETABS	*DDBD*
0.1 g	1246.35	(2,790 kN, 0.12 m)	(4,592 kN, 0.11 m)
0.16 g	1994.16	(3,574 kN, 0.17 m)	(4,647.4 kN, 0.176 m)
0.24 g	2991.25	(3,831 kN, 0.25 m)	(4,720.3 kN, 0.264 m)

The displacement ductility values obtained by the four capacity curves shown in Figure 14, differ marginally. The lowest in case of 1st mode loading pattern and highest in case of building designed for parabolic loading pattern. Building designed for triangular loading pattern had ductility value almost equal to that obtained by DDBD procedure. However, these values were found to be less than the response reduction factor (R) prescribed by code.

It is evident from DDBD procedure also that for building with time period of 1.23 sec under storey drift of 2.4% for 0.24 g level earthquake, the building will experience non-structural damage under low seismic event of 0.1 g as the drift limit of storey exceeds the code limit of 0.004 h (11.2 mm). However, if we consider 0.5% drift limit prescribed for DDBD of infill frame structures (Priestley et al., 2007), then the damage to infill would occur under 0.16 g earthquake as the permissible drift limit increased from 0.4% to 0.5% (14 mm).

8 Summary

The performance of seven storey building located in Zone IV was evaluated for four seismic hazard levels representing the range of PGA values in the country and defined by IS1893 (2002) as seismic hazard for different seismic zones of the country. Thus, building is evaluated for earthquakes of minor to severe intensity. The design of building was carried out according to IS456 (2000) for demands estimated according to the IS1893:2002, IS1893 draft code and EC8 loading patterns. The reinforcement required in the elements differ in each of the three cases with highest requirement for parabolic loading. The detailing of building was done according to IS13920. It is found that the loading pattern proportional to 1st mode results in maximum damage, followed by parabolic and then triangular (Table 9) at performance point. However, the capacity of building is least in case of triangular loading pattern beyond the MCE level base shear of 2,990 kN and results in maximum damage after the MCE level base shear (Figure 3). The detailing requirement for columns which is dependent on the hogging and sagging moments of beams is governed by parabolic loading pattern as it results in maximum internal forces. The building is found to satisfy Life Safety performance level under 0.24 g MCE level earthquake. The inter-storey drifts under all MCE level earthquakes exceed the drift limit of 0.0004h (Figure 11) under all the three lateral loading patterns. Hence, non-structural damage is expected and progresses as the hazard increases. The building designed only for gravity loads is also evaluated with two rebar grade variation and found to satisfy only the Life Safety performance objective for 0.1 g MCE level earthquake [Figure 5(d), Figure 6(a)]. The soft storey effect is also considered for evaluation of gravity designed old building with insufficient shear reinforcements using single strut infill panels [Figure 6(b)]. It was found that the provided reinforcement in beams and columns were not sufficient and members failed in design due to modelling of infill panels. Moreover, the validation of the evaluation procedure carried out in ETABS for range of seismic hazard levels was carried out using DDBD procedure as this procedure has gained significance due to its simple and sound theoretical background as compared to traditional pushover analysis procedure.

9 Conclusions

Well-designed building with efficient energy dissipation characteristics and detailing according to IS13920 (2002) was found to be safe under MCE level earthquake for the region. Non-structural damage, especially to infill walls, occur under a low earthquake of 0.1 g due to storey drift higher than 0.004 h, the limit prescribed by IS1893:2002 for storey drift. The seven storey building, if designed for loading pattern prescribed by IS1893 draft code will not satisfy the performance requirement when evaluated for 1st mode loading pattern of ATC 40 as the demands of the upper stories were not addressed during design stage as the design loading pattern was inverted triangle based on time period of building. Moreover, the procedure led to better understanding of the influence of seismic hazard levels and design procedure on the performance of a building. It also provided insight into the probable damage the building may encounter by evaluating its performance for earthquake events of different intensities and hence an action plan can be put in place for such buildings in the region in order to safeguard the interest of those regions. This study is pertaining to regular reinforced concrete buildings only, hence it can be extended for other structures and seismic hazard can be estimated for various building typologies for range of ground acceleration values expected in the region and the probable damage can be identified before next notable earthquake strikes.

References

Araya, R. and Saragoni, G.R. (1984) 'Earthquake accelerogram destructiveness potential factor', *8th World Conference on Earthquake Engineering*, San Francisco, USA.

ATC 40 (1996) *Seismic Evaluation and Retrofit of Concrete Structures*, Report ATC 40, Applied Technology Council, Redwood City, California, USA.

Bhushan, R. (2014) 'Influence of parameters on performance evaluation of RC structure', *5th Asia Conference on Earthquake Engineering (5ACEE)*, Taipei, Taiwan.

Chopra, S., Kumar, D., Rastogi, B.K., Choudhury, P. and Yadav, R.B.S. (2012b) 'Estimation of seismic hazard in Gujarat region, India', *Natural Hazards*, Vol. 65, No. 2, pp.1157–1178.

Freeman, S.A. (1998) 'The capacity spectrum method as a tool for seismic design', *Proceedings of the 11th European Conference on Earthquake Engineering*, pp.6–11, France.

Freeman, S.A., Nicoletti, J.P. and Tyrell, J.V. (1975) 'Evaluations of existing buildings for seismic risk – a case study of Puget Sound Naval Shipyard, Bremerton, Washington', *Proceedings of US National Conference on Earthquake Engineering*, Berkeley, USA, pp.113–122.

Hoffman, G.W., Kunnath, S.K., Reinhorn, A.M. and Mander, J.B. (1992) *Gravity Load Designed Reinforced Concrete Building: Seismic Evaluation of Existing Construction and Detailing Strategies for Improved Seismic Resistance*, Technical Report, NCREE-92-0016.

IS 13920 (2002) *Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces – Code of Practice*, Bureau of Indian Standards, New Delhi.

IS 1893 Part I (2002) *Criteria for Earthquake Resistant Design of Structures*, Bureau of Indian Standards, New Delhi.

IS 456 (2000) *Plain and Reinforced Concrete – Code of Practice (4th Revision)*, Bureau of Indian Standards, New Delhi.

Iyengar, R.N. and Ghosh, S. (2004) 'Seismic hazard mapping of Delhi City', *13th World Conference on Earthquake Engineering*, Vancouver, Canada, Paper No. 180.

Kanagarathinam, L., Dodagoudar, G.R. and Boominathan, A. (2008) 'Probabilistic seismic hazard studies of East Coast Region of India', *14th World Conference on Earthquake Engineering (14WCEE)*, Beijing, China.

Krawinkler, H. (1996) 'Pushover analysis: why, how and when not to use it', *6th Annual Convention*, Structural Engineers Association of California, Hawaii.

Menon, A., Ornthammarath, T., Corigliano, M. and Lai, C.G. (2010) 'Probabilistic seismic hazard macrozonation of Tamil Nadu in Southern India', *Bulletin of Seismological Society of America*, Vol. 100, No. 3, pp.1320–1341.

Murthy, C.V.R., Goswami, R., Vijayanarayanan, R.A. and Mehta, V.V. (2012) 'Some concepts in earthquake behavior of buildings', *IITK-GSDMA*, National Information Centre of Earthquake Engineering (NICEE), India.

Priestley, M.J.N., Calvi, G.M. and Kowalsky, M.J. (2007) *Displacement Based Seismic Design of Structures*, IUSS Press, Pavia, Italy.

Raghukanth, S.T.G. and Iyengar, R.N. (2006) 'Seismic hazard estimation for Mumbai city', *Current Science*, Vol. 1486, No. 91, p.11.

RC Analysis software (2008) *Program for Moment-Curvature-Shear Analysis of RC Sections*, Virtual Laboratory of Earthquake Engineering (VLEE), Universidad Tecnica Particular De Loja (UTPL).

Shibata, A. and Sozen, M.A. (1976) 'Substitute-structure method for seismic design in R/C', *Journal of the Structural Division*, Vol. 102, No. 1, pp.1–18.

Sitharaman, T.G., James, N., Vipin, K.S. and Raj, K.G. (2012) 'A study on seismicity and seismic hazard for Karnataka State', *Journal of Earth System Science*, Vol. 121, No. 2, pp.475–490.

Sullivan, T.J., Bono, F., Magni, F. and Calvi, G.M. (2012) 'Development of a computer program for direct displacement based design', *15th World Conference on Earthquake Engineering (15WCEE)*, Lisbon.

US Geological Survey (USGS) *Measuring Size of an Earthquake* [online] http://earthquake.usgs.gov (accessed 01.03.15).

Watabe, M. and Tohdo, M. (1979) 'Analysis of various parameters for the simulation of three-dimensional earthquake ground motions', *5th International Conference on Structural Mechanics in Reactor Technology*, Berlin, Germany, No. K1/1 in K(a), pp.1–11.