A Comparative Study of Code Provisions for Ductile RC Frame Buildings

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LISBOA 2012

SUMMARY:
All current seismic design codes are based on a prescriptive Force-Based Design approach. In this approach, a linear elastic analysis is performed and inelastic energy dissipation is considered indirectly, through a response reduction factor (or behaviour factor). Building codes define different ductility classes and specify corresponding response reduction factors based on the material, configuration, and detailing. Codes also differ significantly in specifying the effective stiffness of RC members, procedures to estimate drift, and allowable limits on drift. This paper presents a comparative study of different ductility classes and corresponding response reduction factors, reinforcement detailing provisions, and a case study of seismic performance of a ductile RC frame building designed using four major codes, viz. ASCE7 (United States), EN1998-1 (Europe), NZS 1170.5 (New Zealand) and IS 1893 (India). The performance of the test building is evaluated using the Displacement Modification Method (DMM) as well as the guidelines of ASCE-41.

Keywords: Seismic Design, Design Codes, Ductile RC Frames, Drift Control, Seismic Performance.

1. INTRODUCTION

All modern national seismic design codes converge on the issue of design methodology. These are based on a prescriptive Force-Based Design approach, where the design is performed using a linear elastic analysis, and inelastic energy dissipation is considered indirectly, through a response reduction factor (or behavior factor). This factor, along with other interrelated provisions, governs the seismic design forces and hence the seismic performance of code-designed buildings. However, different national codes vary significantly on account of various specifications which govern the design force level. The response reduction factor, as considered in the design codes, depends on the ductility and overstrength of the structure. Building codes define different ductility classes and specify corresponding response reduction factors based on the structural material, configuration and detailing. Another important issue, which governs the design and expected seismic performance of a building, is control of drift. Drift is recognized as an important control parameter by all the codes; however, they differ regarding the effective stiffness of RC members. Further, the procedures to estimate drift and the allowable limits on drift also vary considerably.

Different codes differ not only with respect to the design base shear but also employ different load and material factors (or strength reduction factors) for the design of members, and hence, the actually provided strength in different codes does not follow the same pattern as the design base shear. This has direct effect on the expected performance of buildings designed using different codes. Further, the other provisions of codes also indirectly govern the seismic performance. In the era of globalisation, there is a need for convergence of design methodologies to result in buildings with uniform risk of suffering a certain level of damage or collapse. A first step in this direction is to compare the expected seismic performance of buildings designed using the provisions of different codes. A detailed comparison of various provisions of different codes and design base shear coefficients obtained for a given hazard were conducted by the authors earlier (Khose et al. 2012). This paper extends the comparison further and presents a comparative study of the expected seismic performance of a ductile RC frame building designed for four major codes, viz. the U.S. American (ASCE 7-10 2010), European (EN1998-1 2004), New Zealand (NZS 1170.5 2004) and Indian code (IS 1893-Part1 2002). The provisions of different codes regarding effective stiffness of RC members, procedure to estimate drift, and allowable drift limits are also compared to each other.

2. BUILDING DUCTILITY CLASSIFICATION AND RESPONSE REDUCTION/BEHAVIOR FACTORS

Currently, all seismic design codes take into account the effect of inelastic energy dissipation by reducing the design seismic force by a 'response reduction factor' (also called 'behavior factor'). Values of response reduction factor are provided for different ductility classes of buildings. In addition to ductility, the response reduction factor also takes into account the effect of overstrength. NZS 1170.5 considers a separate structural performance factor in addition to the ductility factor, which represents the combined effect of the limited number of cycles having peak amplitude, overstrength, redundancy, and over-capacity due to damping in secondary components and in the foundation. Further, only NZS 1170.5 considers the effect of period on the relationship between ductility and the response reduction factor. All other codes provide constant response reduction factors for a particular construction type, irrespective of the period of vibration. ASCE 7 classifies RC frame buildings into three ductility classes: Ordinary Moment Resisting Frame (OMRF), Intermediate Moment Resisting Frames (IMRF) and Special Moment Resisting Frames (SMRF). Eurocode 8 (EN 1998-1) classifies the building ductility as Low (DCL), Medium (DCM) and High (DCH). NZS 1170.5 classifies structures into three ductility classes, namely Ductile Structures (DS), for which the structural ductility factor is greater than 1.25 but less than 6, Structures of Limited Ductility (SLD), which is a subset of DS with structural ductility factor between 1.25 and 3, and Nominal Ductile Structures (NDS), for which the ductility factor is between 1 and 1.25. IS 1893 classifies RC frame buildings as Ordinary Moment Resisting Frames (OMRF) and Special Moment Resisting Frames (SMRF).

Table 2.1. Overview of ductile detailing requirements for RC frame buildings in different seismic design codes (Khose et al. 2012)

Ductile Detailing Criteria		ASCE 7[1)]			Eurocode 8			NZS 1170.5[2)]			IS 1893[3)]	
		OMRF	IMRF	SMRF	DCL	DCM	DCH	NDS	SLD	DS	OMRF	SMRF
Capacity Design	Strong Column Weak Beam	○	○	●	○	●	●	○	●	●	○	○
	Capacity Shear for Column	○	●	●	○	●	●	○	●	●	○	●
	Capacity Shear for Beam	○	●	●	○	●	●	○	●	●	○	●
Special Confinement Reinforcement	Column	○	●	●	○	●	●	○	●	●	○	●
	Beam	○	●	●	○	●	●	○	●	●	○	●
Special Anchorage Requirement	Interior Joint	○	○	●	○	●	●	●	●	●	○	●
	Exterior Joint	○	○	●	○	●	●	●	●	●	○	●
Joint Shear Design		○	○	●	○	○	●	●	●	●	○	○

[1)] ductile detailing as per ACI 318M-08 (2008)
[2)] ductile detailing as per NZS 3101:Part 1 (2006) and NZS 1170.5 (2004)
[3)] ductile detailing of OMRF and SMRF as per IS 456 (2000) and IS 13920 (1993), respectively

○ provision is not available
● provision is available

Seismic design codes either provide guidelines for the design and detailing of RC buildings for different ductility classes or refer to complimentary design codes. These provisions, in general, consist of four requirements: (i) capacity design provisions to achieve a hierarchy of strength in order to avoid brittle failure modes, (ii) provision of special confining reinforcement (in the form of closely-spaced stirrups) at potential plastic hinge locations, (iii) anchorage of beam longitudinal reinforcement into columns, and (iv) design of beam-column joints to avoid shear failure. Table 2.1 summarizes the different provisions in the codes for different ductility classes of RC frame buildings.

It is evident from Table 2.1 that it is not possible to have a one-to-one parity between different ductility classes of various codes. However, three broad categories of ductility can be considered, as shown inTable 2.2, where each category includes building classes with similar ductility provisions. Figure 2.1shows the response reduction/behavior factors for different ductility classes of RC frames, according to different codes. There is a large difference in reduction factors for long-period and short-period structures according to NZS 1170.5, whereas, as mentioned above, other codes do not consider the effect of period on response reduction factors. The reduction factors for medium and high ductility classes of ASCE 7, NZS 1170.5 (for long-period structures) and IS 1893 (no high ductility class is available) are close, whereas the corresponding reduction factors in Eurocode 8 are quite low. For low ductility class, the response reduction factors of NZS 1170.5 and Eurocode 8 are close, whereas the response reduction factors of ASCE 7 and IS 1893 are identical and twice as high as those of Eurocode 8.

Table 2.2. Different ductility categories of RC frame buildings (Khose et al. 2012)

Category	Ductility classes			
	ASCE 7	Eurocode 8	NZS 1170.5	IS 1893
I – Low dissipative structures	OMRF	DCL	NDS	OMRF
II – Medium dissipative structures	IMRF	DCM	SLD	SMRF
III – High dissipative structures	SMRF	DCH	DS	–

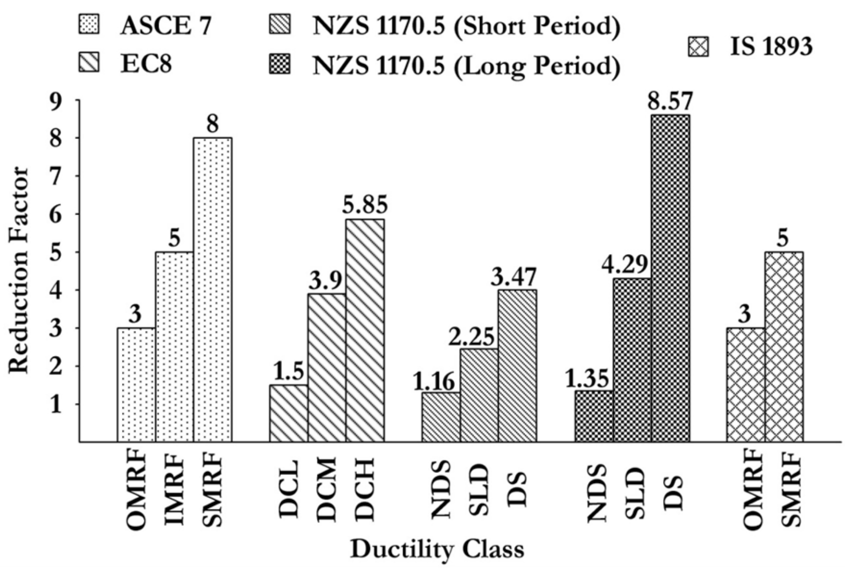

Figure 2.1. Comparison of reduction/behavior factors recommended in different national codes (Khose et al. 2012)

3. DESIGN BASE SHEAR

The design base shear coefficients for buildings of different ductility classes and different fundamental periods (representing different heights) are compared in Tables 3.1-3.5 for different site classes. ASCE

7 and NZS 1170.5 also specify a minimum design base shear coefficient. In ASCE 7 the minimum design base shear coefficient depends on the ductility class and PGA, while in case of NZS 1170.5 it is independent of ductility class.

For the comparison of design base shear, two values of PGA for 2% probability of exceedance, i.e., 0.2*g* and 0.5*g*, have been considered and design periods 0.25, 1.5, 2.5, 3.5 and 4.5 seconds are chosen so that the acceleration-, velocity- and displacement-controlled ranges of the design spectra of all codes are covered. Since each code applies a different classification, ASCE 7 equivalent site classes as described in Khose et al. (2012) have been considered for the current study. It is observed from Tables 3.1-3.5 that large differences exist between the design base shear coefficients obtained from the various codes. In almost all cases, the design base shear coefficients follow a descending order for NZS 1170.5, Eurocode 8, ASCE 7 and IS 1893, except for a few cases where the minimum base shear requirements of ASCE 7 are governing. The differences are not limited to the calculated base shear only, but large differences exist in the specified minimum design base shear as well. Eurocode 8 and IS 1893 require very small design base shear coefficients for taller buildings in some cases, as there is no minimum limit on the design base shear. The comparison shows that buildings designed according to different codes will not have similar performance for the same level of hazard. Therefore, there is a need for harmonization of different codes to result in building designs having comparable risk to suffer different levsl of damage or collapse for a given seismic hazard. A comparison of expected performance of the buildings designed using different codes will be useful in this direction.

Table 2.1. Design base shear coefficients (%) from various codes (for site classes equivalent to ASCE 7 class A)

Ductility Class		PGA (2% PE in 50 years) = 0.2*g*					PGA (2% PE in 50 years) = 0.5*g*				
		Period *T*									
		0.25*s*	1.5 *s*	2.5 *s*	3.5 *s*	4.5 *s*	0.25*s*	1.5 *s*	2.5 *s*	3.5 *s*	4.5 *s*
I	ASCE 7 - OMRF	8.90	2.38	1.43	1.02	_1.00*_	22.23	5.93	3.56	2.54	_1.76_
	EC8 - DCL	16.32	5.13	2.47	1.53	1.53	40.74	12.80	6.13	3.87	3.87
	NZS - NDS	__17.83__	__5.77__	__3.48__	__3.00*__	__3.00*__	__44.58__	__14.43__	__8.66__	__5.25__	__3.40*__
	IS 1893 - OMRF	_8.33_	_2.23_	_1.33_	_0.97_	--	_20.83_	_5.57_	_3.33_	_2.37_	--
II	ASCE 7 - IMRF	5.34	1.43	1.00*	1.00*	_1.00*_	13.34	3.56	2.14	1.53	_1.05_
	EC8 - DCM	6.28	1.97	0.95	0.59	0.59	15.67	4.92	2.36	1.49	1.49
	NZS - SLD	__8.58__	__3.00*__	__3.00*__	__3.00*__	__3.00*__	__28.33__	__6.01__	__3.61__	__3.40*__	__3.40*__
	IS 1893 - SMRF	_5.00_	_1.34_	_0.80_	_0.58_	--	_12.50_	_3.34_	_2.00_	_1.42_	--
III	ASCE 7 - SMRF	_3.34_	_1.00*_	1.00*	1.00*	1.00*	_8.34_	_2.23_	_1.34_	1.00*	1.00*
	EC8 - DCH	4.18	1.32	_0.63_	_0.39_	_0.39_	10.45	3.28	1.57	_0.99_	_0.99_
	NZS - DS	__5.28__	__3.00*__	__3.00*__	__3.00*__	__3.00*__	__17.43__	__3.40*__	__3.40*__	__3.40*__	__3.40*__

* minimum base shear requirements are governing; -- corresponding design spectral values are not available in the code
bold numbers indicate maximum and *italic numbers* indicate minimum base shear for a given category and design period

Table 3.2. Design base shear coefficients (%) from various codes (for site classes equivalent to ASCE 7 class B)

Category and Ductility Class		PGA (2% PE in 50 years) = 0.2*g*					PGA (2% PE in 50 years) = 0.5*g*				
		Period *T*									
		0.25*s*	1.5 *s*	2.5 *s*	3.5 *s*	4.5 *s*	0.25*s*	1.5 *s*	2.5 *s*	3.5 *s*	4.5 *s*
I	ASCE 7 - OMRF	11.10	2.96	1.77	1.27	_1.00*_	27.77	7.40	4.44	3.17	_2.19_
	EC8 - DCL	16.32	5.13	2.47	1.53	1.53	40.74	12.80	6.13	3.87	3.87
	NZS - NDS	__17.83__	__5.77__	__3.48__	__3.00*__	__3.00*__	__44.58__	__14.43__	__8.66__	__5.25__	__3.40*__
	IS 1893 - OMRF	_8.33_	_2.23_	_1.33_	_0.97_	--	_20.83_	_5.57_	_3.33_	_2.37_	--
II	ASCE 7 - IMRF	6.66	1.77	1.06	1.00*	_1.00*_	16.66	4.44	2.66	1.90	_1.32_
	EC8 - DCM	6.28	1.97	0.95	0.59	0.59	15.67	4.92	2.36	1.49	1.49
	NZS - SLD	__8.58__	__3.00*__	__3.00*__	__3.00*__	__3.00*__	__28.33__	__6.01__	__3.61__	__3.40*__	__3.40*__
	IS 1893 - SMRF	_5.00_	_1.34_	_0.80_	_0.58_	--	_12.50_	_3.34_	_2.00_	_1.42_	--
III	ASCE 7 - SMRF	_4.16_	_1.11_	1.00*	1.00*	1.00	_10.41_	_2.78_	1.67	1.19	1.00*
	EC8 - DCH	4.18	1.32	_0.63_	_0.39_	_0.39_	10.45	3.28	_1.57_	_0.99_	_0.99_
	NZS - DS	__5.28__	__3.00*__	__3.00*__	__3.00*__	__3.00*__	__17.43__	__3.40*__	__3.40*__	__3.40*__	__3.40*__

* minimum base shear requirements are governing; -- corresponding design spectral values are not available in the code
bold numbers indicate maximum and *italic numbers* indicate minimum base shear for a given category and design period

Table 3.3. Design base shear coefficients (%) from various codes (for site classes equivalent to ASCE 7 class C)

Category and Ductility Class		PGA (2% PE in 50 years) = 0.2*g*					PGA (2% PE in 50 years) = 0.5*g*				
		Period *T*									
		0.25*s*	1.5 *s*	2.5 *s*	3.5 *s*	4.5 *s*	0.25*s*	1.5 *s*	2.5 *s*	3.5 *s*	4.5 *s*
I	ASCE 7 - OMRF	13.33	4.73	2.84	2.03	*1.40*	27.77	9.62	5.77	4.12	*2.85*
	EC8 - DCL	**19.55**	**7.67**	**3.67**	1.87	1.53	**48.90**	**19.20**	**9.20**	**4.67**	**3.87**
	NZS - NDS	17.83	5.77	3.48	**3.00***	**3.00***	44.58	14.43	8.66	5.25	3.40*
	IS 1893 - OMRF	*8.33*	*2.23*	*1.33*	*0.97*	--	*20.83*	*5.57*	*3.33*	*2.37*	--
II	ASCE 7 - IMRF	8.00	2.84	1.70	1.22	1.00*	16.66	5.77	3.46	2.47	1.71
	EC8 - DCM	7.52	2.95	1.41	0.72	*0.59*	18.81	7.38	3.54	1.79	*1.49*
	NZS - SLD	**8.58**	**3.00***	**3.00***	**3.00***	**3.00***	**28.33**	**6.01**	**3.61**	**3.40***	**3.40***
	IS 1893 - SMRF	*5.00*	*1.34*	*0.80*	*0.58*	--	*12.50*	*3.34*	*2.00*	*1.42*	--
III	ASCE 7 - SMRF	5.00	1.78	1.07	1.00*	1.00*	*10.41*	3.61	*2.17*	1.55	1.07
	EC8 - DCH	5.01	1.97	0.94	0.48	0.39	12.54	4.92	2.36	*1.20*	*0.99*
	NZS - DS	**5.28**	**3.00***	**3.00***	**3.00***	**3.00***	**17.43**	***3.40****	**3.40***	**3.40***	**3.40***

* minimum base shear requirements are governing; -- corresponding design spectral values are not available in the code
bold numbers indicate maximum and *italic numbers* indicate minimum base shear for a given category and design period

Table 3.4. Design base shear coefficients (%) from various codes (for site classes equivalent to ASCE 7 class D)

Category and Ductility Class		PGA (2% PE in 50 years) = 0.2*g*					PGA (2% PE in 50 years) = 0.5*g*				
		Period *T*									
		0.25*s*	1.5 *s*	2.5 *s*	3.5 *s*	4.5 *s*	0.25*s*	1.5 *s*	2.5 *s*	3.5 *s*	4.5 *s*
I	ASCE 7 - OMRF	15.57	5.93	3.56	2.54	1.76	27.77	5.93	3.56	2.54	*1.76*
	EC8 - DCL	18.76	8.80	4.20	2.13	*1.53*	46.86	22.07	10.60	5.40	3.87
	NZS:C - NDS	**22.25**	7.18	**4.37**	**3.00***	**3.00***	**55.71**	**18.06**	**10.88**	**6.66**	**4.00**
	NZS:D - NDS	**28.28**	**11.77**	**7.03**	**4.29**	**3.00***	**70.74**	**29.30**	**17.61**	**10.80**	**6.51**
	IS 1893 - OMRF	*8.33*	*3.03*	*1.80*	*1.30*	--	*20.83*	*7.57*	*4.53*	*3.23*	--
II	ASCE 7 - IMRF	9.34	3.56	2.14	1.53	1.05	16.66	6.67	4.00	2.86	*1.05*
	EC8 - DCM	7.21	3.38	1.62	0.82	*0.59*	18.02	8.49	4.08	2.08	1.49
	NZS:C - SLD	**10.70**	3.00*	**3.00***	**3.00***	**3.00***	**35.40**	**7.52**	**4.53**	**3.40***	**3.40***
	NZS:D - SLD	**13.60**	**3.71**	**3.00***	**3.00***	**3.00***	**44.95**	**12.21**	**7.34**	**4.50**	**3.40***
	IS 1893 - SMRF	*5.00*	*1.82*	*1.08*	*0.78*	--	*12.50*	*4.54*	*2.72*	*1.94*	--
III	ASCE 7 - SMRF	5.84	2.23	1.34	1.00*	1.00*	*10.41*	*4.17*	*2.50*	*1.79*	*1.00**
	EC8 - DCH	*4.81*	*2.26*	*1.08*	*0.55*	*0.39*	12.02	5.66	2.72	1.38	0.99
	NZS:C - DS	**6.58**	**3.00***	**3.00***	**3.00***	**3.00**	**21.78**	**3.76**	**3.40***	**3.40***	**3.40***
	NZS:D - DS	**8.37**	**3.00***	**3.00***	**3.00***	**3.00**	**27.66**	**6.11**	**3.67**	**3.40***	**3.40***

* minimum base shear requirements are governing; -- corresponding design spectral values are not available in the code
bold numbers indicate maximum and *italic numbers* indicate minimum base shear for a given category and design period

Table 3.5. Design base shear coefficients (%) from various codes (for site classes equivalent to ASCE 7 class E)

Category and Ductility Class		PGA (2% PE in 50 years) = 0.2*g*					PGA (2% PE in 50 years) = 0.5*g*				
		Period *T*									
		0.25*s*	1.5 *s*	2.5 *s*	3.5 *s*	4.5 *s*	0.25*s*	1.5 *s*	2.5 *s*	3.5 *s*	4.5 *s*
I	ASCE 7 - OMRF	*18.90*	*9.49*	*5.69*	4.07	2.81	*22.23*	*17.78*	*10.67*	*7.62*	5.27
	EC8 - DCL	21.99	11.73	6.60	*3.40*	*2.07*	54.97	29.35	16.53	8.47	*5.13*
	NZS - NDS	**24.64**	**18.20**	**10.95**	**6.66**	**4.07**	**61.64**	**45.51**	**27.31**	**16.72**	**10.14**
	IS 1893 - OMRF	8.33	3.70	2.23	1.60	--	20.83	9.27	5.57	3.97	--
II	ASCE 7 - IMRF	11.34	5.69	3.42	2.44	1.69	13.34	10.67	6.40	4.57	3.16
	EC8 - DCM	8.46	4.51	2.54	1.31	0.79	21.14	11.29	6.36	3.26	1.97
	NZS - SLD	**12.43**	**5.74**	**3.45**	**3.00***	**3.00***	**31.10**	**14.35**	**8.61**	**5.27**	**3.40***
	IS 1893 - SMRF	*5.00*	*2.22*	*1.34*	*0.96*	--	*12.50*	*5.56*	*3.34*	*2.38*	--
III	ASCE 7 - SMRF	7.09	**3.56**	2.14	1.53	1.05	*8.34*	*6.67*	*4.00*	2.86	1.98
	EC8 - DCH	*5.64*	*3.01*	*1.69*	*0.87*	*0.53*	14.09	7.53	4.24	*2.17*	*1.32*
	NZS - DS	**8.88**	3.00*	**3.00***	**3.00***	**3.00***	**22.21**	**7.18**	**4.31**	**3.40***	**3.40***

* minimum base shear requirements are governing; -- corresponding design spectral values are not available in the code
bold numbers indicate maximum and *italic numbers* indicate minimum base shear for a given category and design period

4. CONTROL OF DRIFT

Performance of structural as well as non-structural components of a building is controlled by interstory drift. Interstory drift also governs the secondary (P-Δ) effects and it is one of the most important design parameters, and governs the member sizes in many cases, particularly in the case of tall buildings. As in the case of design base shear, the various codes differ not only in the limits on interstory drift, but also in the estimation procedure. In ASCE 7, elastic displacement at a floor level is calculated and amplified by a deflection amplification factor depending on the type of building. Limits are provided on amplified interstory drifts, representing the inelastic deformations in the building. Eurocode 8 presents limits on the elastic displacements amplified directly by the behavior factor. NZS 1170.5 requires the elastic displacements to be multiplied by the structural ductility factor and drift modification factor in order to obtain inelastic displacements. The drift modification factor accounts for higher mode effects and depends on the height of the building. IS 1893 provides the drift control limits directly on the elastic displacement at the design load, without any amplification for ductility demand.

ASCE 7 limits story drift according to occupancy category and allows up to 2.5% drift for ordinary multistory RC frame buildings. According to Eurocode 8, the allowable story drift depends on the type of non-structural elements, and for multistory RC framed buildings, the allowable drift is 1% for buildings having brittle non-structural elements, 1.5% for buildings having ductile non-structural elements, and 2% for buildings having non-structural elements which are “ (..) fixed in a way so as not to interfere with structural deformations (..)” or without non-structural elements. NZS 1170.5 allows a story drift of 2.5% for all types of buildings, irrespective of material and occupancy class. IS 1983 limits the interstory drift to 0.4% at the design load level, which renders it dependent on the ductility class of the building (Haldar and Singh 2009). Considering the ductility factor approximately to be equal to the response reduction factor, the effective limits at ultimate displacement are 1.2% and 2% for OMRF and SMRF, respectively. The different limits in case of OMRF and SMRF result in an interstory drift governing the design in case of OMRF but not in case of SMRF (Haldar and Singh 2009).

Consideration of the effective stiffness of RC members is obviously the most crucial step in the estimation of building deformations and interstory drifts. ACI 318M-08 (2008) specifies effective stiffness as 70% and 35% of the gross stiffness for columns and beams, respectively. Eurocode 8 specifies 50% of gross stiffness as the effective stiffness for both columns and beams, while NZS 3101:Part 1 (2006) suggests the effective stiffness for rectangular beams to be 32% and 40% of the gross section stiffness for yield strength (f_y) of reinforcement equal to 500 *MPa* and 300 *MPa*, respectively. The corresponding values for columns vary from 0.40 to 0.80 with a reinforcement ratio for f_y = 300 *MPa*, and from 0.30 to 0.80 for f_y = 500 *MPa*. In addition to the code provisions, several other proposals for the effective stiffness of RC members under seismic loads are available which vary significantly (Kumar and Singh 2010). Therefore, in addition to the design base shear, a consensus on the limits and estimation procedure for permissible drift is also necessary for uniformity in expected performance and associated risk, in buildings designed as per different codes. This is also crucial for the development of future versions of seismic codes based on displacement-based design methodology.

5. SEISMIC PERFORMANCE OF BUILDINGS DESIGNED FOR DIFFERENT CODES

Different codes employ different load and material factors (or strength reduction factors) for the design of members, and hence the actually provided strength in different codes may not follow the same pattern as the design base shear. Further, as discussed earlier, the drift may govern the design in many cases, resulting in further discrepancies in the actually provided strength. Therefore, in this study, the seismic performance of a building designed for the U.S. American (ASCE 7-10 2010), European (EN1998-1 2004)), New Zealand (NZS 1170.5 2004) and Indian (IS 1893-Part1 2002) seismic design codes has been compared. It is assumed that the building is used for residential purpose and is founded on ASCE 7-10 site class D. For residential buildings, all the codes specify an importance factor equal to 1. To have a common hazard level for design, a PGA value of 0.35*g* on soil type B of ASCE 7-10, corresponding to 2% probability of exceedance (PE) in 50 years is considered for the study. The building is designed for medium energy dissipation (i.e. Category II inTable 2.2). Accordingly, ductility classes IMRF, DCM, SLD, and SMRF of ASCE 7-10, Eurocode 8, NZS 1170.5 and IS 1893, respectively, are considered. In case of ASCE 7, the applicable seismic design category for the considered seismic hazard level is D, and IMRF is not permitted for the same. Whereas in other codes, the building classes associated with medium energy dissipation are allowed. Therefore, IMRF as well as SMRF ductility classes of ASCE 7, are considered in the study.

5.1. Building Modeling and Design

An eight-story RC frame building with plan and elevations as shown in Figure 5.1 is considered for study. The building is symmetric in both directions. In transverse direction, corridor beams are not provided and the two blocks separated by the corridor are rigidly connected by the slab. The sStory height is 3.2 *m* for all floors. The 3D model of the building is developed in SAP2000 (2010). Beams and columns have been modeled as frame elements. In-plane rigidity of the slab is simulated using rigid diaphragm action. The columns are assumed to be fixed at the base. The effective stiffness factors for RC members are considered as per respective design codes. Since the design codes do not provide guidelines for modeling of joints, the recommendations of Elwood et al. (2007) regarding modeling of RC beam-column joints have been used, whereas columns are considered as rigid and beams are considered as flexible, within the joint.

The buildings are designed as per the considered seismic codes and the corresponding design codes. The design codes used are ACI 318M-08 (2008), EN1992 (2004), NZS 3101:Part 1 (1995) and IS 456 (2000), respectively. All the considered codes use cylinder strength as the measure of concrete compressive strength, except for the Indian code, which uses cube strength. In the present study, the cube compressive strength f_{ck} is considered as 30 *MPa*, which corresponds to a cylinder strength, f_c' of 24 *MPa*. The values of modulus of elasticity of concrete have been estimated using the relationships provided in the corresponding codes. Specified yield strength, f_y and modulus of elasticity, E_s of reinforcing steel are considered as 500 *MPa* and 2×10^5 *MPa*, respectively.

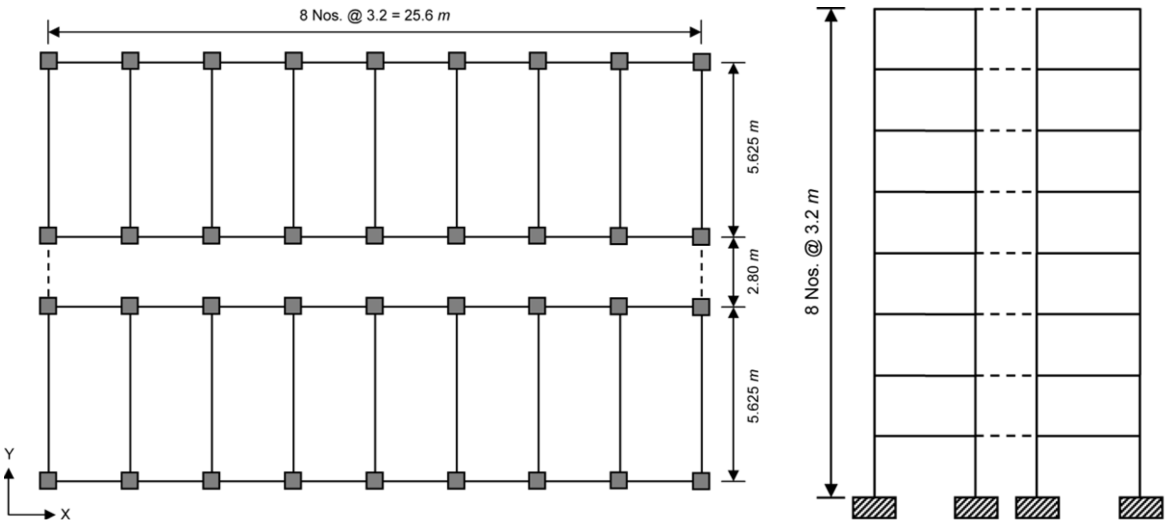

Figure 5.1. Plan (left) and side elevation (right) of the building considered for study. In the transverse direction, beams over corridor are not provided. Two blocks separated by corridor, are interconnected by rigid diaphragms of floor/roof slabs.

The seismic load according to the relevant codes has been estimated and the building is designed for combined effect of gravity and seismic loads, considering all the design load combinations specified in each code. The drift limits for different codes as discussed earlier have also been applied. As mentioned earlier, all the codes considered for the study specify drift limits on the total (inelastic) displacement, except for the Indian code, which specifies drift limit on the elastic displacement. In the present case, the design is governed by strength requirements and drift does not govern the design for any of the considered codes.

5.2. Seismic Performance Evaluation

The expected seismic performance of the building designed for different codes, is evaluated using nonlinear static (pushover) analysis as per ASCE 41-06 (2007). Lumped plasticity models for beams and columns have been used. Moment hinges at the ends of all beams, and interacting P-M-M hinges at the ends of all columns, have been assigned as per ASCE 41. Member yield capacity based on expected strength of material, and generalized force deformation relations as recommended by ASCE 41, have been used. The effective stiffness and performance acceptance criteria for beams and columns, have been considered as per the ASCE 41 update (Elwood et al. 2007). Modulus of elasticity of concrete, E_c have been used as recommended by ACI 318M-08 (2008). Mode proportional load pattern as per ASCE 41 has been used for pushover analysis; and Displacement Modification Method (DMM) has been used to obtain the Performance Point (PP) for DBE (corresponding to 10% probability of exceedance in 50 years) and MCE (corresponding to 2% probability of exceedance in 50 years) hazard levels.

5.3. Results and discussion

Figure 5.2 shows the capacity curves and performance points of the building designed for different codes. It can be observed that there is significant variation in capacity curves and performance points of building designed for codes considered herein. This variation is a result of combined effect of the differences in design spectra, effective member stiffness, response reduction factors, load and material factors, as well as load combinations. The capacity curves of the SMRF building designed for Indian code and IMRF building designed for American code are close, as these are the only codes which apply capping on the design period. The buildings designed for other codes (New Zealand and Eurocode) have significantly lower strengths than the buildings of comparable ductility classes designed for Indian and American codes. In case of DBE, all the considered codes result in Life Safety (LS) or better performance levels in both the directions, except in case of Eurocode 8 in both the directions and NZS 1170.5 in transverse direction. In case of MCE, only ASCE 7 (IMRF) designed building could provide Collapse Prevention (CP) performance level and building designed for all other codes show partial/full collapse under MCE hazard.

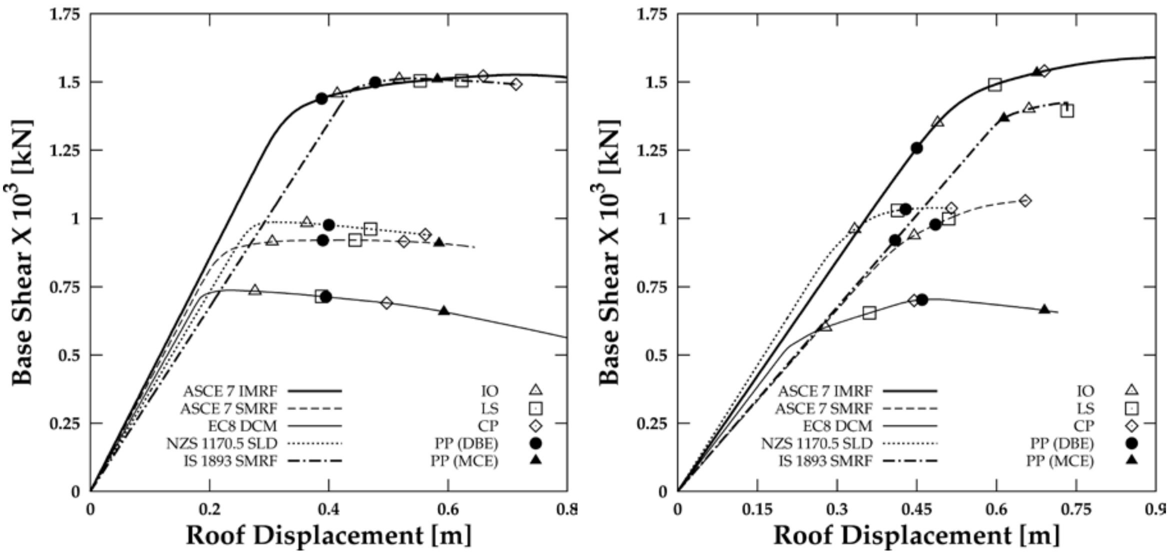

Figure 5.2. Comparison of capacity curves and performance points in longitudinal (left) and transverse (right) directions of the building designed using different codes.

Figures 5.3 and 5.4 show the interstory drift ratios at the performance point of the building designed using different code for DBE and MCE, respectively. There are differences among various codes in case of interstory drift also, but the differences are not as drastic as in case of the capacity curves. However, the peak interstory drift ratio in case of all the codes exceeds 2.5% (the highest limit on design drift among all the considered codes) for DBE, except in case of ASCE 7 (IMRF) in longitudinal direction and IS 1893 in transverse direction. In case of MCE, the peak interstory drift ratio reaches up to or exceeds 4% for most of the codes.

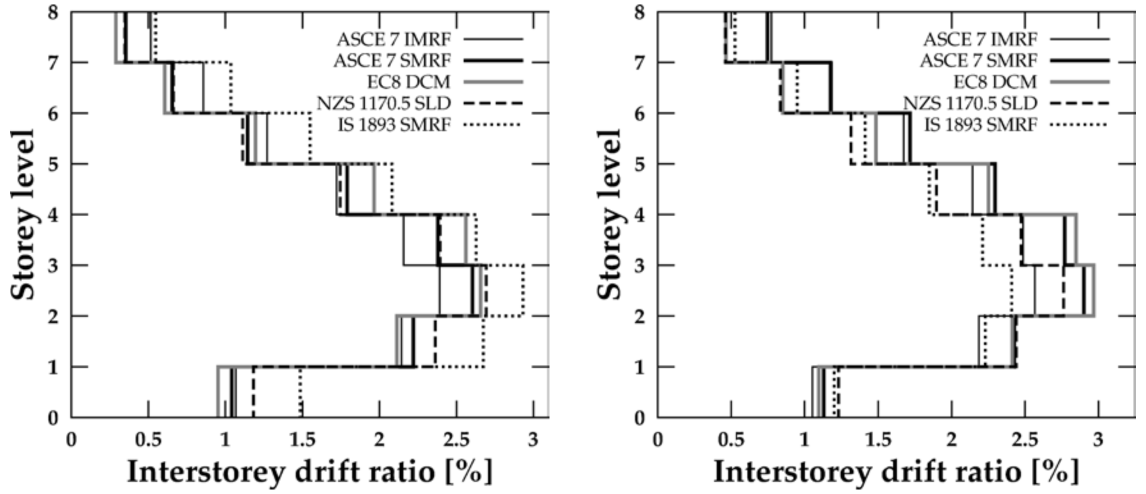

Figure 5.3. Comparison of drift ratios under DBE in longitudinal (left) and transverse (right) directions of the building designed using different codes.

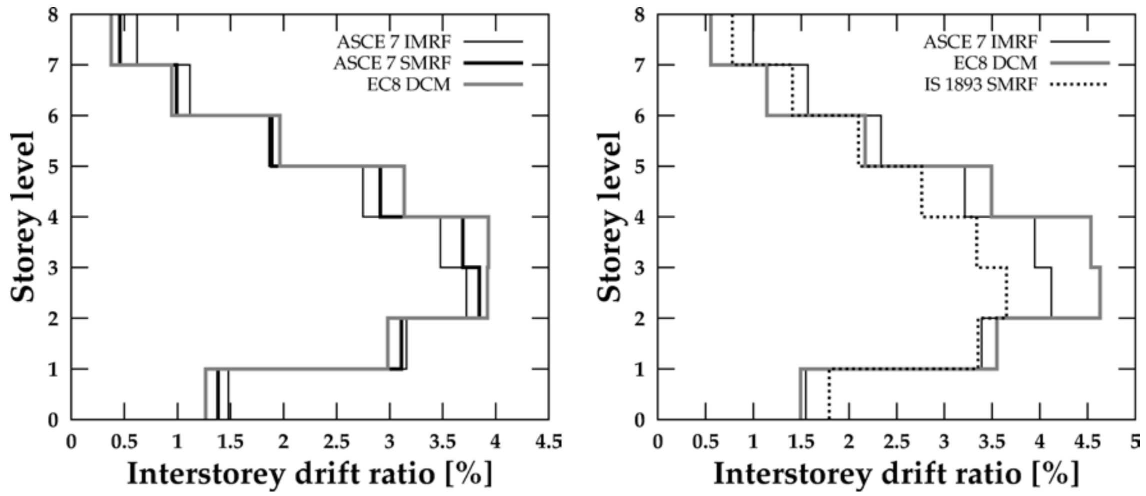

Figure 5.4. Comparison of drift ratios under MCE in longitudinal (left) and transverse (right) directions of the building designed using different codes.

6. CONCLUSIONS

A comparative study of response reduction factors, ductility classes, reinforcement detailing provisions, and seismic performance of a ductile RC frame building designed for four major codes has been performed. The effective stiffness of RC members, procedure to estimate drift, and allowable drift limits are also compared. The comparison of broad ductility classes suggests significant variation in different codes and it is not possible to directly compare the response reduction factors for various ductility classes due to the variation in provisions for reinforcement detailing and capacity design provisions. Most of the codes combine the effect of overstrength and ductility in a single response reduction factor, except for NZS 1170.5, which considers the effect of overstrength separately through a ‘structural performance factor’. Further, only NZS 1170.5 considers the effect of period on response reduction factor. Drift is recognized as an important control parameter by all the codes; however, they differ regarding the effective stiffness of RC members. Further, the procedures to estimate drift and allowable limits on drift also vary. It has been observed that NZS 1170.5 results in the highest design base shear for a given period, for almost all the cases considered in this study. The design base shear as per Eurocode 8 is close to that of NZS 1170.5, while IS 1893 results in the lowest design base shear for a given hazard. The codes also differ significantly on the issue of minimum design base shear, and Eurocode 8 and IS 1893 have no minimum limit on design base shear.

Seismic performance of an eight-story RC frame building designed for different codes has also been compared and it has been observed that the actually provided strength and expected performance of the building is not following the same hierarchy as the design base shear. Further, there is significant variation in the strength capacity of the buildings designed for different codes. The variation in capacity curves may be attributed to differences in design spectra, effective member stiffness values, response reduction or behavior factor, load and material factors, design load combinations, and most importantly, the capping on the design period. All the code designed buildings show Life Safety or better performance level for DBE but show partial/full collapse at MCE. In most of the cases considered in this study, the design was mainly governed by strength while drift was not a governing criteria. However, the interstory drift ratio for most of the code designed buildings is greater than 2.5% (the highest limit on design drift among all the considered codes) for DBE. In case of MCE, the peak interstory drift ratio reaches up to or exceeds 4% for most of the codes.

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